LABORATORY EVALUATION OF RECYCLED CONCRETE AS AGGREGATE IN NEW CONCRETE PAVEMENTS

FINAL PROJECT REPORT

by

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16. Abstract

The Washington State Department of Transportation (WSDOT) has initiated a research project to investigate the use of recycled concrete as aggregates (RCA) in Portland (hydraulic) cement concrete pavements (PCCP). The planned source for the RCA in the project will be from demolished pavements in western Washington, which generally contain very high-quality aggregates. Aggregate quality varies across the state, and concrete made with RCA sourced elsewhere will likely have different properties. This PacTrans proposal is to expand the scope of the WSDOT project to include additional sources of RCA as well as evaluations of the RCA properties for the purpose of establishing performance criteria necessary for successful application in PCCP. The goal of the combined projects is to evaluate the use of RCA for widespread application in concrete pavements in Washington State and beyond.

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EXECUTIVE SUMMARY

This study was performed for the Washington State Department of Transportation (WSDOT) and evaluated the use of recycled concrete as a replacement for natural aggregates in new portland cement concrete pavements. Recycled concrete aggregate (RCA) produced from demolished pavements in three geographically-dispersed locations in Washington state were used to perform tests on aggregate characteristics, fresh concrete properties, and hardened concrete properties. Variables included the source of the RCA, percent replacement of coarse natural aggregate with RCA (0%, 15%, 30% and 45%), and percent replacement of portland cement with type F fly ash (0% or 20%).

Four tests were used to characterize RCA properties including specific gravity, absorption capacity, Los Angeles abrasion loss, degradation value in various conditions, and alkali-silica reactivity. The conditions for which the degradation value was determined included the as-delivered unprocessed RCA, the processed RCA, and processed RCA mixed with natural aggregate at rates of 15%, 30%, and 45%. Overall, tests showed that RCA has a lower specific gravity, greater absorption capacity, and meets the WSDOT requirements on Los Angeles abrasion loss and degradation value once processed. Additional tests may need to be performed to evaluate potential deleterious expansion due to alkali-silica reactivity.

Three tests were used to determine if RCA had any effects on the properties of fresh concrete including slump, air content by the pressure method, and density. Slump and air content were controlled parameters in the batching process, with targets specified by WSDOT of 1-3 inches for slump and 4-7% for air content. It was a goal during the mix process to make concrete mixtures within the low end of each of those ranges. Slump was controlled by withholding mix

water or adding water-reducing admixture (WRA), and air content was controlled by the amount of air entraining admixture used in the batch. RCA was found to decrease the slump and density of fresh concrete. RCA had no significant effect on air content.

Five tests were used to determine the effects of RCA on hardened concrete properties including compressive strength, modulus of rupture, coefficient of thermal expansion, drying shrinkage, and freeze-thaw durability. Test results showed that up to a 45% replacement of coarse natural aggregate with RCA had no significant effect on any of the hardened concrete properties tested. In addition, all samples tested met WSDOT minimum strength requirements for use in concrete pavements. It should be emphasized that these results were obtained using RCA obtained from demolished pavements incorporating high-quality original materials.

The results of this study indicate that RCAs of similar quality to those incorporated in this research would be viable for use in new concrete pavements. RCA had no significant effects on the compressive strength, modulus of rupture, coefficient of thermal expansion, drying shrinkage, or freeze-thaw durability of hardened concrete for up to a 45% replacement of coarse RCA for natural coarse aggregate. In addition, all results from tests on the RCA from the three sources and results concrete incorporating this RCA at up to 45% replacement met WSDOT requirements for use in new concrete pavements.

Further study should be performed to determine the minimum criteria RCA must meet for use as an aggregate in concrete pavements as well as to evaluate higher RCA replacement rates.

CHAPTER 1: INTRODUCTION

1.1 Background

The Washington State Department of Transportation (WSDOT) manages about 2,400 lane-miles of concrete roadway, the majority of which were constructed in the 1950s and 1960s. At that time, the estimated design life of these roadways was only 20 years. As a result, many of these concrete pavements have greatly surpassed their original design life and expected traffic loading, and they are in need of replacement (Washington State Department of Transportation, 2010). Due to costs associated with replacement in addition to economic limitations, much of this needed replacement has been backlogged.

In response to this situation, both the Federal Highway Administration (Wright, 2006) and the WSDOT are interested in alternatives that promote cheaper and more sustainable pavement construction practices. One such alternative is to incorporate recycled concrete as aggregate in new portland cement concrete pavements. According to a report from Iowa State University, recycled concrete aggregates (RCA) can reduce costs, environmental impacts, and project delivery time when used in concrete pavements (Garber, et al., 2011). A WSDOT report adds that dwindling supplies of high-quality natural aggregate, increasingly limited landfill space, swelling disposal costs, emphasis on the conservation of natural resources, and reduced construction costs arise as convincing reasons to consider the use of RCA (Anderson, Uhlmeyer, & Russell, 2009).

A 2004 Federal Highway Administration study found that only 11 states actively use RCA in new portland cement concrete, though 41 were reported to recycle concrete in roadway base construction (Gonzalez & Moo-Young, 2004). This means that, despite being an approach that has been around for several decades, the practice of recycling concrete into new concrete pavement is uncommon.

Previous studies of the effects of using RCA in new portland cement concrete have varied in their conclusions, largely because aggregate quality varied widely in the original concrete from which the RCA was produced. The quality and properties of RCA has been found to be similar to the quality and properties of the original aggregate (Garber, et al., 2011), and RCA quality impacts the quality of the concrete in which it is incorporated (Limbachiya, Meddah, & Ouchagour, 2012). Thus, recycling concrete where the original aggregates were of low quality would likely yield RCA which is inadequate for use.

According to a 2010 report, WSDOT's use of RCA is limited to ballast, gravel base, crushed surfacing, backfill for foundations, walls and drains, select and common borrow foundations, and bank run gravel. The report adds that it is not permitted to use RCA as an aggregate in concrete pavements (Washington State Department of Transportation, 2010). It has been recognized by WSDOT that choosing to not reuse high-quality aggregate could be wasteful, and that exploring RCA could provide relief to the exhaustion of natural aggregate resources and WSDOT's transportation constructions budget. The research in this report was performed at Washington State University with the goal of evaluating the suitability of using recycled concrete as aggregates in new portland cement pavements in Washington State.

1.2 Scope and Objectives

The primary objective of this research is to evaluate the effectiveness of RCA created from demolished concrete pavements in Washington State as aggregates in new portland cement concrete pavements. Three sources of RCA were investigated in this study, incorporating demolished concrete pavements from western, eastern, and central Washington. The variables evaluated in this study included the replacement of coarse natural aggregate with coarse RCA at replacement levels of 0%, 15%, 30% and 45%, the source of the RCA, and a 0% or 20% replacement of portland cement with fly ash. In total, twenty concrete batches were created; two non-RCA mixtures and a series of six mixtures for each source of RCA were evaluated to investigate the effects on concrete properties of the variables described above. Fresh concrete properties evaluated included slump, air content, and density. Hardened concrete properties evaluated included compressive strength, modulus of rupture, coefficient of thermal expansion, drying shrinkage, and freeze-thaw durability. Additional tests were performed on the RCA including absorption capacity, specific gravity, Los Angeles abrasion loss, degradation factor, and alkali-silica reactivity.

Three sources of RCA are investigated in this study to validate the findings of the study across different geographic sources within Washington. Source A came from crushed PCCP roadway panels from I-90 near Roslyn, Washington (Mjelde, 2013). Source B came from crushed PCCP runway panels from Fairchild Air Force Base near Spokane, Washington (Spry, 2013). Source C came from crushed PCCP roadway panels from I-5 near Woodmont Beach, Washington (Boyle, 2013). Tests performed on mixes containing RCA sources A, B, and C were the same with two exceptions. Tests on source A did not include testing for drying shrinkage and freeze-thaw durability tests were performed only for source C.

Conclusions are reached and recommendations provided for the use of RCA as a replacement for natural coarse aggregates in new concrete pavements in Washington State.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

In this chapter, the results of previous research evaluating the characteristics of coarse RCA and its effects on new fresh and hardened concrete are discussed.

2.2 RCA Properties

Properties of RCA are necessary to determine its suitability for use as an aggregate in new concrete pavements and for proper design of the concrete mix design.

2.2.1 Specific Gravity and Absorption Capacity

Previous research has shown that RCA has a lower specific gravity than that of natural aggregates (NA). Typically, the specific gravity of RCA is 2.1 to 2.4, whereas NA is 2.4 to 2.9 (Snyder, 2006). The lower specific gravity exhibited by RCA is due to the adhered mortar portion, which is less dense than NA because of entrained air and porosity from the original concrete structure (Snyder, 2006). This is also the reason for the increased absorption capacity of RCA, which is typically 3.7% to 8.7% according to previous studies. In comparison, NA typically has absorption capacities of 0.8% to 3.7% (Snyder, 2006).

It is important to determine the specific gravities of aggregates to achieve proper proportioning of the mix materials, including the substitution of NA with RCA by volume instead of weight. Substitution by volume will prevent underestimation of overall mix yield in addition to more accurately controlling proportions of water and cement in the mix design.

2.2.2 Los Angeles Abrasion Loss

The Los Angeles abrasion test is a method for determining an aggregate's resistance to abrasion. It can be used as a measure of an aggregate's suitability for use in concrete because higher loss values often indicate undesirable softness of an aggregate (Snyder, 2006). Typical values of mass loss for RCA are 20-45% compared to 15-30% for NA (Snyder, 2006). The values usually indicate internal structural strength of the aggregates and the quality of the aggregate. Additional loss occurs for recycled aggregates in this test, dependent on the amounts of adhered mortar, due to the weakness of mortar-to-aggregate bond strengths (Amorim, de Brito, & Evangelista, 2012).

WSDOT requires that aggregates in new concrete pavements do not exceed a 35% loss from the Los Angeles abrasion test (WSDOT, 2012). According to Snyder's report, discussed earlier, most RCA typically meets this criterion.

2.2.3 Degradation Factor

The degradation factor measures the resistance to abrasion in the presence of water. Based on the aforementioned increased amounts of loss for RCA in the Los Angeles abrasion test, it can be expected that RCA performs slightly worse than NA in tests for the degradation factor. WSDOT requires that aggregates have a minimum degradation factor of 30 in order to allow usage in concrete pavement mixtures (WSDOT, 2012).

2.2.4 Alkali-Silica Reactivity

The alkali-silica reaction (ASR) occurs when aggregates with siliceous composition react to alkalis in concrete paste to create a gel which expands when it absorbs water. This process can create internal pressure and cracking in concrete, and creates durability concerns. ASR can be mitigated through the use of fly ash or low alkali cement.

RCA has been noted as having an increased potential for the ASR in comparison to NA. One study suggests that this is because the crushing process exposes more internal surface area of aggregates, increasing the accessible chemical potential and reactivity of the aggregates (Ideker, Adams, Tanner, & Jones, 2011; Snyder, 2006). In addition, a multi-laboratory study showed that recycled concrete which had already experienced deleterious ASR in the field during the primary service life still had significant potential for expansion (Ideker, Adams, Tanner, & Jones, 2011). In other words, it should be expected that recycled concretes which experienced ASR during their primary service life will also experience ASR during any recycled applications.

ASR is typically not a concern for aggregates in Washington State (Anderson, Uhlmeyer, & Russell, 2009). Thus, it is not expected that RCA from pavements in Washington will experience ASR.

2.3 Fresh Concrete Properties

This section discusses results from previous research in regards to the effect of coarse RCA on the workability, air content, and density of fresh concrete.

2.3.1 Workability

RCA replacement for coarse NA has been shown to decrease workability of fresh concrete mixes. One reason for this is that RCA, depending on the crushing process, has more friction potential due to angular shape and rougher surface conditions than NA (Amorim, de

Brito, & Evangelista, 2012). The greater absorption capacity of RCA can also result in a reduction in workability by effectively reducing the water-cement ratio (Garber, et al., 2011). Several solutions have been suggested to counteract this effect including the use of water reducing admixture, fly ash, or a combination of the two. In addition to reduced workability, fresh concrete mixtures incorporating RCA commonly experience more rapid slump loss due to the increased absorption capacity of RCA (Snyder, 2006).

2.3.2 Air Content

The air contents of concrete mixtures with coarse RCA are slightly higher and more variable than those with only NA. This is attributable to the entrained air and greater porosity of the RCAs due to the adhered mortar (Snyder, 2006). As a result, target air contents should be raised in order for concrete mixtures incorporating RCA to achieve the same durability performance as those with only NA (Snyder, 2006). However, in order to circumvent the variability of this characteristic, it may be better practice to remove as much as possible of the adhered mortar portion from RCA prior to usage.

2.3.3 Density

It can be expected from the discussion in preceding sections that the inclusion of coarse RCA in mix design results in a reduction in mix density. As noted before, RCA has a smaller specific gravity, and is therefore less dense due to the greater amount of entrained air and porosity of the adhered mortar. Consequently, concretes mixtures incorporating coarse RCA will have a reduced density.

2.4 Hardened Concrete Properties

This section discusses the effects of RCA replacement of coarse NA on the performance of hardened concrete as determined by compressive strength, modulus of rupture, drying shrinkage, and durability.

2.4.1 Compressive Strength

The compressive strengths of concretes incorporating coarse RCA, in general, are the same if not slightly lower than those with only NA (Snyder, 2006). The degree to which RCA reduces compressive strength has been a point of disagreement in a number of studies. A 2012 investigation concluded that compressive strength is relatively unaffected by the replacement of NA with coarse RCA, theorizing that strength is maintained because the RCA has better interfacial transition zone with new cement paste as well the possible presence of unhydrated cement on the RCA (Amorim, de Brito, & Evangelista, 2012). A different study found that, in general, compressive strengths were slightly reduced with greater levels of RCA replacement, noting that results were often inconsistent as a result of the RCA's inherent inconsistency (Limbachiya, Meddah, & Ouchagour, 2012). One report suggests that the strength decrease can be explained by increase in air content as a result of the RCA (Snyder, 2006).

2.4.2 Modulus of Rupture

The modulus of rupture (MOR) is a measure of a brittle material's flexural strength. A 2006 report found that coarse RCA replacement of coarse NA reduced the MOR of a concrete mixture by up to eight percent (Snyder, 2006). As with compressive strength, this was partially attributed to the mixture's increase in air content as a result of the RCA. Additionally, the influence of RCA on MOR is heavily dependent upon the mortar-to-aggregate bond strength (Snyder, 2006). Thus, RCA with weaker bond strengths will more heavily reduce the MOR of a mixture's hardened concrete.

2.4.3 Coefficient of Thermal Expansion

The coefficient of thermal expansion (CTE) quantifies a relationship between length change and temperature variation. Aggregate properties have the greatest effect of many influencing factors in a concrete mixture on the coefficient of thermal expansion (Portland Cement Association, 2002). Thus, it is likely that RCA replacement will have some relationship with coefficient of thermal expansion. According to one report, RCA will reduce the CTE of concrete (Smith & Tighe, 2009). This would results in a performance increase of concrete because there would be less expansion and shrinkage with temperature change.

2.4.4 Drying Shrinkage

Drying shrinkage is a long-term property of concrete. It depends upon the amount of excess mix water, paste content, and how well the aggregate restrains paste shrinkage. The use of coarse RCA results in excess water in the pores of the RCA as well as an increase in paste content. Thus, coarse RCA replacement of coarse NA typically results in an increase in drying shrinkage (Snyder, 2006).

2.4.5 Freeze-Thaw Durability

The resistance to degradation and cracking when concrete undergoes shrinkage and expansion associated with freezing and thawing is an important characteristic of concrete mixtures. Typically, concretes with greater amounts of entrained air have better performance because there is more volume into which freezing water can expand (Portland Cement Association, 2002). As a result, coarse RCA replacement typically results in better concrete performance in comparison to concretes containing solely NA (Snyder, 2006).

CHAPTER 3: EXPERIMENTAL PROGRAM

3.1 Introduction

All batch proportions for the mixes of this study were based upon a reference portland cement concrete pavement (PCCP) mix design, C8022, used by Central Pre-Mix and provided by WSDOT. This reference mix is given in Appendix A. Variable investigated included three different sources of RCA, replacement of coarse NA in the reference mix design with coarse RCA at different percentages by volume, and replacement of cement with fly ash by weight. Six batches were performed for each of the three RCA sources with different replacement levels of coarse NA, and two 0% RCA batches were used as baseline mix designs. A summary of the mixes and replacement proportions is shown in Table 3.1. Concrete batches were labeled according to their RCA source, RCA replacement percentage, and fly ash percentage. For example, a 30% RCA replacement from source A includes 20% fly ash substitution and is labeled as A-30-20. Batches which did not include RCA were labeled as source X.

Table 3.1 Concrete Batch Parameters

Mix No.	Percent coarse RCA Substitution	Percent Fly Ash Substitution
1	0%	0%
2	15%	0%
3	30%	0%
4	45%	0%
5	0%	20%
6	15%	20%
7	30%	20%
8	45%	20%

3.2 Materials

Materials in this project were the same as those used in the reference PCCP mix design, with the exception of the substitution of RCA and fly ash. All materials, except for the RCA, met WSDOT requirements.

3.2.1 Natural Aggregates

NAs were obtained from WSDOT-approved aggregate pits located in Spokane, Washington and delivered to Washington State University as five components. Coarse aggregate components conformed to AASHTO No. 467 gradation and included 1 1/2 in. round combined, 3/4 in. round combined, and 3/8 in. round combined. Fine aggregate components conformed to AASHTO Type I gradation and included coarse sand combined and blend sand combined.

In order to accommodate batching procedures, the various components of the original aggregate distribution were mixed into coarse and fine stockpiles that met the aggregate distribution for the reference design mix. The coarse stockpile included all of the 1½ in. round and ¾ in. round components as well as the ⅜ in. round portion that did not pass a No. 4 sieve. The fine stockpile included the ⅜ in. round portion that passed the No. 4 sieve as well as blend sand combined and coarse sand combined. These stockpiles were stored indoors.

3.2.2 RCA

Three sources of RCA were used in this project and were labeled as A, B, and C. Source A came from crushed PCCP roadway panels from I-90 near Roslyn, Washington. Source B came from crushed PCCP runway panels from Fairchild Air Force Base near Spokane, Washington.

Source C came from crushed PCCP roadway panels from I-5 near Woodmont Beach, Washington.

The RCA was produced by crushing the panels using a jaw crusher, and the resulting pieces sieved to 1¼ in. minus for all three sources. Source B was further crushed using a comb crusher. Through this process, the RCA contained both coarse and fine materials. This research investigated only coarse RCA replacement of coarse natural aggregate, so the portion of the RCA passing a No. 4 sieve was discarded. The remainder of the RCA was sieved and washed to remove fines. After drying, the RCA was then recombined to conform to AASHTO Grading No. 467. The processed RCA was then stockpiled indoors for later use.

After the RCAs were sieved and recombined to meet the AASHTO No. 467 size distribution, there was a 31% aggregate yield from piles of source A, 26% yield from piles of source B, and a 68% yield from piles of source C. These yield rates are approximate as the size distribution of the as-delivered RCA varied significantly throughout the piles.

3.2.3 Cementitious Materials

Two cementitious materials were used for this study. The cement used was type I-II portland cement produced by Ash Grove Cement in Durkee, Oregon. The fly ash used as Type F fly ash from Centralia, Washington.

3.2.4 Admixtures

Two admixtures were used in this study. The air-entraining admixture (AEA) was Daravair 1000, and the water-reducing admixture (WRA) was WRDA 64. Both admixtures were the same as those used in the reference design mix and were manufactured by WR Grace & Co.

3.3 Test Methods

This section describes the test methods used in this study to characterize aggregate properties, fresh concrete properties, and hardened concrete properties.

3.3.1 RCA Characteristics Tests

Four tests were used to determine five characteristics of the recycled aggregates. The specific gravity and absorption capacity were determined using methods outlined in AASHTO T 85, “Specific Gravity and Absorption of Coarse Aggregate.” The Los Angeles abrasion loss was determined using AASHTO T 96, “Standard Method of Test for Resistance of Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine.” The degradation value was tested by WSDOT using WSDOT T 113, “Method of Test for Determination of Degradation Value.” ASR reactivity was tested using AASHTO T 303, “Accelerated Detection of Potentially Deleterious Expansion of Mortar Bars Due to Alkali-Silica Reaction.”

3.3.2 Fresh Concrete Tests

Three tests were performed on fresh concrete samples for each concrete mix. Slump, a measure of workability, was determined using AASHTO T 119, “Slump of Hydraulic Cement Concrete.” Air content was determined using AASHTO T 152, “Air Content of Freshly Mixed Concrete by the Pressure Method.” The density of concrete each concrete mix was determined using AASHTO T 121, “Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete.”

3.3.3 Hardened Concrete Tests

Five tests were used to determine the effects of RCA on the hardened concrete, including compressive strength, modulus of rupture, coefficient of thermal expansion, drying shrinkage, and freeze-thaw durability. Separate concrete batches were made for freeze-thaw durability tests, while all other tests were performed on the same concrete batch for each mix design.

The compressive strength of concrete was determined according to AASHTO T 22, "Compressive Strength of Cylindrical Concrete Specimens." In conformance with AASHTO T 22, the loading rate was 47,500-71,250 pounds per minute, corresponding to the 28-42 psi per second loading rate specified by AASHTO T 22. Fourteen compression cylinders per batch, of dimensions 12 in. length and 6 in. diameter, were molded for this test. AASHTO T22 required that moist cured specimens be tested in a moist condition, thus all specimens were tested shortly after their removal from their respective curing tubs. Steel caps lined with neoprene pads were also used to avoid issues with load transfer due to imperfections on the top and bottom surfaces. Compression samples were tested as follows; 3 samples at 7 days of age, 3 samples at 14 days of age, 5 samples at 28 days of age, and 3 samples at 90 days of age. Load was applied using a Tinius Olsen Universal Testing Machine.

The modulus of rupture was determined according to AASHTO T 177, "Flexural Strength of Concrete (Using Simple Beam with Center –Point Loading)." Beams procured for this test were 21 in. long with a 6 in. by 6 in. cross section. For each batch, all 5 beams molded for each batch were tested after 14 days of curing. In accordance with WSDOT Test Method T 808, "Method for Making Flexural Test Beams," beams were tested in a moist condition. Steel rollers were used as supports and for load application. In addition, moist leather strips were placed between rollers and concrete to apply uniform pressure across the loading points. Load was applied using the Tinius Olsen Universal Testing Machine at a rate of 1000-1400 pounds per minute in conformance with the specification.

The CTE of concrete specimen was established using methods outlined in AASHTO T 336, "Coefficient of Thermal Expansion of Hydraulic Cement Concrete." Three samples for each batch were prepared with diameters of 4 in. and heights of 8 in. CTE tests were performed after specimens had cured for 28 days. In order to conform to the specification, the specimens were cut to a 7 in. length prior to testing using a lapidary saw. A stainless steel support frame employing a submersible linear variable differential transformer (LVDT) was used to measure length fluctuation of the specimen while in a temperature controlled water bath. Two thermocouples were used to monitor water bath temperature at shallow and deeper depths to maintain an average bath temperature. Thus, the data acquisition system recorded the nominal length of the specimen as well as the water temperature near the top and near the bottom of the water bath.

Procedures for obtaining measurements of the drying shrinkage of concrete specimens largely followed AASHTO T 160, "Length Change of Hardened Hydraulic Cement Mortar and Concrete." For each batch, three beams with 4 in. by 4 in. cross sections and specimen lengths of 11.25 in. were prepared. Gauge studs were placed in the ends of specimens for the use of a length comparator to monitor length change over time. After 28 days of curing in a lime-saturated bath, drying shrinkage specimens were placed in an environmentally-controlled chamber which maintained a relative humidity of 50% and a temperature of 23 degrees Celsius. However, the relative humidity fluctuated by approximately 5%, and the temperature fluctuated by approximately 3 degrees Celsius. Rates of evaporation and circulation were not monitored, as required in the specification, due to the lack of an atmometer available for use in this study. Measurements were taken, with day 0 beginning upon their removal from lime saturated water baths after 28 days of curing, at ages of 0 days, 4 days, 7 days, 14 days, 28 days, 8 weeks, and 16 weeks.

Testing to determine the effects of RCA on concrete resistance to freezing and thawing cycles was performed based on AASHTO T 161, "Resistance of Concrete to Rapid Freezing and Thawing." Procedure A, rapid freezing and thawing in water, was followed. Six specimens for each of the X-0-0 and C-45-0 batches were prepared for this test. The specimens were 16 in. in length and 3 in. by 4 in. in cross sections. An extra specimen was prepared with a thermocouple cast within it to accurately monitor internal temperatures of the specimens. After 14 days of curing in lime-saturated water, the specimens were tested using a nonstandard nondestructive flexural modulus test in addition to the modal vibration frequency based on methods described in the specification. The nonstandard flexural modulus test was performed using a third point cyclic beam loading while monitoring deflection for 100 load cycles, as shown in Figure 3.1.

Figure 3.1 Nonstandard Flexural Modulus with Third Point Loading Test Setup

The modal frequency test was performed based on the freeze-thaw specification using a modal hammer, an accelerometer, a foam pad to reduce data noise, and computer software which analyzed the output of the system to show the concentrations of various modal frequencies in the concrete specimens, as shown in Figure 3.2.

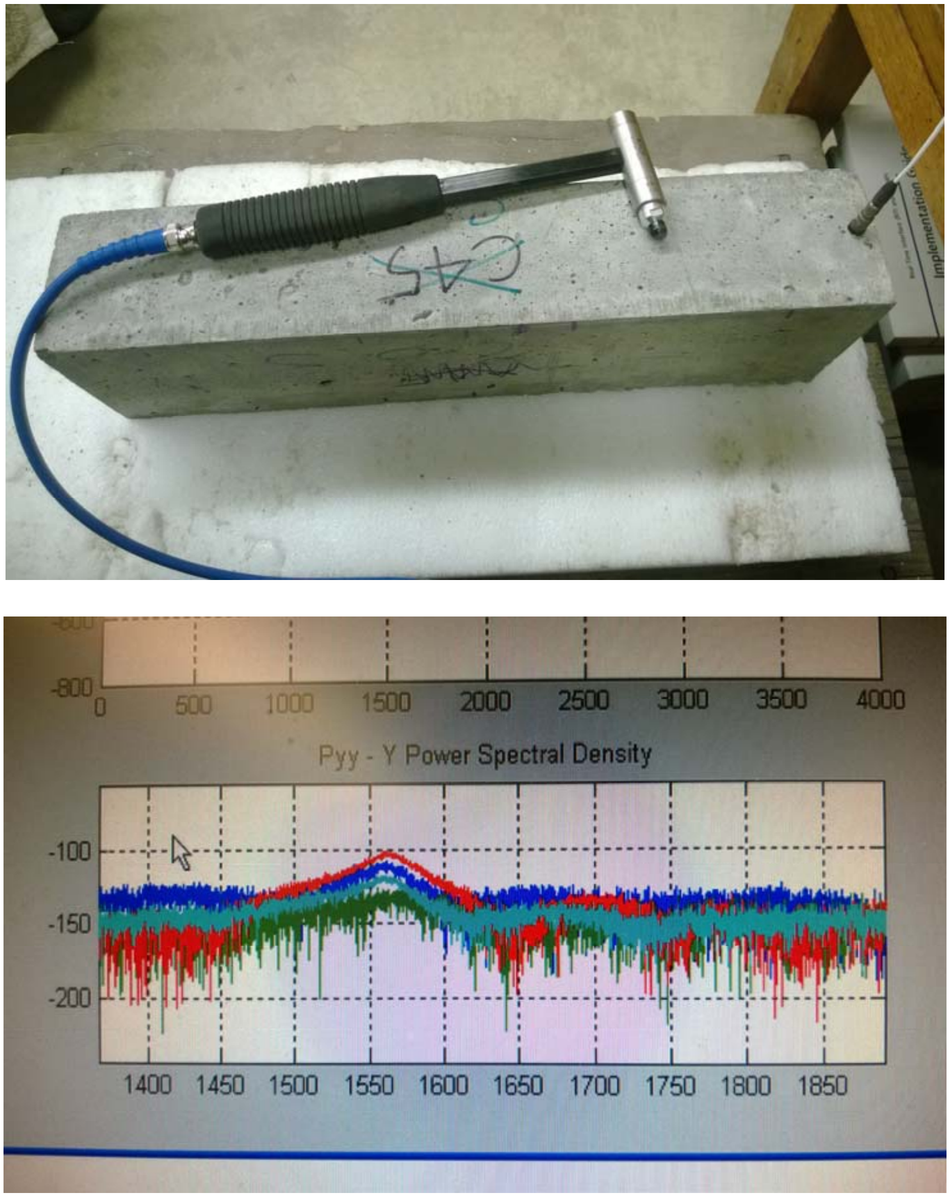


Figure 3.2 Test Setup for Freeze-Thaw Modal Test and Visualized Computer Output

Half of the specimens produced from each batch were then placed in aluminum containers which allowed for a 1/32-1/8 in. gap on all sides. These containers were filled with tap water to undergo freezing and thawing cycles. The remaining three specimens from each batch were kept in the lime-saturated water throughout the experiment to function as control specimens. The cycling of specimen internal temperatures was done with target thaw internal temperatures of 2 to 6 degrees Celsius and target internal freezing temperatures of -15 to -20 degrees Celsius. Freezing and thawing periods were 2 hours each, resulting in a full cycle time of 4 hours. The time required to change internal temperatures of specimens from 2 to -12 degrees Celsius or from -12 to 2 degrees Celsius was at least half of the respective heating or cooling periods. Specimens were removed from the testing chambers and tested for non-standard flexural modulus and modal vibration frequency at room temperature at 50 and 100 cycles. Testing at room temperature differs from the specification and was selected to ensure the curing specimens and freeze-thaw specimens had the same internal temperature during testing.

3.4 Concrete Mixing

This section describes the proportioning of concrete mixtures, the procedures used to produce the concrete mixtures, and the molding and preparation of concrete specimens for testing.

3.4.1 Material Preparation

The aggregate quantities listed in the reference mix design are based upon saturated surface dry (SSD) moisture conditions. SSD means that aggregates are at their absorption capacity with no excess water on their surfaces. Moisture content of the aggregates prior to mixing was not controlled in this study. In order to address this, the SSD bulk specific gravity and absorption capacity of the recombined aggregates of the three stockpiles had been determined prior to mixing.

In order to account for the unknown moisture conditions of the aggregate stockpiles, the following procedures were used. First, the existing moisture condition of the aggregate stockpiles was determined the day prior to mixing. This was done by weighing a sample of the aggregate in each stockpile and then weighing the same sample once completely oven dried. The difference in weight was used to calculate the moisture condition of the aggregate stockpiles.

The absorption capacities and bulk SSD specific gravities for the two NA and the RCA stockpiles were known. The difference between the calculated existing moisture condition and the absorption capacity was calculated. If the existing moisture content was less than SSD condition, water was added to the total mixing water to be weighed out to bring the aggregates to the SSD condition. Aggregate quantities would be reduced in weight equal to the weight of water added in order keep the total weight of materials in the batch constant. If the existing moisture content was greater than SSD condition, water was reduced from the overall mixing water equal to the amount of excess water in the aggregates. In this case, aggregate quantities were increased in weight equal to the weight of water subtracted from the mixture. The adjustment of water and aggregates as described ensured that aggregates effectively achieved SSD condition and that the water specified in the reference mix design was available for the hydrating cementitious materials.

After moisture adjustments were made, the quantities of aggregates, cement, fly ash, water, and admixtures were measured and placed near the concrete mixer.

3.4.2 Concrete Mixing Procedures

The mixing procedure began after all materials were gathered by first using a cement and water mixture to prepare the mixer. The cement and water mixture was used to coat the sides and blades of the mixer drum and prevent the loss of a mixture's materials to the mixer surfaces. The excess cement and water mixture was poured from the mixer drum once full coating had been achieved.

All aggregates were then added to the mixing drum. The mixer was then turned on and lowered while a portion of mix water was added to the aggregates. This amount was subjective, with the goal of providing enough water for aggregates to approach a saturated condition. The aggregates were then mixed for approximately 3-5 minutes so that they were well blended.

Once the aggregates had been mixed, and with the mixer still running, all cementitious materials and the majority of the mixture's remaining water were added. Approximately 2-5 pounds of water was withheld to prevent the mixture from exceeding WSDOT's provided target of a 1-3 in. slump. After approximately 1-2 minutes of mixing, the mixer was turned off so that the sides of the mixing drum could be scraped to remove any materials that had adhered to the sides and were not mixing with the rest of the concrete mixture. The mixer was then turned back on until the mixture had mixed for a total of five minutes. During this time, additional water from the remaining 2-5 pounds was added until the mixture had the qualitative appearance of having reached the minimum slump limit of 1 in.

The mixer was stopped after having mixed for five minutes, and a slump test was performed. Slump was tested in accordance with ASTM C 143, "Standard Test method for Slump of Hydraulic-Cement Concrete." If the minimum slump limitation was achieved, the mixing process proceeded. If the minimum slump was not achieved, additional mixing water was added and the mixture was mixed for an additional 2-3 minutes prior to a second slump test. If all mix water had been added, then WRA was added at the same time as AEA. If the minimum slump had been achieved, no WRA was added to the mixture. Any remaining mix water was weighed and subtracted from the reference mix water amount in order to accurately characterize the effective water and water-cementitious products ratio of a mixture.

Following the preliminary slump tests, the next step was to add all needed admixtures. WRA was included if slump had not been achieved with the full inclusion of all mix water. AEA was also added in this step. The amounts of WRA and AEA in the mix were based on previous experiences and the goal of approaching slump and air content from the lower end of the WSDOT provided limits for air content and slump. WSDOT provided a 4-7% target for air content.

After the admixtures were added, the mixture was again mixed for five minutes and then turned off. At this time the slump and air content of the mixture were measured. Air content was measured using ASTM C231, "Stand Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method." If both slump and air content were within the provided limits, the density of the mixture was then measured. If the slump or air content were still too low, then additional AEA or WRA was added and allowed to mix for three minutes. The slump and air content were measured again to ensure acceptable levels of each had been obtained, and then the density was measured. No mixture in this study required additional mixing periods beyond the second admixture mixing period.

After the density had been measured, the mixed concrete was transported from the mixer to the sample molding area.

3.4.3 Sample Preparation

Samples were prepared in accordance with AASHTO T R 39, "Making and Curing Concrete Test Specimens in a Laboratory." A total of 20 main batches were prepared along with 2 unique freeze-thaw test sample batches. For each of the main batches, the samples prepared included 14 cylinders for compressive strength tests, 3 cylinders for CTE tests, 5 beams for MOR tests, and 3 beams for drying shrinkage testing. The sizes of these samples are described earlier in this section. No drying shrinkage samples were prepared for batches incorporating RCA from source A. In addition, 6 beam samples were prepared for freeze-thaw testing for each of the two freeze-thaw batches.

Cylindrical specimens were rodded 25 times between three equal-height lifts prior to being smoothed with a trowel and then covered with a plastic cap. Beam molds were vibrated and rodded, and then covered with a damp towel and a sheet of plastic. All samples were then allowed to cure for 24 hours.

Following the initial 24-hour curing period, samples were de-molded and transferred into tubs containing lime-saturated water as shown in Figure 3.3.

Figure 3.3 Lime-Saturated Water Curing Tub

CHAPTER 4: TEST RESULTS AND DISCUSION

4.1 Introduction

The results of tests performed on samples from each of three RCA sources and from concrete batches incorporating the RCA are presented and discussed in this chapter. These include the results from tests to determine properties of the natural aggregates and RCA, fresh concrete tests, and hardened concrete tests. The mix quantities for each investigated concrete batch are presented in Appendix B.

4.2 Natural Aggregate Characteristics

Virgin natural aggregates used in this project were acquired from WSDOT-approved aggregate pits located in Spokane, Washington. As a result, all of the NAs met WSDOT aggregate requirements for use in concrete pavements. Sieve analyses showed that the gradations of the NA components used in this study were the same as those in the reference mix design. Sieve analyses also confirmed that the stockpiles of combined coarse aggregate conformed to AASHTO Grading No. 467 and the combined fine aggregate stockpile conformed to Class 1 fine aggregate grading. Values of the SSD bulk specific gravity and absorption capacity are given for the combined coarse and fine stockpiles in Table 4.1

Table 4.1 Properties of Combined Natural Aggregate Stockpiles

Combined Natural Aggregate Type	SSD Bulk SG	Absorption Capacity
Fine Stockpile	2.59	1.96%
Coarse Stockpile	2.63	1.17%

4.3 RCA Characteristics

The SSD bulk specific gravities and absorption capacities of the coarse recombined RCA stockpiles from sources A, B, and C are summarized in Table 4.2.

Table 4.2 Properties of Recombined Coarse RCA Stockpile

Source	SSD Bulk SG	Absorption Capacity
A	2.52	3.87%
B	2.53	3.30%
C	2.57	3.05%

The bulk specific gravity for each RCA source was less than that for the coarse NA by 2-4%, which confirms expectations that RCA is less dense than natural coarse aggregate. Additionally, the absorption capacities of the coarse RCA were 161-231% larger than that of natural coarse aggregate. The minor decrease in specific gravity and large increase in absorption capacity are explained by the lower density and porous entrained air structure of the RCA's adhered mortar.

The durability properties of RCA, which includes results from the Los Angeles wear and degradation factor tests, are summarized in Table 4.3. Degradation value tests were run for a number of conditions and rates of mixture with coarse NA, as listed in Table 4.3.

Table 4.3 Durability Properties of RCA

		Degradation Value Tests				
Source	**Los Angeles Wear Loss**	**Raw As-Delivered RCA**	**100% RCA**	**15% RCA**	**30% RCA**	**45% RCA**
A	29%	15	55	77	75	70
B	20%	37	49	77	75	73
C	21%	40	69	76	76	78

The WSDOT specification requires that aggregates to be used in concrete pavements must have a Los Angeles wear loss that is less than 35%, and that the degradation factor must be greater than 30 (WSDOT, 2012). All three sources of RCA met WSDOT requirements for these tests, with the exception of the raw RCA from source A. The raw RCA for all sources contained fines that were not incorporated in the mixes of this project. For comparison, WSDOT reported that aggregates in the pit from which all of the natural coarse aggregates used in this study were obtained have a Los Angeles abrasion loss of 15% (WSDOT, 2010). The RCA had a Los Angeles abrasion loss which is 33-93% greater than that for the NA, and the degradation factor decreased with increased RCA replacement ratios. These two results are consistent with expectations, and are a result of the adhered mortar present on the RCA and the relatively weak paste-to-aggregate bond strength. These results also show that removing fines in raw RCA increases the degradation performance significantly, by over 200% for RCA from source A. The large increase is due to the fact that, in its raw condition, RCA from source A had a visibly greater amount of fines than the other two RCAs. Based on the degradation factor and Los Angeles abrasion loss, all three RCAs meet WSDOT requirements.

The average 14-day ASR expansions of processed and crushed RCAs are presented in Table 4.4.

Table 4.4 Average 14-day ASR Expansion

Source	**Average 14-Day Expansion**
A	0.068%
B	0.173%
C	0.087%

According to AASHTO T 303, if the average expansion of the mortar bars is above 0.10%, then the aggregate being tested is susceptible to deleterious expansion, and it is likely ASR reactive. Thus, the RCA obtained from sources A and C are not ASR reactive, while RCA from source B may be ASR reactive and is susceptible to deleterious expansion if used in PCCP. RCA from source B would likely require ASR mitigation techniques, such as the use of fly ash in the concrete mixture. An inference from this result is that ASR reactivity may be an issue with using RCA. This could be because of the original or remaining alkali levels in the recycled aggregates, or from the crushing process which can expose new surfaces whose ASR reactivity has not yet been depleted. It is recommended that all sources of RCA be tested individually, or that mitigation techniques be applied universally, under the presumption of ASR reactivity for reasons of simplicity and safety.

4.4 Fresh Concrete Test Results

This section discusses results evaluating the effects of substitutions of RCA for coarse NA and fly ash for portland cement on fresh concrete in terms of slump (workability), air content, and density. The previously defined labeling system was used to identify the source of the RCA, the RCA replacement ratio, and fly ash replacement ratio for each mixture, in that order. In baseline mixtures, where no RCA is incorporated, an X was used in place of the source.

The water-cementitious materials ratios, slumps, air contents, and densities of each batch prepared are presented in Table 4.5.

Table 4.5 Fresh Concrete Measurements

	Water/Cementitious Materials Ratio	Slump (in.)	Air Content	Density (pcf)
X-0-0	0.43	1.63	4.3%	145.8
A-15-0	0.44	1.50	4.9%	144.2
A-30-0	0.43	1.50	4.5%	145.2
A-45-0	0.44	2.25	4.9%	142.8
B-15-0	0.44	1.13	4.1%	146.2
B-30-0	0.43	1.50	5.0%	143.4
B-45-0	0.43	1.25	4.3%	145.8
C-15-0	0.44	2.50	5.6%	144.2
C-30-0	0.44	2.00	5.1%	144.4
C-45-0	0.44	1.50	4.3%	146.2
X-0-20	0.40	1.75	4.1%	146.8
A-15-20	0.40	1.25	4.2%	145.4
A-30-20	0.42	2.00	4.5%	144.8
A-45-20	0.40	1.50	4.5%	144.6
B-15-20	0.41	1.75	4.7%	145.8
B-30-20	0.42	1.75	4.2%	145.4
B-45-20	0.41	2.00	4.7%	143.4
C-15-20	0.39	1.63	4.1%	146.8
C-30-20	0.40	1.63	4.0%	145.8
C-45-20	0.41	1.50	4.0%	145.2

The slumps, air contents, and densities shown in Table 4.1 are the final measurements taken at the conclusion of the mixing process. All mixes met the provided WSDOT targets for air content and slump of 1-3 in. and 4-7%, respectively.

Slump is affected by several properties of the mixture, but the most prominent factor is the water-cementitious materials ratio. However, slump was a controlled measurement for this experiment, and determining how RCA affects the slump of a concrete mixture requires examination of selective sets of data. The A-45-0 and B-45-0 batches were the only batches that required the use of WRA to increases workability in order to obtain the minimum target slump. This was the highest RCA replacement ratio tested, and each batch had the same water-cementitious materials ratio in order to reach the target slump. WRA was only used when the full amount of the available mix water was used and the mixture still had not met the minimum slump. This suggests that a greater replacement of RCA results in further reduction in slump. Indeed, this trend is confirmed by examining results for the C-15-0, C-30-0, and C-45-0 batches. Though the 45% replacement for RCA from source C did not require the use of WRA, the three batches have the same water-cementitious materials ratio and a trend of decreasing slump from 2.50 in. for a 15% replacement, to a 2.00 in. slump for a 30% replacement, and a 1.50 in. slump for a 45% replacement. Opposite to the effects of increasing RCA, fly ash clearly increased the workability of concrete mixtures. This is shown by the lower water-cementitious materials ratios for all 20% fly ash batches, which indicates that water was held back in order to remain below the target maximum slump of 3 in.

AASHTO T 152 required that an aggregate correction factor be obtained for each of the aggregates using the methods described in the standard. For each RCA source and rate of RCA substitution, that factor was determined to be 0.5%. This factor was determined by first testing for the correction factor for natural aggregates and then testing for the correction factor of a mixture with a 45% replacement of RCA. Thus, for replacement ratios between 0% and 45%, the aggregate correction factor was determined to be a constant 0.5%.

The amount of AEA, used to control air content and obtain target values, differed for each mixture. Amounts were estimated prior to mixing based on experience and the final amount determined during mixing by periodic measurements of measured air content. Thus, it is difficult to draw any meaningful conclusions from the data. However, the constancy of the aggregate correction factor for a 0% and 45% replacement of RCA suggests that there may be no significant direct effect of RCA on air content, and thus the structurally entrained air of the aggregates did not result in an increase in the correction factor. Indirect effects, resulting from the effects of RCA on workability and density precipitating to effects on the mixtures air content, cannot be obtained because air content was a controlled parameter.

Figure 4.1 shows the relationship between fresh concrete density and the rate of RCA replacement.

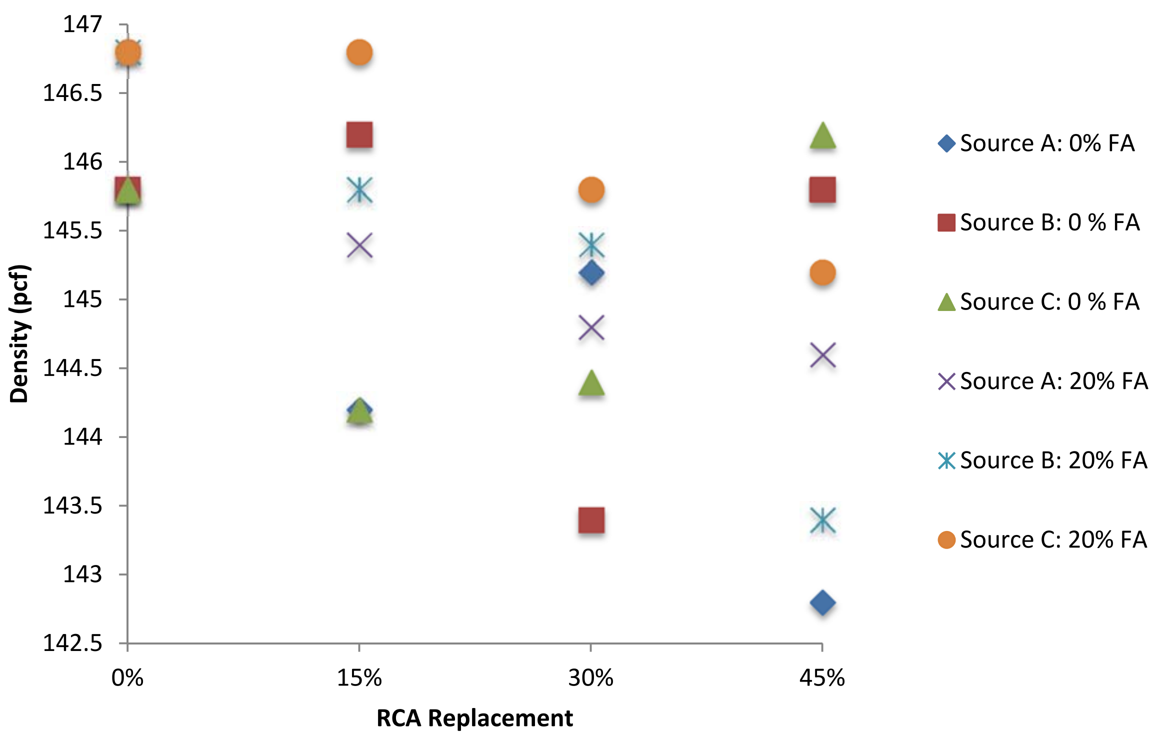


Figure 4.1 Fresh Concrete Density vs. % RCA Replacement

The trends shown in Figure 4.1 indicate a general relationship between increased RCA replacement and reduced fresh concrete density. On average, an increase of 15% in the RCA replacement amount typically resulted in a density decrease of 0.4% to 0.6% relative to batches from the same source. The mixtures that did not follow this trend, the mixes with no fly ash for source C, may have been affected by an unintentional decreasing trend in air content. Unsurprisingly, larger air contents of the fresh concrete correlated to decreased density, as shown in Figure 4.2.

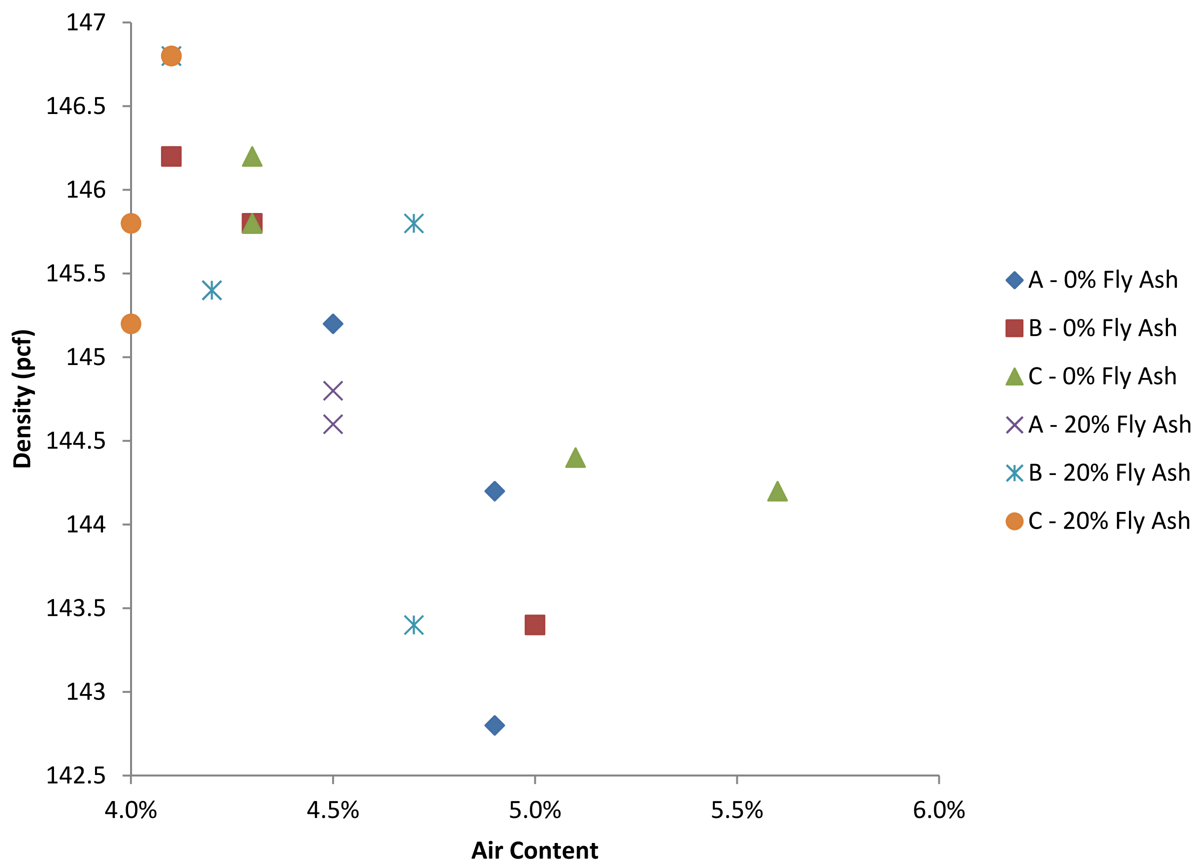


Figure 4.2 Fresh Concrete Density vs. Air Content

4.5 Hardened Concrete Test Results

This section presents and discusses results evaluating the effects of RCA on concrete compressive strength, modulus of rupture, coefficient of thermal expansion, drying shrinkage, and freeze-thaw durability.

In this section, an analysis of various with a 95% confidence interval using Microsoft Excel's "Single Factor ANOVA" function is used to determine if there is a statistical difference between results. Additional consideration of factors such as air content and water-cement ratio is made when a statistical difference is found in order to assess the validity of the determined statistical differences.

4.5.1 Compressive Strength

Test data for all compression samples is shown in Appendix C. The average compressive strengths and coefficients of variation (CoV) at ages of 7, 14, 28, 90 days are given in Table 4.6.

Table 4.6 Average Compressive Strength Results

	7-Day (psi)	CoV	14-Day (psi)	CoV	28-Day (psi)	CoV	90-Day (psi)	CoV
X-0-0	3750	1.8%	4348	2.3%	4834	1.6%	5515	0.7%
A-15-0	3753	4.2%	4180	10.5%	4921	3.3%	5418	1.5%
A-30-0	4330	0.1%	4868	1.4%	5474	4.0%	5901	1.1%
A-45-0	3839	3.4%	4619	0.8%	5130	1.8%	5573	2.6%
B-15-0	3977	4.1%	4877	1.1%	5396	1.0%	6101	2.3%
B-30-0	3867	2.5%	4823	1.2%	5312	2.2%	5787	3.2%
B-45-0	4091	8.0%	5164	2.4%	5515	2.9%	6119	4.8%
C-15-0	3355	1.4%	3808	1.3%	4335	2.9%	4924	5.5%
C-30-0	3521	0.6%	4264	2.0%	4740	2.4%	5323	1.6%
C-45-0	3794	2.3%	4454	3.7%	4937	2.1%	5749	1.4%
X-0-20	3709	4.4%	4568	6.0%	5337	1.6%	6281	1.7%
A-15-20	3904	4.4%	4655	3.4%	5592	2.8%	6555	2.5%
A-30-20	3737	1.7%	4503	2.1%	5290	5.4%	6269	2.5%
A-45-20	3763	4.0%	4497	3.8%	5503	4.0%	6403	1.5%
B-15-20	3618	1.0%	4381	1.1%	5184	0.9%	6208	1.7%
B-30-20	3631	2.2%	4380	3.0%	5222	2.0%	6185	2.9%
B-45-20	3303	1.3%	4089	1.0%	4756	1.7%	5795	0.4%
C-15-20	3435	2.4%	4106	2.2%	4682	3.8%	5524	2.1%
C-30-20	3614	1.1%	4347	1.4%	4813	1.2%	5752	1.1%
C-45-20	3893	1.6%	4391	1.1%	5151	2.1%	6401	3.0%

The coefficients of variation ranged from 0.1% to 10.5%, indicating that scatter occurring in the results for a particular mix is relatively small. Figure 4.3 is a plot of average 28-day compressive strength versus RCA substitution rate.

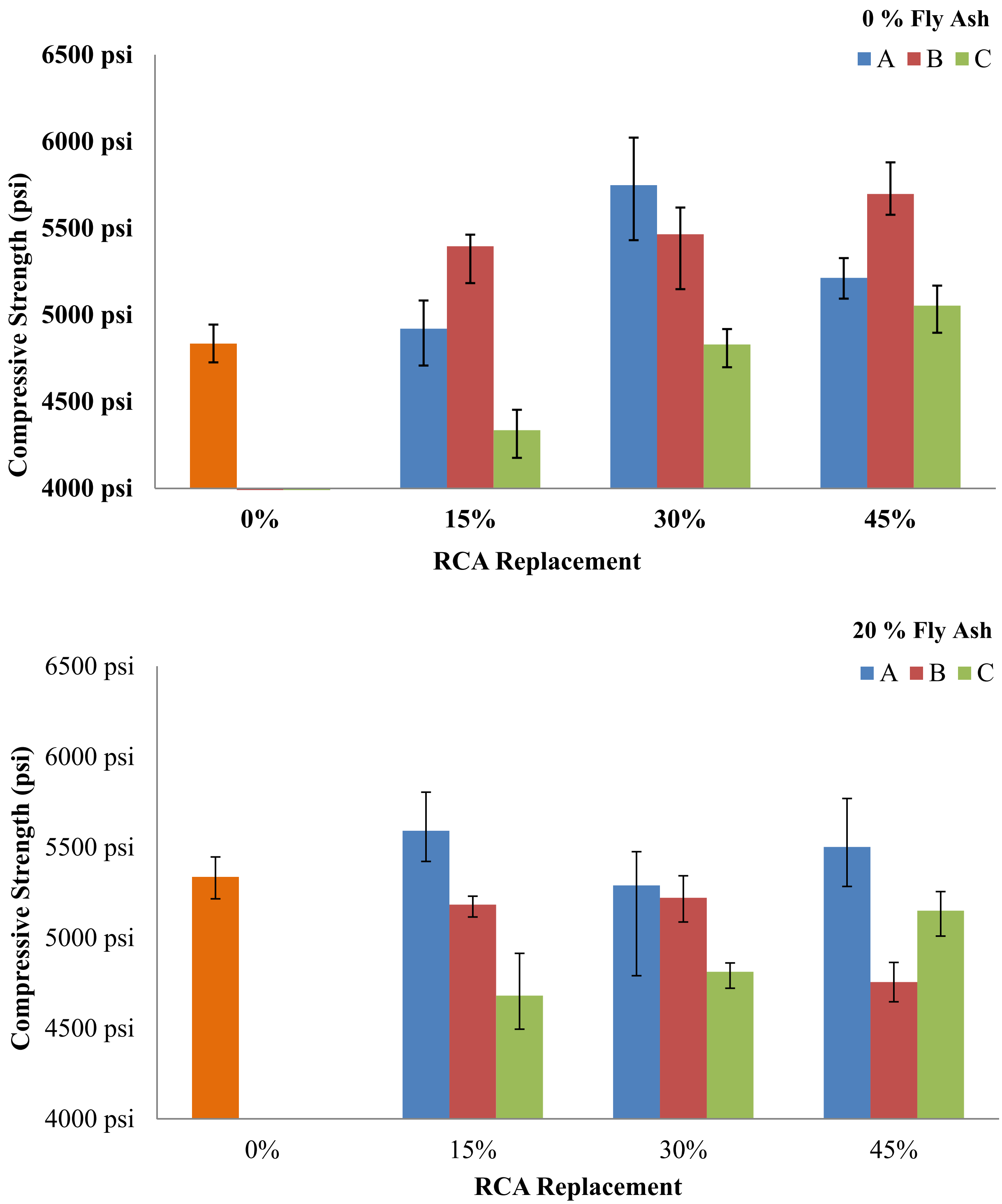


Figure 4.3 Average 28-Day Compressive Strength vs. % RCA Substitution: (top) 0% Fly Ash; (bottom) 20% Fly Ash

The WSDOT minimum 28-day compressive strength for PCCP of 4,000 psi is shown at the horizontal axes in Figure 4.3. The range of compressive strengths for each data set is indicated by the black data range bars. All samples tested met the WSDOT minimum compressive strength requirement.

A qualitative assessment of Figure 4.3 shows that there is no obvious relationship between RCA replacement ratios and 28-day compressive strength, and individual ANOVA analyses confirm this. There is no trend of statistically significant differences as a result of greater RCA replacement, which indicates that the differences likely do not occur due to the presence of RCA. For example, while ANOVA analysis for A-15-0 and A-30-0 concludes that there is a statistically significant difference between the two data sets, no such statistically significant different exists between A-15-0 and A-45-0 despite a greater replacement of RCA. This shows that differences are most likely due to the effects of other characteristics of the mix such as air content, water-cementitious materials ratio.

The hypothesis that the statistical differences are due to parameters other than RCA, especially air content, is supported by the differences existing between C-15-0, C-30-0, and C-45-0. This series had an unintentional trend of decreasing air contents of 5.6%, 5.1%, and 4.3%. Each difference that exists between the three batches correlates to a trend that the concrete is stronger with less air content and more RCA. Ultimately, there is no statistical difference of 28-day compressive strength between X-0-0 and C-45-0 despite similar air contents, indicating that RCA was not the significant factor in compressive strength. This does not prove that RCA has zero effect on compressive strength. However, it does show that the effect is very small in magnitude compared to other known factors on concrete compressive strength such as water-cement ratio and air content.

The performance differences between each of the sources was not significant, though compressive strengths for source C were consistently less than the strengths of concretes from the other two sources. A conclusion regarding the whether RCA was the cause of existing differences cannot be made regarding the differences in ultimate strengths between the three sources because of the variability of other parameters.

Averaging data across the same replacement ratios to decrease the variation caused by other variables suggests that there is a possible 0-5% gain in compressive strength for a 45% replacement and 0% fly ash. Oppositely, when a 20% fly ash substitution is used, averaged data suggests that there is a 0-5% loss in compressive strength with a 45% replacement of RCA. 20% Fly ash concrete samples were, on average, 10% stronger than 0% fly ash samples after 90 days, despite being approximately the same strength after 28 days. Ultimately, the variability in concrete strength cannot be contributed to RCA alone, and it is more likely that other effects are the cause. It is important to note that the very high quality of the RCA used in this project could be a reason why the effects of RCA on concrete compressive strength were insignificant.

The rate of strength gain of concrete compressive strength was also not significantly affected by the presence of RCA. Figure 4.4 and Figure 4.5 are plots of strength as a percentage of ultimate 28-day strength versus the amount of replaced RCA. The plots show that the rate of strength gain was fairly consistent regardless of the rate of mixture. Data suggests that late age strength gain may be slightly negatively affected by the presence of RCA. The observation that

RCA has little or no effect on strength gain is confirmed numerically in Tables 4.7 and 4.8, which compare strengths for 0% RCA and 45% RCA batches relative to their 28-day strengths.

Furthermore, Table 4.7 and Table 4.8 show that, because 28-day strengths were largely the same for both fly ash and non-fly ash concretes, samples with fly ash had a lower short-term compressive strength gain at 14 days and a larger long-term compressive strength gain. For 20% fly ash replacement, concrete samples had 7-day and 14-day compressive strengths that were 68-76% and 83-86% of 28-day compressive strength, respectively. This was generally lower than the 0% fly ash samples, which achieved 73-79% of 28-day strength after 7 days and 85-94% of 28-day strength after 14 days. The hypothesis that fly ash causes greater long-term strength gain is substantiated by the observation, despite having the similar average 28-days ultimate strengths, 20% fly ash samples achieved 90-day strengths that were 116%-124% of 28-day strength while 0% fly ash samples only achieved 90-day strengths which were 108-116% of 28-day strengths.

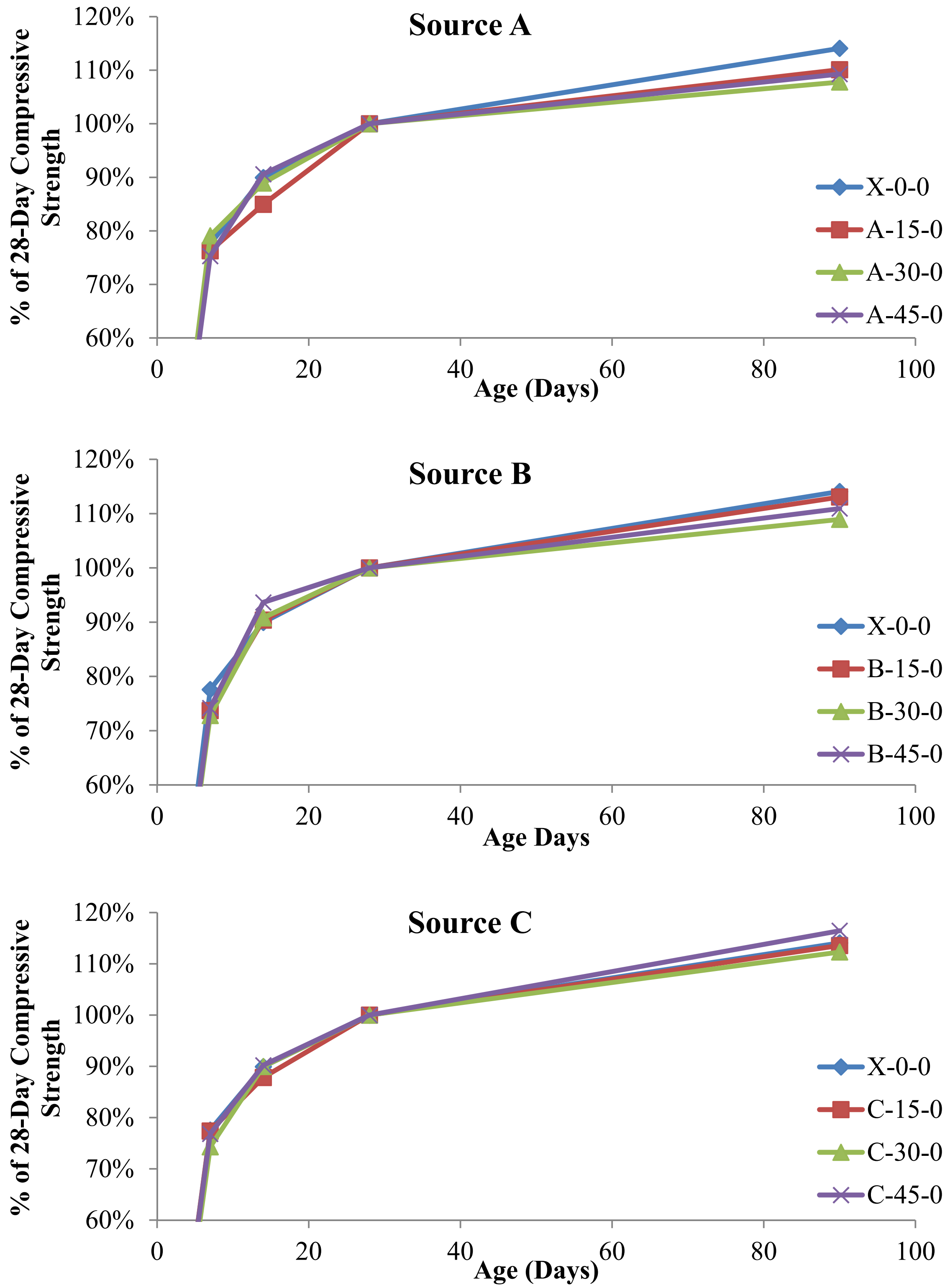
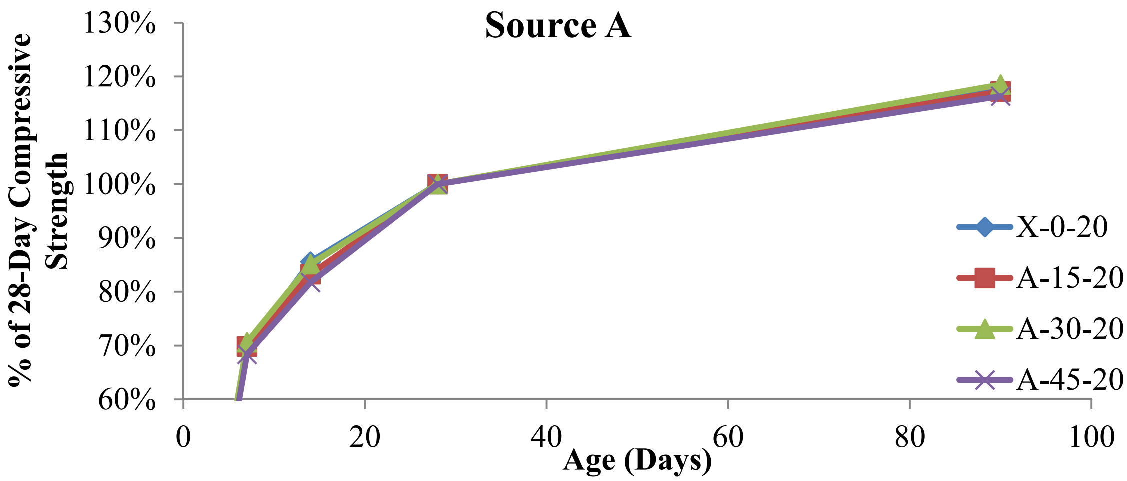
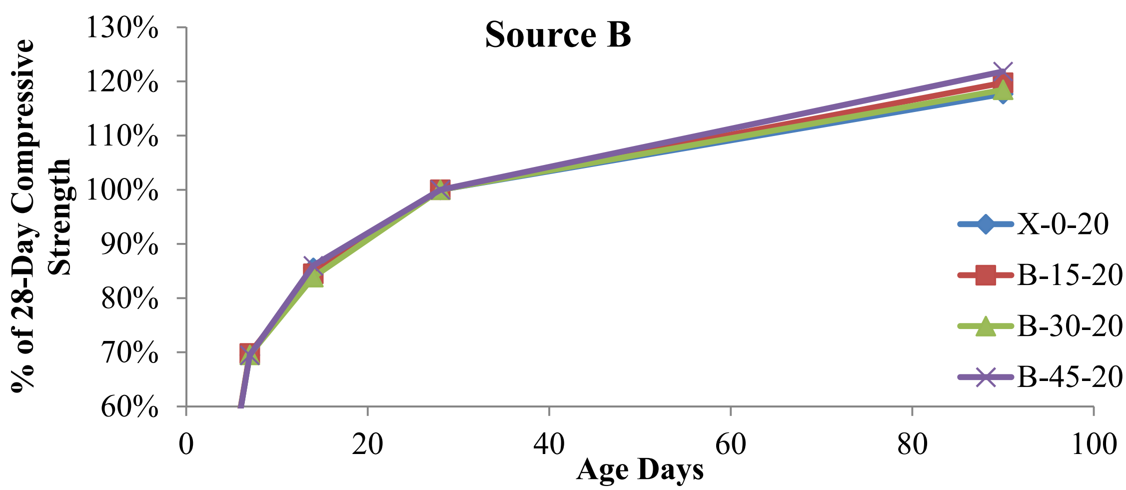


Figure 4.4 Concrete Compressive Strength Ratio to 28-Day Strength vs. Age (days)


Source A
% of 28-Day Compressive Strength
130%
120%
110%
100%
90%
80%
70%
60%
0
20
40
60
80
100
Age (Days)
X-0-20
A-15-20
A-30-20
A-45-20


Source B
% of 28-Day Compressive Strength
130%
120%
110%
100%
90%
80%
70%
60%
0
20
40
60
80
100
Age Days
X-0-20
B-15-20
B-30-20
B-45-20

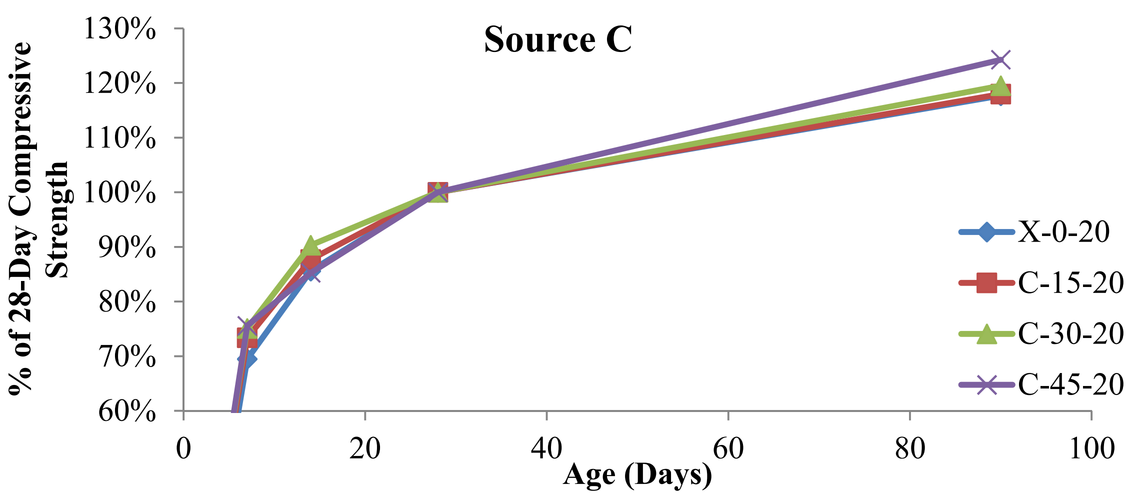

Source C
% of 28-Day Compressive Strength
130%
120%
110%
100%
90%
80%
70%
60%
0
20
40
60
80
100
Age (Days)
X-0-20
C-15-20
C-30-20
C-45-20

Figure 4.5 Concrete Compressive Strength Ratio to 28-Day Strength vs. Age (days)

Table 4.7 Compressive Strength as a Percentage of 28-Day Strength, 0% Fly Ash

Source	Age (Days)	A	B	C	Average
0% RCA	0	0%			0%
	7	78%			78%
	14	90%			90%
	28	100%			100%
	90	114%			114%
15% RCA	0	0%	0%	0%	0%
	7	76%	74%	77%	76%
	14	85%	90%	88%	88%
	28	100%	100%	100%	100%
	90	110%	113%	114%	112%
30% RCA	0	0%	0%	0%	0%
	7	79%	73%	74%	75%
	14	89%	91%	90%	90%
	28	100%	100%	100%	100%
	90	108%	109%	112%	110%
45% RCA	0	0%	0%	0%	0%
	7	75%	74%	77%	75%
	14	91%	94%	90%	91%
	28	100%	100%	100%	100%
	90	109%	111%	116%	112%

Table 4.8 Compressive Strength as a Percentage of 28-Day Strength, 20% Fly Ash

Source	Age (Days)	A	B	C	Average
0% RCA	0	0%			0%
	7	69%			69%
	14	86%			86%
	28	100%			100%
	90	118%			118%
15% RCA	0	0%	0%	0%	0%
	7	70%	70%	73%	71%
	14	83%	85%	88%	85%
	28	100%	100%	100%	100%
	90	117%	120%	118%	118%
30% RCA	0	0%	0%	0%	0%
	7	71%	70%	75%	72%
	14	85%	84%	90%	86%
	28	100%	100%	100%	100%
	90	119%	118%	120%	119%
45% RCA	0	0%	0%	0%	0%
	7	68%	69%	76%	71%
	14	82%	86%	85%	84%
	28	100%	100%	100%	100%
	90	116%	122%	124%	121%

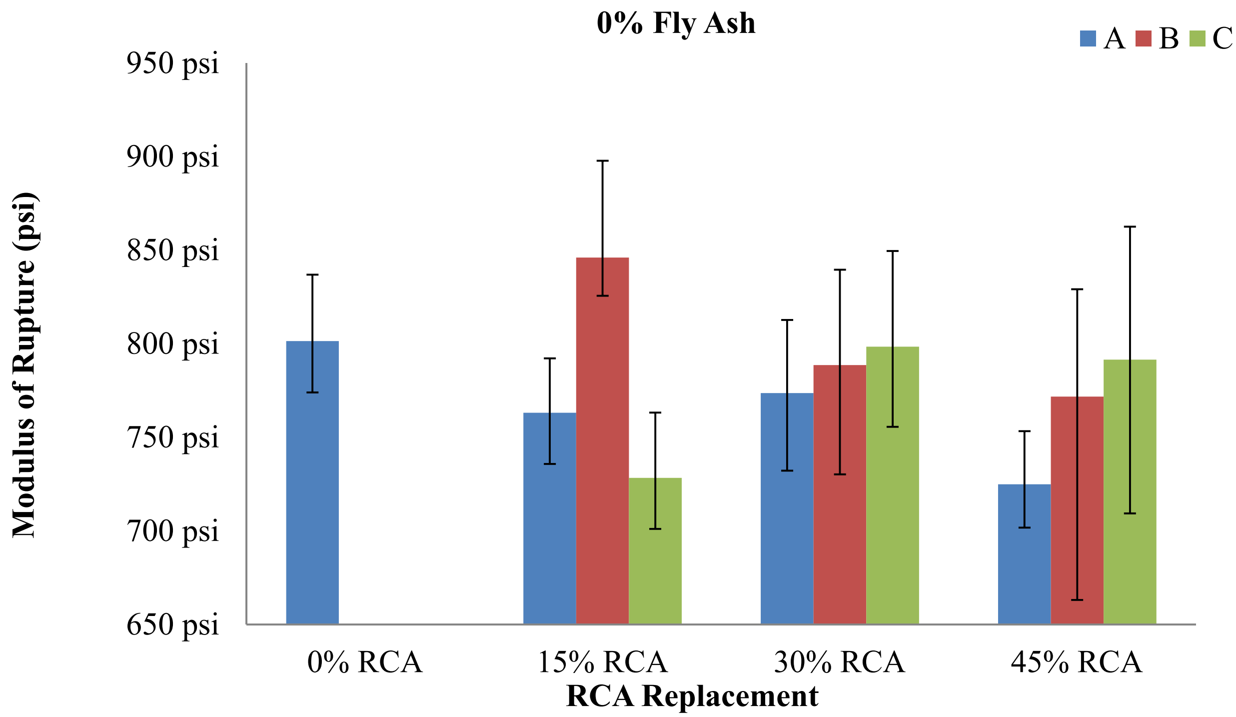
4.5.2 Modulus of Rupture

All MOR test results are presented in Appendix D. Table 4.9 shows the average 14-day MOR values and coefficients of variation for all the mixes tested.

Table 4.9 Average 14-Day MOR Results

	Average MOR (psi)	**CoV**
X-0-0	801 psi	2.9%
A-15-0	763 psi	2.7%
A-30-0	773 psi	5.0%
A-45-0	725 psi	3.5%
B-15-0	846 psi	3.5%
B-30-0	788 psi	5.2%
B-45-0	771 psi	8.5%
C-15-0	728 psi	4.2%
C-30-0	798 psi	4.6%
C-45-0	791 psi	7.7%
X-0-20	776 psi	6.3%
A-15-20	780 psi	2.8%
A-30-20	720 psi	6.3%
A-45-20	747 psi	4.9%
B-15-20	774 psi	6.9%
B-30-20	777 psi	4.3%
B-45-20	725 psi	2.2%
C-15-20	782 psi	7.3%
C-30-20	761 psi	3.3%
C-45-20	805 psi	2.6%

The coefficients of variation ranged 2.2% and 8.5%, indicating relatively small variations in tests results for a given mix.   Figure 4.6 is a plot of the MOR versus RCA replacement percentage.


0% Fly Ash
A
B
C
950 psi
900 psi
850 psi
800 psi
750 psi
700 psi
650 psi
Modulus of Rupture (psi)
0% RCA
15% RCA
30% RCA
45% RCA
RCA Replacement

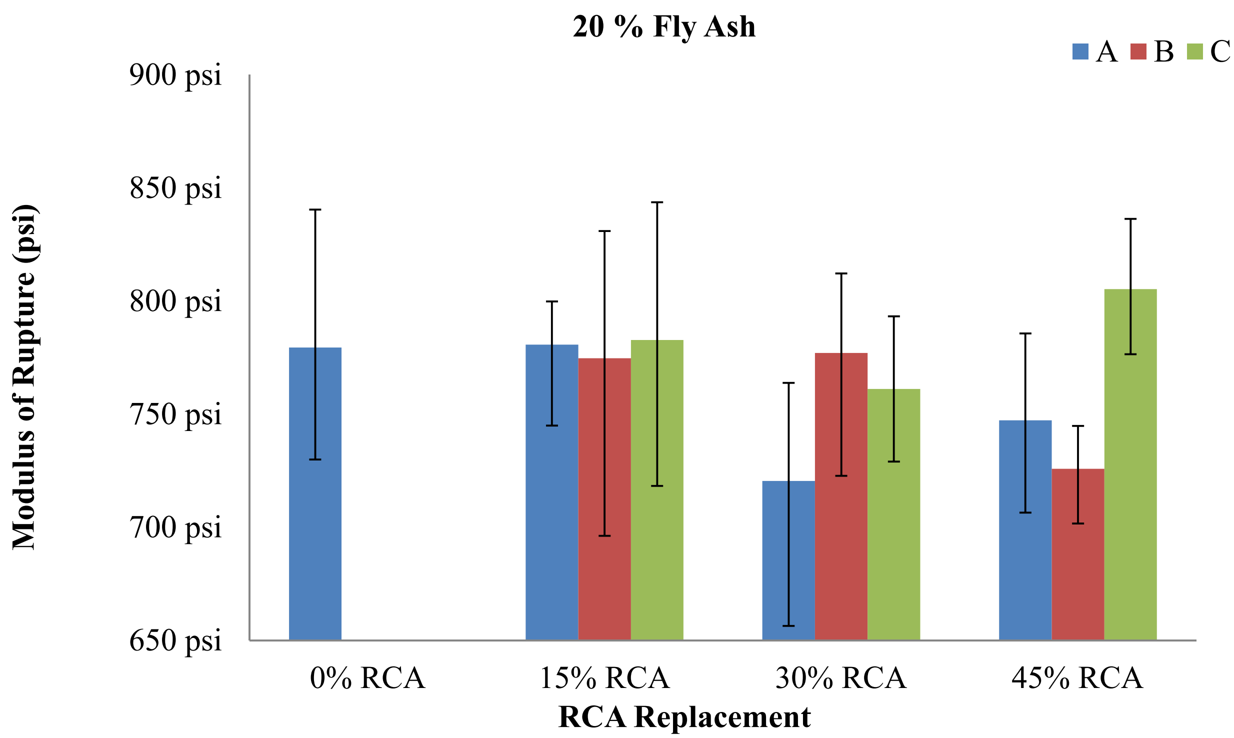
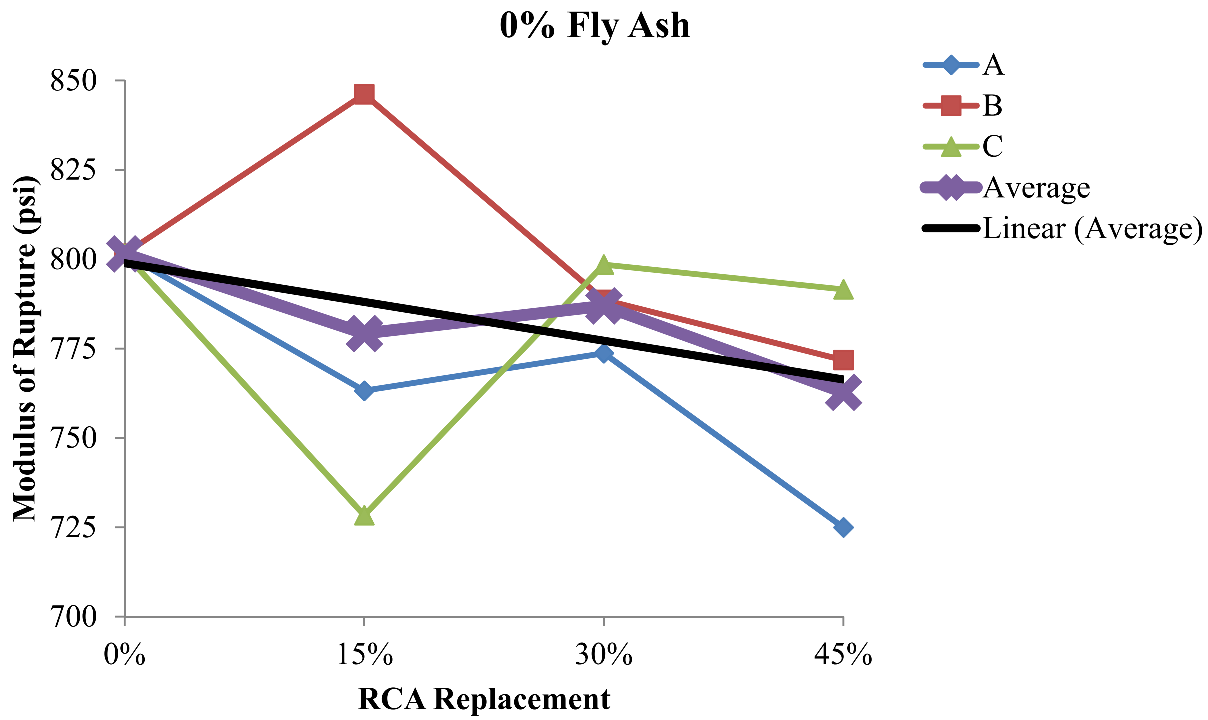

20 % Fly Ash
A
B
C
900 psi
850 psi
800 psi
750 psi
700 psi
650 psi
Modulus of Rupture (psi)
0% RCA
15% RCA
30% RCA
45% RCA
RCA Replacement

Figure 4.6 Modulus of Rupture vs. % RCA Replacement

WSDOT requires the PCCP mixtures have a minimum 14-day MOR of 650 psi, which is shown as the horizontal axes in Figure 4.6. High and low values of test results are indicated by the black data range bars. All samples tested above the WSDOT MOR limit.

A qualitative visual assessment of Figure 4.6 indicates the possibility of a slight decrease in MOR with higher RCA replacement ratios. ANOVA analysis of each of the six series of batches shows that there are some statistical differences in the 0% fly ash A and C series. ANOVA suggests a statistically significant difference in the decrease between A-15-0 and A-45-0, which both have a 4.9% air content and a 0.44 water-cementitious products ratio. The existence of this statistically significant difference in samples with the same air content and water-cementitious products suggests that RCA could reduce concrete MOR. However, the lack of consistent evidence across the other data sets means that this effect is likely to be small. Averaging the difference in MOR between a 0% replacement and 45% replacement for each source suggests that there is an approximate 0-10% decrease in MOR for both 0% and 20% fly ash batches with a 45% RCA replacement. This hypothesis is shown in the trend lines of the average MORs in Figure 4.7, which display a 3-5% average reduction in MOR for a 45% RCA replacement. The plots show some highly variable data displayed as peaks in the data, which can be attributed to other factors have a more significant effect than RCA. Additionally, it is important to note than the vertical axis of the data is over a small range, further indicating the insignificance of any effects that RCA may have on MOR.


0% Fly Ash
Modulus of Rupture (psi)
850
825
800
775
750
725
700
0%
15%
30%
45%
RCA Replacement
A
B
C
Average
Linear (Average)

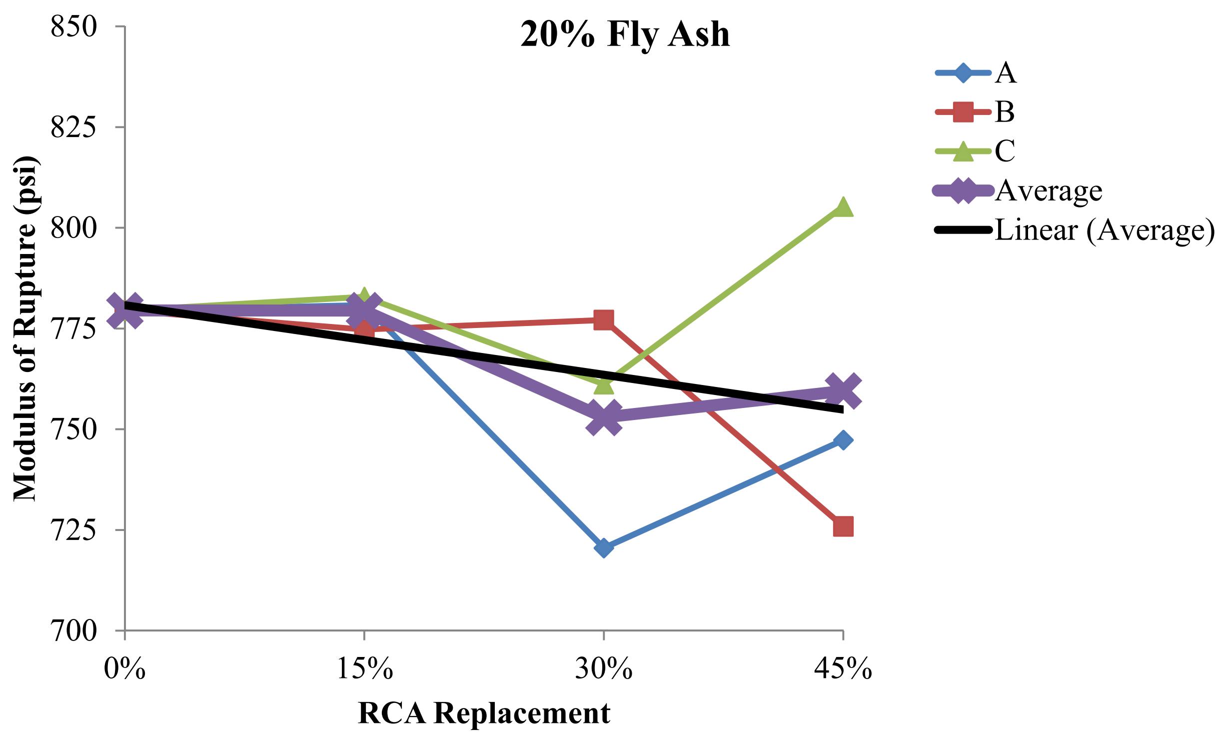

20% Fly Ash
Modulus of Rupture (psi)
850
825
800
775
750
725
700
0%
15%
30%
45%
RCA Replacement
A
B
C
Average
Linear (Average)

Figure 4.7 Average MOR vs. % RCA Replacement and Cross-Series Average Trend line

4.5.3 Coefficient of Thermal Expansion

All data from tests on the coefficient of thermal expansion of the concrete specimens is presented in Appendix E. A summary of CTE results and coefficients of variation is presented in Table 4.10.

Table 4.10 Coefficient of Thermal Expansion Test Results

	Average CTE (in./in. per °C)	COV
X-0-0	9.3E-06	5%
A-15-0	8.2E-06	4%
A-30-0	8.6E-06	7%
A-45-0	8.8E-06	2%
B-15-0	8.9E-06	8%
B-30-0	9.3E-06	2%
B-45-0	8.9E-06	5%
C-15-0	8.9E-06	2%
C-30-0	8.4E-06	15%
C-45-0	8.3E-06	7%
X-0-20	9.3E-06	3%
A-15-20	8.4E-06	1%
A-30-20	8.5E-06	6%
A-45-20	8.8E-06	9%
B-15-20	9.1E-06	2%
B-30-20	9.1E-06	6%
B-45-20	9.9E-06	3%
C-15-20	1.0E-05	16%
C-30-20	8.6E-06	6%
C-45-20	5.2E-06	20%

The coefficients of variation ranged from 1-20% but generally fell below 10%. There is no immediately visible trend between CTE and RCA replacement. The results are consistent with the CTE values of normal ordinary concrete of 5.8 to 12.6 millionths per degree Celsius

(Kosmatka, Kerkhoff, & Panerese, 2002). ANOVA analysis between baseline batches and 45% batches yields no statistically significant differences except in the case of X-0-20 and C-45-20. However, results for C-45-20 were highly variable and should be excluded. All three specimens for the C-45-20 data set had extraneous results, ranging from 4.8 to 28 millionths per degree Celsius. Excluding results for C-45-20, the CTE was reduced 4-10% from a 45% RCA replacement on average. This reduction remained, somewhat inconsistently, with 15% and 30% RCA replacements. Thus, the data shows that RCA may slightly reduce CTE values by anywhere from 0% to 10% for a 45% RCA replacement.

4.5.4 Drying Shrinkage

All data for tests on drying shrinkage is presented in Appendix F. This test was only performed for RCA from sources B and C. The average drying shrinkage results beginning after 28 days of age and coefficients of variation are given in Table 4.11. A negative value indicates shrinkage of the specimen.

Table 4.11 Drying Shrinkage Test Results

	Average Shrinkage Strain (in./in.)							
Day	**X-0-0**	**CoV**	**C-15-0**	**CoV**	**C-30-0**	**CoV**	**C-45-0**	**CoV**
28	0.0E+00	0%	0.0E+00	0%	0.0E+00	0%	0.0E+00	0%
32	-8.7E-05	18%	-7.7E-05	20%	-6.7E-05	9%	-1.3E-04	15%
35	-1.8E-04	13%	-1.2E-04	26%	-1.5E-04	28%	-2.1E-04	25%
42	-3.4E-04	9%	-2.5E-04	15%	-2.4E-04	13%	-3.3E-04	15%
56	-4.4E-04	8%	-3.6E-04	12%	-4.0E-04	14%	-5.0E-04	13%
84	-5.3E-04	19%	-4.7E-04	10%	-5.5E-04	12%	-6.8E-04	13%
140	-6.5E-04	5%	-5.9E-04	7%	-6.7E-04	5%	-7.0E-04	11%
		Day	**B-15-0**	**CoV**	**B-30-0**	**CoV**	**B-45-0**	**CoV**
		28	0.0E+00	0%	0.0E+00	0%	0.0E+00	0%
		32	-1.5E-04	14%	-1.5E-04	29%	-1.2E-04	22%
		35	-1.7E-04	3%	-1.7E-04	30%	-2.1E-04	5%
		42	-1.9E-04	19%	-4.2E-04	11%	-2.2E-04	3%
		56	-3.4E-04	6%	-5.1E-04	11%	-3.6E-04	6%
		84	-4.6E-04	8%	-6.0E-04	5%	-5.2E-04	2%
		140	-5.5E-04	5%	-6.8E-04	9%	-5.9E-04	2%
Day	**X-0-20**	**CoV**	**C-15-20**	**CoV**	**C-30-20**	**CoV**	**C-45-20**	**CoV**
28	0.0E+00	0%	0.0E+00	0%	0.0E+00	0%	0.0E+00	0%
32	-5.7E-05	20%	-5.3E-05	60%	-1.1E-04	10%	-1.1E-04	24%
35	-1.8E-04	10%	-2.0E-04	8%	-1.7E-04	6%	-2.1E-04	7%
42	-3.4E-04	4%	-3.4E-04	6%	-3.9E-04	4%	-3.4E-04	7%
56	-4.3E-04	2%	-4.5E-04	9%	-4.7E-04	0%	-4.3E-04	7%
84	-5.2E-04	3%	-5.3E-04	7%	-6.1E-04	1%	-5.6E-04	5%
140	-6.0E-04	5%	-6.0E-04	6%	-6.7E-04	3%	-6.2E-04	4%
		Day	**B-15-20**	**CoV**	**B-30-20**	**CoV**	**B-45-20**	**CoV**
		28	0.0E+00	0%	0.0E+00	0%	0.0E+00	0%
		32	-7.7E-05	59%	-2.7E-05	87%	-2.0E-04	46%
		35	-1.7E-04	24%	-2.0E-04	13%	-1.9E-04	46%
		42	-2.8E-04	16%	-2.3E-04	12%	-3.6E-04	25%
		56	-4.7E-04	15%	-3.8E-04	9%	-5.3E-04	16%
		84	-5.4E-04	10%	-4.5E-04	10%	-6.6E-04	11%
		140	-6.6E-04	10%	-6.0E-04	5%	-7.1E-04	13%

The tabulated results don't show any obvious trend. Initial shrinkage values had several moderately high coefficients of variation, but data became more consistent as the concrete specimens aged. Shrinkage values displayed in Table 4.11 are such that the zero shrinkage point occurs after 28 days of curing, when the specimens were first removed from the lime-saturated curing water.

Ordinary concrete drying shrinkage strains are typically between 4E-4 and 8E-4 (Kosmatka, Kerkhoff, & Panerese, 2002). All specimens had drying shrinkage strains that are within the range of ordinary concrete at the 140-day shrinkage measurement.

Figure 4.8 and Figure 4.9 show the average drying shrinkage versus age.

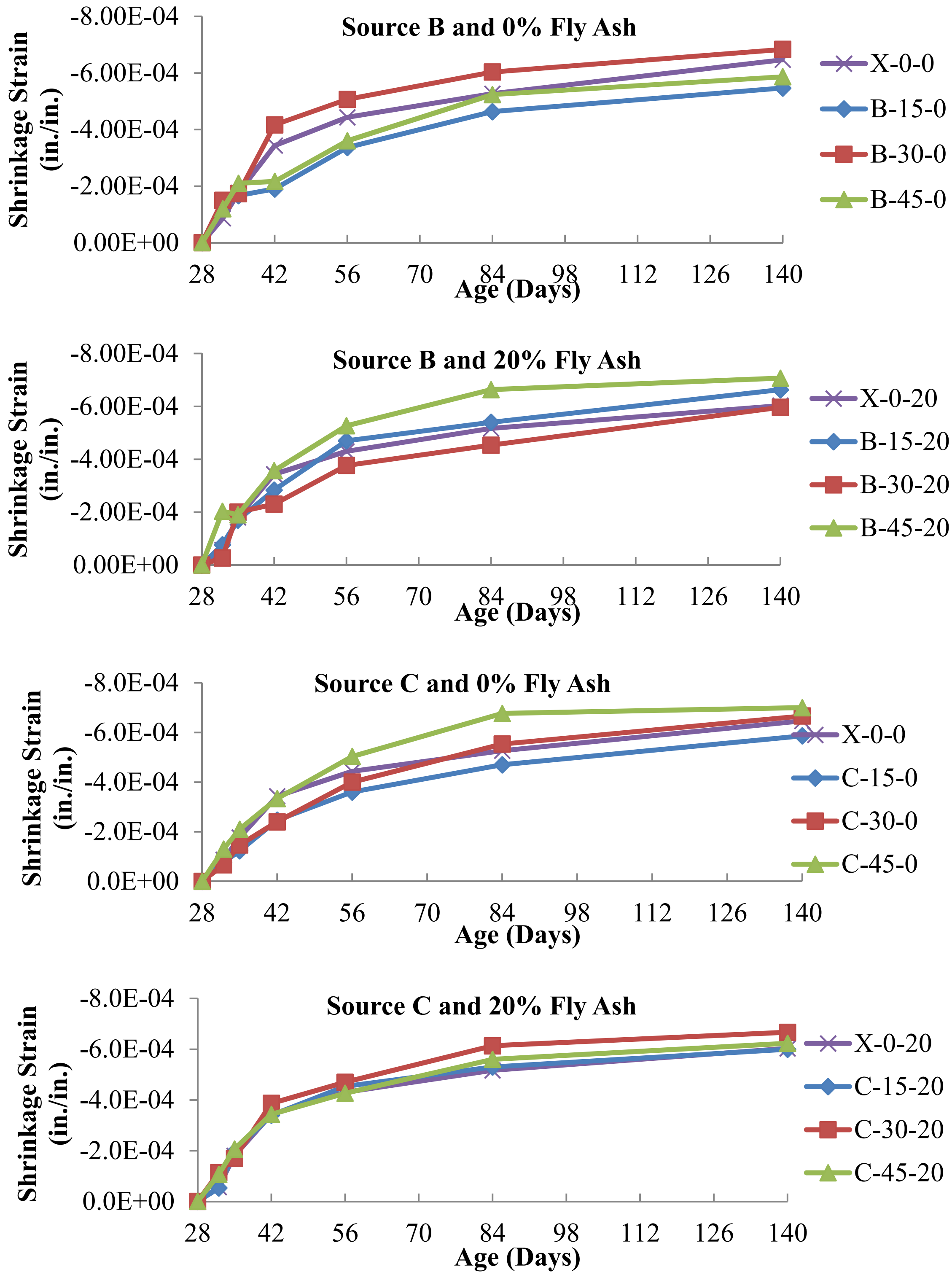


Figure 4.8 Shrinkage Strain vs. Age Results

Long-term shrinkage appears to increase with greater RCA replacement levels, though on a somewhat inconsistent basis. This indicates that RCA may have an effect on drying shrinkage, but there are also other properties of the mix that may have relatively larger effects than RCA on this property. A primary example of the shrinkage being larger for a greater replacement level occurs in the 0% Fly Ash series for source C. Initial shrinkage for X-0-0 is significantly larger than C-30-0, but the shrinkage in C-30-0 eventually surpasses X-0-0. Additionally, assessment of all four series shown in Figure 4.8 concludes that the 30% or 45% replacements generally have greater shrinkage strain than the 0% or 15% RCA replacements.

ANOVA analyses show that some of the visible differences in Figure 4.8 are also statistically significant differences. Statistically significant differences exist at 140 days of age between X-0-0 and B-45-0 based on a 95% confidence interval. Of note is that additional significant differences based on a 90% confidence interval exist at 84 days of age between X-0-0 and C-45-0, X-0-20 and B-45-20, and X-0-20 and C-45-20. The batches did not statistically differentiate themselves from each other in the earlier ages, but developed statistically significant differences after 84 days of age. These differences are less consistent in lesser replacements, suggesting that RCA is one of several factors influencing drying shrinkage. The drying shrinkage of concrete incorporating RCA could be greater because of the porosity and absorption of the adhered mortar portion of the RCA compared to natural aggregates.

The data shows that a 45% replacement of RCA from may result in a 0-30% increase in drying shrinkage, while remaining typical of normal ordinary concrete. Data also showed that replacements of 15% and 30% may result in a 0-20% increase in drying shrinkage.

4.5.5 Freeze-Thaw Durability

Freeze-thaw durability tests were limited to 100 cycles, so results reported here should be considered as preliminary results. However, because the 0% and 45% RCA batches both had an air content of 4.3% and a water-cementitious products ratio of 0.44, it is likely that the experimental differences are due to the use of RCA. Summarized results for tests of nonstandard flexural modulus and modal frequency for all specimens are displayed in Table 4.12. Note that the control specimens were not subjected to freeze-thaw cycles.

Table 4.12 Nonstandard Flexural Modulus and Modal Frequency Results

	Sample	Nonstandard Flexural Modulus (psi)	Modal Frequency
0 cycles	C-0-1	97.4	1557 Hz
	C-0-2	91.6	1563 Hz
	C-0-3	89.7	1568 Hz
	C-0-4 (control)	100.7	1530 Hz
	C-0-5 (control)	98.2	1535 Hz
	C-0-6 (control)	80.0	1530 Hz
	C-45-1	82.8	1525 Hz
	C-45-2	97.9	1521 Hz
	C-45-3	94.1	1510 Hz
	C-45-4 (control)	88.2	1542 Hz
	C-45-5 (control)	86.4	1523 Hz
	C-45-6 (control)	78.1	1530 Hz
50 cycles	C-0-1	91.8	1540 Hz
	C-0-2	86.0	1541 Hz
	C-0-3	84.9	1552 Hz
	C-0-4 (control)	96.0	1558 Hz
	C-0-5 (control)	88.5	1559 Hz
	C-0-6 (control)	95.6	1554 Hz
	C-45-1	84.5	1518 Hz
	C-45-2	89.5	1513 Hz
	C-45-3	88.3	1498 Hz
	C-45-4 (control)	88.8	1560 Hz
	C-45-5 (control)	79.5	1544 Hz
	C-45-6 (control)	74.2	1551 Hz
100 cycles	C-0-1	93.8	1550 Hz
	C-0-2	75.8	1552 Hz
	C-0-3	88.6	1564 Hz
	C-0-4 (control)	103.8	1559 Hz
	C-0-5 (control)	83.4	1572 Hz
	C-0-6 (control)	89.3	1564 Hz
	C-45-1	85.7	1522 Hz
	C-45-2	84.8	1518 Hz
	C-45-3	93.3	1505 Hz
	C-45-4 (control)	94.6	1572 Hz
	C-45-5 (control)	74.7	1555 Hz
	C-45-6 (control)	91.7	1563 Hz

If one of three samples for each condition exhibited behavior opposite to the other two or created a coefficient of variation greater than 100%, it was considered an outlier data point and removed from the data set. Following removal of the outliers, the results shows that, after 100 cycles, the flexural modulus of the concrete samples incorporating RCA suffered less deterioration under cycles of freezing and thawing. On average, freeze-thaw specimens with 0% RCA had a nonstandard flexural modulus decrease of 2.5%, while 45% RCA specimens had an increase of 1.3%. However, the difference in flexural stiffness change was small. The increase in flexural stiffness that occurred due to aging of the concrete during freeze-thaw cycles was greater for RCA concrete, on average, and could have caused the difference. Tests on the modal frequency of freeze-thaw specimens showed that the effects of freezing and thawing cycles may be reduced with a 45% RCA replacement. After 100 cycles, the average modal frequency for 0% RCA specimens decreased by 7.3 Hz while 45% RCA specimens suffered a decrease of 3.7 Hz. This difference is further justified by the fact that increases in the modal frequency of cured specimen due to curing were the about the same regardless of the rate of RCA replacement, as shown in Figure 4.9.

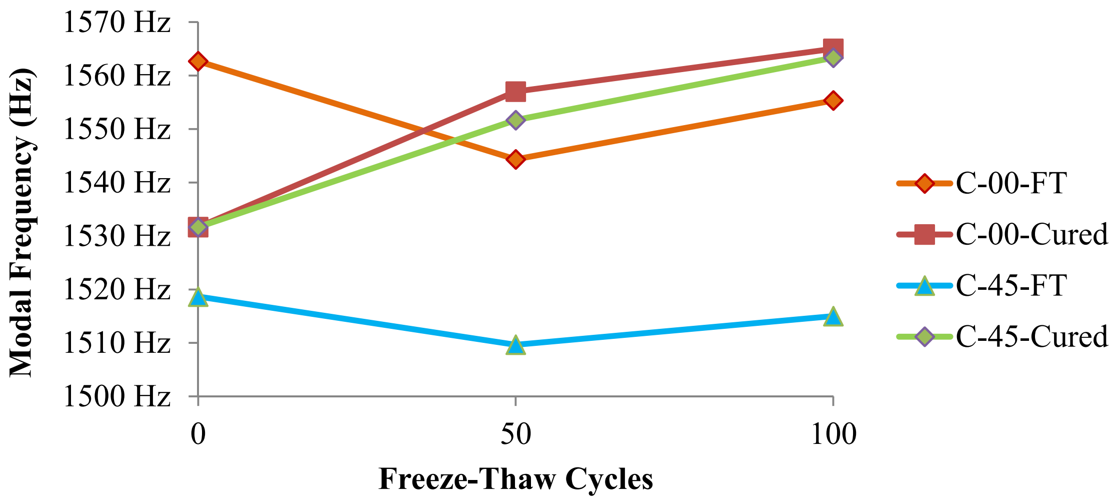


Figure 4.9 Modal Frequencies vs. Freeze-Thaw Cycles

It is difficult to determine if the initial overall difference between the modal frequencies of specimens was due to the presence of RCA or from some other factor. Cured specimens had the same initial value. Figure 4.9 does show that the 50-cycle decrease due to freezing and thawing was smaller for the 45% RCA concrete, and the 100-cycle overall decrease was greater for 0% RCA concrete. Ultimately, no conclusion can be reached in terms of the effects of RCA on the performance of concrete under freezing and thawing cycles because the differences in performance between a 0% and a 45% RCA concrete were insignificant and because the experiment was limited to 100 cycles. However, the data shows that, if there is an effect, RCA may slightly improve freeze-thaw durability performance on the basis that stiffness and modal frequency were not affected as much if RCA was used.

4.6 Summary and Conclusions

The coarse RCA used in this study came from panels obtained from interstate roadways (I-5 and I-90) and from a US Air Force runway. As such, the original materials (including the natural aggregates) are of high quality. RCA from all three sources had lower specific gravities and higher absorption capacities than those for the coarse natural aggregates due to the presence of adhered mortar on the RCA. The adhered mortar was also largely the cause of the increased Los Angeles abrasion loss for the RCA in comparison to that for the natural aggregate. Despite this, the processed RCA still was below the WSDOT maximum allowable loss value for aggregates in PCCP. Results for tests for the degradation value showed that RCA should not be expected to meet WSDOT minimum requirements in its raw condition. However, processed 100% RCA and various mixtures of coarse natural aggregates and coarse RCA all performed similarly with degradation values meeting WSDOT requirements. This showed that washing and sieving of RCA to remove fines is important for its suitability as an aggregate. Alkali-silica reactivity concern varied for the three RCAs in this project, indicating that the alkali-silica reaction potential may be an issue when using RCA and that additional tests may be required for RCAs whose original virgin aggregates were reactive.

Higher rates of RCA substitution resulted in reduced workability of concrete mixtures. Oppositely, a 20% fly ash substitution increased the workability, suggesting that it would serve well as a counteracting agent for RCA's effect to reduce workability. It could not be concluded if RCA had any effect on the air contents of concrete mixtures; however, the lack of effect the RCA had on the aggregate correction factor in the air content test method suggests that any effects of RCA on air content exist are either insignificant or indirect. RCA was less dense than coarse natural aggregate. Thus, greater rates of RCA substitution results in lower values of fresh concrete density.

All individual concrete samples exceeded the WSDOT minimum required 28-day compressive strength of 4,000 psi. RCA appeared to cause a small decrease in late age strength gain, but it was determined to not be statistically significant. Fly ash decreased early-age compressive strengths while increasing later-age compressive strength.

All individual concrete samples exceeded the WSDOT minimum required 14-day modulus of rupture of 650 psi. RCA did not have a statistically significant effect on concrete MOR.

All individual concrete samples had coefficients of thermal expansion that were typical of that for ordinary concrete. While still falling within the range of ordinary concrete values, RCA may slightly reduce the coefficient of thermal expansion by up to10%.

The drying shrinkage strain values for all individual concrete specimens fell within a range that was typical of that for ordinary concrete after 140 days. While still falling in the range of ordinary concrete values, RCA appeared to increase drying shrinkage by up to 30% for a 45% replacement.

Preliminary results for tests on freeze-thaw durability showed that RCA was not a significant factor influencing durability performance. The results suggest that RCA may slightly improve freeze-thaw performance.

Overall, no significant differences were observed in the properties of concrete incorporating the three different sources of RCA. It should again be noted that all three sources of RCA used in this study were very high quality.

The results of this study show that RCA replacement rates of up to 45% have no significant negative effects on concrete properties when the RCA is of similar quality to those used in this project. A previous study suggested limiting RCA replacement rates to 30% (Limbachiya, Meddah, & Ouchagour, 2012). All individual samples incorporating a 45% RCA substitution in this study met all WSDOT requirements, showing that a 30% limit on RCA substitution may overly restrictive for high quality RCAs.

CHAPTER 5: SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary

The primary objective of this research was to evaluate the suitability of incorporating RCA produced from demolished concrete pavements in Washington State as an aggregate in new portland cement concrete pavements. This was done through the investigation of the properties of RCA and evaluating its effects on concrete properties relating to the performance of new PCCP. Three sources of RCA were investigated in this study, incorporating demolished concrete pavements from western, eastern, and central Washington. All three RCA sources were produced from demolished pavements containing high-quality original materials.

The variables investigated in this study included varying levels of replacement of coarse natural aggregate with coarse RCA, the source of the RCA used, and a replacement of portland cement with fly ash. In total, twenty concrete batches were produced. For each batch, fresh concrete samples were tested for slump, air content, and density. Hardened concrete samples were tested for compressive strength, modulus of rupture, coefficient of thermal expansion, drying shrinkage, and freeze-thaw durability. Additional tests were performed on RCA from each of the three sources to determine specific gravity, absorption capacity, Los Angeles abrasion loss, degradation values, and alkali-silica reactivity.

5.2 Conclusions

The major conclusions reached in this study investigating the use of RCA as aggregates in new portland cement pavements are given in this section.

Effect of RCA on Degradation Value: RCA should be sieved and washed to remove fine materials in order to meet WSDOT minimum requirements for degradation value. Once fines are removed, degradation values were found to be nearly constant regardless of mixture ratios of RCA with coarse natural aggregate.

Effect of RCA on Fresh Concrete Workability: RCA reduces the workability of fresh concrete in mixes in which it is incorporated. Oppositely, fly ash increases workability and counteracts a slump reduction resulting from the incorporation of RCA.

Effect of RCA on Fresh Concrete Air Content: No conclusions about the effect of RCA on fresh concrete air content can be reached because the volume of AEA was varied in this study in order to reach a target range of air content values. It is unlikely that RCA has any significant direct effects on air content because the aggregate correction factor for a 45% replacement of RCA was the same as that of a 0% replacement.

Effect of RCA on Fresh Concrete Density: Greater rates of RCA substitution correlate to a decrease in fresh concrete density.

Effect of RCA on Concrete Compressive Strength: RCA does not have a significant effect on the compressive strength of hardened concrete for up to a 45% replacement for coarse natural aggregate. All concrete samples incorporating RCA met WSDOT compressive strength requirements of 4,000 psi for PCCP. A 20% fly ash substitution resulted in lower early-age strengths, nearly equivalent 28-day strengths, and greater long-term strengths.

The effect of RCA on compressive strength gain was also insignificant. A 20% fly ash substitution resulted in lower early-age compressive strength gain and greater later-age compressive strength gain.

Effect of RCA on Concrete Modulus of Rupture: RCA does not have a significant effect on the modulus of rupture for concretes incorporating up to a 45% substitution of RCA for coarse natural aggregate. Concretes tested in this study met WSDOT modulus of rupture requirement of 650 psi for PCCP for up to a 45% replacement of coarse natural aggregate with coarse RCA.

Effect of RCA on Concrete Coefficient of Thermal Expansion: RCA does not have a significant effect on concrete coefficient of thermal expansion, resulting in values that are typical of ordinary concrete. On average, concretes incorporating RCA with a 45% rate of substitution of coarse natural aggregate results in up to a 10% reduction in the resulting coefficient of thermal expansion.

Effect of RCA on Concrete Drying Shrinkage: RCA was found to cause up to a 30% increase in the drying shrinkage of PCCP for a 45% replacement of coarse natural aggregate with coarse RCA.

Effect of RCA on Concrete Freeze-Thaw Durability: RCA did not have a significant effect on the performance of concrete undergoing cycles of freezing and thawing. Preliminary test results show that, if anything, RCA may improve durability performance.

5.3 Recommendations

Based on the results and performance of concretes incorporating RCA in this study, the following recommendations are made:

- RCA produced from recycled concrete pavements incorporating high-quality original materials can be an acceptable source of aggregate for new PCCP at up to 45% replacement of natural coarse aggregate.
- RCA should be sieved and washed in order to remove fine material (< No. 4) for use in new PCCP.
- Properties of the RCA can be characterized using standards tests, including resistance to abrasion and the amount of adhered mortar.
- While no maximum effective RCA substitution rate was established in this study, the results show that a 45% substitution of coarse natural aggregate with high-quality RCA meets all WSDOT requirements for use in new PCCP.
- WRA and fly ash can be used to negate the effects of RCA on fresh concrete workability.
- In order to address performance concerns related to the alkali-silica reactivity of RCA, it is recommended that each RCA source be tested for alkali-silica reactivity following the crushing process and mitigated as necessary.

- Construction of a test section incorporating RCA is recommended to address the constructability related to the use of RCA.
- Additional research is recommended to explore the performance of PCCP incorporating substitution rates greater than 45%, to establish minimum performance criteria for RCA properties, and to perform a more rigorous freeze-thaw durability performance with more than 300 freeze-thaw cycles.

ACKNOWLEDGEMENTS

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APPENDIX A: REFERENCE MIX DESIGN

Washington State Department of Transportation

Concrete Mix Design

Contractor	Submitted By	Date
Acme Concrete Paving	Craig L. Matteson Central Pre-Mix Concrete Co.	7/8/2011

Concrete Supplier	Plant Location
Central Pre-Mix Concrete Co.	1901 N. Sullivan Road or Crestline & Magnesium

Contract Number	Contract Name
8022	Sullivan To Barker Road - Additional Lanes

This mix is to be used in the following Bid Item No(s): 6.0 Sack14 Day Cement Concrete Pavement

Concrete Class: *(check one only)*

☐ 3000 ☐ 4000 ☐ 4000D[a] ☐ 4000P[a] ☐ 4000W ☐ Concrete Overlay ☒ Cement Concrete Pavement[d]
☐ Other

Remarks: To be used for slip-form and mixer placements with air content adjustments.

Mix Design No. 320244 **Plant No.** 1, 2 or 4

Cementitious Materials	Source	Type, Class or Grade	Sp. Gr.	Lbs/cy
Cement	Ash Grove Durkee, OR	Type I-II	3.15	564
Fly Ash[a]				
GGBFS (Slag)				
Latex				
Microsilica				

Concrete Admixtures	Manufacturer	Product	Type	Est. Range (oz/cy)
Air Entrainment	WR Grace	Daravair 1000		2 to 25
Water Reducer	WR Grace	WRDA 64	A & D	15-35
High-Range Water Reducer				
Set Retarder	WR Grace	Recover *(if needed)*	D	0-15
Other				

Water (Maximum) 248 lbs/cy Is any of the water Recycled or Reclaimed? ☐ Yes[e] ☒ No

Water Cementitious Ratio (Maximum) .44 Mix Design Density 144.4 +/- lbs/cf[d]

Design Performance	1	2	3	4	5	Average[f]
28 Day Compressive Strength (cylinders) psi	5,090	5,050	5,020	4,840	4,740	4,950
14 Day Flexural[d] Strength (beams) psi	875	885	905	875	910	890

Agency Use Only (Check appropirate Box)

☑ **This Mix Design MEETS CONTRACT SPECIFICATIONS and may be used on the bid items noted above**
☐ **This Mix Design DOES NOT MEET CONTRACT SPECIFICATIONS and is being returned for corrections**

Reviewed By: [signature] 7/13/2011
PE Signature Date

DOT Form 350-040 EF
Revised 6/05

Distribution: Original - Contractor
Copies To - State Materials Lab-Structural Materials Eng. ; Regional Materials Lab; Project Inspector

Mix Design No. 320244 **Plant No.** 1, 2 or 4

Aggregate Information

Concrete Aggregates	Component 1	Component 2	Component 3	Component 4	Component 5	Combined Gradation
WSDOT Pit No.	PS C-173	PS C-173	PS C-173	PS C-173	PS C-297 & PS C-120	
WSDOT ASR 14-day Results (%) [b]	☒ Yes ☐ No	☒ Yes ☐ No	☒ Yes ☐ No	☒ Yes ☐ No	☒ Yes ☐ No	
Grading[c]	11/2" Round Combined	3/4" Round Combined	3/8" Round Combined	Coarse Sand Combined	Blend Sand Combined	
Percent of Total Aggregate	23	34	07	11	25	100%
Specific Gravity	2.69	2.68	2.67	2.64	2.64	
Lbs/cy (ssd)	700	1040	220	350	770	1.5" NMA Specification

Percent Passing

	Component 1	Component 2	Component 3	Component 4	Component 5	Combined Gradation	1.5" NMA Specification
2 inch						100	100
1-1/2 inch	100					100	87-100
1 inch	39.7	100				86.0	
3/4 inch	4.9	95.9				77.0	62-88
1/2 inch	1.1	55.2	100			62.0	
3/8 inch	.8	25.5	99.8	100	100	51.8	43-64
No. 4		1.1	33.4	98.1	99.4	38.4	29-47
No. 8		.9	3.2	59.9	96.1	31.1	19-34
No. 16		.8	.9	24.9	83.9	24.0	12-25
No. 30		.7	.8	8.9	53.1	14.4	7-18
No. 50		.6	.7	4.1	19.8	5.5	3-14
No. 100		.5	.6	2.2	6.3	2.0	0-10
No. 200	.5	.4	.5	1.5	3.5	1.3	0-2.0

Fineness Modulus: N/A (Required for Class 2 Sand)

ASR Mitigation Method Proposed [b]: **Using Low Alkali Cement**

Notes:

a Required for Class 4000D and 4000P mixes.

b Alkali Silica Reactivity Mitigation is required for sources with expansions over 0.20% - Incidate method for ASR mitigation.
For expansion of 0.21% - o.45%, acceptable mitigation can be the use of low alkali cement or 25% type F fly ash.
Any other proposed mitigation method or for pits with greater than 0.45% expansion, proof of mitigating measure, either ASTM C1260 / AASHTO T303 test results must be attached.
If ASTM C 1293 testing has been submitted indicating 1-year expansion of 0.04% or less, mitigation is not required.

c AASHTO No. 467, 57, 67, 7, 8; WSDOT Class 1, Class 2; or combined gradation. See Standard Specification 9-03.1.

d Required for Cement Concrete Pavements.

e Attach test results indicating conformance to Standard Specification 9-25.1.

f Actual Average Strength as determined from testing or estimated from ACI 211.

DOT Form 350-040 EF
Revised 6/06

APPENDIX B: MIX QUANTITIES FOR 1 CY

	SSD Coarse Aggregate (lb/cy)	SSD Fine Aggregate (lb/cy)	SSD RCA (lb/cy)	Cement (lb/cy)	Fly Ash (lb/cy)	Effective Mix Water (lb/cy)	AEA (oz./cy)	WRA (oz./cy)
Reference Mix Design	1898	1183	0	564	0	248	2-25	15-35
X-0-0	1898	1183	0	564	0	244	6.0	0
A-15-0	1613	1183	273	564	0	248	9.0	0
A-30-0	1328	1183	545	564	0	241	9.6	0
A-45-0	1044	1183	818	564	0	248	8.0	2.9
B-15-0	1613	1183	273	564	0	246	8.0	0
B-30-0	1325	1183	545	564	0	245	8.9	0
B-45-0	1044	1183	818	564	0	243	8.2	1.5
C-15-0	1613	1183	273	564	0	248	11.1	0
C-30-0	1325	1183	545	564	0	248	9.5	0
C-45-0	1044	1183	818	564	0	248	8.5	0
X-0-20	1898	1183	0	451	113	225	8.0	0
A-15-20	1613	1183	273	451	113	226	8.8	0
A-30-20	1328	1183	545	451	113	238	7.6	0
A-45-20	1044	1183	818	451	113	225	8.3	0
B-15-20	1613	1183	273	451	113	233	8.2	0
B-30-20	1328	1183	545	451	113	235	7.4	0
B-45-20	1044	1183	818	451	113	232	8.3	0
C-15-20	1613	1183	273	451	113	222	10.2	0
C-30-20	1328	1183	545	451	113	225	10.2	0
C-45-20	1044	1183	818	451	113	228	8.5	0

APPENDIX C: COMPRESSIVE STRENGTH TEST DATA

X-0-0		
DAY	Ultimate Load	Compressive Strength
7	103823 lb	3672 psi
	106939 lb	3782 psi
	107313 lb	3795 psi
14	120610 lb	4266 psi
	126165 lb	4462 psi
	122035 lb	4316 psi
28	139780 lb	4944 psi
	133635 lb	4726 psi
	136844 lb	4840 psi
	137103 lb	4849 psi
	136009 lb	4810 psi
90	156359 lb	5530 psi
	156747 lb	5544 psi
	154675 lb	5471 psi

A-15-0		
DAY	Ultimate Load	Compressive Strength
7	101590 lb	3593 psi
	110579 lb	3911 psi
	106203 lb	3756 psi
14	126511 lb	4474 psi
	124079 lb	4388 psi
	103995 lb	3678 psi
28	143709 lb	5083 psi
	140039 lb	4953 psi
	135952 lb	4808 psi
	142845 lb	5052 psi
	133102 lb	4708 psi
90	155783 lb	5510 psi
	152559 lb	5396 psi
	151206 lb	5348 psi

B-15-0		
DAY	Ultimate Load	Compressive Strength
7	110824 lb	3920 psi
	117689 lb	4162 psi
	108809 lb	3848 psi
14	136470 lb	4827 psi
	137693 lb	4870 psi
	139549 lb	4936 psi
28	150660 lb	5329 psi
	153193 lb	5418 psi
	154459 lb	5463 psi
	151681 lb	5365 psi
	152862 lb	5406 psi
90	168188 lb	5948 psi
	173254 lb	6128 psi
	176075 lb	6227 psi

C-15-0		
DAY	Ultimate Load	Compressive Strength
7	96002 lb	3395 psi
	95165 lb	3366 psi
	93410 lb	3304 psi
14	106023 lb	3750 psi
	108363 lb	3833 psi
	108608 lb	3841 psi
28	125618 lb	4443 psi
	119560 lb	4229 psi
	118049 lb	4175 psi
	125892 lb	4453 psi
	123762 lb	4377 psi
90	146299 lb	5174 psi
	140154 lb	4957 psi
	131174 lb	4639 psi

A-30-0		
DAY	Ultimate Load	Compressive Strength
7	122395 lb	4329 psi
	122567 lb	4335 psi
	122294 lb	4325 psi
14	135462 lb	4791 psi
	139103 lb	4920 psi
	138369 lb	4894 psi
28	154833 lb	5476 psi
	145810 lb	5157 psi
	157942 lb	5586 psi
	162533 lb	5748 psi
	152775 lb	5403 psi
90	165181 lb	5842 psi
	168836 lb	5971 psi
	166519 lb	5889 psi

B-30-0		
DAY	Ultimate Load	Compressive Strength
7	107874 lb	3815 psi
	112436 lb	3977 psi
	107687 lb	3809 psi
14	137779 lb	4873 psi
	134671 lb	4763 psi
	136671 lb	4834 psi
28	152243 lb	5385 psi
	146026 lb	5165 psi
	149609 lb	5291 psi
	148530 lb	5253 psi
	154531 lb	5465 psi
90	157783 lb	5580 psi
	164893 lb	5832 psi
	168160 lb	5947 psi

C-30-0		
DAY	Ultimate Load	Compressive Strength
7	99106 lb	3505 psi
	99329 lb	3513 psi
	100259 lb	3546 psi
14	121431 lb	4295 psi
	122438 lb	4330 psi
	117790 lb	4166 psi
28	136398 lb	4824 psi
	136067 lb	4812 psi
	130771 lb	4625 psi
	130296 lb	4608 psi
	136542 lb	4829 psi
90	152991 lb	5411 psi
	150185 lb	5312 psi
	148343 lb	5247 psi

A-45-0		
DAY	Ultimate Load	Compressive Strength
7	111213 lb	3933 psi
	104300 lb	3689 psi
	110148 lb	3896 psi
14	130569 lb	4618 psi
	129562 lb	4582 psi
	131677 lb	4657 psi
28	147407 lb	5213 psi
	144500 lb	5111 psi
	142442 lb	5038 psi
	145795 lb	5156 psi
	140801 lb	4980 psi
90	154790 lb	5475 psi
	155740 lb	5508 psi
	162187 lb	5736 psi

B-45-0		
DAY	Ultimate Load	Compressive Strength
7	110579 lb	3911 psi
	110033 lb	3892 psi
	126410 lb	4471 psi
14	142010 lb	5023 psi
	148818 lb	5263 psi
	147234 lb	5207 psi
28	158892 lb	5620 psi
	155294 lb	5492 psi
	149307 lb	5281 psi
	161093 lb	5698 psi
	155121 lb	5486 psi
90	180552 lb	6386 psi
	174377 lb	6167 psi
	164101 lb	5804 psi

C-45-0		
DAY	Ultimate Load	Compressive Strength
7	109730 lb	3881 psi
	107212 lb	3792 psi
	104878 lb	3709 psi
14	131116 lb	4637 psi
	124597 lb	4407 psi
	122067 lb	4317 psi
28	141276 lb	4997 psi
	138211 lb	4888 psi
	140427 lb	4967 psi
	135189 lb	4781 psi
	142874 lb	5053 psi
90	164893 lb	5832 psi
	160273 lb	5669 psi
	162518 lb	5748 psi

X-0-20		
DAY	Ultimate Load	Compressive Strength
7	103080 lb	3646 psi
	110047 lb	3892 psi
	101443 lb	3588 psi
14	125316 lb	4432 psi
	124122 lb	4390 psi
	138024 lb	4882 psi
28	147479 lb	5216 psi
	150890 lb	5337 psi
	150070 lb	5308 psi
	154013 lb	5447 psi
	152045 lb	5377 psi
90	179227 lb	6339 psi
	179529 lb	6350 psi
	174031 lb	6155 psi

A-15-20		
DAY	Ultimate Load	Compressive Strength
7	108838 lb	3849 psi
	106432 lb	3764 psi
	115861 lb	4098 psi
14	131879 lb	4664 psi
	127072 lb	4494 psi
	135937 lb	4808 psi
28	155869 lb	5513 psi
	153308 lb	5422 psi
	164130 lb	5805 psi
	160935 lb	5692 psi
	156258 lb	5527 psi
90	184393 lb	6522 psi
	181169 lb	6408 psi
	190466 lb	6736 psi

B-15-20		
DAY	Ultimate Load	Compressive Strength
7	102997 lb	3643 psi
	102821 lb	3637 psi
	101055 lb	3574 psi
14	124525 lb	4404 psi
	124813 lb	4414 psi
	122236 lb	4323 psi
28	146141 lb	5169 psi
	147896 lb	5231 psi
	146889 lb	5195 psi
	147263 lb	5208 psi
	144644 lb	5116 psi
90	172822 lb	6112 psi
	174952 lb	6188 psi
	178781 lb	6323 psi

C-15-20		
DAY	Ultimate Load	Compressive Strength
7	96235 lb	3404 psi
	99738 lb	3528 psi
	95355 lb	3372 psi
14	113170 lb	4003 psi
	117099 lb	4142 psi
	117991 lb	4173 psi
28	134484 lb	4756 psi
	127115 lb	4496 psi
	133836 lb	4733 psi
	138959 lb	4915 psi
	127446 lb	4507 psi
90	159899 lb	5655 psi
	153855 lb	5442 psi
	154790 lb	5475 psi

A-30-20		
DAY	Ultimate Load	Compressive Strength
7	105011 lb	3714 psi
	107716 lb	3810 psi
	104239 lb	3687 psi
14	124942 lb	4419 psi
	130137 lb	4603 psi
	126899 lb	4488 psi
28	135477 lb	4792 psi
	150070 lb	5308 psi
	153135 lb	5416 psi
	154344 lb	5459 psi
	154833 lb	5476 psi
90	181400 lb	6416 psi
	172650 lb	6106 psi
	177687 lb	6284 psi

B-30-20		
DAY	Ultimate Load	Compressive Strength
7	105248 lb	3722 psi
	101698 lb	3597 psi
	101048 lb	3574 psi
14	125878 lb	4452 psi
	126050 lb	4458 psi
	119560 lb	4229 psi
28	149991 lb	5305 psi
	143867 lb	5088 psi
	146817 lb	5193 psi
	151077 lb	5343 psi
	146500 lb	5181 psi
90	180148 lb	6371 psi
	170074 lb	6015 psi
	174449 lb	6170 psi

C-30-20		
DAY	Ultimate Load	Compressive Strength
7	101626 lb	3594 psi
	103518 lb	3661 psi
	101396 lb	3586 psi
14	123561 lb	4370 psi
	120898 lb	4276 psi
	124228 lb	4394 psi
28	136052 lb	4812 psi
	137419 lb	4860 psi
	137463 lb	4862 psi
	133505 lb	4722 psi
	135995 lb	4810 psi
90	160849 lb	5689 psi
	162576 lb	5750 psi
	164461 lb	5817 psi

A-45-20		
DAY	Ultimate Load	Compressive Strength
7	101623 lb	3594 psi
	110004 lb	3891 psi
	107543 lb	3804 psi
14	122049 lb	4317 psi
	131677 lb	4657 psi
	127734 lb	4518 psi
28	149408 lb	5284 psi
	151868 lb	5371 psi
	163151 lb	5770 psi
	157899 lb	5585 psi
90	183458 lb	6489 psi
	178219 lb	6303 psi
	181414 lb	6416 psi

B-45-20		
DAY	Ultimate Load	Compressive Strength
7	92505 lb	3272 psi
	94803 lb	3353 psi
	92900 lb	3286 psi
14	115070 lb	4070 psi
	114796 lb	4060 psi
	117012 lb	4138 psi
28	135520 lb	4793 psi
	133447 lb	4720 psi
	134498 lb	4757 psi
	131389 lb	4647 psi
	137549 lb	4865 psi
90	163151 lb	5770 psi
	163929 lb	5798 psi
	164432 lb	5816 psi

C-45-20		
DAY	Ultimate Load	Compressive Strength
7	111616 lb	3948 psi
	108118 lb	3824 psi
	110450 lb	3906 psi
14	124050 lb	4387 psi
	122827 lb	4344 psi
	125618 lb	4443 psi
28	148616 lb	5256 psi
	148605 lb	5256 psi
	144241 lb	5101 psi
	141665 lb	5010 psi
	145047 lb	5130 psi
90	176895 lb	6256 psi
	178910 lb	6328 psi
	187118 lb	6618 psi

APPENDIX D: MODULUS OF RUPTURE TEST DATA

X-0-0				
Sample Name	Ultimate Load	Actual Depth	Actual Width	Modulus of Rupture
X-0-0-MOR-14-1	6373 lb	6.02 in.	6.00 in.	792 psi
X-0-0-MOR-14-2	6776 lb	6.04 in.	6.00 in.	837 psi
X-0-0-MOR-14-3	6273 lb	6.04 in.	6.00 in.	774 psi
X-0-0-MOR-14-4	6446 lb	6.02 in.	6.00 in.	801 psi
X-0-0-MOR-14-5	6516 lb	6.04 in.	6.00 in.	804 psi

X-0-20				
Sample Name	Ultimate Load	Actual Depth	Actual Width	Modulus of Rupture
X-0-20-MOR-14-1	6591 lb	6.05 in.	6.00 in.	812 psi
X-0-20-MOR-14-2	6745 lb	6.02 in.	6.00 in.	838 psi
X-0-20-MOR-14-3	6202 lb	5.99 in.	6.00 in.	778 psi
X-0-20-MOR-14-4	5905 lb	6.04 in.	6.00 in.	729 psi
X-0-20-MOR-14-5	5838 lb	6.01 in.	6.00 in.	727 psi

A-15-0				
Sample Name	Ultimate Load	Actual Depth	Actual Width	Modulus of Rupture
A-15-0-MOR-14-1	6199 lb	6.05 in.	6.04 in.	758 psi
A-15-0-MOR-14-2	6217 lb	6.04 in.	6.06 in.	759 psi
A-15-0-MOR-14-3	6498 lb	6.06 in.	6.03 in.	792 psi
A-15-0-MOR-14-4	6152 lb	6.00 in.	5.99 in.	770 psi
A-15-0-MOR-14-5	5945 lb	6.05 in.	5.96 in.	736 psi

A-30-0				
Sample Name	Ultimate Load	Actual Depth	Actual Width	Modulus of Rupture
A-30-0-MOR-14-1	6233 lb	6.00 in.	6.00 in.	779 psi
A-30-0-MOR-14-2	6595 lb	6.03 in.	6.04 in.	813 psi
A-30-0-MOR-14-3	6593 lb	6.04 in.	6.04 in.	809 psi
A-30-0-MOR-14-4	5916 lb	6.02 in.	6.00 in.	736 psi
A-30-0-MOR-14-5	5956 lb	6.03 in.	6.04 in.	732 psi

A-45-0				
Sample Name	Ultimate Load	Actual Depth	Actual Width	Modulus of Rupture
A-45-0-MOR-14-1	6032 lb	6.00 in.	6.02 in.	753 psi
A-45-0-MOR-14-2	5732 lb	6.03 in.	6.07 in.	702 psi
A-45-0-MOR-14-3	6031 lb	6.00 in.	6.03 in.	752 psi
A-45-0-MOR-14-4	5711 lb	6.04 in.	6.00 in.	704 psi
A-45-0-MOR-14-5	5734 lb	6.02 in.	6.00 in.	713 psi

A-15-20				
Sample Name	Ultimate Load	Actual Depth	Actual Width	Modulus of Rupture
A-15-20-MOR-14-1	6548 lb	6.10 in.	6.06 in.	784 psi
A-15-20-MOR-14-2	6623 lb	6.12 in.	6.13 in.	779 psi
A-15-20-MOR-14-3	6514 lb	6.04 in.	6.06 in.	796 psi
A-15-20-MOR-14-4	6201 lb	6.10' in.	6.04 in.	745 psi
A-15-20-MOR-14-5	6582 lb	6.07 in.	6.03 in.	800 psi

A-30-20				
Sample Name	Ultimate Load	Actual Depth	Actual Width	Modulus of Rupture
A-30-20-MOR-14-1	5639 lb	6.02 in.	6.40 in.	656 psi
A-30-20-MOR-14-2	5952 lb	6.20 in.	5.96 in.	701 psi
A-30-20-MOR-14-3	5672 lb	6.00 in.	5.93 in.	717 psi
A-30-20-MOR-14-4	6101 lb	6.00 in.	5.99 in.	764 psi
A-30-20-MOR-14-5	6221 lb	6.05 in.	6.01 in.	764 psi

A-45-20				
Sample Name	Ultimate Load	Actual Depth	Actual Width	Modulus of Rupture
A-45-20-MOR-14-1	5854 lb	6.04 in.	6.04 in.	718 psi
A-45-20-MOR-14-2	6307 lb	6.01 in.	6.00 in.	786 psi
A-45-20-MOR-14-3	6036 lb	6.04 in.	6.03 in.	741 psi
A-45-20-MOR-14-4	5747 lb	6.03 in.	6.04 in.	707 psi
A-45-20-MOR-14-5	6428 lb	6.06 in.	6.04 in.	784 psi

B-15-0				
Sample Name	Ultimate Load	Actual Depth	Actual Width	Modulus of Rupture
B-15-0-MOR-14-1	6810 lb	6.03 in.	6.00 in.	844 psi
B-15-0-MOR-14-2	6709 lb	6.05 in.	6.00 in.	826 psi
B-15-0-MOR-14-3	6778 lb	6.04 in.	6.00 in.	837 psi
B-15-0-MOR-14-4	7214 lb	6.01 in.	6.00 in.	898 psi
B-15-0-MOR-14-5	6647 lb	6.02 in.	6.00 in.	826 psi

B-30-0				
Sample Name	Ultimate Load	Actual Depth	Actual Width	Modulus of Rupture
B-30-0-MOR-14-1	6784 lb	6.03 in.	6.00 in.	840 psi
B-30-0-MOR-14-2	6292 lb	6.04 in.	6.00 in.	777 psi
B-30-0-MOR-14-3	6505 lb	6.01 in.	6.00 in.	810 psi
B-30-0-MOR-14-4	6328 lb	6.02 in.	6.00 in.	785 psi
B-30-0-MOR-14-5	5944 lb	6.05 in.	6.00 in.	730 psi

B-45-0				
Sample Name	Ultimate Load	Actual Depth	Actual Width	Modulus of Rupture
B-45-0-MOR-14-1	6148 lb	6.04 in.	6.00 in.	760 psi
B-45-0-MOR-14-2	6571 lb	6.04 in.	6.00 in.	812 psi
B-45-0-MOR-14-3	6478 lb	6.06 in.	6.00 in.	795 psi
B-45-0-MOR-14-4	5307 lb	6.00 in.	6.00 in.	663 psi
B-45-0-MOR-14-5	6744 lb	6.05 in.	6.00 in.	829 psi

B-15-20				
Sample Name	Ultimate Load	Actual Depth	Actual Width	Modulus of Rupture
B-15-20-MOR-14-1	6429 lb	6.02 in.	6.00 in.	800 psi
B-15-20-MOR-14-2	6054 lb	6.04 in.	6.00 in.	747 psi
B-15-20-MOR-14-3	5626 lb	6.03 in.	6.00 in.	696 psi
B-15-20-MOR-14-4	6733 lb	6.04 in.	6.00 in.	831 psi
B-15-20-MOR-14-5	6503 lb	6.05 in.	6.00 in.	800 psi

B-30-20				
Sample Name	Ultimate Load	Actual Depth	Actual Width	Modulus of Rupture
B-30-20-MOR-14-1	6237 lb	6.00 in.	6.00 in.	780 psi
B-30-20-MOR-14-2	6587 lb	6.04 in.	6.00 in.	812 psi
B-30-20-MOR-14-3	6359 lb	6.01 in.	6.00 in.	791 psi
B-30-20-MOR-14-4	6238 lb	6.00 in.	6.00 in.	780 psi
B-30-20-MOR-14-5	5767 lb	5.99 in.	6.00 in.	723 psi

B-45-20				
Sample Name	Ultimate Load	Actual Depth	Actual Width	Modulus of Rupture
B-45-20-MOR-14-1	6028 lb	6.04 in.	6.00 in.	745 psi
B-45-20-MOR-14-2	5819 lb	6.03 in.	6.00 in.	719 psi
B-45-20-MOR-14-3	5895 lb	6.02 in.	6.00 in.	732 psi
B-45-20-MOR-14-4	5973 lb	6.06 in.	6.00 in.	732 psi
B-45-20-MOR-14-5	5668 lb	6.03 in.	6.00 in.	702 psi

C-15-0				
Sample Name	Ultimate Load	Actual Depth	Actual Width	Modulus of Rupture
C-15-0-MOR-14-1	5705 lb	6.06 in.	5.97 in.	703 psi
C-15-0-MOR-14-2	5751 lb	6.02 in.	5.99 in.	715 psi
C-15-0-MOR-14-3	5680 lb	6.04 in.	5.99 in.	701 psi
C-15-0-MOR-14-4	6121 lb	6.00 in.	6.02 in.	763 psi
C-15-0-MOR-14-5	6187 lb	6.06 in.	5.99 in.	759 psi

C-30-0				
Sample Name	Ultimate Load	Actual Depth	Actual Width	Modulus of Rupture
C-30-0-MOR-14-1	6452 lb	6.01 in.	5.95 in.	811 psi
C-30-0-MOR-14-2	6449 lb	6.02 in.	5.97 in.	805 psi
C-30-0-MOR-14-3	6207 lb	6.06 in.	5.93 in.	771 psi
C-30-0-MOR-14-4	6174 lb	6.06 in.	6.00 in.	756 psi
C-30-0-MOR-14-5	6821 lb	6.03 in.	5.97 in.	850 psi

C-45-0				
Sample Name	Ultimate Load	Actual Depth	Actual Width	Modulus of Rupture
C-45-0-MOR-14-1	5726 lb	6.05 in.	5.95 in.	709 psi
C-45-0-MOR-14-2	6172 lb	6.03 in.	6.01 in.	762 psi
C-45-0-MOR-14-3	6394 lb	6.05 in.	5.99 in.	786 psi
C-45-0-MOR-14-4	6913 lb	6.03 in.	5.96 in.	863 psi
C-45-0-MOR-14-5	6812 lb	6.02 in.	6.05 in.	837 psi

C-15-20				
Sample Name	Ultimate Load	Actual Depth	Actual Width	Modulus of Rupture
C-15-20-MOR-14-1	6774 lb	6.03 in.	5.97 in.	842 psi
C-15-20-MOR-14-2	6861 lb	6.05 in.	6.00 in.	844 psi
C-15-20-MOR-14-3	5858 lb	6.05 in.	6.03 in.	718 psi
C-15-20-MOR-14-4	6078 lb	6.02 in.	6.02 in.	752 psi
C-15-20-MOR-14-5	6178 lb	6.05 in.	6.02 in.	758 psi

C-30-20				
Sample Name	Ultimate Load	Actual Depth	Actual Width	Modulus of Rupture
C-30-20-MOR-14-1	6170 lb	6.03 in.	6.07 in.	754 psi
C-30-20-MOR-14-2	6854 lb	6.20 in.	6.07 in.	793 psi
C-30-20-MOR-14-3	6331 lb	6.05 in.	6.00 in.	778 psi
C-30-20-MOR-14-4	6178 lb	6.14 in.	6.06 in.	729 psi
C-30-20-MOR-14-5	6221 lb	6.07 in.	6.08 in.	751 psi

C-45-20				
Sample Name	Ultimate Load	Actual Depth	Actual Width	Modulus of Rupture
C-45-20-MOR-14-1	6360 lb	6.06 in.	6.02 in.	777 psi
C-45-20-MOR-14-2	6819 lb	6.05 in.	6.02 in.	836 psi
C-45-20-MOR-14-3	6552 lb	6.05 in.	6.04 in.	802 psi
C-45-20-MOR-14-4	6537 lb	6.03 in.	6.01 in.	807 psi
C-45-20-MOR-14-5	6596 lb	6.07 in.	6.02 in.	804 psi

APPENDIX E: COEFFICIENT OF THERMAL EXPANSION TEST DATA

Sample Name	28-Day Coefficient of Thermal Expansion (in./in. °C)
X-0-0-CTE-28-1	9.21E-06
X-0-0-CTE-28-2	9.81E-06
X-0-0-CTE-28-3	8.86E-06
X-0-20-CTE-28-1	9.57E-06
X-0-20-CTE-28-2	1.93E-05
X-0-20-CTE-28-3	9.11E-06
A-15-0-CTE-28-1	8.20E-06
A-15-0-CTE-28-2	8.46E-06
A-15-0-CTE-28-3	7.85E-06
A-30-0-CTE-28-1	8.24E-06
A-30-0-CTE-28-2	9.27E-06
A-30-0-CTE-28-3	8.32E-06
A-45-0-CTE-28-1	8.96E-06
A-45-0-CTE-28-2	8.69E-06
A-45-0-CTE-28-3	8.78E-06
A-15-20-CTE-28-1	1.01E-05
A-15-20-CTE-28-2	8.35E-06
A-15-20-CTE-28-3	8.45E-06
A-30-20-CTE-28-1	7.89E-06
A-30-20-CTE-28-2	8.92E-06
A-30-20-CTE-28-3	8.74E-06
A-45-20-CTE-28-1	9.52E-06
A-45-20-CTE-28-2	7.99E-06
A-45-20-CTE-28-3	8.96E-06

Sample Name	28-Day Coefficient of Thermal Expansion (in./in. °C)
B-15-0-CTE-28-1	9.45E-06
B-15-0-CTE-28-2	1.06E-05
B-15-0-CTE-28-3	8.41E-06
B-30-0-CTE-28-1	9.51E-06
B-30-0-CTE-28-2	9.20E-06
B-30-0-CTE-28-3	9.33E-06
B-45-0-CTE-28-1	9.47E-06
B-45-0-CTE-28-2	8.56E-06
B-45-0-CTE-28-3	8.72E-06
B-15-20-CTE-28-1	8.89E-06
B-15-20-CTE-28-2	9.16E-06
B-15-20-CTE-28-3	9.13E-06
B-30-20-CTE-28-1	9.36E-06
B-30-20-CTE-28-2	8.48E-06
B-30-20-CTE-28-3	9.50E-06
B-45-20-CTE-28-1	9.65E-06
B-45-20-CTE-28-2	1.03E-05
B-45-20-CTE-28-3	9.92E-06

Sample Name	28-Day Coefficient of Thermal Expansion (in./in. °C)
C-15-0-CTE-28-1	8.83E-06
C-15-0-CTE-28-2	9.09E-06
C-15-0-CTE-28-3	8.80E-06
C-30-0-CTE-28-1	9.33E-06
C-30-0-CTE-28-2	7.54E-06
C-30-0-CTE-28-3	5.44E-06
C-45-0-CTE-28-1	8.08E-06
C-45-0-CTE-28-2	9.04E-06
C-45-0-CTE-28-3	7.93E-06
C-15-20-CTE-28-1	8.85E-06
C-15-20-CTE-28-2	1.11E-05
C-15-20-CTE-28-3	1.30E-05
C-30-20-CTE-28-1	9.08E-06
C-30-20-CTE-28-2	8.07E-06
C-30-20-CTE-28-3	8.76E-06
C-45-20-CTE-28-1	4.48E-06
C-45-20-CTE-28-2	2.88E-05
C-45-20-CTE-28-3	5.94E-06

APPENDIX F: DRYING SHRINKAGE TEST DATA

X-0-0				
Day	Sample 1	Sample 2	Sample 3	Reference
1	0.1554 in.	0.1409 in.	0.1382 in.	0.0973 in.
28	0.1246 in.	0.1102 in.	0.1074 in.	0.0660 in.
32	0.1234 in.	0.1093 in.	0.1063 in.	0.0658 in.
35	0.1224 in.	0.1084 in.	0.1052 in.	0.0657 in.
42	0.1208 in.	0.1069 in.	0.1042 in.	0.0660 in.
56	0.1192 in.	0.1055 in.	0.1024 in.	0.0654 in.
84	0.1172 in.	0.1040 in.	0.1007 in.	0.0645 in.
140	0.1174 in.	0.1036 in.	0.1003 in.	0.0655 in.

X-0-20				
Day	Sample 1	Sample 2	Sample 3	Reference
1	0.1396 in.	0.1305 in.	0.1115 in.	0.0966 in.
28	0.1300 in.	0.1298 in.	0.1112 in.	0.0973 in.
32	0.1296 in.	0.1292 in.	0.1108 in.	0.0974 in.
35	0.1281 in.	0.1282 in.	0.1093 in.	0.0973 in.
42	0.0982 in.	0.0978 in.	0.0795 in.	0.0689 in.
56	0.0945 in.	0.0944 in.	0.0759 in.	0.0662 in.
84	0.0928 in.	0.0925 in.	0.0742 in.	0.0653 in.
140	0.0916 in.	0.0920 in.	0.0733 in.	0.0653 in.

B-15-0				
Day	Sample 1	Sample 2	Sample 3	Reference
1	0.1399 in.	0.1315 in.	0.1739 in.	0.0970 in.
28	0.1111 in.	0.1018 in.	0.1449 in.	0.0682 in.
32	0.1102 in.	0.1006 in.	0.1441 in.	0.0687 in.
35	0.1099 in.	0.1006 in.	0.1438 in.	0.0687 in.
42	0.1075 in.	0.0980 in.	0.1406 in.	0.0662 in.
56	0.1054 in.	0.0960 in.	0.1388 in.	0.0657 in.
84	0.1021 in.	0.0925 in.	0.1352 in.	0.0635 in.
140	0.1018 in.	0.0927 in.	0.1361 in.	0.0646 in.

B-30-0				
Day	Sample 1	Sample 2	Sample 3	Reference
1	0.1379 in.	0.1323 in.	0.1615 in.	0.0972 in.
28	0.1106 in.	0.1045 in.	0.1340 in.	0.0687 in.
32	0.1093 in.	0.1033 in.	0.1320 in.	0.0687 in.
35	0.1090 in.	0.1032 in.	0.1317 in.	0.0687 in.
42	0.1042 in.	0.0981 in.	0.1268 in.	0.0662 in.
56	0.1030 in.	0.0968 in.	0.1254 in.	0.0658 in.
84	0.0995 in.	0.0935 in.	0.1224 in.	0.0635 in.
140	0.0996 in.	0.0942 in.	0.1225 in.	0.0646 in.

B-45-0				
Day	Sample 1	Sample 2	Sample 3	Reference
1	0.1108 in.	0.1138 in.	0.1265 in.	0.0971 in.
28	0.0812 in.	0.0842 in.	0.0969 in.	0.0681 in.
32	0.0808 in.	0.0834 in.	0.0960 in.	0.0686 in.
35	0.0796 in.	0.0828 in.	0.0954 in.	0.0687 in.
42	0.0772 in.	0.0801 in.	0.0928 in.	0.0662 in.
56	0.0753 in.	0.0781 in.	0.0906 in.	0.0656 in.
84	0.0723 in.	0.0751 in.	0.0878 in.	0.0643 in.
140	0.0719 in.	0.0747 in.	0.0876 in.	0.0646 in.

B-15-20				
Day	Sample 1	Sample 2	Sample 3	Reference
1	0.1314 in.	0.1174 in.	0.1566 in.	0.0972 in.
28	0.1035 in.	0.0890 in.	0.1281 in.	0.0685 in.
32	0.1001 in.	0.0865 in.	0.1251 in.	0.0663 in.
35	0.0991 in.	0.0854 in.	0.1241 in.	0.0662 in.
42	0.0975 in.	0.0839 in.	0.1226 in.	0.0658 in.
56	0.0955 in.	0.0824 in.	0.1205 in.	0.0658 in.
84	0.0928 in.	0.0794 in.	0.1178 in.	0.0637 in.
140	0.0924 in.	0.0792 in.	0.1174 in.	0.0646 in.

B-30-20				
Day	Sample 1	Sample 2	Sample 3	Reference
1	0.1278 in.	0.1685 in.	0.1464 in.	0.0973 in.
28	0.0996 in.	0.1402 in.	0.1175 in.	0.0688 in.
32	0.0966 in.	0.1372 in.	0.1149 in.	0.0662 in.
35	0.0951 in.	0.1358 in.	0.1126 in.	0.0662 in.
42	0.0944 in.	0.1351 in.	0.1119 in.	0.0658 in.
56	0.0929 in.	0.1339 in.	0.1105 in.	0.0659 in.
84	0.0897 in.	0.1311 in.	0.1076 in.	0.0637 in.
140	0.0898 in.	0.1304 in.	0.1072 in.	0.0648 in.

B-45-20				
Day	Sample 1	Sample 2	Sample 3	Reference
1	0.1365 in.	0.1738 in.	0.1533 in.	0.0973 in.
28	0.1062 in.	0.1446 in.	0.1223 in.	0.0662 in.
32	0.1046 in.	0.1415 in.	0.1209 in.	0.0662 in.
35	0.1039 in.	0.1409 in.	0.1202 in.	0.0654 in.
42	0.1025 in.	0.1394 in.	0.1187 in.	0.0656 in.
56	0.1014 in.	0.1386 in.	0.1179 in.	0.0664 in.
84	0.0975 in.	0.1356 in.	0.1147 in.	0.0644 in.
140	0.0978 in.	0.1349 in.	0.1144 in.	0.0646 in.

C-15-0				
Day	Sample 1	Sample 2	Sample 3	Reference
1	0.0804 in.	0.1175 in.	0.0923 in.	0.0659 in.
28	0.0792 in.	0.1163 in.	0.0897 in.	0.0641 in.
32	0.0779 in.	0.1151 in.	0.0887 in.	0.0637 in.
35	0.0772 in.	0.1148 in.	0.0883 in.	0.0637 in.
42	0.0762 in.	0.1140 in.	0.0873 in.	0.0640 in.
56	0.0755 in.	0.1134 in.	0.0867 in.	0.0645 in.
84	0.0745 in.	0.1125 in.	0.0856 in.	0.0646 in.
140	0.0743 in.	0.1119 in.	0.0856 in.	0.0655 in.

C-30-0				
Day	Sample 1	Sample 2	Sample 3	Reference
1	0.1045 in.	0.0889 in.	0.1324 in.	0.0661 in.
28	0.1027 in.	0.0870 in.	0.1304 in.	0.0636 in.
32	0.1021 in.	0.0864 in.	0.1299 in.	0.0637 in.
35	0.1010 in.	0.0861 in.	0.1292 in.	0.0638 in.
42	0.1005 in.	0.0854 in.	0.1285 in.	0.0641 in.
56	0.0987 in.	0.0841 in.	0.1268 in.	0.0641 in.
84	0.0976 in.	0.0832 in.	0.1257 in.	0.0646 in.
140	0.0973 in.	0.0830 in.	0.1255 in.	0.0655 in.

C-45-0				
Day	Sample 1	Sample 2	Sample 3	Reference
1	0.0810 in.	0.1265 in.	0.1080 in.	0.0653 in.
28	0.0806 in.	0.1264 in.	0.1079 in.	0.0636 in.
32	0.0795 in.	0.1255 in.	0.1066 in.	0.0638 in.
35	0.0793 in.	0.1241 in.	0.1064 in.	0.0640 in.
42	0.0783 in.	0.1232 in.	0.1055 in.	0.0643 in.
56	0.0769 in.	0.1216 in.	0.1043 in.	0.0646 in.
84	0.0760 in.	0.1202 in.	0.1032 in.	0.0652 in.
140	0.0758 in.	0.1202 in.	0.1030 in.	0.0653 in.

C-15-20				
Day	Sample 1	Sample 2	Sample 3	Reference
1	0.1694 in.	0.1104 in.	0.1283 in.	0.0655 in.
28	0.1682 in.	0.1097 in.	0.1279 in.	0.0639 in.
32	0.1679 in.	0.1095 in.	0.1271 in.	0.0640 in.
35	0.1666 in.	0.1083 in.	0.1262 in.	0.0643 in.
42	0.1659 in.	0.1072 in.	0.1252 in.	0.0648 in.
56	0.1656 in.	0.1066 in.	0.1245 in.	0.0654 in.
84	0.1648 in.	0.1061 in.	0.1238 in.	0.0655 in.
140	0.1640 in.	0.1053 in.	0.1230 in.	0.0654 in.

C-30-20				
Day	Sample 1	Sample 2	Sample 3	Reference
1	0.0968 in.	0.1141 in.	0.1062 in.	0.0655 in.
28	0.0962 in.	0.1135 in.	0.1055 in.	0.0639 in.
32	0.0950 in.	0.1123 in.	0.1045 in.	0.0639 in.
35	0.0946 in.	0.1120 in.	0.1041 in.	0.0641 in.
42	0.0931 in.	0.1103 in.	0.1026 in.	0.0647 in.
56	0.0922 in.	0.1095 in.	0.1015 in.	0.0646 in.
84	0.0916 in.	0.1090 in.	0.1010 in.	0.0655 in.
140	0.0908 in.	0.1085 in.	0.1004 in.	0.0654 in.

C-45-20				
Day	Sample 1	Sample 2	Sample 3	Reference
1	0.1853 in.	0.1321 in.	0.1121 in.	0.0656 in.
28	0.1845 in.	0.1310 in.	0.1114 in.	0.0637 in.
32	0.1839 in.	0.1299 in.	0.1105 in.	0.0639 in.
35	0.1827 in.	0.1293 in.	0.1099 in.	0.0641 in.
42	0.1822 in.	0.1282 in.	0.1089 in.	0.0646 in.
56	0.1813 in.	0.1274 in.	0.1084 in.	0.0647 in.
84	0.1807 in.	0.1269 in.	0.1079 in.	0.0655 in.
140	0.1802 in.	0.1262 in.	0.1069 in.	0.0654 in.