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Evaluation of Low-Quality Recycled Concrete Pavement Aggregates for Portland Cement-Treated Base

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16 Abstract

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PREFACE

The Kansas Department of Transportation's (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative, and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

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Abstract

The Kansas Department of Transportation (KDOT) has begun to explore the incorporation of recycled concrete aggregates (RCAs) into portland cement-treated base (PCTB). KDOT currently employs a freeze-thaw method that includes 90 days of curing and 660 freeze-thaw cycles to determine the durability of concrete mixtures for pavements and stabilized bases. An experimental study was conducted to determine if low-quality RCA could adequately replace virgin aggregates in PCTB. Two sources of D-cracked aggregates were used to batch the PCTB mixture, while control samples were batched using virgin aggregates to mimic the gradation of the two RCA sources. Following the procedure specified by KDOT, the RCA and control samples were tested for freeze-thaw durability. Results showed that increasing the total amount of cementitious binder in the PCTB mixture increased the durability and that, at low binder contents, the RCA source influences the performance of the PCTB mixture with RCA. The RCA and control mixtures performed similarly, proving that RCA could be a viable aggregate source for PCTB. The conclusion was made, however, that new failure criteria must be developed for the freeze-thaw durability of PCTB mixture with RCA since the mass loss was an overriding issue.

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Chapter 1: Introduction

1.1 Background

Durability cracking or D-cracking is a progressive structural deterioration of concrete caused by freezing and thawing. D-cracking begins in coarse aggregate below the pavement surface at joints that permit moisture intrusion and then progress inward and upward. This distress has caused millions of dollars of damage to concrete pavements in Kansas to date. Because concrete pavements in Kansas use concrete mixtures made from limestone, they are highly susceptible to D-cracking when subjected to freeze-thaw conditions.

The Kansas Department of Transportation (KDOT) follows two procedures to assess the freeze-thaw performance or durability of coarse aggregates in concrete mixtures for pavements. The first method, KTMR-21 (1999), determines aggregate resistance to disintegration by freezing and thawing. The second method, KTMR-22 (2015), consists of a 90-day concrete curing period with a 21-day drying period after 67 days of curing in 100% humidity. This method is a modified version of ASTM C666 (2015). Dry curing removes moisture that would cause damage during freezing, and the low degree of saturation decreases deterioration during 660 cycles of freeze-thaw (which may take up to three months) required by KTMR-22. The combined durations of KTMR-22 curing and freeze-thaw testing may result in a 6-month aggregate qualification procedure.

1.2 Problem Statement

Recycled concrete aggregate (RCA) obtained from recycling portland cement concrete (PCC) pavements can be put back into new concrete pavements. However, concrete with more than 10% to 20% fine aggregates from RCA has demonstrated increased water demand for a given slump, resulting in decreased concrete quality. Although on-site processed RCA is a cost-effective solution for projects where suitable quality aggregate is not readily available, only minimal information is known about the suitability of lower quality recycled PCC pavement aggregates, such as those from pavements with D-cracking.

1.3 Objectives

The primary objectives of this study included the following:

1. Determine the freeze-thaw durability of RCA aggregates using KTMR-21;
2. Determine the strength of a portland cement-treated base (PCTB) mixture that incorporates RCA;
3. Determine the freeze-thaw durability of a PCTB mixture with RCA; and
4. Compare the durability of a PCTB mixture with RCA to control samples with virgin aggregates.

1.4 Study Method

This research was divided into two studies. The first study, the D-cracked RCA portland cement-treated base study, consisted of tasks to meet objectives 1 and 2. RCA was obtained from D-cracked pavement sources in Topeka and Kansas City. These aggregates were used to batch three PCTB mixtures per RCA source. All mixes followed KDOT's 90-day curing outlined in KTMR-22, including 67 days in a wet room with 100% humidity and drying in a 73 °F room with 50% relative humidity for 21 days. After a day of curing, the samples were immersed in 70 °F water for 24 hours, followed by immersion in 40 °F water. Upon completion of curing, all samples were subjected to cycles of freezing in air and thawing in water per ASTM C666 Procedure B (KTMR-22, 2015). Multiple measurements of mass, relative dynamic modulus of elasticity (RDME), and length change for each sample were made.

The second part of the study compared samples of concrete mixture with RCA to control samples. The control concrete samples consisted of virgin aggregates that had not been previously used in any concrete mixing. The virgin aggregates and concrete mixtures were placed under the same conditions as the RCA to limit any variability.

Chapter 2: Literature Review

2.1 Recycled Concrete Aggregate

PCC pavements can be entirely recyclable (Verian et al., 2013). RCA, the product of demolished concrete pavements or structures, is categorized by a minimum of 90% mass from portland cement-based wastes and natural aggregates (Silva et al., 2016). Because properties of fresh or hardened concrete consisting of 20% coarse RCA are identical to conventional concrete, RCA use in construction has become prevalent in the global building industry. In Great Britain, 10% of incorporated aggregates are RCA, while the Netherlands utilized 78,000 tons of RCA in 1994 (de Vries, 1996).

2.1.1 Physical Properties of RCA

An abundance of research has evaluated the physical properties of RCA. RCA particles are often derived from pavements in which reinforcement has been removed, and the pavement is then crushed to a specific size and gradation (ACI Committee E-701, 2016). Recycled materials can be crushed to nearly any gradation, and the occasional resulting residual dust on aggregate surfaces is typically non-problematic. The crushed material has a lower relative density than virgin material. RCA has a specific gravity of 2.2–2.5 in the saturated surface-dry (SSD) condition, but relative density decreases as the aggregate size decreases. For example, recycled sand has a relative density of approximately 2.0–2.3 (in SSD condition) due to increased absorption caused by cement mortar adhering to particles (ACI Committee E-701, 2016). RCA absorption is typically 2%–6% for coarse aggregate, with even higher percentages for sand, which can decrease the workability of the concrete mixture. Experience has shown that neither abrasion loss nor sulfate soundness is a concern for RCA. Residual chlorides in RCA containing deicing salts are usually below threshold values of up to 0.05% by mass for fine and coarse aggregates and are, therefore, not a concern (ACI Committee E-701, 2016). Although 100% of coarse aggregates can be recycled, the percentage of fine aggregates in RCA is usually limited to 10% to 20% (ACI Committee E-701, 2016).

2.1.2 RCA Production

RCA is produced by crushing existing concrete slabs or structures and sorting them into desired aggregate sizes. The recycling process includes primary and secondary crushing stages. Jaw crushers are typically used in the preliminary crushing stage to achieve optimal size distribution and reduce the material size to 3–4 inches. Secondary crushing is then done to achieve desired maximum size of coarse aggregate as well as round, less angular particles (Silva et al., 2016). Three main types of concrete recycling crushers (jaw, cone, and impact) each employ unique methods for concrete crushing, and each crushing process removes varying amounts of mortar from the original aggregate particles. The number of processing stages and the type of crushing device used to break down large concrete pieces dictate the size and shape of resulting aggregates (Verian et al., 2013).

2.2 D-Cracking of Coarse Aggregates

D-cracking occurs in coarse aggregates that have been subjected to cycles of freezing and thawing. Although D-cracking has happened since the 1930s when concrete pavements first came into general use, there is little agreement on the term's origin or the meaning of the "D," which varyingly denotes distress, discoloration, or deterioration, or the shape of the pieces that break off from the pavement (O'Doherty, 1987). D-cracking is widely known as durability cracking in Kansas. D-cracking starts at the bottom of the slab near the joints and can take several years to progress up to the top of the slab, where it first becomes visible (O'Doherty, 1987).

2.2.1 Mechanism of D-Cracking

D-cracking is related to moisture and freezing conditions. Aggregates that cause D-cracking absorb moisture from the pavement base and surface water entering through cracks and, most prominently, joints. If the aggregate pores are full when freezing occurs, internal pore pressure cracks the particles, causing the mortar to crack. Additional cracks develop with repeated freezing and thawing cycles (Whitehurst, 1980). Water expands approximately 9% upon freezing, so if a given capillary space is filled more than 91.7%, the volume of ice will be greater than the volume of pore space, and pressure will develop as the ice crystal outgrows the cavity (Thompson et al., 1980).

Powers and Helmuth (1953) proposed the ice accretion/osmotic pressure theory to explain experimental results inconsistent with the hydraulic pressure theory. The osmotic pressure theory states that, during freezing, water moves from gel pores to capillary pores according to the laws of thermodynamics and the osmosis theory (Powers & Helmuth, 1953). During this process, a concentrated alkali solution develops as ice is produced. Unfrozen water travels toward the freezing site due to differences in solute concentrations.

2.2.2 Appearance and Progression of D-Cracking

D-cracking is initiated by moisture, usually at the bottom of a slab where contact with the base inhibits drying. It is accentuated by poor drainage and cracks and joints that provide paths for water to the base. In addition, moisture penetrates tiny pores in the particles of coarse sedimentary aggregates (O'Doherty, 1987). D-cracking typically appears at transverse-longitudinal joint intersections and occasionally at the intersection of longitudinal joints, transverse cracks, and outside corners of a pavement slab (Whitehurst, 1980). As cracking progresses along the joints, the resulting crack pattern forms a nearly continuous network confined to the slab's peripheral areas. With continued deterioration, the pattern encroaches rapidly on the remaining portion of the slab (Whitehurst, 1980). In general, cracking can progress upward for several years before it becomes visible as a series of small cracks at the top of the slab, often preceded by dark discoloration of the concrete surface. D-cracking is usually only detected in the early stages by core drilling (O'Doherty, 1987).

2.2.3 Conditions for D-Cracking

Three primary conditions are needed to produce D-cracking. First, as stated in Section 2.1.1, moisture must be present and available so that the particles are more than 91.7% saturated. Second, the particles of coarse aggregate must be susceptible to cracking. Limestone and fine-grained sedimentary rocks are notorious for having planes of weakness and deleterious pore sizes (0.4–2.0 microns). The larger the particle, the longer the water path and the more cracks will form. Third, freezing and thawing must occur—the problem is less prevalent in very cold regions where fewer freezing and thawing cycles occur (O'Doherty, 1987). Not all sedimentary aggregates cause

D-cracking; however, aggregates with low permeability, high porosity, and small pore size are most likely to cause D-cracking (Schwartz, 1987).

2.2.4 Noninfluential Factors

A study conducted at the University of Illinois determined that using entrained air nearly eliminated the freeze-thaw deterioration of cement paste (Thompson et al., 1980). Another study found that fine aggregate properties, type and amount of cement, pavement design, and traffic only minimally influence D-cracking. At the same time, a positive underdrain system postpones but does not sufficiently prevent D-cracking (Schwartz, 1987).

2.2.5 Prevention

Some states have adopted a source-acceptance approach to avoid using aggregates that are susceptible to D-cracking. Acceptance criteria include performance histories and results of extensive testing. Besides approving general source locations, highway agencies may even identify specific acceptable ledges within a source for crushed stone (Schwartz, 1987). Aggregates such as high-quality dolomite and rocks of igneous origin are less prone to cracking, while aggregates with high proportions of chert and limestone have an increased risk of cracking. Although improved drainage of the base and sealing of joints significantly increase aggregate performance freezing-thawing, they cannot prevent deterioration (O'Doherty, 1987). Mechanical separation can be utilized to separate aggregates based on mechanical properties or specific gravity. Separation by specific gravity, the most common method, utilizes heavy fluids to float off materials with specific gravities lower than the specified gravity, typically 2.5. The assumption behind the use of heavy media separation to improve aggregate quality is that particles with low specific gravities are less durable than particles with high specific gravities (Thompson et al., 1980).

2.3 Freeze-Thaw Resistance of Concrete

In addition to durable aggregate for adequate freeze-thaw performance, a durable cement paste matrix is needed in concrete. This section describes the mechanisms of internal damage, surface scaling to hardened cement paste, and drying.

2.3.1 Mechanisms of Internal Damage

If the aggregates used in concrete are frost resistant, then the freeze-thaw resistance of the cement paste determines overall concrete resistance to freezing and thawing. If the aggregate is susceptible to freeze-thaw damage, the use of that aggregate can increase concrete deterioration. Several theories have been proposed to explain concrete damage due to freezing and thawing (Tanesi, & Meininger, 2006). For example, the critical saturation theory states that concrete will only suffer damage from freezing when the capillaries in the cement paste are more than 91.7% full of water (Powers, 1945). This theory is based on the fact that water expands in volume by approximately 9% when it freezes. Stress is generated when the capillary pores are saturated with water and the water freezes. However, if the pores are only partially filled, expansion resulting from ice formation may be accommodated (Tanesi, & Meininger, 2006).

Similarly, the hydraulic pressure theory states that a buildup of hydraulic pressure from resistance to the flow of unfrozen water in cement paste capillaries causes damage from freezing as the cement paste does not expand to accommodate the water as it freezes. The unfrozen water is pushed through the capillary pores, away from the sites of freezing, similar to water flowing through a pipe (Powers & Willis, 1950).

2.3.2 Effects of Surface Scaling

Compared to internal cracking, scaling is only a surface phenomenon; however, scaling is the most apparent form of deterioration, typically occurring only when concrete freezes in water and increases due to deicer salts (Pigeon et al., 1986). When deicer salt particles encounter water, the freezing point of water decreases; therefore, an increase in deicing chemicals corresponds to reduced hydraulic ice formation of ice. However, chloride ions in deicing salts can become physically or chemically bound by cement hydration products, thereby causing concrete expansion (Wu et al., 2015). In addition, excessive bleeding, bad finishing and overworking of the surface, plastic shrinkage cracking, lack of curing, and early exposure to relatively high temperatures can all weaken the concrete surface and indirectly cause rapid scaling when concrete is exposed to freezing in water (Pigeon & Pleau, 1995). Mass loss during freeze-thaw is driven by surface scaling, but this mass loss can increase significantly when severe internal cracking begins to disintegrate test specimens. Proper air entrainment improves concrete performance in deicer salt scaling but does not prevent scaling in concrete with a high water-to-cement ratio (Pigeon et al., 1986). A high water-to-cement ratio causes a high paste permeability and low strength. As a result, air entrainment does not effectively reduce surface scaling.

2.3.3 Effect of Drying

Periods of concrete drying can be very beneficial for freeze-thaw durability because drying causes pores in the concrete to become enlarged and interconnected, thereby increasing permeability. This increased water permeability allows water to reenter the concrete faster than before the drying period (Pigeon & Pleau, 1995). Drying eliminates the threat of internal damage caused by the expansive hydraulic pressure of freezing water. However, if the concrete is again exposed to water for an extended period, the benefits of drying are negated. Re-saturation of the pore network is a slower process due to various flow characteristics of the partially dried concrete. For intermediate re-saturation, the implication of slower re-saturation is a net increase in freeze-thaw resistance due to partial saturation of the pore network (Ranaivomanana et al., 2011).

2.4 Durable Aggregate and Concrete Identification Procedures

The American Society for Testing and Materials (ASTM) and other organizations have developed many standard test methods to measure and predict the freeze-thaw behavior of aggregate and concrete. Since D-cracking is critical for aggregate usage, aggregates resistant to freeze-thaw must be identified.

2.4.1 KTMR-21: Soundness & Modified Soundness of Aggregates by Freezing and Thawing

KDOT uses the KTMR-21 (1999) standard to determine aggregate resistance to disintegration by freezing and thawing. The procedure, which applies to coarse Class 1 aggregates, or Official Quality Aggregates, including Official Quality Sand-Gravel, requires a design gradation determined by the gradation of the coarse aggregate. The testing procedure begins with a soaking period, and then the material, while in a saturated and drained condition, is placed a freezing equipment that maintains a temperature between -20 °F and 0 °F (KTMR-21, 1999). The sample remains in the cool equipment until frozen, but in no case is freezing time less than 2 hours long. The aggregate is then placed in a tap water bath at 70–80 °F for 40 minutes. One freezing period and one thawing period is one cycle; the test concludes after 25 cycles. The aggregate is washed over a US No. 12 sieve, and the freeze-thaw loss ratio is calculated by dividing the cumulative mass of the sample at the end of the test by the cumulative mass of the sample at the beginning of the test. The aggregates must have a loss ratio of 85 or better to be tested under KTMR-22.

2.4.2 ASTM C666: Rapid Freezing and Thawing of Concrete

Most transportation organizations use ASTM C666 as the standard method to test the freeze-thaw resistance of concrete because it provides a process for freeze-thaw testing of concrete specimens and a process for determining specimen durability. ASTM C666 includes two procedures: Procedure A requires the samples to be immersed in water during the freeze-thaw process, while Procedure B uses air to freeze the pieces, followed by thawing in water. Each freeze-thaw cycle must last 2–5 hours. The temperature at the center of the concrete specimens must be 40±3 °F at the beginning of each cycle, but then the temperature is lowered to 0±3 °F and raised to 40±3 °F at the end of the cycle (ASTM C666, 2015).

Durability readings must be recorded for concrete specimens once every 36 cycles or less. If a sample is suspected of degrading quickly, readings may be taken more frequently. ASTM C666 requires that mass and resonant frequency be measured at every interval. Mass loss is typically attributed to surface scaling or internal cracks forming within the samples, while the mass gain is attributed to water absorption into sample pores. Cement hydration may be another factor of mass gain. The resonant frequency of a concrete specimen is the natural frequency at which the maximum amplitude of an induced mechanical wave occurs. The initial frequency is measured before the samples have been subjected to freeze-thaw and then is monitored at some intervals. The specimen's RDME is calculated based on the resonant frequency and is proportionally related to the resonant frequency. Mass and frequency readings cease when the RDME reaches 60%, and the final number of freeze-thaw cycles is documented. However, ASTM C666 allows other failure limits to be specified. Chapter 4 explains how to obtain and calculate a specimen's resonant frequency.

ASTM C666 also provides an optional length change procedure that may be conducted at every testing interval to determine specimen durability. When internal cracks form in aggregate or paste, the volume of concrete increases and expands. In addition to freeze-thaw cycling, this expansion eventually causes sample failure. ASTM C666 suggests an expansion of 0.10% of a specimen's original length as a failure limit. The procedure for determining expansion is discussed in Chapter 4.

2.4.3 ASTM C88: Aggregate Sulfate Soundness

ASTM C88 sets a procedure for testing aggregates to estimate their soundness when subjected to weathering action in concrete or other applications. The test method utilizes repeated immersion of the aggregates in saturated sulfate sodium or magnesium sulfate solutions, followed by oven drying to wholly or partially dehydrate the salt precipitated in the permeable pore spaces. The expansive internal force, derived from the rehydration of the salt upon re-immersion, simulates the expansion of water upon freezing (ASTM C88, 2013). This test is useful when the aggregate has no known performance records or when little to no information about durability is available.

2.4.4 ASTM C295: Petrographic Examination of Aggregates for Concrete

ASTM C295 (2019) outlines procedures such as microscopy, x-ray diffraction analysis, infrared spectroscopy, and differential thermal analysis for petrographic examination of materials proposed for aggregates. An experienced petrographer must conduct the tests. The purpose of the study is to determine the physical and chemical characteristics of the intended material and identify the portion of each coarse aggregate composed of weathered particles, as well as the extent of weathering. This determination is essential for aggregates that will be subjected to freezing and thawing since finely porous and highly weathered or otherwise altered rocks are susceptible to freeze-thaw damage.

2.4.5 Washington Hydraulic Fracture Test

The Washington Hydraulic Fracture Test (WHFT) predicts aggregate freeze-thaw performance by inducing pressure in aggregate pore walls to simulate the expansion of water due to freezing and thawing. The procedure utilizes compressed nitrogen to force water into aggregate pores, and then the pressure is released to allow air expansion within the pores. Water is expelled, which induces internal stress on the pore walls, similar to the stress of freezing and thawing. Pore walls fracture when the wall structure cannot rapidly dissipate the pressure. The test severity can be used to predict the freeze-thaw resistance of the aggregates (Embacher & Snyder, 2003).

2.5 Curing Methods for Freeze-Thaw Testing

Increased resistance to physical and chemical deterioration is the primary goal of an effective concrete curing process. Specimen curing typically involves a lime-water bath or a moisture room for a specified time. Two unconventional curing methods are discussed in the following sections.

2.6 Modified Versions of ASTM C666

Many departments of transportation (DOTs) use ASTM C666 to identify aggregate and concrete suitable for freeze-thaw. However, since DOT regions vary, some agencies have modified the tests to obtain results that correlate well with field records.

2.6.1 Kansas Department of Transportation

KDOT utilizes KTMR-22 as the standard test method to determine aggregate. This method outlines the curing and testing procedures for concrete specimens subjected to ASTM C666 Procedure B. Procedure B utilizes air freezing and immersion in water to thaw. The specimens undergo a 90-day curing method, which entails 67 days in a 100% moisture room. The specimens are then transferred to a curing room at 50% humidity for 21 days, followed by total immersion in water for two days. The 2-month period in the moisture room allows extensive cement hydration, leading to a more durable paste matrix. The extensive moist curing period is used so that freeze-thaw damage primarily occurs in the coarse aggregate. The 21-day dry curing period removes excess moisture from the aggregate pores, thereby reducing expansive pressures caused by freezing water.

KDOT has also increased the maximum number of cycles from ASTM C666 from 300 to 660. When tests were only conducted through 300 cycles, results could have accurately predicted performance in the field. KDOT selected 660 cycles based on weather data and pavement service life. Weather data shows that Kansas experiences 33 freeze-thaw cycles per year, and the design life of a pavement is 20 years, so multiplying 33 by 20 produces 660, the number of cycles specified in KTMR-22. Aggregate is deemed durable if the samples maintain an expansion ≤ 0.025% and an RDME ≥ 95% after the completion of 660 cycles (KTMR-22, 2015).

2.6.2 Other Organizations' Methods

Although the Michigan Department of Transportation (MDOT) also uses ASTM C666 Procedure B, MDOT specifies a 14-day curing period. The samples are covered with wet burlap for the first 24 hours after casting and then de-molded and immersed in saturated lime water for 12 days before being placed in 40 °F water for 16 hours. However, MDOT uses only 300 cycles or until expansion reaches 0.10%, and MDOT does not require measurement of mass or RDME (MTM 115 Michigan Test Method, 2015).

The Ohio Department of Transportation (ODOT) also uses ASTM C666 Procedure B, but the samples are placed in a moist room for 24 hours for curing while still in the molds. The samples are then de-molded and immersed in water for 14 days (Woodhouse, 2005). ASTM C666 must be performed on all 0.75-inch and 1-inch aggregates. Aggregate durability is determined by calculating the area under the curve obtained by plotting expansion versus the number of freeze-thaw cycles for a given test specimen. For aggregate sources approved by ODOT within one year of testing, this area may be at most 2.05 after a maximum of 350 cycles. Between 1 and 2 years after approval, this area may not exceed 1.00 after 350 cycles (ODOT, 2013).

2.7 Summary

Internal damage to the concrete from freeze-thaw occurs when hydraulic pressures develop in the pores of the cement. If the concrete surface is exposed to water and deicing chemicals, then the concrete will likely be susceptible to surface scaling. Many standard test procedures have been developed to predict and measure the durability of aggregate and concrete exposed to freeze-thaw conditions. KDOT uses an extended curing period, more than double the cycles specified by ASTM C666 Procedure B, and more stringent RDME acceptance criteria. KDOT's standards are stricter than other DOTs due to more extended curing, increased cycles, and higher RDME acceptance criteria. Other states like Michigan and Ohio have adopted modified curing methods, freeze-thaw test durations, and performance criteria based on ASTM C666. Curing studies have shown that temperature elevation during curing can accelerate concrete deterioration. However, although the studies simulate field conditions, results show that exposing concrete to drying periods during curing increases durability after a specified number of freeze-thaw cycles.

Chapter 3: Materials

This study utilized two sources of RCA from D-cracked pavements to test freeze-thaw durability. The first RCA source was taken from Topeka Boulevard in Topeka, Kansas. The second source was taken from runway reconstruction at the Kansas City International Airport in Kansas City, Missouri. Information about the source of the Topeka aggregate was unknown; however, the airport aggregates, quarried from the Martin Marietta Sunflower Quarry, consisted of limestone (calcium carbonate), minor clay minerals (aluminum silicates), and possible minor dolomite (magnesium carbonate). The two RCA sources were categorized as coarse and fine aggregates based on the washed sieve analysis according to the Kansas test method KT-2 (2016). In addition to the RCA, virgin coarse aggregate and fine aggregate were obtained as control samples to compare specimen performance. Midwest Concrete Materials (MCM), a local supplier, provided the virgin coarse aggregate. Aggregate sources were first subjected to the Los Angeles abrasion test following ASTM C131 (2014) to ensure suitability for pavement applications. Then test method KTMR-21 (1999) was used to determine the freeze-thaw durability of the base aggregates. Table 3.1 shows the sieve analysis for the RCA and virgin aggregates in control PCTB.

Table 3.1: Sieve Analysis of All Aggregates

Aggregate Source	1 in.	3/4 in.	1/2 in.	3/8 in.	#4	#8	#40	#100	#200
KC	0	6.2	27.4	40.9	62.2	72.7	94.4	94.5	100
Topeka	0	6.2	21.6	35.1	56.8	68.3	92.4	97.2	100
CS-2	0	0	0	0.2	0.5	32.9	65.3	90.8	100
CA-5	0	4.9	42.7	67.4	95.6	98.8	98.8	98.8	100

3.1 Coarse Aggregate

Aggregate properties such as Saturated Surface Dry (SSD) bulk specific gravity and absorption were obtained with ASTM C127 (2015) for preliminary analysis of aggregate types in the RCA. The aggregate was immersed in water for 24 hours to saturate the pores; then the coarse aggregates were towel-dried and weighed in SSD condition to obtain the aggregate's apparent mass in water. Then, the mass was recorded after oven-drying for 24 hours. SSD bulk specific gravity was determined by comparing the SSD and apparent masses, and absorption was found by comparing the SSD and oven-dried masses (ASTM C127, 2015). Aggregate properties for the coarse aggregate in the RCA are displayed in Table 3.2.

Table 3.2: Coarse Aggregate Properties

Aggregate Source	SSD Bulk Specific Gravity	Absorption (%)	LA Abrasion (%)	Soundness (KTMR-21)
Topeka RCA	2.39	5.71	32.0	0.96
Kansas City RCA	2.52	2.89	52.0	0.94
Midwest Concrete Materials-Coarse Aggregate (CA-5)	2.61	2.86	33.7	0.97

3.2 Fine Aggregate

The properties of the fine aggregate in the RCA were determined using the gravimetric procedure outlined in ASTM C128 (2015). The fine aggregate was immersed in water for 24 hours and then dried to SSD condition. Surface moisture was tested by placing fine aggregates in a cone-shaped mold with two open ends. A tamper was used to consolidate the aggregate, which was considered SSD when it deformed slightly upon mold removal. Table 3.3 shows the properties of the fine aggregates.

Table 3.3: Fine Aggregate Properties

Aggregate Source	SSD Bulk Specific Gravity	Absorption (%)
Topeka RCA	2.02	5.71
Kansas City RCA	2.45	2.89
KDOT Fine Aggregate (CS-2)	2.51	3.69

3.3 Cement

Type I/II cement from Central Plains Cement Company was used in all PCTB mixes for tests with KTMR-22 specifications. The average chemical and physical compositions of the cement are shown in Table 3.4.

Table 3.4: Mill Report Results for Cement in the Study

Property		Spec Limit	Reported Value
SiO_2 (%)		None	20.2
Al_2O_3 (%)		6.0 max	4.7
Fe_2O_3 (%)		6.0 max	3.1
CaO (%)		None	64.0
MgO (%)		6.0 max	1.2
SO_3 (%)		3.0 max	3.0
Loss on Ignition (%)		3.5 max	2.6
Insoluble Residue (%)		1.5 max	0.27
CO_2 (%)		None	1.8
Limestone (%)		5.0 max	4.5
$CaCO_3$ in Limestone (%)		70 min	93
Adjusted Potential Phase Compositions (C150)	C_3S (%)	None	55
	C_2S (%)	None	17
	C_3A (%)	Eight max	7
	C_4AF (%)	None	9
	$C_3S+4.75C_3A$ (%)	100 max	90
Air Content of Mortar (%)		12 max	8
Blaine Fineness (m^2/kg)		260–430	361
-325 (%)		None	95.8
Autoclave Expansion (%)		0.80 max	-.01
Compressive Strength	One day	None	2140
	Three days	1740 min	3800
	Seven days	2760 min	5060
	28 days	None	7050
Time of Setting (minutes)		45–375	97
Mortar Bar Expansion (%)		0.020 max	0.005
Specific Gravity		None	3.15

3.4 Fly Ash

Class C fly ash from Kansas City Fly Ash, LLC, was used as a pozzolan for PCTB mixtures designed for fly ash and cement because Class C has cementitious properties and may enhance the 7-day compressive strength of the specimens. The average chemical and physical compositions of the fly ash used for the study are shown in Table 3.5.

Table 3.5: Mill Report Results for Fly Ash in this Study

Property	Value	ASTM C 618 Class C	AASHTO M295 Class C
SiO2 (%)	35.70	None	None
Al2O3 (%)	18.90	None	None
Fe2O3 (%)	6.12	None	None
SiO2+Al2O3+Fe2O3 (%)	60.72	50 min	50 min
CaO (%)	26.24	None	None
MgO (%)	5.14	None	None
SO3 (%)	1.93	5.0 max	5.0 max
Moisture Content (%)	0.06	3.0 max	3.0 max
Loss on Ignition (%)	0.43	6.0 max	5.0 max
Na2O (%)	1.47	None	None
K2O (%)	0.47	None	None
325 Sieve, % Passing	8.2	34 max	34 max
Density	2.61	None	None
Strength Activity Index with Portland Cement at 7 Days (%)	103	None	None
Water Requirement (%)	94	105 max	105 max
Autoclave Expansion (%)	0.07	0.8 max	0.8 max

Chapter 4: Methods

This chapter describes the methodologies followed in this research, specifically all laboratory tests.

4.1 PCTB Mix Design

All PCTB mixes in this study were based on the optimum moisture content and density requirements outlined in KT-37. A minimum of four points were needed to determine the optimum moisture for each blend of PCTB. The 7-day unconfined compressive strength (UCS) of the PCTB specimens was expected to be 650–1,600 psi (KT-37, 2016). Proportions for each mixture are summarized in Table 4.1. A pozzolan in the form of Class C fly ash was utilized to increase the paste content of the samples without significantly increasing the compressive strength. Cementitious material content has been shown to highly influence specimen performance in freeze-thaw tests (Ashraf et al., 2018).

Table 4.1: Blend Proportions for Cement-Treated Base Mixtures

PCTB Blend	RCA (%)	Cement (%)	Class C Fly Ash (%)
Topeka 100% Cement	93	7	-
Topeka 50% Cement 50% Fly Ash	91	9	-
Topeka 35% Cement 65% Fly Ash	90	5	5
Kansas City 100% Cement	88	6	6
Kansas City 50% Cement, 50% Fly Ash	86	4.9	9.1
Kansas City 35% Cement 65% Fly Ash	86	4.9	9.1

4.2 Moisture-Density Curve

Optimum moisture content was determined using the procedure outlined in KT-37 (2016). First, the point on a moisture-density curve corresponding to the maximum dry density was determined; the curve outlines the point at which the optimum moisture content and the highest density are highest for the mixture. Then at least five samples weighing 15.5 lb each were obtained for testing. Finally, all samples were compacted in 16 inch × 6 inch Humbolt molds, using three lifts with 56 blows of a standard Proctor hammer distributed uniformly over the layer surface. As per KT-37, a slump of less than 1 inch was used. Samples initially contained 5% moisture, and 2%–3% moisture was added for subsequent samples. A polynomial fit was done, and the optimum moisture was determined. Figure 4.1 shows the moisture-density curve for the mixture with the Kansas City (KC) aggregates and 100% cement.

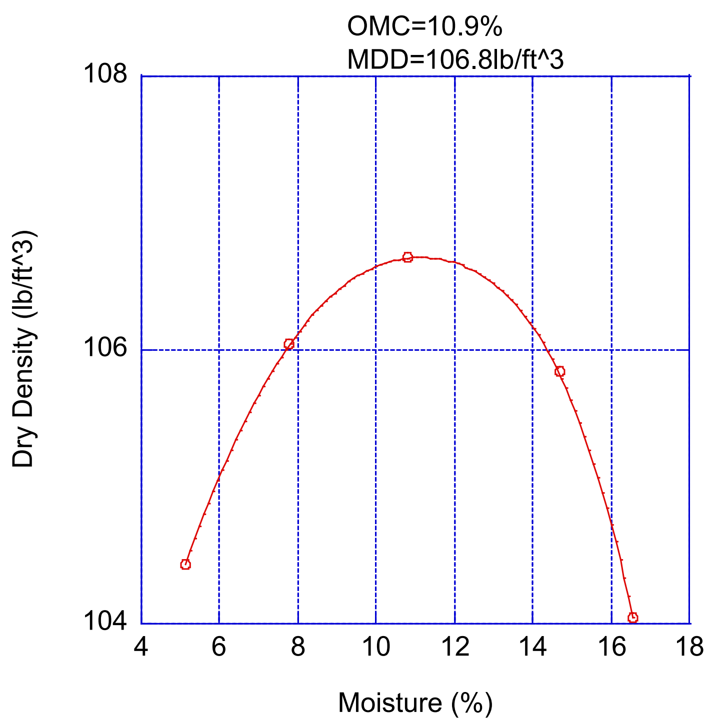

Figure 4.1: Moisture Density Curve for KC Aggregate Mixture with 100% Cement

Moisture density curve tests were conducted for all RCA and control blends of PCTB. The RCA blends had negligible differences in optimum moisture, but the control blends did vary in optimum moisture (about 1%).

4.3 Compression Testing

A cement-treated base mix must be tested for UCS. KDOT requires a minimum of 650 psi for 7-day UCS and a maximum of 1,600 psi (KDOT, 2015). A review of PCTB mix designs used by KDOT showed an average 7-day UCS of 950 psi. Thus, this study aimed to reach a minimum strength near the KDOT average. Figure 4.2 shows an example of a KDOT blend for PCTB. At least three samples were prepared for each PCTB mixture. Table 4.2 summarizes the final mixture proportions and the UCS determined from testing. This study utilized a Humboldt-calibrated cylindrical mold of predetermined volume measuring approximately 6 inches in diameter and 6 inches in height and equipped with a removable base plate. Because the samples were mixed at optimum moisture, the slump was negligible. Therefore, a 5.5-lb compaction hammer was used to compact each layer with 25 blows distributed uniformly over the surface. Materials for three layers of approximately the same height were placed and compacted similarly.

DTMT259 STATE OF KANSAS - DEPARTMENT OF TRANSPORTATION PAGE: 1

DATE: 01/30/17 TIME: 08:33 AM LIST OF CEMENT TREATED BASE MIX DESIGNS

DISTRICT : 1

CTB MIX #: 1CES220A E MATERIAL CODE: 260200011 NAME: CTB (MODIFIED) LEAN

% CEMENT/FLY ASH: 5.8 % MOIST: 11.4 EFFECTIVE DATE: 05/01/14 TERMINATION DATE: 01/01/20

AGGR	NAME	PROD #	PROD NAME	% BLEND
005010017	AGGR/PCTB(LIMESTONE)	00800977	HAMM QUARRY (044)	100.0
				0.0
				0.0

CEMENT	NAME	PROD #	PROD NAME	% BLEND
161060100	CEMENT TY1/2(MH) BULK	00007603	CENTRAL PLAINS (MO)	100.0
				0.0

ADDITIVE	NAME	PROD #	PROD NAME	% BLEND
				0.00
				0.00

SIEVE	1.5	1	3/4	1/2	#4	#8	#40	#100	#200
SINGLE PT.	0.0	2.0	7.0	0.0	54.0	66.0	81.0	0.0	86.0
TOLERANCE	0.0	2.0	10.0	0.0	10.0	10.0	10.0	0.0	5.0
TEST GRADATION	0	2	7	23	54	66	81	85	86.3

COMPRESSIVE STRENGTH	A	B	C	AVG
	750	732	716	733

REMARKS: EMERY-SAPP PUGMILL @ MENOKEN RD., TOPEKA.

Figure 4.2: KDOT PCTB Blend

Table 4.2: Design Binder Content and Compressive Strength for PCTB

PCTB Blend	Binder Content (%)	Average 7-Day UCS (psi)	Std. Deviation of UCS (psi)
Topeka 100% Cement	7	870	44.5
KC 100% Cement	9	928	85.6
Topeka 50% Cement, 50% Fly Ash	10	931	65.0
KC 50% Cement 50% Fly Ash	12	980	88.3
Topeka 35% Cement 65% Fly Ash	14	931	85.0
KC 35% Cement 65% Fly Ash	14	877	3.5

4.4 PCTB Batching

Materials were prepared in accordance with KTMR-22 before the mixtures were batched. Materials were batched in a 2 cubic foot pan mixer using the procedure provided in ASTM C192 (2015):

1. Place coarse aggregates in the mixer;
2. Add approximately half of the mixing water;
3. Start mixer;
4. Add fine aggregates;
5. Add cement;
6. Add the remaining half of the mixing water;
7. Start the timer and mix for 3 min with the lid open;
8. Stop the mixer and let concrete sit still for 3 min with the lid closed; and
9. Start the mixer and mix for an additional 2 min with the lid open.

4.5 Preparation of PCTB Prisms

Concrete prisms with dimensions of 3 × 4 × 16 inches were made after batching. As the cement base was mixed with optimum moisture, the mixture became very stiff, preventing rodding or vibrating as a compaction method. Approximately half the prism height was filled with a new cement base and compacted with the proctor hammer 25 times. All faces of the prism mold were struck with a mallet several times before spading the PCTB along the prism edges. This process was repeated after filling the second half of the mold. A wooden trowel was used to smooth the exposed face of the prism. The KTMR-22 curing method was used for RCA and control/virgin aggregates. As mentioned, KTMR-22 (2015) outlines KDOT's standard 90-day curing procedure for samples subjected to freezing and thawing. These samples were placed in a 100% moist room for 67 days. Then they were transferred to a room with approximately 50% relative humidity for 21 days before two consecutive 24-hour soaking periods in 70 °F and 40 °F water, respectively. Ice was added to the soaking tank, and a temperature gauge was used to maintain a water temperature of 40 °F.

4.6 ASTM C666 Testing

Freeze-thaw testing was conducted in accordance with the ASTM C666 Procedure B (2015). A freeze-thaw machine manufactured by Scientemp Corporation was used to automatically cycle concrete specimens through temperatures specified by ASTM C666. The chamber in this machine has a slot capacity of 20 substantial prisms. Two of the prism slots contained control prisms to monitor internal concrete temperatures via thermocouple wires, and test specimens occupied the remaining 18 slots. The freeze-thaw machine was programmed to complete one cycle every three hours. Specimen temperatures were 40 ± 3 °F at the beginning of each cycle, and the specimens were subjected to freezing in the air for 110 minutes until they reached 0 ± 3 °F. The chamber was then filled with tempered water, which allowed the samples to thaw.

Mass, resonant frequency, and expansion readings were recorded for all prisms before the beginning of the first freeze-thaw cycle. These values were recorded in intervals of 30–36 cycles. Testing continued until samples completed 660 cycles, as specified by KTMR-22 (2015), or until the samples became too deteriorated to obtain meaningful measurements.

4.6.1 PCTB Prism Sample Mass Measurements

In compliance with ASTM C666 requirements, PCTB prism sample mass was recorded using a scale with a capacity of 30 kg. ASTM C666 requires that the mass of the specimens be at most 50% of the scale's capacity. The mass of each specimen ranged from 6 to 8 kg. A towel was used to remove surface water from the prisms to maintain consistent conditions when recording mass. Change in mass was calculated using Equation 4.1:

$$Change\ in\ Mass\ (\%) = \left(\frac{m_x - m_o}{m_o}\right) * 100$$

Equation 4.1

Where:

m_x = mass at freeze-thaw cycle *x* (kg), and

m_o = initial mass (kg).

4.6.2 Resonant Frequency Measurements

The transverse resonant frequency was obtained using a James E-Meter™ Mk II equipped with an impactor and an accelerometer, as shown in Figure 4.3.

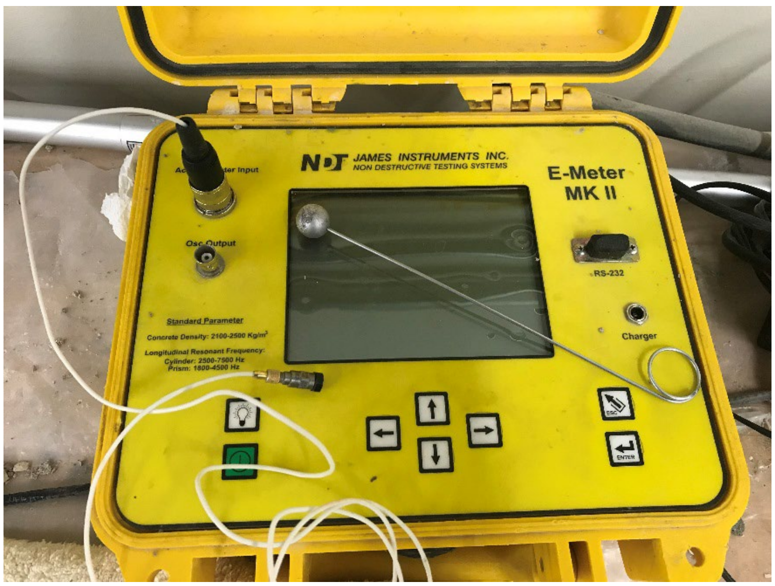

Figure 4.3: James E-Meter Mk II with Accelerometer and Impactor

The prism transverse resonant frequency was obtained following a modified method of ASTM C215 as specified in KTMR-22. Prism dimensions and mass were entered into the meter. The accelerometer was placed 25 mm away from the end of the prism, and then the impactor was used to strike the prism 25 mm from the opposite end. The meter then computed the frequency. Equation 4.2 shows how the transverse frequency values were used to calculate the RDME of each sample:

$$RDME\ (\%) = \frac{n_x^2}{n_o^2} * 100$$

Equation 4.2

Where:

n_x = transverse frequency at freeze-thaw cycle *x* (Hz), and

n_o = initial transverse frequency (Hz) (before freeze-thaw cycling starts).

Figures 4.4 and 4.5 depict the transverse frequency waves as those propagated through the specimen. The wave was a sinusoidal decay wave, so the wave amplitude became smaller and smaller as the specimen deteriorated, as shown in Figure 4.5.

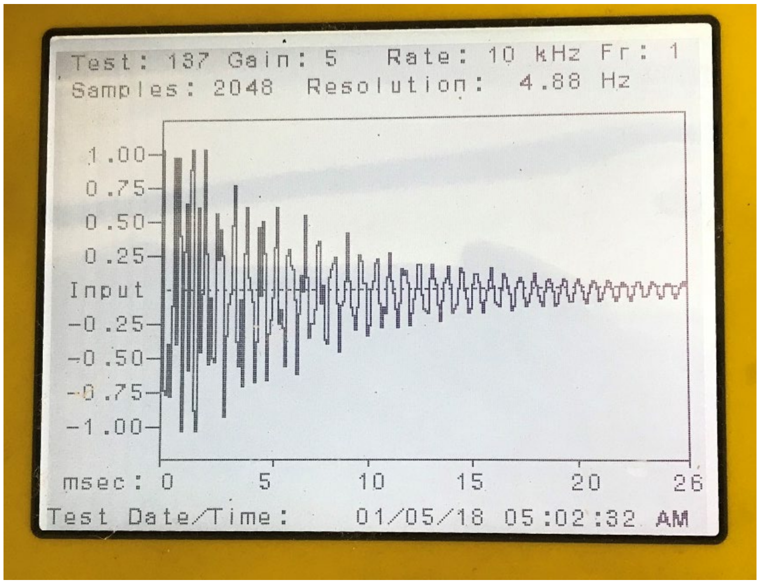

Figure 4.4: Transverse Frequency Wave of Sample Before Testing

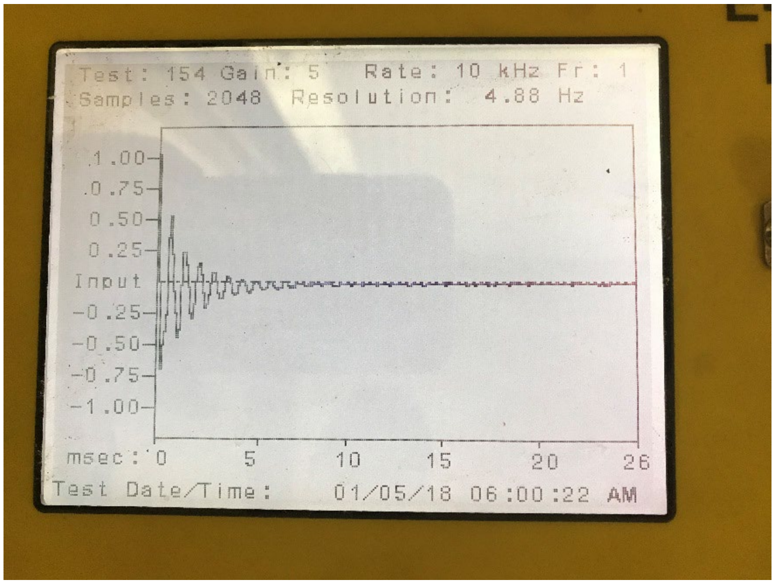

Figure 4.5: Transverse Frequency Wave of Sample Near Failure

Once the data were accepted, an image showing the transverse frequency appeared. The most significant frequency was the recorded value. Figure 4.6 shows a tested sample and recorded frequency.

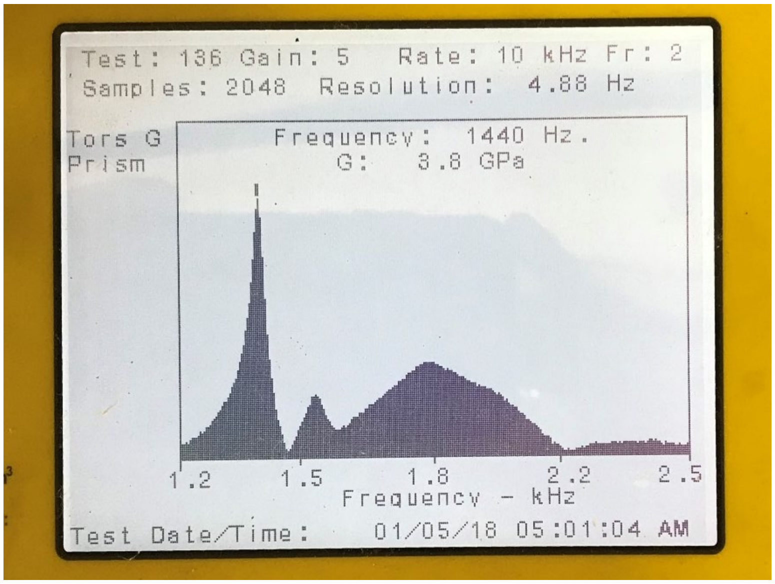

Figure 4.6: Typical Sample Transverse Frequency

4.6.3 Expansion Measurements

Steel gage studs were installed at both ends of the prisms during mixing to measure the length change, or expansion, of the prisms. The molds allowed the studs to remain constant during curing. Each prism's initial length was measured before the first freeze-thaw cycle. The difference in size between each prism and a reference was compared by placing the reference bar in a length comparator equipped with a digital indicator. The indicator was set to zero to establish a reference point from which the prism length could be measured. The reference bar was then removed and replaced by the prism. The precision of the digital indicator was 0.00001 inches. Change in size was monitored continuously throughout testing and calculated using Equation 4.3:

$$Length\ change(\%) = \left(\frac{l_x - l_o}{l_o}\right) * 100$$

Equation 4.3

Where:

l_x = indicator reading at freeze-thaw cycle *x* (inch),

l_o = initial indicator reading (inch), and

l_i = initial prism length (inch).

Chapter 5: D-Cracked RCA PCTB Results

Mass change, RDME, and length change/expansion were measured for each sample until 660 freeze-thaw cycles or until excessive deterioration prevented the recording of such measurements. Due to the decreased quality of D-cracked aggregates, none of the samples reach the minimum 660 cycles specified by KDOT. The mass change, RDME, and length change during testing for each blend of PCTB are presented in Appendix A.

5.1 Change in Mass

A summary of the change in mass for all samples at their terminal freeze-thaw cycle is shown in Table 5.1. In the first column, "Topeka" and "KC" denote the aggregate source. The percentages indicate the binder blend within the PCTB mixture. For example, Topeka 50% Cement and 50% Fly Ash means that the RCA source was from Topeka and the total binder content consists of 50% cement and 50% fly ash.

Table 5.1: Summary of Mass Change Results for Concrete Mixtures with D-Cracked RCA

PCTB Blend	Average Final Mass Change (%)	Std. Dev. of Mass Change (%)
Topeka 100% Cement	0.70	0.4
Topeka 50% Cement 50% Fly Ash	10.7	6.5
Topeka 35% Cement 65% Fly Ash	6.64	8.4
KC 100% Cement	6.71	4.0
KC 50% Cement 50% Fly Ash	15.6	5.1
KC 35% Cement 65% Fly Ash	12.8	6.1

5.2 Relative Dynamic Modulus of Elasticity

The dynamic modulus of elasticity, the proportion of stress to strain when the stress is least under dynamic loads, can be measured via longitudinal or flexural vibration. This modulus signifies the elasticity performance of the material, similar to the initial tangent modulus under static loads. The decrease in the dynamic modulus of elasticity with freeze-thaw cycles indicates the loss of elasticity performance (Shang & Yi, 2013). Table 5.2 summarizes the RDME results.

Table 5.2: Summary RDME Results for Concrete Mixtures with D-Cracked RCA

PCTB Blend	Average Final RDME (%)	Std. Dev. of RDME (%)	Average No. of Cycles	Std. Dev. of Average No. of Cycles
Topeka 100% Cement	52.3	6.7	111	23
Topeka 50% Cement 50% Fly Ash	44.8	19.1	200	44
Topeka 35% Cement 65% Fly Ash	45.8	11.6	285	10
KC 100% Cement	43.6	17.0	35	7
KC 50% Cement 50% Fly Ash	51.2	8.5	169	43
KC 35% Cement 65% Fly Ash	50.9	8.1	214	12

5.3 Expansion

Measurement of length change permits assessing the potential for volumetric expansion or contraction of existing mortar or concrete due to causes other than applied force or temperature change. Table 5.3 summarizes the expansion results.

Table 5.3: Summary of Length Change Results for Concrete Mixtures with D-Cracked RCA

PCTB Blend	Average Final Expansion (%)	Std. Dev. of Expansion (%)
Topeka 100% Cement	0.15	0.066
Topeka 50% Cement 50% Fly Ash	0.28	0.55
Topeka 35% Cement 65% Fly Ash	0.17	0.091
KC 100% Cement	0.39	0.28
KC 50% Cement 50% Fly Ash	0.36	0.20
KC 35% Cement 65% Fly Ash	0.08	0.048

However, results for the final average expansion may not entirely represent the expansion behavior of the PCTB mixtures with RCA because, during testing, the most significant mass loss occurred on the ends of the prism, causing the length-change studs to fall out of the specimens. These studs could not be replaced. Blends with 35% cement and 65% fly ash experienced less stud loss than the other two blends with less binder. Because the expansion measurement is an optional test within ASTM C666 (2015), test results could be solely based on mass change and RDME.

Chapter 6: Control PCTB Mixture Results

An aggregate coarse and fine aggregate blend was determined for the control samples based on the washed sieve analysis of the RCA and virgin aggregates. The mix proportion for the Topeka control samples was 52% coarse aggregate and 48% fine aggregate, and the mix proportion for the KC control samples was 65% coarse aggregate and 35% fine aggregate. Both curves were plotted on the same graph, and the blend that best fits the curve was used as the control blend. Figures 6.1 and 6.2 show the gradation for the Topeka and KC control samples, respectively.

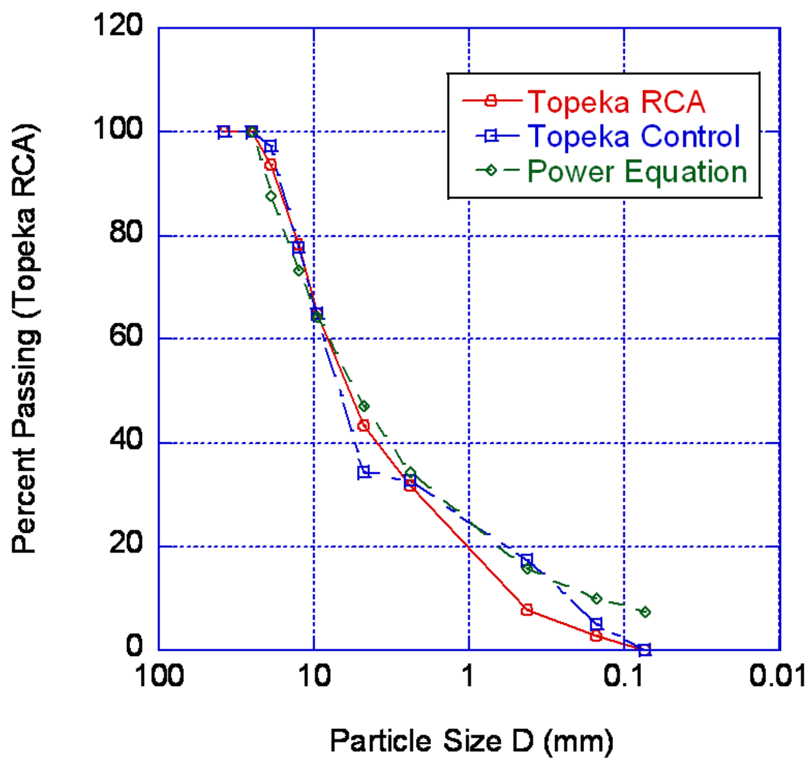

Figure 6.1: Topeka Control Gradation

The power equation in Equation 6.1 represents the 0.45 power maximum density curve:

$$p = \left(\frac{d}{D}\right)^{0.45}$$

Equation 6.1

Where:

p = % finer than the sieve,

d = aggregate size being considered, and

D = maximum aggregate size to be used in the blend.

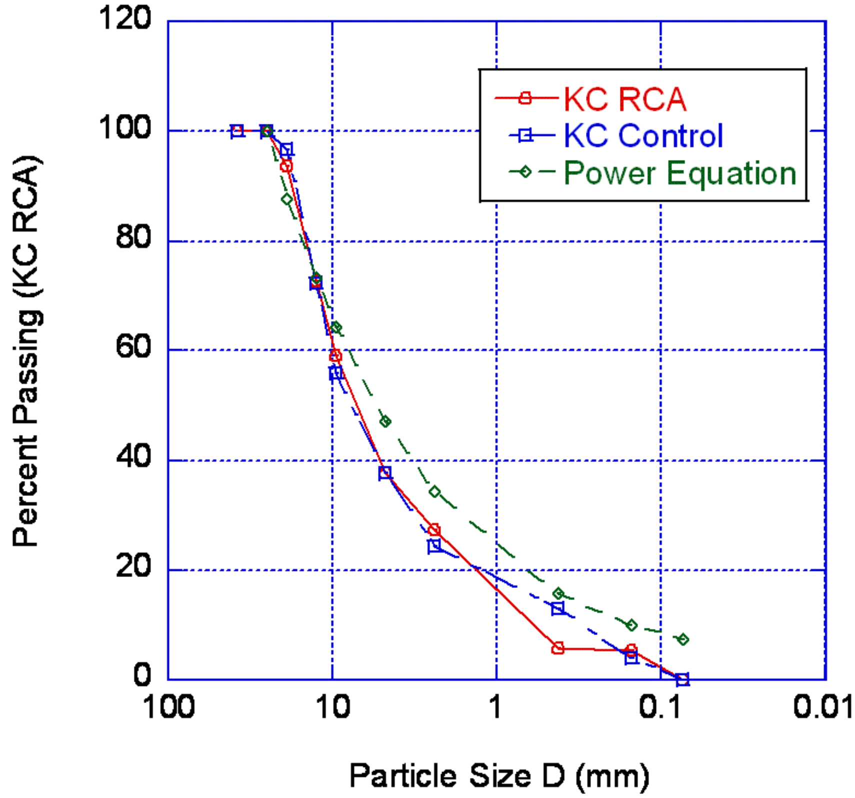

Figure 6.2: KC Control Gradation

The compressive strength of the control samples was determined using the same testing and mixing procedures as in the RCA PCTB to achieve consistency. The results were similar but different. As expected, the control sample compressive strength was more significant at lower percentages of binder, as shown in Table 6.1. However, all control samples had the same or lower binder rates compared to the PCTB mixtures with RCA.

Table 6.1: UCS of Control Samples

PCTB Blend	Binder (%)	Average 7-Day UCS (psi)	Std. Deviation of UCS (psi)
Topeka 100% Cement	7	924	24.5
KC 100% Cement	7	1,017	48.5
Topeka 50% Cement 50% Fly Ash	9	1,155	42.5
KC 50% Cement 50% Fly Ash	9	940	10
Topeka 35% Cement 65% Fly Ash	11	1,087	25.5
KC 35% Cement 65% Fly Ash	11	1,080	55

Mass change, RDME, and length change were measured for each control sample using the procedure described for the RCA mixture samples (Chapter 5) until 660 freeze-thaw cycles or excessive deterioration prevented the recording of such measurements. The control samples did not reach the maximum 660 cycles specified by KDOT for concrete pavements, as was the case for the samples with RCA. However, only the 100% cement and 50% cement 50% fly ash samples were tested, and time constraints resulted in the 35% cement 65% fly ash sample testing being postponed. Representations of mass change, RDME, and length change during testing for each control mixture are presented in Appendix A.

6.1 Mass Change

A summary of the change in mass for all samples at their terminal freeze-thaw cycle is shown in Table 6.2. Mass changes of the control specimens were lower than the PCTB mixtures with RCA, indicating virgin aggregates resulted in lower mass changes.

Table 6.2: Summary of Results of Control Mass Change

Sample Set	Average Final Mass Change (%)	Std. Dev. of Mass Change (%)
Topeka Control 100% Cement	0.3	0.22
Topeka Control 50% Cement 50% Fly Ash	4.3	0.07
KC Control 100% Cement	0.5	0.43
KC Control 50% Cement 50% Fly Ash	5.0	1.3

6.2 Relative Dynamic Modulus of Elasticity

The average final RDME results are shown in Table 6.3. The final RDME values for the control specimens are similar to or lower than the PCTB mixtures with RCA depending on the cementitious material contents.

Table 6.3: Summary of Results of Control RDME

Sample Set	Average Final RDME (%)	Std. Dev. of RDME (%)	Final Cycle	Std. Dev. of RDME
Topeka Control 100% Cement	38.0	16.7	40	6
Topeka Control 50% Cement 50% Fly Ash	56.5	1.3	196	18
KC Control 100% Cement	41.1	18.7	44	6
KC Control 50% Cement 50% Fly Ash	55.3	2.6	166	17

6.3 Expansion

The average expansion results are shown in Table 6.4. As shown, the expansion values of control mixtures are lower than the PCTB mixtures with RCA, indicating that virgin aggregates result in decreased expansion.

Table 6.4: Summary of Results of Control Length Change

Sample Set	Average Final Expansion (%)	Std. Dev. of Final Expansion (%)
Topeka Control 100% Cement	0.06	0.003
Topeka Control 50% Cement 50% Fly Ash	0.02	0.009
KC Control 100% Cement	0.05	0.062
KC Control 50% Cement 50% Fly Ash	0.03	0.004

Chapter 7: Analysis and Discussion

This chapter documents a detailed analysis and discussion of the results of the tests outlined in Chapter 6.

7.1 RCA PCTB Mixture

As anticipated, PCTB mixtures with D-cracked aggregates did not reach the 660 cycles specified in KTMR-22. Unlike concrete, hardened PCTB is a very porous material that allows excess water to penetrate the sample and accelerates deterioration during freeze-thaw conditions. Figure 7.1 shows the average cycle in which the samples fell below 60% RDME.

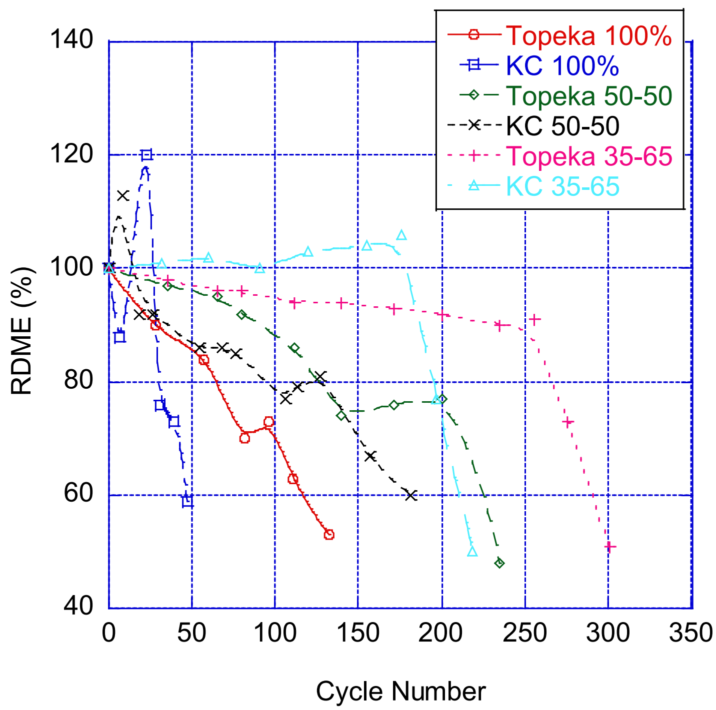

Figure 7.1: Average RCA RDME Trend During Study

Figure 7.1 shows that an increase in cementitious material content (i.e., cement and fly ash) results in increased durability of the samples, potentially due to higher quality calcium silicate hydrates in the hydrated paste in the PCTB. This hydrated paste phase may limit freeze-thaw deterioration. The figure also shows that existing pavement composition significantly impacts sample durability, as evidenced by the unique performances of the Topeka and KC samples. The

KC samples with 100% cement unexpectedly displayed a rapid increase in RDME at the beginning of testing between 10 and 25 cycles. This behavior could be attributed to initial cracks caused by the freeze-thaw cycling, which allowed excess water to penetrate the samples but not escape. This additional mass can cause the RDME to increase as the sample becomes denser, consequently allowing for the wave from the impact of the hammer to propagate faster through the sample. Appendix B shows the change in RDME over testing for each sample.

The Topeka samples displayed their own unique behavior. The 35% cement and 65% fly ash samples appeared to develop a yield point, demonstrated by steady linear performance until a specific number of cycles was reached. After that, the specimens began to deteriorate rapidly. Figure 7.2 depicts the mass change during the study. As noted, an increase in mass was observed at the beginning of freeze-thaw cycling. An increased RDME occurred shortly before the samples rapidly degraded and failed. While this trend was less prevalent in the 35% cement 65% fly ash blend, a noticeable change occurred in the two other blends for the Topeka samples.

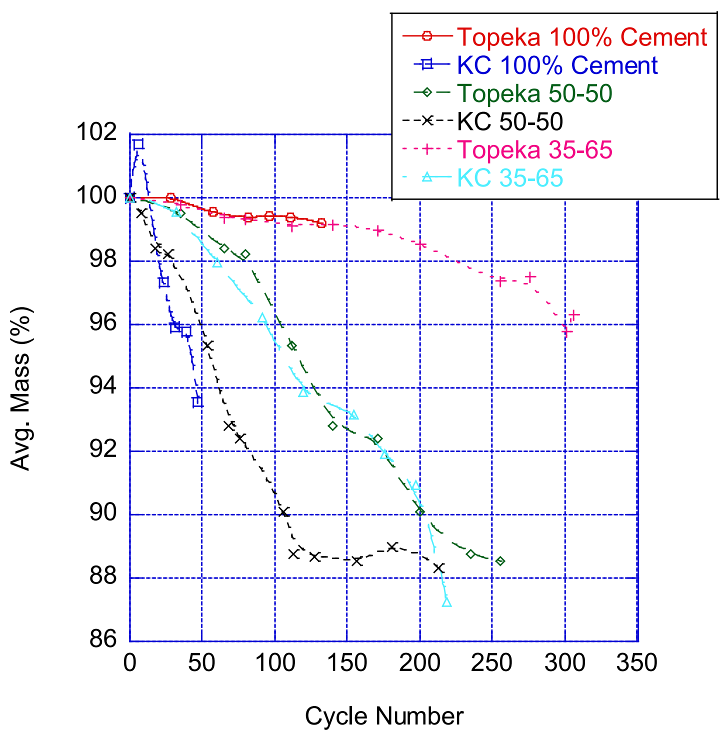

Figure 7.2: RCA Mass Change during Study

The Topeka 100% cement samples significantly outperformed the KC 100% cement samples. The average number of cycles at failure for the Topeka samples was 111 but 35 for the KC samples. It needs to be clarified why the Topeka samples lasted over three times as long as the KC samples. The mass loss for Topeka samples with 100% cement was nearly negligible. The KC samples lost approximately 7% of their mass. The expansion of the Topeka samples was also less than that of the KC samples.

The Topeka samples outperformed the KC samples when comparing the 50% cement and 50% fly ash samples. The average cycle at failure for the Topeka samples was 200, while the average process at loss for the KC samples was 169. Convergence of the performance is likely due to the increased binder content of the samples.

The average final cycles for the Topeka and KC 35% cement 65% fly ash samples were 285 and 214, respectively. As in the previous samples, an increase in total binder content increased the freeze-thaw durability of the samples. As shown in Figure 7.1, the Topeka and KC samples exhibited similar behavior. A linear portion of change was evident in the RDME, followed by a sharp decrease until failure. The assumption was made that higher cementitious material percentages would result in improved freeze-thaw performance.

7.2 Control PCTB Mixture

Similar to the RCA, the control samples did not meet the 660-cycle criteria specified by KDOT. However, except for the Topeka 100% cement samples, the control samples had comparable durability to the PCTB samples. The KC 100% cement control samples had higher durability and less mass loss than the RCA samples, but the 50%-50% blends for the control samples underperformed compared to the RCA samples. The control samples, however, had less binder and failed an average of 3.5 cycles sooner than the RCA samples. As expected, the control samples displayed similar performance, presumably due to their identical coarse and fine aggregates. Figure 7.3 shows the average RDME trend over the life of the control samples, and Figure 7.4 compares the average RDME of the KC control and RCA samples. The RCA samples displayed unpredictable behavior during freeze-thaw testing, whereas the control samples followed a linear trend. Both, however, showed similar durability in terms of the RDME limit. Appendix E includes comparisons of the PCTB with RCA and the control specimens with error bars that the control samples had less variability in behavior and performance than the RCA samples.

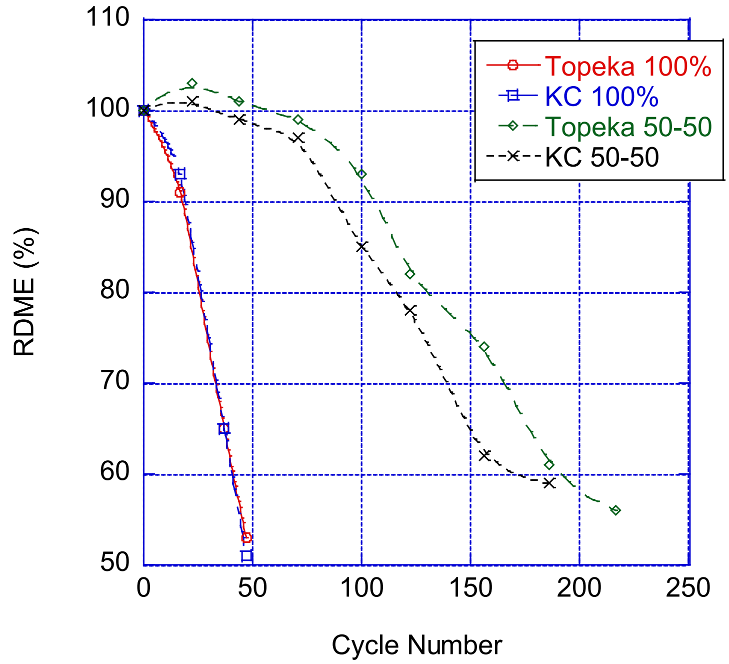

Figure 7.3: Control Sample RDME Trend during Study

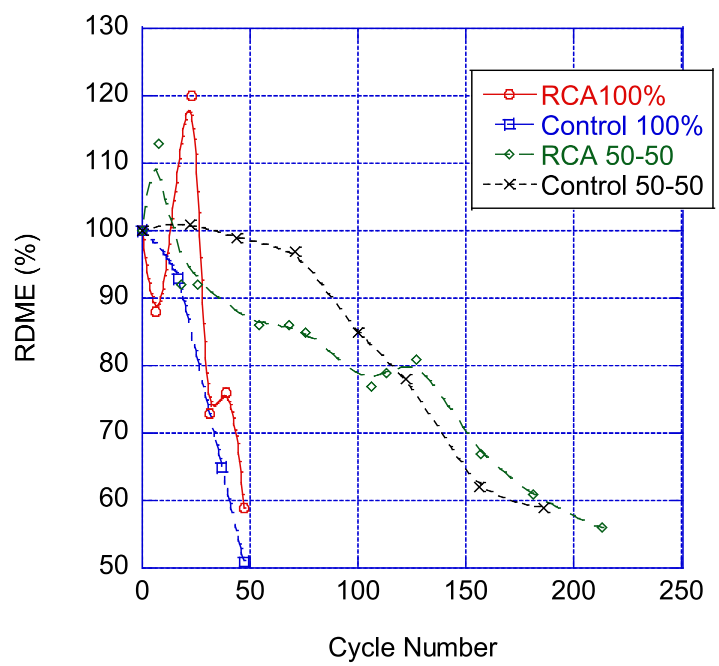

Figure 7.4: KC RCA and Control RDME Trend Comparison

Chapter 8: Conclusions and Recommendations

8.1 Conclusions

Mixtures with increased binder content resulted in higher freeze-thaw testing durability than those with lower binder contents. At lower binder contents, aggregate composition in RCA was shown to be a controlling factor in durability. Increased binder content decreased the influence of the coarse aggregate since the samples were less porous. None of the models reached the 660 cycles during freeze-thaw testing. Because the mixing was conducted at optimum moisture content, the result was a more porous material than used in concrete pavement.

Except for the Topeka 100% cement samples, other control samples demonstrated similar durability. However, testing on the 35% cement and 65% fly ash for both Topeka and KC control samples must be conducted to determine if the trend of comparable durability continues. The correlation between D-cracked RCA and control samples is promising since D-cracked RCA was expected not to perform as well as the control samples. Results showed that total binder percentage is the primary controlling factor in PCTB mixture performance in freeze-thaw conditions.

8.2 Recommendations

Freeze-thaw results for the 35% cement and 65% fly ash are yet to be completed, and these results could verify or disprove the close correlation between the RCA and control samples. Different blends of cement and fly ash, such as 75% cement, 25% fly ash, or 25% cement and 75% fly ash, should be done to determine an optimum blend of cement and fly ash for the freeze-thaw performance of PCTB with RCA. Dry sieving over a US No. 18 (1.00 mm) sieve to remove excess fine aggregate before testing for optimum moisture and compressive strength could also help achieve a more homogeneous gradation for RCA since gradation can vary drastically due to crushing, transportation, and storage of the RCA, as well as settlement of fine aggregates. KDOT currently does not have specifications for the freeze-thaw durability of PCTB. The inability to complete the 660 cycles for the control samples and the observed significant mass loss for the RCA samples prove that shorter cycle criteria and mass loss requirements are needed to define the durability of PCTB more accurately. Testing of more samples is warranted to develop such specifications.

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Appendix A: Moisture Density Curves for RCA Mixtures

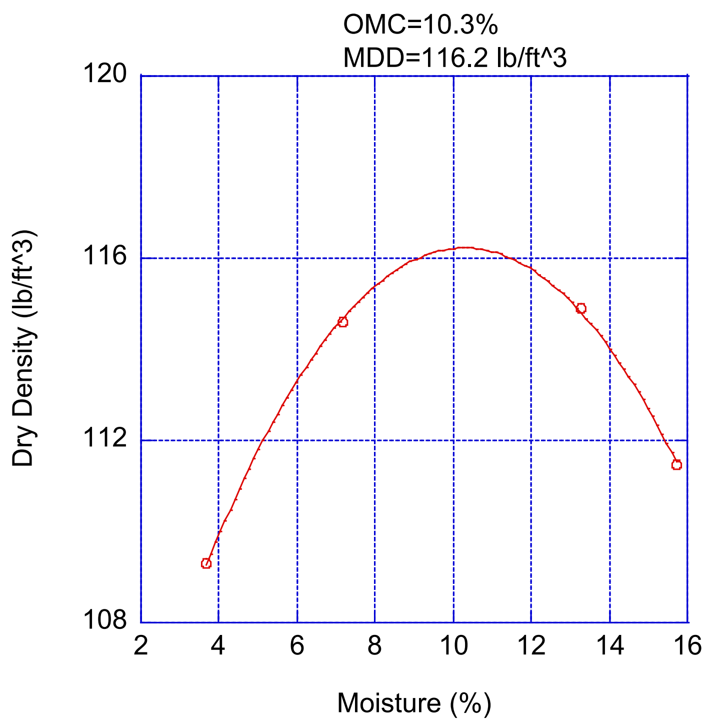

Figure A.1: Topeka 100% Cement Moisture Density Curve

Appendix B: ASTM C666 Results for RCA Mixture

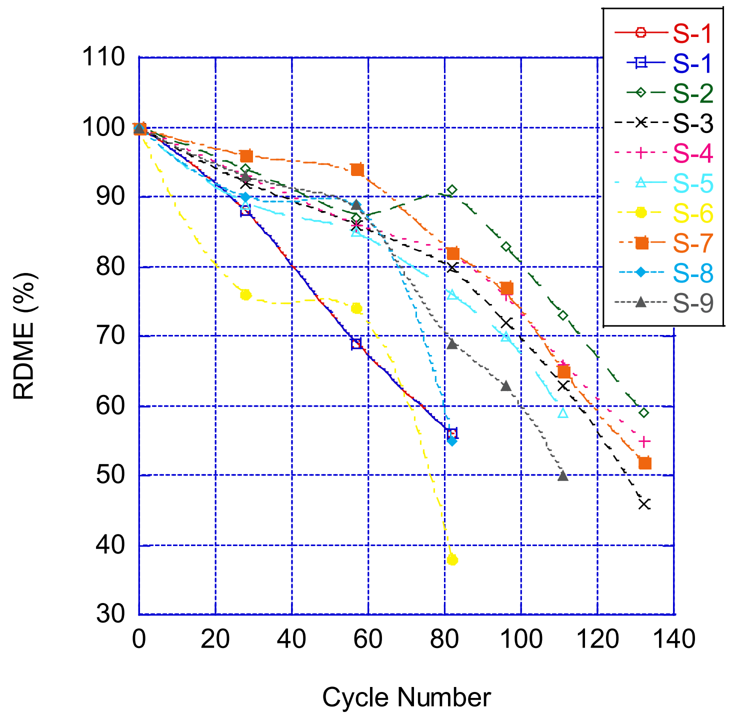

Figure B.1: Topeka 100% Cement Freeze-Thaw Trend

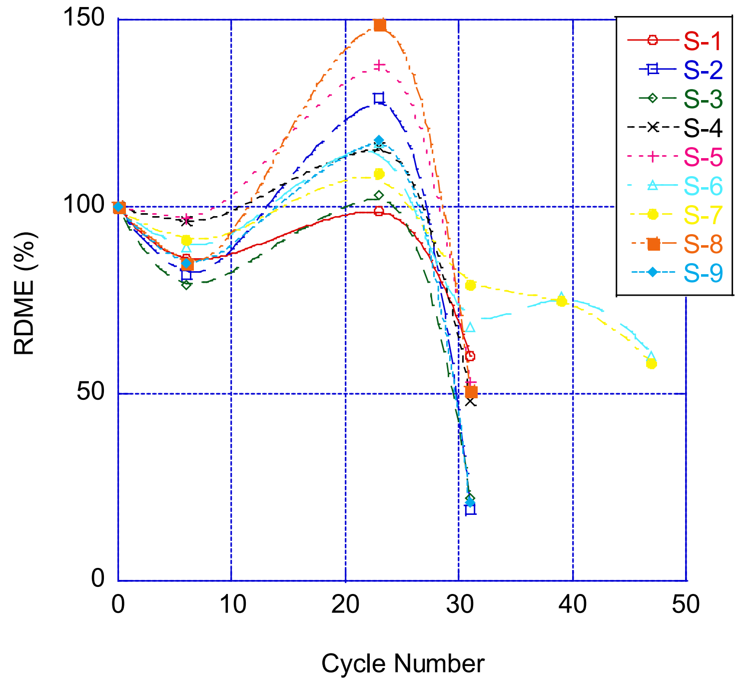

Figure B.2: KC 100% Cement Freeze-Thaw Trend

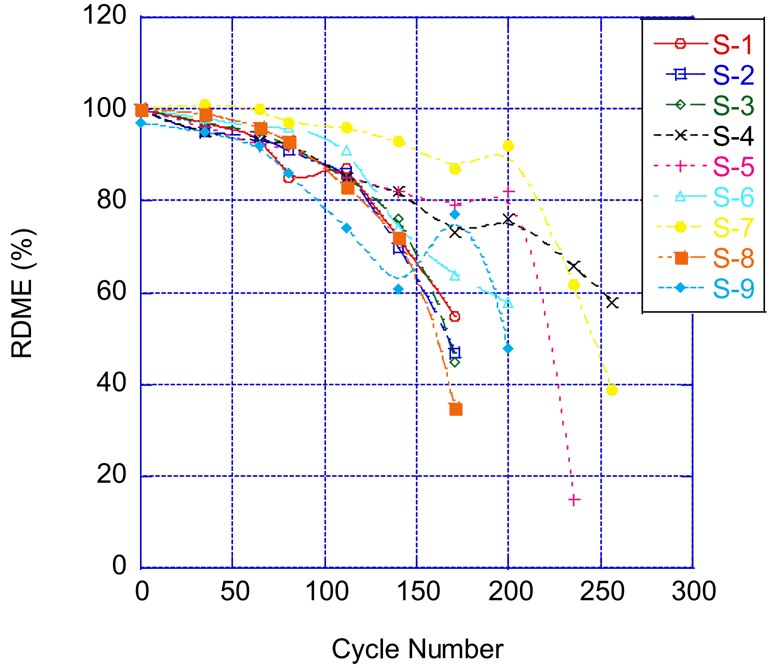

Figure B.3: Topeka 50% Cement 50% Fly Ash Freeze-Thaw Trend

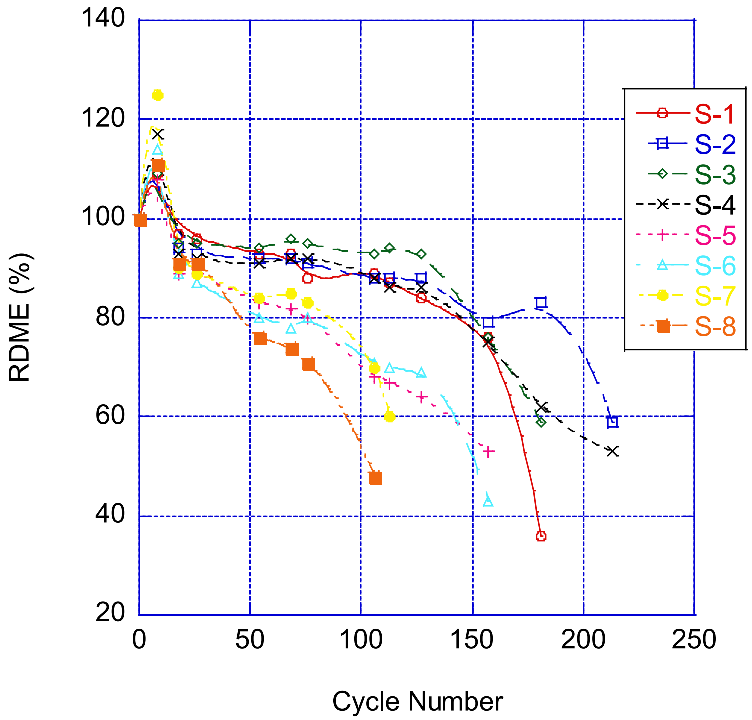

Figure B.4: KC 50% Cement 50% Fly Ash Freeze-Thaw Trend

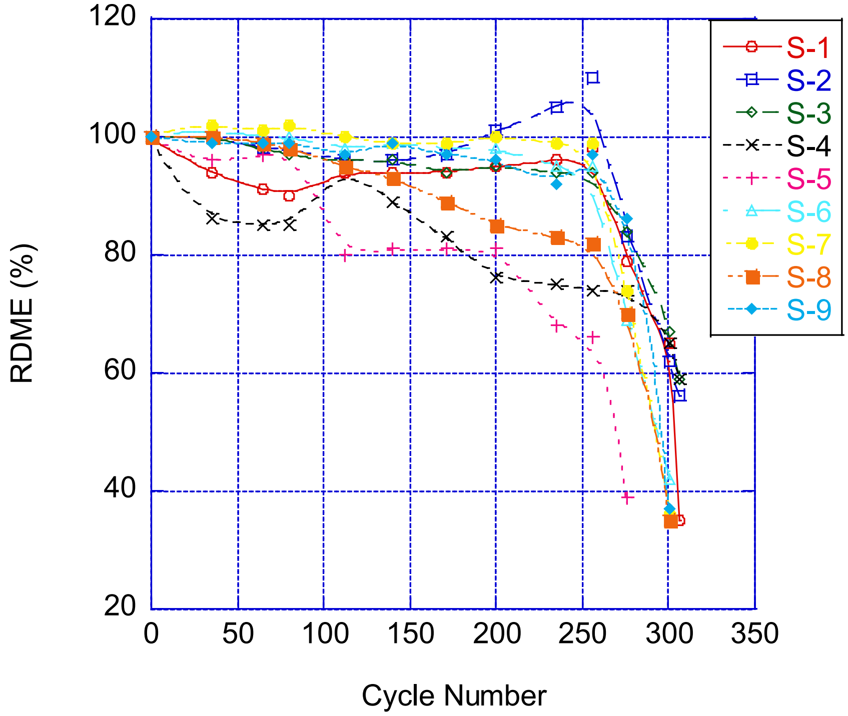

Figure B.5: Topeka 35% Cement 65% Fly Ash Freeze-Thaw Trend

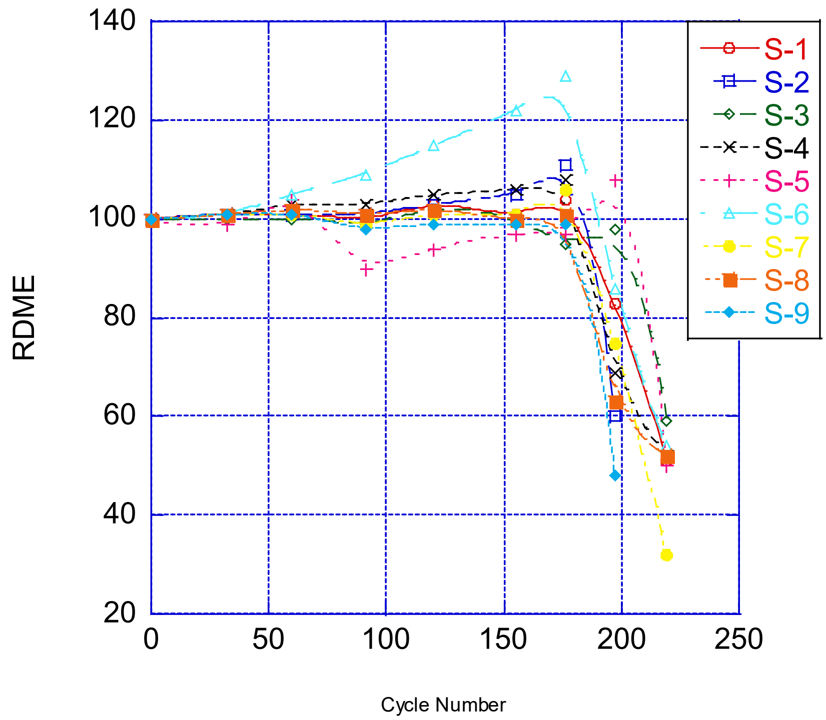

Figure B.6: KC 35% Cement 65% Fly Ash Freeze-Thaw Trend

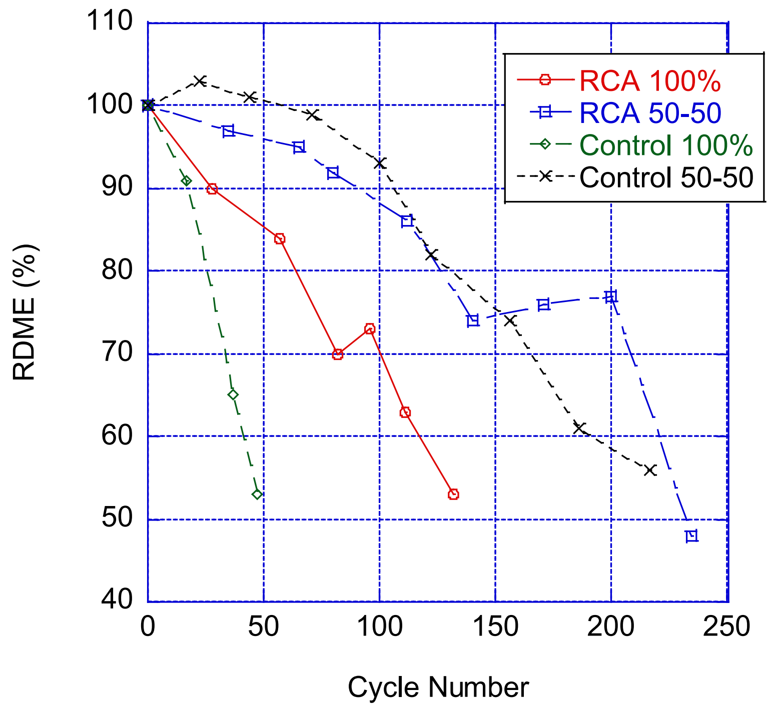

Figure B.7: Topeka RCA and Control Comparison

Appendix C: Control Moisture Density Curves

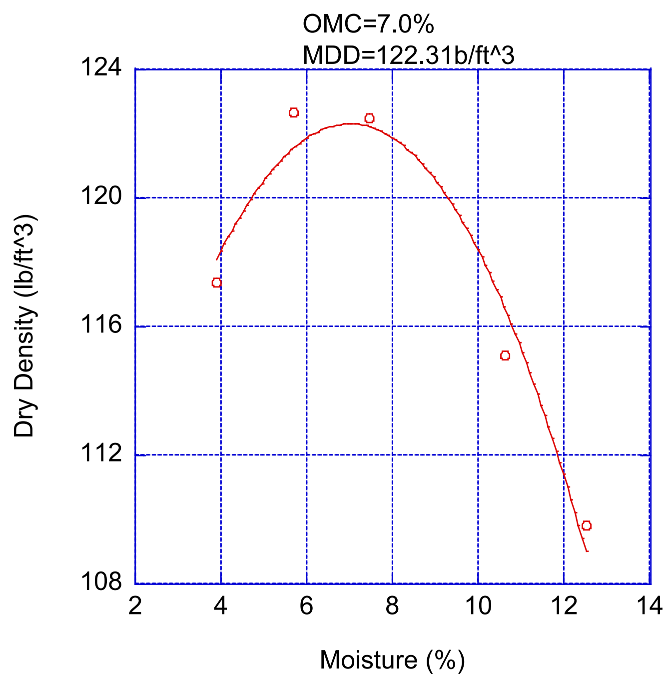

Figure C.1: Topeka Control 100% Cement Moisture Density Curve

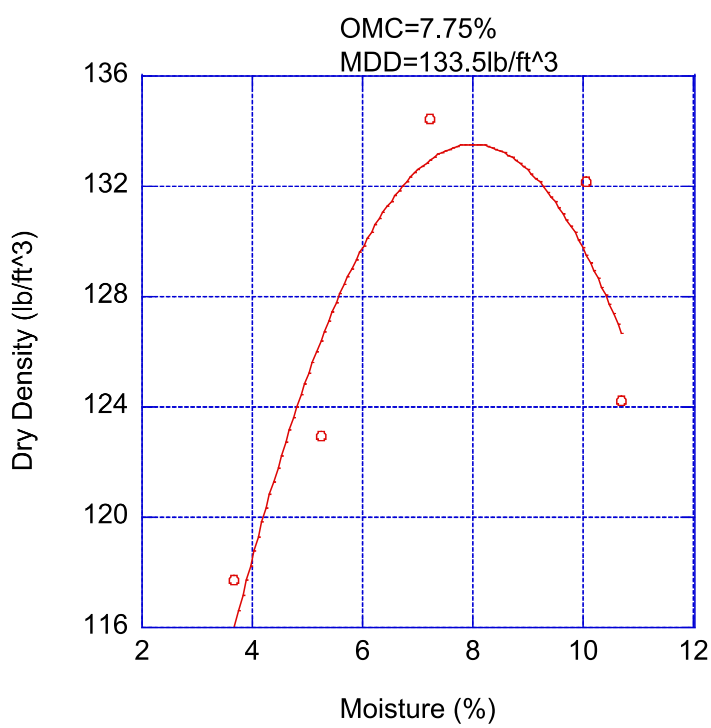

Figure C.2: KC Control 100% Moisture Density Curve

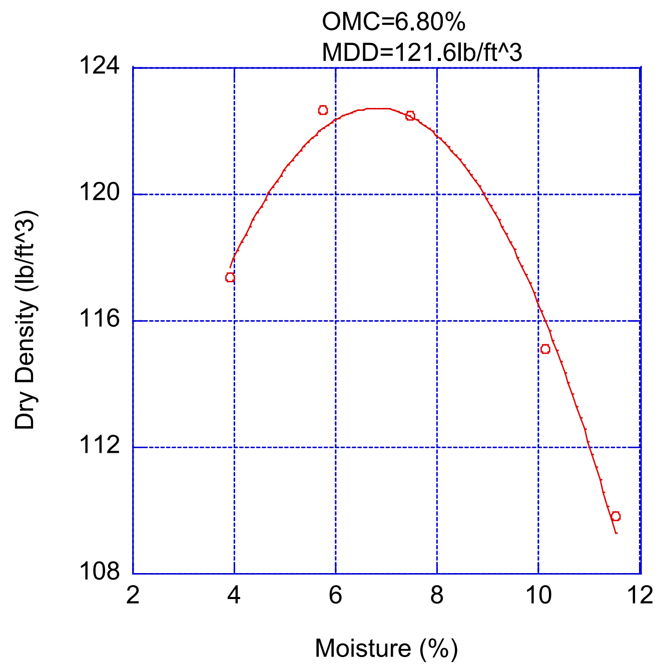

Figure C.3: Topeka Control 50%-50% Moisture Density Curve

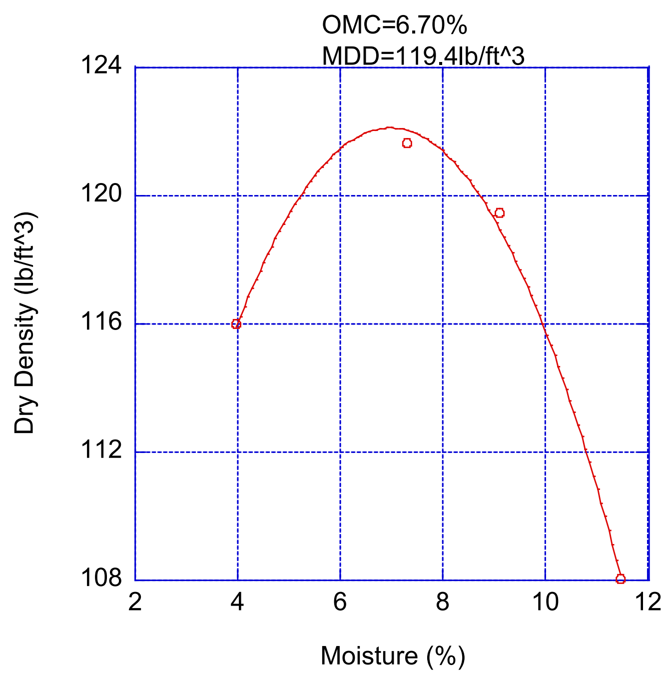

Figure C.4: KC Control 50%-50% Moisture Density Curve

Appendix D: ASTM C666 Results for Control Mixtures

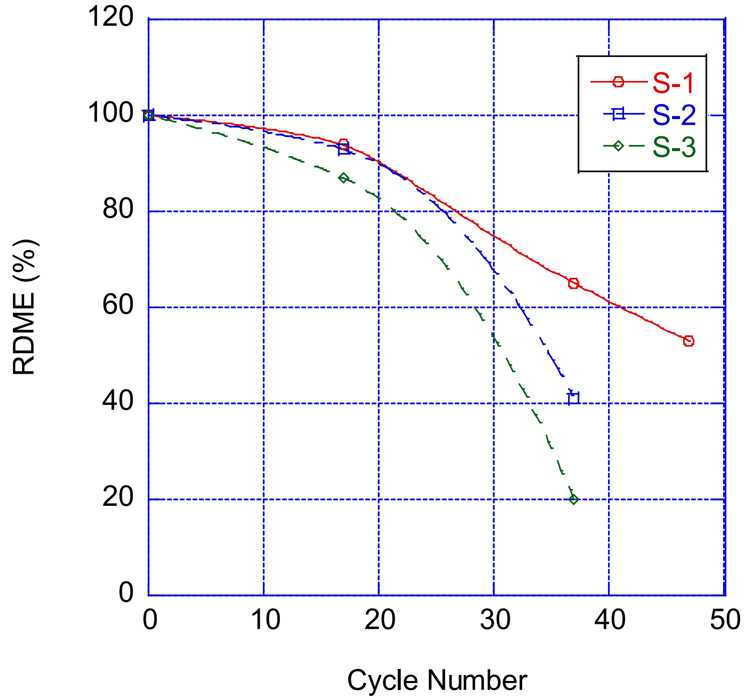

Figure D.1: Topeka Control 100% Cement Freeze-Thaw Trend

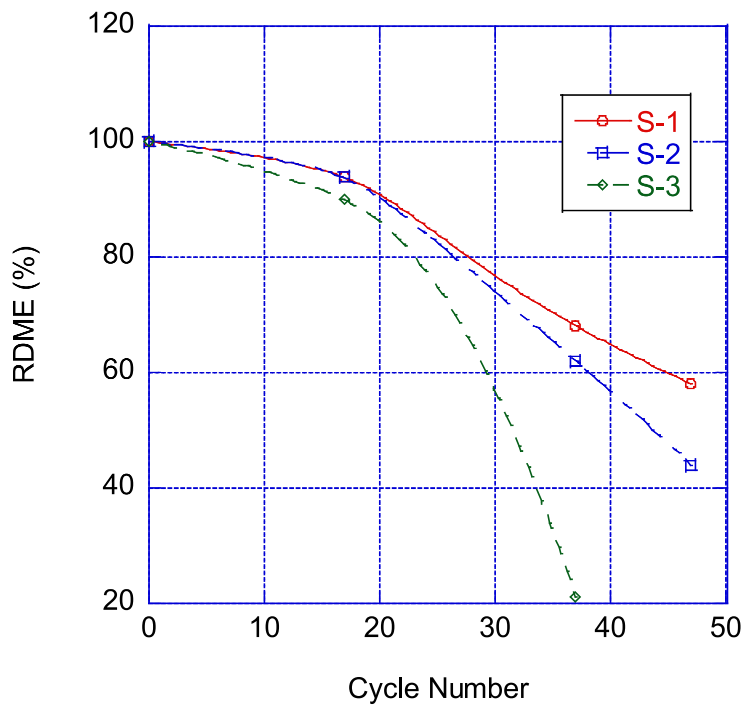

Figure D.2: KC Control 100% Cement Freeze-Thaw Trend

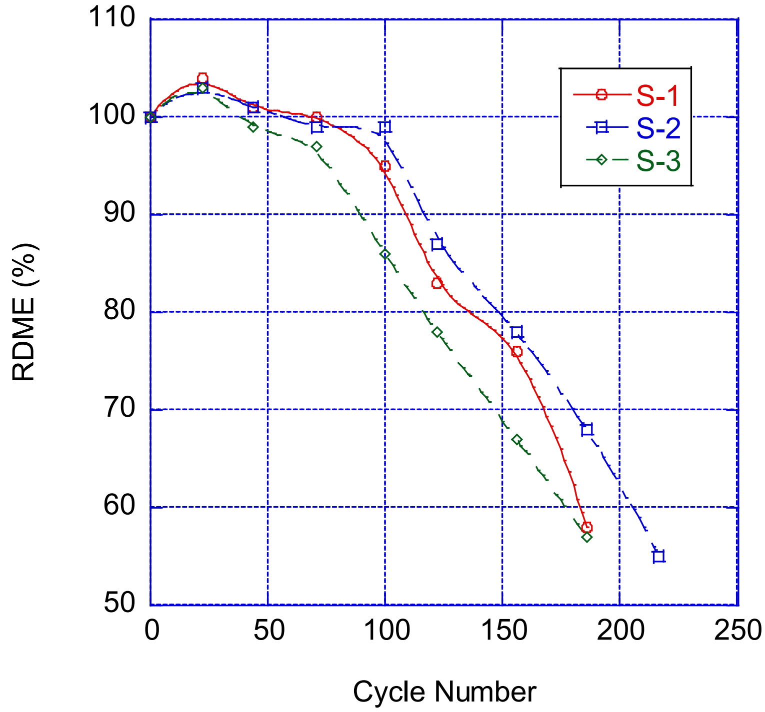

Figure D.3: Topeka Control 50%-50% Freeze-Thaw Trend

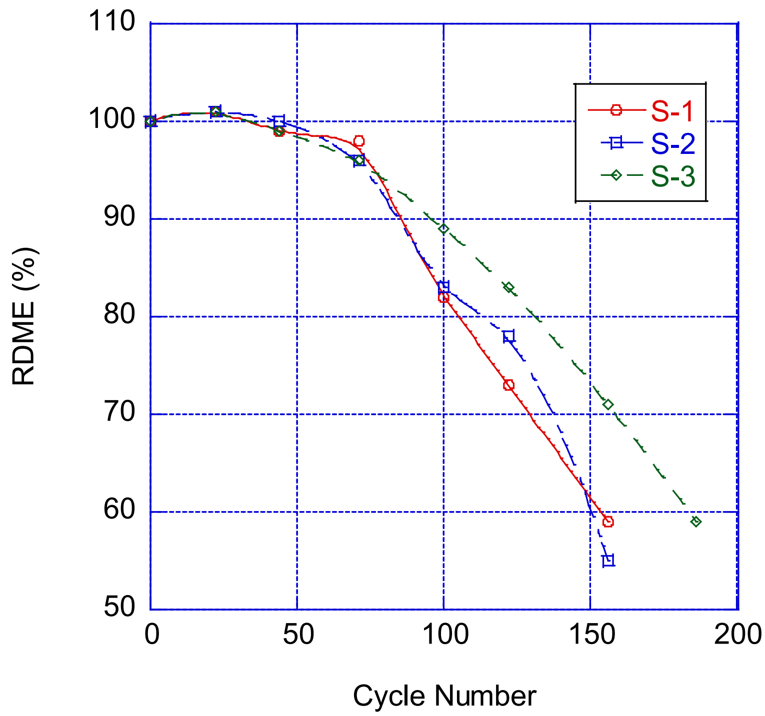

Figure D.4: KC Control 50%-50% Freeze-Thaw Trend

Appendix E: Comparison of PCTB with RCA to Control

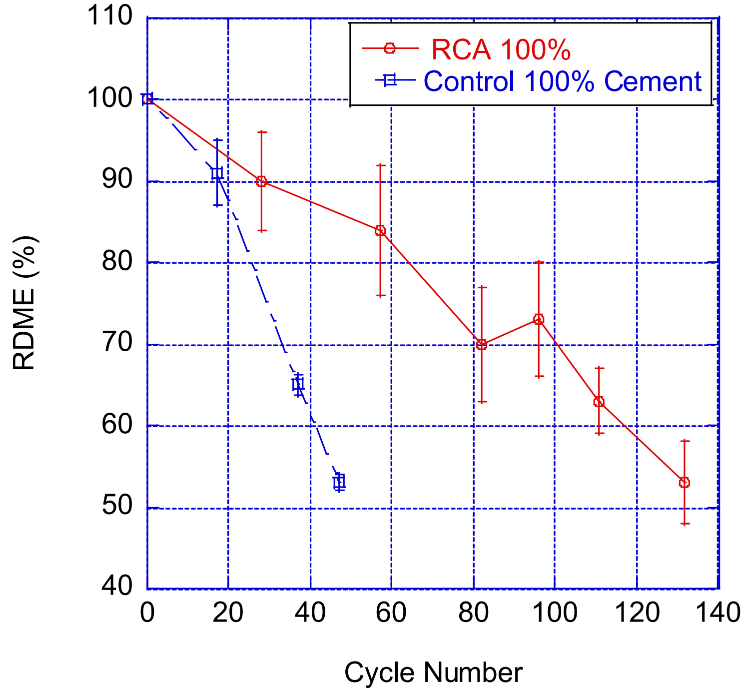

Figure E.1: Topeka 100% Cement Comparison

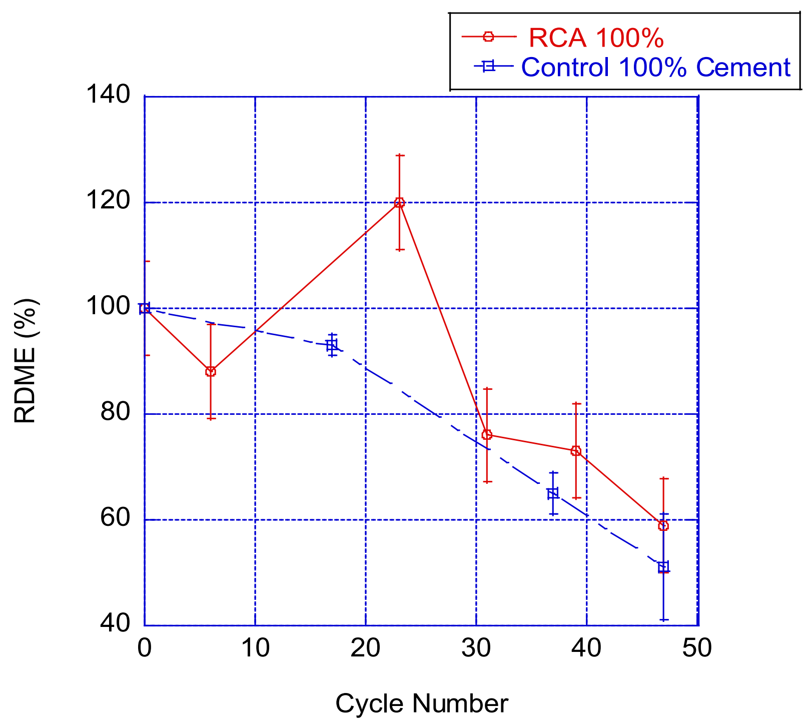

Figure E.2: KC 100% Cement Comparison

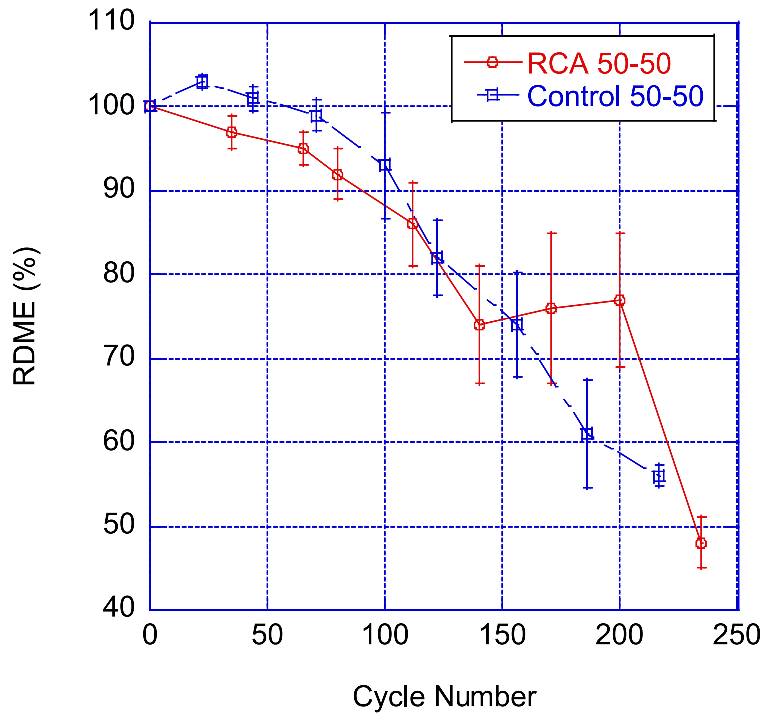

Figure E.3: Topeka 50%-50% Comparison

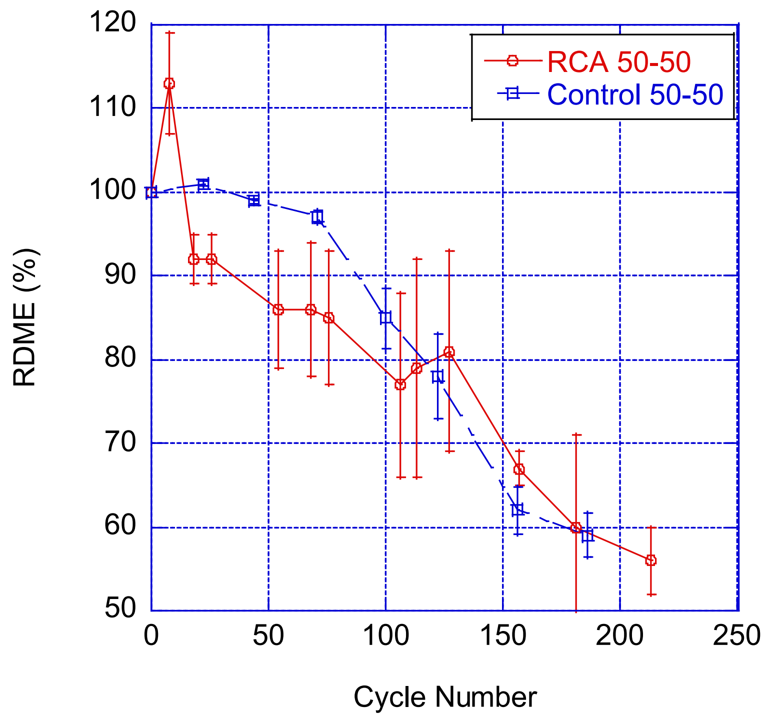

Figure E.4: KC 50%-50% Comparison

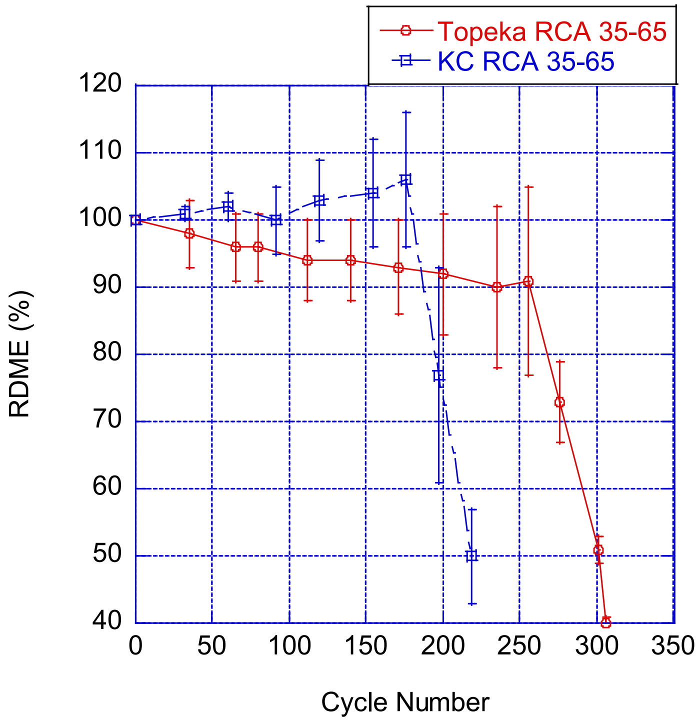

Figure E.5: Topeka & KC 35%-65% Comparison

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