RECYCLED BRICK MASONRY AGGREGATE CONCRETE: USE OF RECYCLED AGGREGATES FROM DEMOLISHED BRICK MASONRY CONSTRUCTION IN STRUCTURAL AND PAVEMENT GRADE PORTLAND CEMENT CONCRETE

by

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ABSTRACT

TARA LANI CAVALLINE. Recycled brick masonry aggregate concrete: use of recycled aggregates from demolished brick masonry construction in structural and pavement grade portland cement concrete. (Under the direction of DR. DAVID C. WEGGEL)

Reuse of construction waste as aggregates is becoming increasingly popular for a number of environmental and economic reasons. In this study, structural- and pavement-grade portland cement concrete (PCC) mixtures were developed using crushed recycled brick masonry from a demolition site as a replacement for conventional coarse aggregate. Prior to developing concrete mixtures, testing was performed to determine properties of whole clay brick and tile, as well as the crushed recycled brick masonry aggregate (RBMA). Concrete mixtures exhibiting acceptable workability and other fresh concrete properties were obtained, and tests were performed to assess mechanical properties and durability performance of the hardened concrete. Results indicated that recycled brick masonry aggregate concrete (RBMAC) mixtures can exhibit mechanical properties and durability performance characteristics comparable to that of structural- and pavement-grade PCC containing conventional coarse aggregates. Based on current North Carolina Department of Transportation (NCDOT) requirements, the suitability of RBMAC for use in pavement applications was evaluated, and the Mechanistic-Empirical Pavement Design Guide procedure was used to compare the potential performance of RBMAC pavement to conventional PCC pavement. Results indicated that RBMAC provides acceptable performance in pavement applications, where its thermal properties produce thinner pavement sections than PCC. This research gives designers a first look at some of the salient material properties that will influence future use of RBMA and RBMAC.

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AER	air exclusion rating
ACI	American Concrete Institute
ASTM	American Society for Testing and Materials
BIA	Brick Industry Association
BTU	British Thermal Unit
C	Coulomb
ºC	degrees Celcius
cf	cubic feet
con’t	continued
CTE	coefficient of thermal expansion
DOE	Department of Energy
DOT	Department of Transportation
ºF	degrees Fahrenheit
FHWA	Federal Highway Administration
ft	foot
Hg	mercury
hr	hour
ICM	Integrated Climatic Model
in	inch
K	Kelvin

kg	kilogram
l	length
L_a	actual length change
L_0	measured length of specimen at room temperature
lb	pound
lb-f	pound-force
LEED	Leadership in Energy and Environmental Design
LTPP	Long Term Pavement Performance Program
m	meter
M-EPDG	Mechanistic-Empirical Pavement Design Guide
mi	mile
min	minute
mm	millimeter
MOR	modulus of rupture
MPa	megapascal
N	newton
NCDOT	North Carolina Department of Transportation
oz	ounce
P	probability
Pa	Pascal
PCC	portland cement concrete
pcy	pounds per cubic yard
psf	pounds per square foot

psi	pounds per square inch
RBMA	recycled brick masonry aggregate
RBMAC	recycled brick masonry aggregate concrete
RAC	recycled aggregate concrete
RCA	recycled concrete aggregate
sec	second
sf	square feet
SSD	saturated surface dry
T	temperature
TGA	thermogravimetric analyzer
V	volume
WAR	water absorption rate
w/c	water to cementitious material ratio
wt	weight
Δ	change in
µL	microliter

CHAPTER 1: INTRODUCTION

1.1 Background

Use of recycled material as aggregates in civil engineering applications is beneficial because it reduces the environmental impact and economic cost of quarrying operations, processing, and transport. Reuse of construction and demolition waste is becoming increasingly desirable due to rising hauling costs and tipping fees for putting this material into landfills (Robinson et al. 2004). In recent years, sustainable construction initiatives have also made reuse of construction and demolition debris (as aggregates and otherwise) an appealing option when considering design alternatives for many types of structures (Desai 2004).

Incorporating recycled aggregates into cementitious materials is practical, as cementitious materials are non-homogeneous composites that allow material of different sizes and compositions to be bound in a cementitious matrix. Much research has been performed on the use of recycled concrete aggregates (RCA) and “mixed rubble” (concrete, concrete block, brick, and other materials) in portland cement concrete (PCC), but in the United States, very little research has been done on the use of recycled brick masonry as aggregates for concrete.

The use of brick as aggregate in concrete is far from a novel concept. After World War II, in England and Germany, rubble from brick buildings damaged or destroyed by bombs was crushed at rubble recycling plants producing crushed brick aggregate that was used in new concrete construction. Since this time, brick aggregate has not commonly been used in concrete in most Western countries. Brick aggregate concrete has, however, become common in locations where sources of natural aggregate are not available and the cost of importing natural aggregate is prohibitive. In these areas brick aggregate concrete is most often used in non-structural or non-critical applications. Existing studies on brick aggregate concrete often include only new or discarded brick that does not include the mortar fraction. In recent studies abroad on concrete that includes reclaimed brick masonry demolition waste, the bricks and mortar have properties that differ from those of the materials typically used in the United States.

With the large amount of brick masonry rubble produced in the United States each year, this material may provide a significant source of aggregates that can be used to produce more sustainable concrete. In addition to reducing the amount of waste that is landfilled (or used in low-grade applications such as roadbed gravel), other benefits can be realized. Brick aggregates are lighter than normalweight aggregates, and would provide haul cost savings. Concrete that incorporates brick aggregates is also lighter than normalweight concrete, would also be cheaper to transport, and can significantly reduce the self-weight of a structure. Brick aggregates have the potential to enhance the fire performance of concrete due to their thermal expansion and conductivity properties. Unfortunately, however, recycled brick masonry aggregates, or RBMA, also have undesirable characteristics such as high absorption and angular particle shapes, which must be addressed in the development of concrete mixture designs. As an additional concern, the characteristics of the brick masonry obtained from demolition sites vary, making a thorough understanding of this material, as well as the performance of concrete incorporating this material, critical.

If RBMA can show mechanical properties and durability performance similar to conventional PCC, it is envisioned that concrete incorporating RBMA, (or recycled brick masonry aggregate concrete, RBMAC) could provide advantages, both economic and otherwise, in several structural and pavement applications. To maximize the potential cost-savings, RBMA produced from a demolished structure could be re-used either in new concrete needed on-site, or in nearby construction (minimizing haul costs). Mobile crushers can be brought to a demolition site to produce the RBMA, and the RBMAC could be produced either in an on-site batch plant or hauled to a local ready-mixed plant. New RBMAC construction could include pavements, either as the wearing surface or as the lower lift in a two-lift pavement. Use in structural applications, particularly in precast concrete construction, could also be possible. Initiatives that promote sustainable design and construction practices, such as the Leadership in Energy and Environmental Design (LEED) and Greenroads programs, may provide other incentives for use of RBMAC in new construction. Sustainable design challenges designers to find site-specific solutions, and therefore optimum material use may take different forms at different sites.

1.2 Objectives and Scope

The key objective of this work was to develop concrete mixtures, using crushed brick masonry as a replacement for normal weight coarse aggregate, that exhibit acceptable mechanical properties and durability performance characteristics comparable to that of structural- and pavement-grade PCC containing conventional coarse aggregates. This research gives designers a first look at some of the material properties that will influence how RBMA and RBMAC can be managed and used in sustainable construction. In order to obtain the 'cleanest' demolished brick masonry for this study that can be achieved via readily employable source separation techniques, the crushed brick aggregate was made from brick masonry obtained from a single demolition site, Idlewild Elementary School in Charlotte, North Carolina.

Prior to developing the concrete mixtures, tests were performed to characterize whole brick and clay tile as well as the crushed RBMA. Mixture designs were developed in accordance with ACI 211.2, "Standard Practice for Selecting Proportions for Structural Lightweight Concrete." Issues anticipated and addressed in development of the mixture designs included the high absorption and angularity of the brick aggregate. After concrete mixtures exhibiting acceptable workability and fresh properties were developed, tests were performed on hardened concrete specimens to assess mechanical properties and durability performance.

A second objective of this work was to assess recycled brick masonry aggregate concrete (RBMAC) for suitability in pavement applications for North Carolina Department of Transportation (NCDOT) use as well as in the new Mechanistic-Empirical Pavement Design Guide (M-EPDG) procedure. Results of characterization and testing of the RBMA and RBMAC were compared to current and proposed NCDOT standard specifications, and potential limitations were identified. A proposed full-scale field test was developed as part of this work to assist in evaluating the performance of an in-situ pavement comprised of RBMAC. The proposed full-scale field test pavement section was designed using M-EPDG, which allows designers to incorporate specific aggregate properties into the design and analysis procedure. Characteristics of the RBMA and performance data for the RBMAC were used as inputs to the M-EPDG software.

1.3 Organization of Contents

This work is presented in eight chapters. A review of available relevant literature is presented in Chapter 2. Literature surveyed included publications on reuse of construction waste as aggregates in PCC and mortars, use of brick and recycled brick masonry in PCC, previous research on brick aggregate concretes and mixtures, and the potential for widespread acceptance and use of brick aggregate concretes and mortars. Research needs are also identified and stated in this chapter.

The testing program for characterization of recycled materials is presented in Chapter 3. This chapter begins with an overview of the demolition site used to obtain the RBMA used for this study. Experimental procedures for characterization of whole clay brick and tile, as well as the RBMA are also presented.

In Chapter 4, the components of the PCC mixtures developed as part of this study are described, as well as some of their properties that may influence the performance of the mixtures.

The development of concrete mixture proportions is presented in Chapter 5. This chapter begins with an overview of the strategy utilized for developing these mixtures, and anticipated challenges, design variables and constraints, and information pertaining to the design approach are subsequently presented. Preliminary and final mixture proportions are also included in this chapter.

The testing program for the baseline (final) RBMAC mixtures is described in Chapter 6, which also includes the batching procedure and mixing method. Methodology for tests on fresh and hardened concrete is outlined, and the experimental results from the testing program are subsequently presented.

In Chapter 7, use of RBMAC in pavement applications is studied. Potential use of RBMAC in NCDOT pavement applications is explored, and compliance with current and proposed NCDOT requirements and standards is discussed. Some effects of using measured RBMAC properties in M-EPDG pavement designs are identified, and a proposed test pavement project utilizing RBMAC is presented.

Chapter 8 provides a summary of the work performed, identifies findings and conclusions, and recommendations for future work.

CHAPTER 2: LITERATURE REVIEW

Use of crushed construction and demolition waste as aggregates in new concrete has gained increased interest in recent years for reasons related to both economics and environmental sustainability. Demolition waste materials used for production of recycled aggregates can include crushed concrete, crushed brick masonry rubble, and crushed mixed rubble from various sources including demolished buildings and roadways. Recycled aggregates have been used in new concrete in both vertical and horizontal construction applications. Concrete that contains recycled aggregates is called recycled aggregate concrete (RAC).

Concrete that has been taken out of service, or returned unused from a jobsite and allowed to harden, can be crushed and made into recycled aggregate called recycled concrete aggregate (RCA). The suitability of RCA for use in new concrete has been extensively studied by a number of researchers (Topçu and Sengel 2004, Obla et al. 2007, Etxeberria et al. 2007 and others), and new concrete produced using RCA has been used in a number of field applications and products. Guidelines for use of RCA in new concrete have been published by a number of agencies, including the Federal Highway Administration (FHWA 2008). Several state Departments of Transportation (DOTs), including Michigan (Michigan Department of Transportation 2003) and Texas (Texas Department of Transportation 2004), allow use of RCA in some applications, with requirements outlined in their specifications.

Although a significant amount of research and a number of field studies have been performed on concrete produced with RCA, use of recycled aggregates produced from demolished brick masonry construction has not been extensively studied, particularly in the United States. A significant amount of brick masonry rubble is produced in the United States, and this material may provide a viable source of RBMA that can be used to produce sustainable concrete.

In subsequent sections, a review of literature relevant to this research study is presented. Information on the current reuse of construction waste as aggregates in portland cement concrete applications is provided, along with a discussion on the advantages and challenges related to the use of this material. The potential for future use of this material on a local, regional, and national scale is also discussed. A general overview of the current use of crushed brick material (including both virgin brick material and recycled brick masonry) as aggregates in portland cement concrete is provided, as well as a summary of results of the limited number of research studies performed on this material. This is followed by a discussion on the potential for widespread acceptance and use of brick aggregate and RBMA in concrete applications, including practical challenges and economic considerations. Finally, a summary of research needs related to use of RBMA in concrete is presented.

2.1 Reuse of Construction Waste as Aggregates in Portland Cement Concrete and Mortars

In the construction industry, a significant amount of waste material is generated during demolition of unwanted facilities including roads and buildings. Much of this waste material is being placed into landfills, but some hardscape rubble is being transported to public and private facilities, where it is crushed into graded or ungraded aggregate material. Advantages of use of aggregates made from crushed construction waste material in new concrete are related to economy as well as sustainability.

Although a number of challenges associated with use of this material have been identified, many of these challenges are being addressed through tactics including process controls (implemented during demolition, transport, and manufacturing of aggregates) and through requirements outlined in specifications. Successful performance of RAC (particularly concrete made with RCA) has been demonstrated in a number of research studies and field installations. However, despite an increased understanding of the performance of RAC, acceptance and use of RAC has been relatively slow.

2.1.1 Advantages

Use of recycled materials as aggregates in concrete can provide a number of advantages to stakeholders, which include owners, contractors, and the ready-mixed concrete and precast concrete industries. From an economic standpoint, recycled aggregates can be cheaper than conventional (natural and manufactured lightweight) aggregates. Use of aggregates made from crushed construction and demolition debris may become an increasingly attractive alternative due to rising landfill tipping fees, diminishing landfill space, and rising cost of virgin natural aggregate material (Tam and Tam 2006).

From the standpoint of sustainability, use of recycled materials as aggregates provides several advantages. Landfill space used for disposal is decreased, and existing natural aggregate sources are not as quickly depleted (Kutegeza and Alexander 2004). Use of recycled aggregates in lieu of virgin quarried aggregates can potentially result in a lower embodied energy of the concrete, although this is often dependent on hauling costs (Chong and Hermreck 2010). This particularly holds true if the methodology used to compute the embodied energy of a structure accounts for the “recovery” of energy at the end of its service life (Thormark 2002).

2.1.2 Challenges and Current Applications

Challenges inhibiting the use of recycled aggregates in concrete applications are generally related to “lack of awareness, lack of government support, (and) non-existence of specifications/codes for reusing these aggregates in new concrete,” according to Rao et al. (2007). Although published studies provide guidance regarding use of recycled aggregates in concrete, specifying agencies often do not allow for use of this material due to risks perceived by the designer (Desai 2004) or contractor. Of these risks, one of the most prominent is related to the consistency of the recycled aggregate source material (Hansen 1986) and its impact on the performance of the new concrete in-situ. Certification systems for recycled aggregates, such as that discussed by Hendriks (1994), may help to address this barrier. Ultimately, an increased number of successful field implementations will help to increase the understanding of the performance of RAC, particularly from a durability standpoint, which will help to increase its acceptance and use (Olorunsogo and Padayachee 2001).

Research and development, and consequently field implementation, for concrete containing recycled aggregates has progressed more swiftly in Europe than in the United States, particularly during the 1980’s and 1990’s (Transport and Road Research Laboratory 1981, Williams 1996). This is partly due to the fact that many European nations do not have readily available land for landfills and quarries (Oikonomou 2005).

However, in the United States, research on RCA has taken significant strides over the past decade, which has in turn increased the use of RAC in field applications such as pavements and other structures.

Recognizing the need to gain a comfort level with use of recycled materials in concrete applications in an expedient manner, the Federal Highway Administration (FHWA) has promoted a number of research initiatives, including a study on using accelerated aging techniques to assess the long-term performance of recycled materials in concrete (Eighmy et al. 2001). Recently, FHWA and many state DOTs have gained an increasing level of comfort with RCA concrete, permitting it to be used in a number of transportation applications (Texas Department of Transportation 2004, Michigan Department of Transportation 2003). Initiatives to promote a sustainability rating system for roadways, such as the proposed Greenroads program (Anderson et al. 2011), have also brought increased attention to use of RAC.

The ready-mixed concrete industry, realizing cost savings and other benefits to their members, has also promoted extensive research and development to support the use of RCA (Obla et al. 2007). Sustainable building design and construction initiatives, including the Leadership in Energy and Environmental Design (LEED) building rating system promoted by the United States Green Building Council, have provided incentives for use of recycled material, including RAC, in building construction in recent years (Kibert 2008).

2.1.3 Forecast for Use on a Local, Regional, and National Scale

As discussed in the previous section, increased interest in use of RAC, from both a profitability and sustainable perspective, has resulted in increased research and understanding of material performance. Sustainability initiatives, economic considerations, and an increased comfort level with material performance will likely increase the use of RAC on a national scale.

From a regional and local perspective, use of recycled aggregates in concrete will depend on the ability of recycled aggregates to compete with conventional aggregate sources in the regional or local construction market. As landfill space and virgin aggregate become more costly in some markets in the United States (Robinson et al. 2004), it is likely that use of recycled aggregates such as RCA and RBMA in concrete applications will increase here as well. In addition to reducing the amount of waste that is landfilled, potential cost savings can be realized by owners, contractors and/or material suppliers.

Locally, the Mecklenburg County Land Use and Environmental Services Agency (2006) reported that approximately 8% of construction and demolition waste was composed of concrete and other hardscape rubble. Much of this material is currently utilized in low-grade uses such as gravel for landfill haul roads, and the recent economic downturn has resulted in a deficit of crushed recycled aggregate material to meet the low-grade demands. However, improved economic conditions will likely bring about a resurgence of construction activity, and findings of a recent study by Tempest et al. (2010) indicated that “if the supply and consistency of demolition rubble increases, there should be improved market interest in RA (recycled aggregates).”

2.2 General Overview - Use of Brick and Recycled Brick Masonry as Aggregate in Portland Cement Concrete

Use of construction and demolition waste, including brick, in cementitious materials is far from a novel concept. In Roman times, structures such as buildings and water supply channels were constructed using natural pozzolans and crushed brick (Hansen 1992). A large amount of brick masonry rubble is produced in the United States each year. This material may provide a significant source of RBMA that can be used to produce more sustainable concrete. In the following sections, a general overview of past and present use of recycled brick and recycled brick masonry as aggregate in PCC is presented. First however, it is necessary to provide definitions of certain terminology that will be used.

Review of related papers revealed that, with a few exceptions, most previous work published between the 1940's and present day was done using aggregate comprised solely of crushed brick. In this literature review, this material will subsequently be referred to as "brick aggregate" or "brick aggregate concrete" when brick aggregate comprises a substantial amount of the concrete filler. In the relatively few instances where researchers used recycled brick masonry material which included the mortar fraction (and potentially other debris from the demolition process) of the construction, the terminology "recycled brick masonry aggregate" (abbreviated RBMA) or "recycled brick masonry aggregate concrete" (abbreviated RBMAC) is used.

A number of studies use "masonry rubble" which can be loosely defined as including "conventional concrete and concrete block, clay brick, sand-lime brick, lightweight concrete and block of various types, and natural stone (ECCO 1999)." In addition to the materials listed above, Hansen (1992) includes aerated concrete blocks, blastfurnace slag brick and blocks, burnt clay materials (roofing tiles and shingles) as materials collectively included in “masonry rubble.” Materials collectively called “contaminants” can include “metals, asphalt, timber, plastics, glass and plaster (Hansen 1992).”

2.2.1 Early History

The earliest use of crushed brick in cementitious materials using portland cement occurred in Germany in 1860 (Devenny and Khalaf 1999, Hansen 1992). In Europe, many of the buildings damaged or destroyed by bombs during World War II included brick masonry. As part of rebuilding the damaged cities in Germany, rubble recycling plants were created, producing crushed RBMA that was used in new concrete construction. The RBMAC produced by these plants was used to construct 175,000 new housing units (Hansen 1992). After debris from the bombing was cleared, bricks were no longer widely used as concrete aggregates (Khalaf and DeVenny 2004).

Crushed brick masonry was also used as aggregates in concrete in Great Britain after World War II. Newman (1946) performed testing on concrete and mortars made from brick rubble obtained from demolished bomb shelters. Newman’s RBMAC mixtures were made (1) using brick as both fine and coarse aggregate, and (2) using brick as coarse aggregate with river sand as the fine aggregate. Mixtures were created using brick from demolished shelters from eight cities (in England, Scotland and Wales), with w/c ratios varied from approximately 0.70 to 1.05 in order to maintain consistent workability (as measured by slump). Compressive strengths resulting from these mixtures varied based upon water cement ratio and the source of RBMA, and ranged from 875 to 5,770 psi (6.0 to 39.8 MPa) for mixtures using RBMA as both fine and coarse aggregate, and from 945 to 5,690 psi (6.5 to 39.2 MPa) for mixtures using RBMA only as coarse aggregate. Other research performed in the 1940s produced RBMAC with compressive strengths between 15 and 25 MPa (approximately 2,175 to 3,625 psi) (Hansen 1992). Khalaf (2006) attributes the low strengths obtained in these studies to use of weaker cement produced during this era or impurities present in the brick aggregates used.

A relatively large gap in time exists between the testing performed on RBMAC created using post-World War II rubble and the “next generation” of testing on brick aggregates used in cementitious materials. It seems that in Europe, once rubble from structures destroyed during World War II was utilized, interest in this area diminished. The next available publications regarding brick aggregate and RBMA use in cementitious materials were published in the late 1970’s and early 1980’s.

Economic conditions and lack of suitable natural aggregates seems to have resulted in brick being used as aggregate in developing nations before being used in developed ones. Khan and Choudrhy (1978) discuss how in Bangladesh, brick chips have been used as aggregate for a number of years prior to publication of their paper. They describe the three manufacturing processes used for making brick in Bangladesh and discuss the large variation in quality and mechanical properties of bricks made from these methods. General batching processes are described, and data for several brick aggregate concrete mixtures (compressive strengths ranging from 3,950 to 6,260 psi, or 27.2 to 43.2 MPa) is presented.

2.2.2 Current Use of Brick Aggregate in Portland Cement Concrete

Currently, in many locales (both domestic and abroad), crushed brick aggregate and RBMA is often used in non-cementitious, low-grade applications (Tempest et al. 2010). This material is often in demand for use in roadbeds and as fill material, and after being used for these low-grade applications, is not available for use in portland cement concrete applications. Khalaf (2006) indicates that in recent years, brick aggregate concrete has been limited to "low level uses such as pipe bedding or site fill," and states that "this is mainly due to impurities in the material, lack of knowledge of its performance in concrete, lack of available standards on the use of recycled aggregates in concrete and high water absorption characteristics of recycled brick aggregates." He further states that "only a small amount of work has been carried out using the types of brick that are commonly used in construction today and there is little knowledge on the subject in the United Kingdom and other countries."

Publications regarding the current use of RBMA from demolition sites in portland cement concrete are far less prevalent than publications on RCA in portland cement concrete. However, a number of publications regarding the current use of brick (either standard brick or discarded brick) as aggregate in portland cement concrete exist. Use of brick aggregate in concrete has become common in locations where sources of natural aggregate are not available and the cost of importing natural aggregate is prohibitive or politically unfeasible. An example is Bangladesh, where the land primarily consists of a deltaic plain (Mazumder et al. 2006).

2.2.3 Current Use of Recycled Brick Masonry as Aggregate in Portland Cement Concrete

In the United States, RBMA is not being used in portland cement concrete. Worldwide, lack of knowledge of performance of brick aggregate concrete is an obstacle for reuse of brick waste (Debieb and Kenai 2006). However, with increased environmental awareness during the past decades, and economic incentives to re-use waste, use of RCA and masonry rubble is once again receiving attention from the technical community. Studies abroad conducted recently indicate that use of RBMA in non-structural applications such as paving blocks is of interest in Yugoslavia (Jankovic et al. 2012), Spain (Lopez et al. 2011), Iran (Rafsnajani et al. 2012) and China (Li et al. 2012). Demand for low cost non-quarried aggregates continues to promote research into brick aggregate concrete in Bangladesh (Ahmad and Roy 2011). The construction boom in China has driven increased interest in alternative sources of building material. Studies on RBMAC have recently been performed by Yang et al. (2011) and Ho and Tsai (2011). Although RBMAC is currently not used in the United States, an understanding of the performance of RBMAC could lead to its use in non-structural applications, and subsequently in structural and pavement applications. Test data on RBMAC produced in the United States is needed to spur an interest in use of this material in sustainable design.

2.3 Previous Research on Brick Aggregate Concretes and Mortars

Some research that was performed abroad exists regarding the mechanical properties, and to some extent, the durability characteristics of brick aggregate concrete and RBMAC. The mechanical properties and durability characteristics of brick aggregate concrete and RBMAC are often compared to the properties of similar concrete mixtures that contain virgin natural aggregate. Most of the studies included in this literature review investigated the effect of varying percentages of brick aggregate or RBMA replacement (for both fine and coarse aggregates) on mechanical properties such as compressive strength, tensile strength, flexural strength, and modulus of elasticity of brick aggregate concrete and mortar. Durability studies on brick aggregate concrete and RBMAC are far less common, and often a few durability tests were performed as part of a study that largely focused on mechanical properties. A few studies were performed on the pozzolanic reactions that can occur when brick is in contact with cementitious materials, and microstructural characteristics of brick aggregate concretes and mortars. Findings of these studies are summarized in subsequent sections of this literature review.

2.3.1 Mixture Design and Fresh and Hardened Performance Characteristics

Studies related to mixture design, mixing procedure, and hardened performance characteristics for brick aggregate and RBMA concrete and mortar typically focus on several areas. These areas include water demand, mechanical properties (compressive strength, tensile strength, flexural strength), modulus of elasticity, and shrinkage. Most of these studies have been performed on concrete and mortars incorporating brick aggregate, not RBMA. Typically, only part of a fine or coarse aggregate component has been replaced with brick aggregate or RBMA, and in only a very few studies was brick aggregate or RBMA used at 100% replacement rates for a natural aggregate. In almost all studies, water reducing admixtures were not used to assist with achieving adequate workability and strength at relatively low w/c ratios. Subsequently, cement contents were relatively high and it is likely that the permeability of the paste matrix was high due to the relatively high water content in the mixture.

A key part of designing brick aggregate and RBMA concrete and mortar mixtures is addressing the high water demand of the very porous brick aggregates. Additional water needs to be added to mixtures in order to provide the required workability, but this often comes at the expense of strength of the hardened concrete or mortar. Researchers have tried to address this issue by either pre-soaking the aggregate prior to mixing or by modifying the mixing procedure to ensure that the water used in the mixture has adequate contact time with the cement (Khaloo 1994). Hansen (1992) performed a study in which mixtures were prepared using finely crushed brick as a replacement for some of the sand. Hansen (1992) indicated that if pre-soaking the brick aggregate is done to help overcome workability problems, the brick aggregates will be almost completely saturated after 30 minutes of soaking.

Relatively few comprehensive studies on brick aggregate and RBMA concrete mixtures exist. Most studies focus on testing of compressive strength and modulus of elasticity, and possibly several other mechanical properties. Two relatively extensive studies on brick aggregate concrete were performed by Akhtaruzzaman and Hasnat (1983 and 1986) and Khalaf (2006). The results of these studies are often cited in other works, and the findings are used as a comparison to the results of many other researchers. Therefore, these two studies are subsequently discussed in this literature review with a significant amount of detail. It is noted that, like most other studies, testing performed by Akhtaruzzaman and Hasnat (1983) and Khalaf (2006) was on concrete that contained crushed brick aggregate, not RBMA, and therefore none of the concrete in either of these studies contains the mortar fraction typically included with crushed brick masonry as well as other contaminant material that may be inadvertently mixed with the RBMA.

Akhtaruzzaman and Hasnat (1983) tested several concrete mixture designs that used brick aggregate as coarse aggregate. The bricks used by Akhtaruzzaman and Hasnat (1983) were manually crushed to sizes ranging from 3/16 to ¾ in (4.8 to 19.1 mm), with most of the brick aggregate falling within the range of 3/8 to ¾ in (9.5 to 19.1 mm). The compressive strength of the bricks used to make the aggregate was approximately 5,300 psi (36.5 MPa). A table included in this paper is useful in presenting some of the fundamental differences between the brick aggregate and the sand, with key differences including the absorption (11.20 percent for brick aggregate vs. 3.10 for sand) and the bulk specific gravity (1.93 for brick aggregate vs. 2.57 for sand). Water-cement ratios ranged from 0.54 to 0.88, and the compressive strengths of the mixtures ranged from 2,300 to 5,500 psi (15.9 to 37.9 MPa). Unit weights of the concrete mixtures ranged from 125 to 130 pounds per cubic foot (pcf), (or 2002 to 2082 kilograms per cubic meter, kg/m^3), which is lower than the unit weight of normal weight concrete (typically 140 to 150 pcf, or 2,242 to 2,402 kg/m^3) and higher than the unit weight of lightweight concrete with traditional lightweight aggregates such as expanded shale (typically 110 to 120 pcf, or 1,762 to 1,922 kg/m^3).

Akhtaruzzaman and Hasnat (1983) found that "in general, for the same grade of concrete the modulus of elasticity is about 30% lower and the tensile strength is about 11% higher than normal weight concrete." Additionally, test results indicated that the brick aggregate concrete produced during this study had a modulus of elasticity that was about 30% lower and a modulus of rupture that was about 10% greater than that compared to normal weight concrete of similar strength. The authors also provide general equations for the compressive strength, modulus of elasticity, modulus of rupture, and splitting tensile strength. These equations are presented in a format similar to the equations used in the American Concrete Institute (ACI) Building Code, with the constants adjusted to reflect the results of the test data obtained using their brick aggregate concrete.

Further studies on beams cast using brick aggregate concrete were performed by Akhtaruzzaman and Hasnat (1986). The beams did not contain web reinforcement, and were tested in shear and flexure using two-point loading tests. Test results indicated that "the shear strength of brick-aggregate concrete beams without web reinforcement is higher than that of normal weight concrete beams computed on the basis of the 1983 building code equation." The authors further stated that this "indicated that brick-aggregate concrete beams will require less web reinforcement." For beams failing in flexure, the experimental moment capacities are in close agreement with the computed values. Akhtaruzzaman and Hasnat (1986) conclude that "although further research is needed, it can be concluded from this investigation that brick-aggregate concrete is a suitable structural material. Economy in construction would also be achieved because of its higher shear and tensile strengths and lower unit weight."

Perhaps the most extensive recent study on brick aggregate concrete was published by Khalaf (2006). This study was also done on brick aggregate concrete made using coarse aggregates made from crushed new bricks that did not include a mortar fraction or impurities that would be found in demolished brick rubble. The new bricks used for this study have several compressive strengths, and companion mixtures were made using a granite natural aggregate. Gradations of both types of aggregates were kept constant in the mixture designs in order to assist with the direct comparison of test results. Additionally, admixtures were incorporated into some of the mixtures. An air entraining admixture was used to entrain 5% air. A superplasticizing admixture was used to assist in achieving suitable workability and to assist in maintaining the desired water to cement ratios for the mixtures (which varied from 0.55 to 0.7, relatively high for modern-day concrete).

Khalaf (2006) attempted to understand and refine the mixing procedure for brick aggregate concrete during his study. For mixtures that did not use a superplasticizing admixture, the author indicated that the brick aggregate was pre-wet prior to mixing in order to minimize initial absorption of the mix water. For the concrete mixtures in which a superplasticizer was used, “the aggregate was not presoaked before mixing as it was hoped that the superplasticizer would improve the workability and allow the presoaking procedure to be omitted.”

Khalaf’s targeted compressive strengths of the mixtures at 28-days were 43 MPa (approximately 6,235 psi) and 50 MPa (approximately 7,250 psi). It was hoped by the author that the mixtures could reach 50 MPa (approximately 7,250 psi) and 63 MPa (approximately 9,135 psi) during the course of the study. These compressive strengths are significantly higher than the compressive strengths targeted in many earlier studies, and are consistent with the design strengths of many structural concrete mixtures used today.

Khalaf (2006) presents results for workability, compressive strength, and flexural strength. Densities of the brick aggregate concrete mixtures were approximately 8 to 15% lower than the natural aggregate concrete mixtures. Brick aggregate concretes produced in this study gained strength at rates similar to natural aggregate concrete, and the targeted compressive strengths were achieved. It is noted that the compressive strengths of the mixtures were influenced by the compressive strength of the bricks used as the coarse aggregates. Like many of the studies conducted outside of the United States, compressive strength tests were performed using cubes, rather than the cylindrical test specimens more commonly used in the United States for testing structural concrete.

Khalaf (2006) states that paste-aggregate bonds were good for the brick aggregate, citing that in many compression testing specimens, fractures propagated through brick aggregate particles rather than around aggregate-paste interfaces. Khalaf states that the targeted 5% air contents were typically met, indicating that “air-entrained concrete containing crushed brick aggregate can be successfully produced using the Building Research Establishment mix design method.” For mixtures that did not contain a superplasticizer, workability was judged to be best at w/c ratios of approximately 0.55, which is in agreement with the results of Akhtaruzzaman and Hasnat (1983). Slumps for these mixtures were typically around ½ in, which is extremely low for modern concrete mixtures. By using a superplasticizer, slumps of up to 2¾ in (69.9 mm) were achieved, although the authors noted that “the effect of the superplasticizer only lasted for about 15 minutes, after that the concrete became difficult to work with.”

Flexural strength testing results from Khalaf’s (2006) study indicated “that there was about an 8% reduction in flexural strength when crushed brick aggregate was used in place of granite as the coarse aggregate.” This differs from the results reported by Hansen (1992) and Khaloo (1994) who indicated that brick aggregates provide a 10% and 15% increase in flexural strength, respectively.

Other studies on brick aggregate and RBMA tend to focus only on strength (compressive, splitting tensile, and/or flexural), with a limited number of other tests for mechanical properties and durability performance performed. Work done by Schulz and Hendricks (1992) indicated that, although use of brick aggregates often requires significant changes in the w/c ratio of concrete, compressive strength values higher than that of conventional aggregate concrete can be achieved. Data compiled by Hansen (1992) (consisting mainly of post-WWII brick aggregate concrete) shows a weak correlation between the density of brick aggregate and compressive strength. He presents similar relationships for other mechanical properties such as creep, shrinkage, and modulus of elasticity.

Khaloo (1994) used clinker brick, a waste product from the brick making industry in Iran, as coarse aggregate in the brick aggregate concrete produced for his study. Brick aggregate concrete was produced using clinker brick at varying levels of replacement for natural aggregate. A table showing the chemical analysis of the soil used to create the bricks used in this study is provided, information which is typically left out of publications pertaining to other studies. Testing was performed to determine the compressive, tensile, and flexural strengths. Khaloo (1994) found that "All the 100 percent crushed brick concretes present compressive, tensile, and flexural strengths higher than those for 50 percent… This may be explained by the higher uniformity in a concrete mix of 100 percent crushed clinker brick compared with 50 percent clinker brick plus 50 percent crushed stone aggregates." Khaloo also notes an "increase in tensile and flexural strengths of 100 percent crushed clinker brick concrete compared to crushed stone concrete is due to the rough surface of the crushed bricks, which provides better bond between the concrete matrix and the crushed bricks." This study is one of a very few studies which looked at the abrasion resistance of the brick aggregate concrete, with Khaloo (1994) commenting that the "abrasion resistance of the… crushed brick is less than that of the crushed stone aggregates."

Kesegic et al. (2008) performed tests on three different mixture designs to compare the performance of brick aggregate concrete, RCA concrete, and concrete containing natural aggregates. The w/c ratio was kept constant. Compressive and flexural strength tests were performed, and the results indicated that at 28 days, the compressive strengths of the brick aggregate concrete and tile aggregate concrete were about 23.8% and 32.7% lower (respectively) than the compressive strength of the concrete containing natural aggregate.

In a number of studies, brick aggregate is used as a partial or complete replacement for natural coarse aggregate. Typically in these publications, a discussion on challenges related to mixture design and batching is presented, and then the results of various mechanical property tests at different ages are presented. Studies that focused on comparing the performance (particularly strength) of brick aggregate concrete to concrete containing natural aggregates include those performed by Ramamurthy and Gumaste (1998) and DeVenny and Khalaf (1999). Ramamurthy and Gumaste (1998) compared the compressive strength of brick aggregate concrete mixtures to similar natural aggregate and RCA concrete mixtures. They noted that acceptable compressive strengths were obtained even when the brick aggregates did not pass the current British standards for impact and crushing. DeVenny and Khalaf (1999) used crushed brick (new bricks broken up by hand) in concrete mixtures, comparing the compressive strength results with similar mixtures using natural aggregate. Interestingly, in certain cases, the brick aggregate mixtures had strengths that exceeded some of the natural aggregate mixtures. Examples of other studies on coarse aggregate replacement include those done by Kibriya and Speare (1996), Mansur et al. (1996 and 1999), Zakaria (1999), Padmini (2001 and 2002), and Jankovic (2002), the key findings of which are presented in the following paragraphs.

Kibriya and Speare (1996) tested brick aggregate concrete produced using three different types of brick aggregate. In this study, the brick aggregate concrete that was produced had compressive, tensile, and flexural strengths that were comparable to those of the concrete that contained natural coarse aggregate. However, the modulus of elasticity was drastically reduced. At 90 days, the brick aggregate concrete had significantly higher shrinkage than the conventional concrete, and the creep at one year was slightly higher.

Crushed brick (new bricks rejected from a factory) were used as a coarse aggregate replacement by Mansur et al. (1996 and 1999) in brick aggregate concrete, and compressive strength results were compared with similar mixtures using natural aggregate. A superplasticizer was used to assist in obtaining the desired workability of each mixture. Similar to the findings of Kibriya and Speare (1996), Mansur et al. (1999) state that "results indicate that brick-concrete can attain the same compressive strength, gives a higher tensile strength, a lower drying shrinkage and almost identical creep strains when compared to conventional concrete. However, it exhibits a substantially smaller elastic modulus." Equations expressing the relationships between mechanical properties are presented.

In a study by Zakaria (1999), the tensile strength of brick aggregate concrete was found to be higher than that of the natural aggregate concrete and “the modulus of rupture of brick aggregate concrete is, on the average, 10 percent higher than for gravel aggregate concrete.” He also found that both the static modulus of elasticity and dynamic modulus of elasticity of brick aggregate concrete were lower than that of the natural aggregate concrete.

Padmini et al. (2001) tested brick aggregate concrete mixtures generated using a five-factor factorial design in order to isolate the characteristics most influential in governing the compressive strength of the concrete. It was found that cement content, moisture content of the brick aggregate (pre-soaked vs. not pre-soaked), and strength of the brick aggregate had the most influence on compressive strength, while sand content and aggregate size had minimal influence. It is noted that cement content had a significantly greater influence on compressive strength than any of the other four factors. Subsequently, additional testing was done to further explore the effects of variation of the three parameters deemed most influential (listed above) on compressive, flexural, and splitting tensile strengths. The data obtained were used to create relationship curves that could be used as guidelines for developing new mixture designs for brick aggregate concrete based on the ACI 211.2 procedure for structural lightweight aggregate concrete.

Jankovic (2002) performed tests on brick aggregate concrete that included varying dosages of polymer admixture. Some mixtures included brick aggregate as both fine and coarse aggregates, while other mixtures used brick aggregate as only the coarse aggregate, with a river sand as the fine aggregate. Results from her study indicated that increasing levels of polymer admixture generally result in increases in compressive and flexural strength, while reducing the modulus of elasticity. The polymer-modified brick aggregate concrete mixtures also had lower absorptions, displayed greater creep, and reduced shrinkage. This work was based on earlier studies by Muravljov, Pakvor, and Jankovic (1998) that studied the curing requirements of polymer modified brick aggregate concrete. A study by Drpic and Jankovic (1999) on polymer modified brick aggregate concrete focused on the thermal conductivity of the mixtures, finding that it decreased with an increase in polymer or with a decrease in cement content.

Brick aggregate is used as a partial or complete replacement for fine aggregate in a number of studies. Examples of these studies on fine aggregate replacement include those done by Corinaldesi et al. (2002), Corinaldesi and Moriconi (2002), and Poon and Chan (2007), the key findings of which are presented in the following paragraphs.

The use of crushed brick as fine aggregate, as well as the use of brick fines obtained from the crushing process, in mortar was explored by Corinaldesi et al. (2002). In these mixtures, polypropylene and stainless steel fibers were added to study the effects on compressive strength, flexural strength, bond strength and shrinkage. Stainless steel fibers provided improved mechanical properties, while the polypropylene fibers did not have beneficial effects. It was concluded that, in many circumstances, the cost of the addition of fibers would be offset by the lower cost of the brick aggregates. Corinaldesi and Moriconi (2002) performed additional studies on the use of brick aggregates in normal concrete mixtures as well as the addition of brick fines to self-consolidating concrete. Both types of mixtures developed as part of this study were deemed to have acceptable performance according to the authors.

Poon and Chan (2007) evaluated concrete mixtures in which up to 20% of the fine aggregate content was replaced by crushed brick waste and crushed tile waste (in separate mixtures). In this study, the two-stage mixing process was used and resulted in lower initial slumps and reduced slump loss. Poon and Chan (2007) indicate that mixtures in which the two-stage mixing process was used had higher compressive strengths than those that were prepared using the traditional mixing process. Reductions in density, compressive strength, and modulus of elasticity correlated with increases in the percentage of recycled aggregate used. Poon and Chan (2007) state that, based on their studies, "coarse and fine recycled aggregate should not be used together to entirely replace both the coarse and fine natural aggregate in concrete mixes because the strength and durability of concrete would be adversely affected. Nevertheless, the coarse and fine recycled aggregate could be used separately to replace the coarse and fine natural aggregate, respectively, in different concreting projects under a variety of circumstances."

Debieb and Kenai (2008) performed testing on brick aggregate concrete that used crushed brick as partial or complete replacement for both coarse and fine aggregates. Hardened concrete testing was performed to assess the performance of brick aggregate concrete compressive and flexural strengths, as well as porosity, water absorption, and shrinkage. It was found that "the percentage of entrained air increases as the percentage of substitution of recycled aggregates increases." Due to the high absorption of the brick material, workability was an issue, and some segregation of the 100% coarse and 100% fine aggregate replacement mixture occurred. Compressive strengths of the brick aggregate concrete mixtures were lower than the normal aggregate concrete, with greater strength reductions occurring with higher levels of brick aggregate replacement. Similar reductions in the modulus of elasticity were also observed to correlate with an increasing percentage of brick aggregate. Use of a plasticizing admixture helped increase strengths by improving workability while reducing water content of the mixtures. The results obtained by Debeib and Kenai (2008) in this study contradicted the results of Akhtaruzzaman and Hasnat (1983) in that flexural strengths of the brick aggregate concrete were lower than normal aggregate concrete.

In further studies, Debeib and Kenai (2008) determined that, for brick aggregate concrete containing crushed brick as a fine aggregate replacement, "shrinkage at early age is almost six times higher than for the natural aggregates concrete and continues with the same rate of increase up to 90 days. On the other hand, when both coarse and fine crushed brick are used together as aggregates, the shrinkage of the recycled concrete is stabilized at early age (up to 7 days) and becomes comparable with that of the natural concrete. At later age, shrinkage of recycled aggregates concrete is again higher than that of the natural concrete." Additional study to clarify these findings is recommended by the authors. Debeib and Kenai (2008) concluded that, overall, the brick aggregate concrete produced as part of this testing program had performance characteristics similar to natural aggregate concrete when the replacement of crushed brick was limited to 25% or less for coarse aggregate and less than 50% for fine aggregate.

In a few studies, such as those by Pakvor et al. (2004), Chen et al. (2003), Jones et al. (2004), Levy and Helene (2004), de Brito et al. (2005), Poon and Chan (2006) and Cachim (2009), RBMA was used as a partial or complete replacement for fine and/or coarse aggregate in concrete and mortar. The findings of these studies are particularly relevant to the work performed as part of the research work reported here, as RBMA includes the mortar fraction of the recycled brick masonry, as well as the potential for contaminant material to be present. The extent of testing performed in these studies varies, and the key details related to the testing protocol and findings are presented in the following paragraphs.

The performance of RBMAC mixtures that had varying gradations of fine and coarse brick aggregate were compared by Pakvor et al. (1994). The brick aggregates used were obtained from a demolished structure, and therefore likely included some mortar fraction, although the percentage of mortar was not indicated. Compressive strength, flexural strength, modulus of elasticity, and shrinkage test results were compared. Pull-out tests were performed to assess the bond between the RBMAC and steel reinforcing bars. Additionally, several reinforced RBMAC beams were cast and tested to evaluate response to load. The authors conclude that "reinforced structures can be successfully made using crushed brick, all in accordance with the principles of reinforced concrete theory and practice."

Chen et al. (2003) performed a study of RAC and mortar mixtures in which the recycled aggregate consisted of both recycled concrete and recycled brick and tile. The percentage of recycled brick and tile included in the overall recycled aggregate content was varied. This study found that at the replacement levels used (0 to 67% of RCA replaced by RBMA), effects on compressive strength, flexural strength, and modulus of elasticity were limited. Chen et al. (2003) also studied the effects of using both unwashed and washed recycled aggregate in concrete mixtures. Using unwashed recycled aggregates resulted in lower strengths, particularly flexural strength, with a more readily observable effect at lower water/cement ratios.

Jones et al. (2004) used RBMA as fine and coarse aggregates in precast concrete masonry blocks. The recycled masonry aggregate was obtained from a demolition operation, and although primarily composed of brick and mortar, contained a significant amount of plaster, tile, glass, metal, paper and wood. This study included testing for the total sulfate content of the recycled aggregate via acid extraction. It was shown that precast concrete blocks of acceptable compressive strength could be created using this recycled masonry aggregate.

RBMA was used as a fine and coarse aggregate replacement in concrete produced by Levy and Helene (2004). The authors state that "concrete made with recycled aggregates (20%, 50%, and 100% replacement) from old masonry or from old concrete can have the same fresh workability and can achieve the same compressive strength of concrete made by natural aggregates in the range of 20 to 40 MPa (2,900 to 5,800 psi) at 28-days." Research by de Brito et al. (2005) on RBMA made from crushed ceramic brick from building partition walls showed (in some cases) adequate compressive strength, flexural strength, and abrasion resistance for use in pavement applications.

Poon and Chan (2006) studied the use of RCA and RBMA as replacement for 70 to 100% of normal aggregate in concrete used to form paving blocks. Paving block specimens produced using brick aggregate concrete were tested for density, compressive strength, splitting tensile strength, transverse breaking load, skid and abrasion resistances, and absorption. Crushed brick used as aggregate for this study contained "a high amount of adhered brick mortar and other impurities such as tile, wood, and dust," which led to a high absorption value and subsequent water demand of the concrete mixtures. The inclusion of such contaminants may be responsible for the significant reduction in the strengths of the recycled aggregates (alone) after soaking. Fly ash was used as a supplementary cementitious material in some of the mixtures, and a lower water demand for these mixtures was "attributed to the effect of the replacement of the coarse and fine aggregates by the fine fly ash particles which filled the voids within the mixture."

The mixtures prepared by Poon and Chan (2006) had brick aggregate replacements varying from 25% to 75%, with the rest of the aggregate being RCA. The mixtures that did not contain fly ash had 28-day compressive strengths ranging from 24.6 to 39.6 MPa (approximately 3,570 to 5,740 psi). Splitting tensile strength results were not provided for these mixtures. The mixtures containing fly ash resulted in 28-day compressive strengths ranging from 39.2 to 55.4 MPa (approximately 5,685 to 8,033 psi) and splitting tensile strengths of 2.4 to 2.8 MPa (approximately 350 to 400 psi), respectively. They concluded that "it was feasible to produce paving blocks prepared with 25% crushed clay brick that satisfied the compressive strength requirement for paving blocks (for trafficked areas in Hong Kong)."

Cachim (2009) performed a study on the mechanical properties of RBMAC, with RBMA replacement of natural coarse aggregates at levels of 15% and 30%. The RBMA was tested to determine the initial absorption capacity, measuring absorption percentages for the first 2 minutes and 5 minutes, in order to determine "a curve of the evolution of the water absorption with time." He found that at least 75% of total water absorption by the RBMA occurred within the first 2 minutes, while at least 91% of the total water absorption occurred within the first 5 minutes. Cachim used this information to establish a mixing protocol that eliminated pre-saturation of the aggregates. The w/c ratio was computed without including water absorbed by the aggregates. Then, during mixing, additional water equal to a certain percentage of the total absorption of the brick aggregates was added in order to ensure that “aggregates absorbed part of the water and that the pores were at least partly saturated” prior to adding cement. This was done in lieu of pre-soaking the aggregates, as has been done in many previous studies. Cachim’s (2009) results showed reductions in compressive strengths for the brick aggregate concrete that were similar to the findings of de Brito et al. (2005) and Akhtaruzzaman and Hasnat (1983), although Cachim (2009) states that “the attained strength showed that crushed bricks can be used to substitute natural aggregate in concrete (24.5 to 38.5 MPa (approximately 3,550 to 5,580 psi) at 28 days) for percentages about 15% to 20%.”

2.3.2 Durability of Brick Aggregate Concretes and Mortars

Although durability concerns related to brick aggregates and RBMA in concrete and mortar are cited by a number of researchers, relatively little work has been done in this area. Many stakeholders are reluctant to use any RAC (including RBMAC) due to concerns about its durability performance, and in order for this material to become a viable alternative to natural aggregate concrete, further understanding in this area will be critical. Studies that were identified as part of this literature review that included durability performance testing of brick aggregate concrete and RBMAC have typically focused on permeability, freeze-thaw resistance, and susceptibility to chemical attack.

Permeability is a key parameter influencing the durability performance of concrete and cementitious materials. High air and water permeability facilitate ingress of deleterious substances, such as chlorides, into the concrete. Increased porosity and permeability of brick aggregates and mortar included with RBMA can cause brick aggregate concrete and RBMAC to exhibit higher permeability, and hence reduced durability performance. Some research (Padmini et al. 2002, Zakaria and Cabrera 1996, Olorunsogo and Padayachee 2002) has supported this, with test results indicating that brick aggregate concrete exhibits higher permeability than concrete made with natural aggregates, sometimes up to 50% higher. After assessing the permeability of brick aggregate concrete, Khalaf and DeVenny (2002) established new tests to assist in calculating the porosity and water absorption of brick aggregates. The authors recommended a vacuum saturation test for porosity and a modified 5 hr boiling test for absorption.

Although the high porosity of brick particles contributes to higher permeability, the porosity of the particles has also been shown to provide enhanced durability performance in freeze-thaw testing. Most studies in this area have been performed on very small brick particles used as a partial replacement for fine aggregate. Litvan and Sereda (1977) added brick particles approximately 0.5 mm in size to mortar and concrete mixtures in order to assess whether the porosity of the brick would enhance freeze-thaw durability. Testing was performed in accordance with ASTM C666, and ASTM C456 procedures to characterize the air void parameters including spacing factor. It was found that incorporation of small particles (0.4 mm to 0.8 mm) with high porosity improved the freeze-thaw resistance of the mixtures. Bektas et al. (2009) performed tests on crushed brick used as a partial replacement for fine aggregate, and observed that freeze-thaw durability of brick aggregate mortars was increased, and it was indicated that “the highly porous nature of crushed brick might provide a similar air entraining action and reduce the freeze-thaw expansion."

Mulheron and O'Mahony (1988 and 1990) performed freeze-thaw durability tests on concrete made from recycled aggregate that included some fraction of clay brick material. Additional information characterizing their recycled aggregate, including the brick fraction, is presented in a subsequent study (Mulheron and O'Mahony 1990). They concluded that, although the concrete mixtures with the recycled aggregates had lower strengths and densities than concrete made with natural aggregates, "the durability of concretes manufactured with these recycled aggregates is similar to that of conventional concretes."

The susceptibility of RBMAC to chemical attack, as well as problems with secondary chemical reactions in the hardened state, is attributed by several researchers to components such as chlorides and sulfates incorporated in the RBMA and in contaminant materials introduced during demolition and processing (Sherwood 1995 and Hansen 1992). Sulfate attack is caused by secondary ettringite crystals forming in the hardened concrete or mortar matrix, causing cracking. In his post-World War II studies, Newman (1946) cautioned about use of brick aggregate concrete in moist or wet environments due to the inherent sulfate content of the bricks and indicates that care should be taken to minimize the mortar fraction included with brick aggregates in the concrete mixtures. Sulfates are of particular concern due to the common intermingling of brick and gypsum plaster (sheetrock) in demolition waste.

Reactions between certain soluble silica materials and alkalis in the hardened cementitious matrix can cause formation of an expansive gel, which causes cracking.

This reaction, known as alkali-silica reactivity or ASR, may also be a potential problem depending on the form of silica in the bricks. ASR testing performed by Litvan and Serada (1977) indicated that some brick particles may be susceptible to reaction with sodium and potassium alkalis from the cementitious matrix. Based on expansions observed in the ASR testing, Bektas et al. (2009) propose that a pessimum content of brick aggregate (corresponding to the highest measured expansions) in the mortar occurs at around 30%. Lower expansions occurred at replacement percentages of brick both higher and lower than 30%. The authors recommend further study to clarify the findings of this portion of the study. In a similar study on concrete made using crushed tile coarse aggregate, Khaloo (1995) noted that the high alkali content of the tile could make it susceptible to ASR.

Other studies that include durability testing have been performed on brick aggregate concrete. Kibriya and Speare (1996) performed testing on brick aggregate and natural aggregate concrete mixtures to compare mechanical properties (compressive strength, tensile strength, flexural strength, modulus of elasticity), time-dependent properties (creep and shrinkage) as well as durability performance in chloride ion diffusion, freeze-thaw, and sulfate attack tests. Water reducing admixtures were not used, resulting in relatively high-permeability paste matrices. Chloride diffusivity of the brick aggregate concrete mixtures were greater than the control natural aggregate mixture. However, the brick aggregate concrete mixtures performed better than the control mixture in both the freeze-thaw testing and the sulfate attack testing. It is noted that the freeze-thaw tests and the sulfate attack tests performed by Kibriya and Speare (1996) monitored the reduction in the dynamic modulus of elasticity of the specimens rather than the performance characteristics typically monitored for these tests in the United States (mass loss for freeze-thaw resistance and percent expansion for sulfate attack).

Another study was performed by Levy and Helene (2004). This study focused on the durability performance of concrete made using recycled concrete, masonry, and brick aggregates obtained from demolished structural components. These aggregates included some mortar fraction. Testing was focused on three characteristics specifically linked to the durability performance of concrete: water absorption (permeability), porosity, and carbonation. Results indicated that the lowest water absorption and porosity for the recycled aggregate concrete mixtures was observed at 20% replacement for both the concrete containing RCA and the RBMAC. Testing also indicated that the carbonation depth decreases with increasing percentages of coarse and fine recycled concrete and masonry aggregates. The authors link this performance to the increased cement contents of these mixtures (compared to the reference mixtures prepared with natural aggregate), with the higher cement concrete responsible for the formation of a denser paste microstructure, as well as providing an "alkaline reserve" capable of resisting carbonation. Overall, Levy and Helene (2004) conclude that "when the natural aggregate is replaced by 20% of the recycled aggregates from old concrete or old masonry, the resulting recycled concrete will likely present the same, and sometimes better, behavior than the reference concrete made with natural aggregates in terms of the properties studied in this investigation."

Recently, the behavior of concrete members in fire is of increasing concern. The coefficient of thermal expansion (CTE) and coefficient of thermal conductivity of concrete is greatly influenced by the same properties of its constituents, including the aggregates (Riley 1991). Brick aggregates have the potential to enhance the fire performance of concrete because its thermal expansion and conductivity may lead to lower internal stresses at high temperatures, as well as a reduced transfer of heat through its mass. Khoury (1992) cites data from previous research performed by others that indicates that "concrete containing brick aggregate (coarse and fine) exhibited no loss in the residual (i.e. after cooling) compressive strength for test temperatures up to 600°C."

2.3.3 Pozzolanic Reactions and Microstructural Characteristics of Brick Aggregate Concretes and Mortars

The pozzolanic nature of natural clays has been used to create hydraulic mortars for centuries. According to Baronio and Binda (1997), "the two fundamental characteristics of pozzolans are usually defined as: (a) ability to react with lime, (b) ability to form insoluble products with binding properties." Incorporation of brick aggregate into concrete and cementitious materials can provide supplemental and enhanced hydration reactions to strengthen and enhance the performance of the resulting product. This could be particularly beneficial in brick aggregate concretes and mortars that contain crushed brick as fine aggregate.

Böke et al (2004 and 2006) performed a study on brick used as aggregate in plaster and brick-lime mortar in historic brick bath houses built during the Ottoman Empire. The materials were studied at a microscale level, and their results indicated that the mortars contained "crushed brick powders that have good pozzolanicity," chiefly attributed to the amorphous clay mineral dissolution products. The pozzolanic nature of the finely ground brick resulted in beneficial hydration products formed between the lime binder and the brick aggregates “at the brick-lime interface and the pores of the bricks.”

The potential pozzolanicity of modern brick, has been questioned due to the relatively higher firing temperatures used in recent decades. Baronio and Binda (1997) studied the pozzolanicity of both ancient and modern bricks, finding that the finely-ground clay minerals, mainly silicates and aluminates, are pozzolanic “when burnt at temperatures in the range of 600-900°C.” However, when bricks are fired at temperatures greater than 900°C (1652°F), they do not display pozzolanicity. Most modern bricks are fired at temperatures greater than 900°C (1652°F), and therefore Baronio and Binda (1997) conclude that “modern bricks are seldom pozzolanic, not only because they are fired at high temperature, but also because they can be made of materials which do not contain or have a low content of clays.”

Conflicting results, however, have been found by a number of researchers using brick fines, or fine brick material that is incorporated into the cementitious material with larger brick particles. Hamassaki et al. (1996) used crushed recycled brick as fine aggregate in rendering of masonry mortar. When compared to mortars prepared with natural aggregates, the brick aggregate mortars had higher compressive strengths. Klimesch and Ray (2001) and Klimesch et al. (2003 and 2004) used brick fines along with portland cement and quartz sand to manufacture block products using an autoclave for curing. In their study, they determined that brick fines can be used as a partial replacement for sand, with blocks having compressive strengths that were comparable or better than mixtures without the brick fines. Other studies on brick aggregate concrete (Khatib 2005, Zakaria and Cabrera 1996) and on concrete block (Schuur 2000) that incorporated brick fines also found higher compressive strengths at later ages for the cementitious materials that incorporated brick fines. Zakaria and Cabrera (1996) indicate that this finding is in agreement with researchers in Germany (Shulz and Hendricks 1992), who "attributed this to the pozzolanic effect of the finely ground portion of the burnt brick."

Other studies have been performed on the microscale to assess pozzolanicity (Marrocchino et al. 2004), and establish the mineralogic characteristics that promote hydration and a denser interfacial transition zone between the cement paste and the brick particles (Corinaldesi et al. 2002). A better understanding of the brick aggregate-paste interface, and reactions that occur in this region, is required in order to facilitate confidence in our understanding of the potential performance of brick aggregate concretes and mortars.

2.4 Potential for Widespread Acceptance and Use of Brick Aggregate Concretes and Mortars

Given the current economic and social climate, additional research to understand the performance of brick aggregate used in cementitious materials is both timely and imperative. Benefits other than a reduced amount of landfilled waste can be realized if brick rubble from construction and demolition waste can be reused as aggregate in concrete and other cementitious materials. Brick aggregate concrete mixtures are lighter than similar normal aggregate concrete mixtures, which can significantly reduce the self-weight of a concrete structure. Khalaf (2006) notes that a key advantage of brick aggregate concrete is its relatively low density, and "by using concrete of low density, smaller sections (for structural members) can be used and consequently the size of foundations can be reduced, representing a financial savings."

Based on the papers surveyed as part of this literature review, it is easier to predict increased acceptance of this brick aggregate and RBMA concretes in locales outside of the United States, particularly in developing areas of the world where sources of natural aggregates are scarce and/or landfill space is limited. Due to the capacity of landfills in Hong Kong being overcome within the foreseeable future, use of recycled aggregates (particularly RCA) is being promoted extensively (Poon and Chan 2007). In some countries, such as South Africa, illegal dumping of C&D waste is becoming a problem in some locales as some entities attempt to avoid landfill tipping fees (Kutegeza and Alexander 2004).

Several studies (Lopez et al. 2011, Jankovic et al. 2012) indicate that, worldwide, the potential for use of RBMA in low-grade concrete may be enticing. Chitaranjan (1997) used demolished brick masonry as aggregate for manufacturing hollow blocks and bamboo-reinforced roof panels in a study on techniques that could be used for economical construction of mass housing. It is also possible that other reuse applications may divert this waste material from concrete applications, however. For example, Galbenis and Tsimas (2004) have been performing research to characterize recycled concrete aggregates and recycled brick aggregates to assess their suitability for use as raw materials in the manufacture of cement.

Recently, in the United States and abroad, the concept of internal curing has been of interest to many in the concrete industry. Internal curing occurs when water contained in a high-absorption aggregate, typically fine aggregate, provides a reservoir of water that promotes hydration reactions after conventional curing measures (misting, wet burlap on surface, etc.) have been stopped or removed. Several researchers have suspected that the high absorption and porosity of RBMA provided internal curing benefits in portland cement concrete and mortar batched in their studies. For example, Cachim (2009) suspected that in his study, the saturated brick aggregates provided a measure of internal curing due to the water available to facilitate hydration after the mixture has used up the readily available mix water. Cachim concludes "Thus, the moderate use of saturated bricks in concrete act as [a] self-curing agent for concrete… The correct balance between the reduced strength of crushed brick aggregate sand and its effectiveness as [a] self-curing agent is the key point to use them as aggregates in concrete."

Other studies have shown the potential of brick aggregate to contribute to internal curing as well. Bektas et. al. (2009) performed studies on durability characteristics of mortar produced using crushed clay brick aggregate. In mortar mixtures prepared as part of this study, the fine aggregate was replaced at levels of 10% and 20% with crushed brick aggregate. The authors stated that "mortar containing 10% crushed brick showed the highest shrinkage whereas the 20% crushed brick replacement resulted in the lowest shrinkage value," attributing this to the brick aggregate acting as an internal curing agent. Kumar et al. (1988) performed testing on mixtures that contained brick aggregate, "muck" aggregate from demolished buildings that presumably contained a variety of materials, and natural aggregate concrete. Test results indicated that shrinkage was delayed because of the increased moisture content of the brick aggregate. Khatib (2005) also found that increased percentages of brick aggregate result in lower shrinkage.

Interestingly, when specimens were placed in water for an extended period, an expansion occurred, attributed by the author to formation of a pozzolanic gel that imbibes water.

As mentioned earlier, in order to feel comfortable with use of RBMA, similar steps of testing and field implementation are necessary. In the following sections, practical challenges that inhibit widespread acceptance and use of brick aggregate and RBMA in concrete are discussed. Economic considerations are also outlined.

2.4.1 Practical Challenges

Robinson et al. (2004) state that factors limiting the reuse of concrete as aggregates (and, it can be presumed, the reuse of brick and recycled brick masonry as aggregates) include "processing costs, quality and performance issues, and lack of large quantities where needed." Performance factors that limit the use of recycled concrete aggregate include specific gravity, absorption, soundness (resistance to environmental conditions such as chemical and physical weathering), gradation (grain-size distribution), and contaminant solubility and the potential for groundwater contamination." The authors indicate that external factors that influence the limited use of recycled aggregates include "cost, state specifications, and environmental regulations."

Use of demolished brick and concrete masonry as concrete aggregates is complicated due to the presence of contaminants in the demolition debris. Bricks may be contaminated with plaster and sheetrock. Fragments of concrete block and timber may be incorporated into the demolition debris, and the fragments of brick inherently include some fractions of attached and detached mortar (Tam and Tam 2006). The increased prevalence of on-site separation of demolition waste is helping this situation. In addition to source-separation techniques that have been shown to provide relatively "clean" material (Tempest et al. 2010), several processes (wet, dry, and thermal) exist to remove impurities from crushed masonry demolition rubble (Hansen 1992).

The mortar fraction present in RBMA will vary based on the source construction, and influences the performance of RBMAC. In order to assist in evaluating RBMA for use in RBMAC, it is likely that the quantity of mortar (as well as its quality) will need to be characterized. Methods used to quantify the amount of mortar present in recycled aggregates vary. Currently, removal of the mortar from crushed brick is accomplished for both testing and reuse using one of several means. Using a thermal method, interfacial stress is created when mortar-covered brick is saturated and then heated to several hundred degrees Celcius, resulting in cracking and separation of the mortar from the brick aggregates (Tam and Tam 2006). Recently, De Juan and Gutierrez (2009) performed a study that compared three methods that can be used to quantify the mortar content of recycled concrete aggregates: acid treatment, encapsulation of the aggregates in new colored concrete (and subsequent visual analysis), and a thermal treatment similar to that used by Tam and Tam (2006). De Juan and Gutierrez (1999) indicated that the thermal treatment provides the most reliable results.

Ultimately, to promote widespread use of RBMA, guidelines for characterizing RBMA need to be established. Guidelines for the assessment of RBMA for suitability for use in concrete would provide some assurance to stakeholders. The characteristics and quality of the original brick weigh heavily on the performance of recycled brick aggregate, and this quality is dependent on the initial source mineralogy as well as the firing process. These guidelines have not been established. However, Luco and Castellote (2004) provide an outline of criteria that need to be investigated in order to assess the suitability of a waste material for inclusion in a cementitious matrix. The authors indicate that the key considerations include:

- geometrical and physical considerations (size, shape, texture, density, porosity, and absorption)
- effects of a waste material on the properties of concrete in its fresh (unhardened) state (water demand, workability, segregation of mixture components, initial and final set times, heat of hydration)
- effects of a waste material on the properties of concrete in its hardened state (strength, modulus of elasticity, CTE, drying shrinkage), and
- effects of a waste material on durability of the hardened concrete (porosity, permeability, freeze/thaw resistance, resistance to chemical attack, leaching of contaminants contained within the waste).

2.4.2 Economic Considerations

Marrocchino et al. (2004) state that "recycling strategies" need to be "ecologically correct and economically advantageous." In some areas of the United States, it is becoming increasingly difficult to establish new natural aggregate quarries due to zoning restrictions and public outcry (ECCO 1999). As landfill fees and quarrying costs increase, the potential for acceptance of RBMA for use in concrete applications will also increase.

In their article "Recycling of construction debris as aggregate in the Mid-Atlantic Region, USA," Robinson et al. (2004) present a compilation of information pertaining to reuse of reclaimed asphalt pavement and RCA in Maryland and Virginia. The paper explores "the cost, availability, and performance factors of recycled aggregate relative to natural aggregate," related to use of demolished masonry rubble, particularly brick, used in concrete as a replacement of virgin natural coarse aggregate material. The authors indicate that RCA is economically viable in areas where its unit cost is comparable to the unit cost of natural aggregate. If the RCA is produced on-site using portable equipment, transportation costs can be minimized, and costs may be comparable to conventionally produced concrete.

The introduction of a new aggregate source such as RBMA to a ready-mixed or precast concrete plant results in additional costs. Similar to the characteristics identified by Luco and Castellote (2004), Brown (1996) lists characteristics that need to be considered (and potentially tested) in order for potential recycled and secondary aggregates to be successfully used in concrete: geometric properties (gradation, shape, fines content, particle density, absorption), physical properties (strength, freeze/thaw resistance, abrasion resistance, resistance to polishing, volume stability), and chemical properties (chloride content, sulfate content, contaminants, alkali reactivity). Costs associated with this testing and characterization of RBMA and RBMAC would be incurred by the concrete producer and/or material supplier. In addition, Brown (1996) warns that aggregates that can only be used within a narrow range of a concrete producer's product line may not receive consideration due to the economic hurdles imparted by use of additional storage and batching system components.

2.5 Research Needs

Although research has been done with regard to the use of RCA in concrete, far fewer studies have focused on the performance of concrete made with other recycled aggregates, such as brick aggregates. Worldwide, very little research has been done on concrete and mortar produced with RBMA. No studies on RBMA or RBMAC have been performed in the United States. Many studies on RBMAC (performed abroad), surveyed as part of this literature review, focused on producing concrete paving blocks and other non-structural elements. Local (North Carolina and/or Southeastern United States) brick was not used in the studies reviewed. A database of material properties of RBMA and RBMAC produced in the United States does not exist, and would be useful to designers considering use of these materials in sustainable construction.

Although tests of RBMAC in simple structural members (beams) have been performed, the potential use of RBMAC in pavement applications has not been studied. A key consideration for use in pavement applications, in addition to strength, is the abrasion resistance of RBMAC. The abrasion resistance of RBMAC has not been extensively studied using methods currently used in the United States. Abrasion testing using ASTM C944 (rotary cutter method) has drawn increased attention by the FHWA to evaluate concrete used in transportation applications, and this test method was not used in any of the studies in the literature.

Variations in the performance of RBMAC due to the use of different compositions of RBMA have not been extensively studied. The percentage of mortar included in the RBMA was generally not reported in the literature, and the influence of mortar attached to the brick aggregate on the fresh and hardened properties of brick aggregate concrete has been studied by relatively few. Attached mortar is a source of sulfate (Newman 1946, de Juan and Gutierrez 2009, among others), and significantly influences the absorption of the brick aggregate. The role of the mortar fraction in recycled brick aggregates needs to be further defined in order to make brick aggregate concrete a viable construction material. De Juan and Gutierrez (2009) provided relationships between mortar content and aggregate fraction size, mortar content and absorption, mortar content and sulfate content, and mortar content and abrasion resistance. This study did not, however, link the mortar content to the test results of other mechanical or durability properties.

Shrinkage of RBMAC has not been included in many studies, and the possibility of RBMA being used to provide internal curing has not been well evaluated. Additionally, a possible improvement in mechanical properties could be realized by incorporation of an optimized RBMA gradation in the concrete, which has not been investigated in previous studies.

The effects of the addition of supplementary cementitious materials (such as silica fume, fly ash, and ground granulated blast furnace slag) and chemical additives (such as silicate densifiers) on RBMAC have not been thoroughly investigated. Additionally, admixtures (such as water reducers and viscosity modifiers) were not often utilized in previous studies. Specifically, the effects of these materials (and admixtures) on the fresh properties of the mixture and on the mechanical properties and durability performance of the hardened RBMAC need to be identified and better understood.

Durability performance of RBMAC, particularly with regards to freeze-thaw behavior (using current ASTM test methods) and chemical attack (particularly sulfate attack and secondary ettringite formation), has not been extensively studied. Microstructural characterization of the paste and brick aggregate bond, including pozzolanic effects between brick particles and cement paste has not been studied to an extent where it is adequately understood. Thermal characteristics of RBMA may lead to advantageous performance in concrete pavement applications. Similarly, the thermal characteristics of brick aggregates may potentially provide increased fire resistance of brick aggregate concrete and RBMAC. However, based on the literature, fire resistance testing has not been performed on RBMAC.

CHAPTER 3: TESTING PROGRAM FOR CHARACTERIZATION OF RECYCLED MATERIALS

3.1 Overview of Recycled Materials

Recycled brick masonry aggregate (RBMA) used in this work was obtained from a single demolition site, Idlewild Elementary School in Charlotte, North Carolina. This demolition site was used as a case study for research on the reuse of construction and demolition waste, as part of a Department of Energy grant, DOE Project #DE-FG26-08NT01982, “Building Materials Reclamation Program.”

3.1.1 Demolition Site

As stated previously, the RBMA for this study was obtained from a single demolition site, Idlewild Elementary School in Charlotte, North Carolina. Effort was made to obtain the ‘cleanest’ demolished brick masonry that can be achieved via readily employable source separation techniques. A typical brick masonry wall is shown in Figure 3-1. Walls are mainly clay brick, with clay tile comprising the lower portion of some walls.

Figure 3-1: Cafeteria at Idlewild Elementary School prior to demolition.

3.1.2 Demolition Sequence and Material Handling

Impurities in the brick aggregate were minimized by ensuring components such as sheetrock, acoustical tile, roof material, and other interior building components were removed prior to demolition of the brick masonry walls. The demolition contractor, D. H. Griffin Co., utilized the “top down” demolition sequence in which the concrete slab-on-grade was allowed to remain in place while other building components were sequentially demolished from the roof down (shown in Figure 3-2). This facilitated separation of building components on a “clean” surface as they were demolished, and the brick masonry was removed with minimal contamination from other building components. The brick masonry that was used in this study was monitored by UNC Charlotte personnel during the demolition process, during loading and transport to a local crushing yard, and during transport to UNC Charlotte laboratories.

Figure 3-2: Demolition of the brick masonry walls at Idlewild Elementary School.

3.1.3 Obtaining Whole Clay Brick and Tile Specimens

Prior to transport to the crushing and grading facility, samples of whole brick and whole clay tile were obtained from the demolished rubble in order to determine properties in accordance with ASTM C67, “Standard Test Methods for Sampling and Testing Brick and Structural Clay Tile.” The whole brick and whole clay tile samples were returned to UNC Charlotte’s laboratories, where most of the attached mortar was removed by mechanical means.

3.1.4 Obtaining Recycled Brick Masonry Aggregate

The RBMA material used for this study was produced at D.H. Griffin Crushing and Grading, in Charlotte, North Carolina. This facility specializes in creating crushed aggregate material from construction and demolition waste. Materials typically produced at this facility are recycled concrete aggregate and recycled aggregate comprised of mixed rubble (concrete masonry, brick masonry, and concrete).

To facilitate production of a “clean” RBMA, the demolished brick masonry material was transported to the facility in a separate dumptruck, without intermingling the material with other construction waste. This operation is shown in Figure 3-3. Once the demolished brick masonry arrived at the facility, it was stockpiled separately from other materials, as shown in Figure 3-4.

Figure 3-3: Demolished brick masonry from Idlewild Elementary School, loaded into a separate dumptruck for transport to D.H. Griffin Crushing and Grading.

Figure 3-4: Demolished brick masonry from Idlewild Elementary School, stockpiled at D.H. Griffin Crushing and Grading, prior to being crushed into aggregate.

The recycled brick masonry was crushed in the rotary crusher apparatus (shown in Figure 3-5), and was subsequently mechanically separated into three gradations: AASHTO M43 #78, AASHTO M43 #67, and smaller size material that may have a gradation similar to ASTM C33 sand. For this study, the crushed brick masonry aggregate was the material mechanically separated into the AASHTO M43 #78 gradation (nominal size ½” to No. 8). As can be seen in Figure 3-6, the #78 gradation material came off of the conveyor belt and was subsequently shoveled into barrels and returned to UNC Charlotte laboratories.

Figure 3-5: Rotary crusher apparatus at D.H. Griffin Crushing and Grading.

Figure 3-6: After falling from the conveyor belt, recycled brick masonry aggregate was shoveled into barrels and returned to UNC Charlotte laboratories.

The crushing operation generates fine material that clings to the aggregates. These fines were not washed off of the recycled brick masonry aggregate at the D. H.

Griffin Crushing and Grading facility. However, this fine material was washed from the RBMA when it was returned to UNC Charlotte's laboratories, prior to testing.

3.2 Whole Clay Brick and Tile

Due to differences in source mineralogy and production processes, the mechanical properties and thermal characteristics of clay brick and tile produced in different locations can vary greatly. In order to provide information regarding the demolished clay brick and tile used in the RBMA produced from Idlewild Elementary school, whole clay brick and tile were taken to UNC Charlotte for laboratory testing.

3.2.1 Experimental Procedures

Tests to determine the mechanical properties and thermal characteristics of the whole clay brick and tile were performed as outlined in the following sections. Prior to testing, adhered mortar was removed using mechanical means and plastic bristle brushes. Care was taken not to damage the specimens during removal of the mortar, and therefore trace amounts of mortar remained on some areas of some of the bricks and tiles tested. Typical test specimens are shown in Figure 3-7 and in Appendix A (Figures A-1 and A-2).

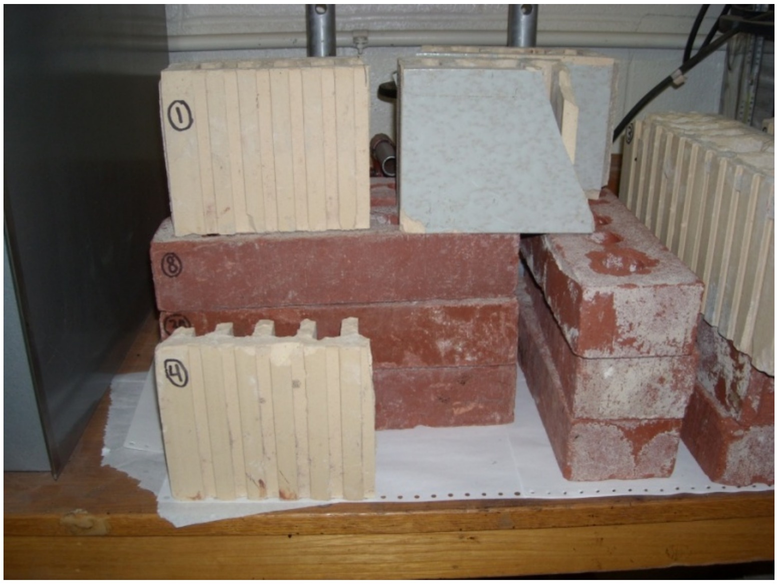

Figure 3-7: Whole brick and clay tile subjected to testing.

3.2.1.1 Unit Weight

Unit weight of whole clay brick and tile specimens was determined in general accordance with ASTM C67, "Standard Test Methods for Sampling and Testing Brick and Structural Clay Tile." Specimens were dried in a ventilated oven at 230°F (110°C) until a constant weight was achieved. Specimens were then cooled at approximately 70°F (21.1°C) and relative humidity between 30% and 70% for at least four hr prior to weighing. The oven-dry weights of 31 brick specimens and 6 clay tile specimens were determined. The corresponding unit weights were then computed.

As the clay bricks and tiles obtained from Idlewild Elementary School had cores, it was necessary to account for the volume of cores in determining the unit weight of the clay bricks and tiles. The volume of the cores was obtained by measuring representative bricks and tiles, and these volumes were used for all bricks and tiles when computing their unit weights.

3.2.1.2 Absorption

Absorption of whole clay brick and tile specimens was determined in general accordance with ASTM C67, "Standard Test Methods for Sampling and Testing Brick and Structural Clay Tile." Five whole brick specimens and three whole clay tile specimens were tested utilizing the 24-hr soak procedure outlined in ASTM C67.

3.2.1.3 Suction

Initial rate of absorption, or suction, of the whole clay brick and tile specimens was determined in general accordance with ASTM C67, "Standard Test Methods for Sampling and Testing Brick and Structural Clay Tile." After ambient air conditioning, six whole brick specimens and two whole clay tile specimens were weighed and tested. The procedure measures the weight gain of test specimens submerged in water to a specified depth for a specified period of time. Specimens were placed into water so that it covered the entire test surface (bottom face) and an additional 1/8 in of height, and they were allowed to remain in the water for one minute (±1 second) prior to reweighing. The weight gain is then corrected to a basis of 30 in^2 for reporting purposes.

3.2.1.4 Compressive Strength

Compressive strength of the whole clay brick and tile specimens was determined in general accordance with ASTM C67, "Standard Test Methods for Sampling and Testing Brick and Structural Clay Tile." After conditioning, six whole bricks and three whole tiles were sawcut in half and a sulfur capping compound was applied. Compressive strength testing was performed utilizing a UTM machine. As specified in ASTM C67, the speed of testing was controlled in a manner in which half of the failure load was applied at a convenient rate, and then the remaining load was applied at a uniform rate that resulted in the specimen failing in not less than 1 min but not more than 2 min.

As the clay bricks and tiles obtained from Idlewild Elementary School had cores, it was necessary to account for the volume of cores in determining their compressive strengths. The volume of the cores was obtained by measuring representative bricks and tiles, and these volumes were subtracted from the gross volumes of the specimens to adjust resulting compressive strengths.

3.2.1.5 Modulus of Rupture

The modulus of rupture of whole clay brick and tile specimens was determined in general accordance with ASTM C67, “Standard Test Methods for Sampling and Testing Brick and Structural Clay Tile.” After conditioning, five bricks and two clay tiles were tested.

3.2.1.6 Thermal Characteristics

Thermal characteristics of the brick and clay tile obtained from Idlewild Elementary School were determined using the procedures outlined in the subsequent sections.

3.2.1.6.1 Coefficient of Thermal Expansion

There is not currently an accepted method for performing testing to determine the CTE for brick materials. Currently, the accepted test method for determining the CTE for concrete is AASHTO T336, “Standard Method of Test for Coefficient of Thermal Expansion of Hydraulic Cement Concrete.” Equipment required to perform AASHTO T336 is not currently available at UNC Charlotte. Therefore, testing to determine a reasonable value for CTE of a portion of a brick specimen was performed utilizing a methodology that used specimen conditioning and temperature ranges similar to those used in the AASHTO T336 method. Clay tiles were deemed too thin to survive the required mounting of metal reference points, and thus were not tested.

To perform this testing for the brick, reference points were mounted in a piece of brick at an approximate distance of 4 in. This test specimen is shown in Figure 3-8. The test specimen was saturated in lime water at 68°F (20°C) until a constant mass was achieved. An initial length measurement between the reference points was then obtained using a mechanical strain gage apparatus. Prior to obtaining the initial measurement (and subsequent measurements), the mechanical strain gage was calibrated (zeroed at 4 in length) using the reference bar provided by the equipment manufacturer. This was done in lieu of an accepted standard or calibration specimen for this test method.

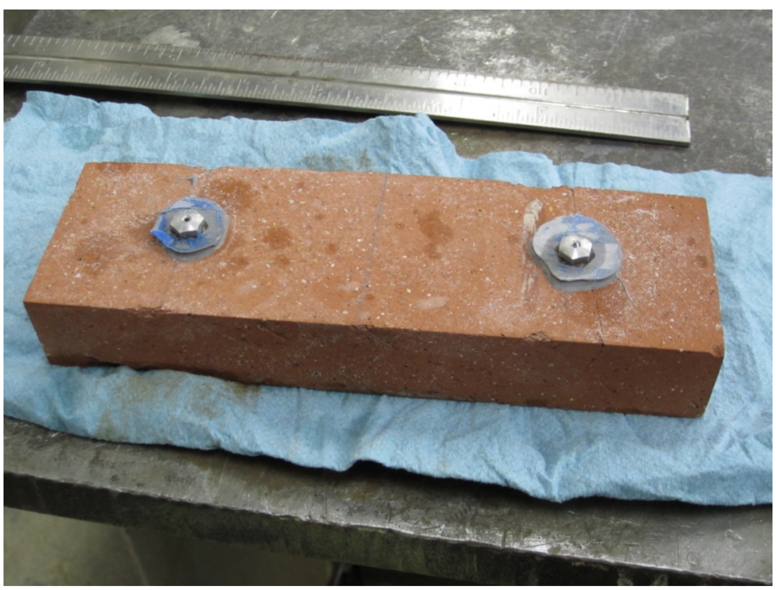

Figure 3-8: Coefficient of thermal expansion testing of brick

The test specimen was then placed back in the saturated lime water, the container sealed, and placed in an environmental chamber. The environmental chamber was set to 122°F (50°C). Once a sensor in the water surrounding the test specimen registered 122°F (50°C), the specimen was allowed to remain in the lime water bath in the environmental chamber (at 122°F, or 50°C) overnight. The following morning, the specimen was removed from the lime water bath, and the mechanical strain gage was used to obtain a length measurement between the reference points. An infrared thermometer was used to obtain the temperature of the specimen at the time of measurement. The specimen was returned to the lime water bath, the container was sealed, and the container was placed back into the environmental chamber. The environmental chamber was then set to 50°F (10°C).

Once the sensor in the water surrounding the test specimen registered 50°F (10°C), the specimen was allowed to remain overnight in the lime water bath in the environmental chamber at 50°F (10°C). The following morning, a mechanical strain gage was again used to obtain the length of the specimen between the reference points, and a temperature reading was taken with the infrared thermometer. The CTE was then computed using the formula

$$CTE = \frac{\left(\frac{\Delta L_a}{L_o}\right)}{\Delta T} \tag{3-1}$$

where ΔL_a is the actual length change of the specimen during the temperature change (measured on concrete surface) ΔT and L_0 is the measured length of the specimen at room temperature. For the three conditioning temperatures used in this test (50ºF, 68ºF, and 122ºF) Eq. 3-1 is modified as

$$CTE = \frac{\left(\frac{L_{122°F}-L_{50°F}}{L_{68°F}}\right)}{T_{122°F}-T_{50°F}} \tag{3-2}$$

where $L_{122°F}$, $L_{68°F}$, and $L_{50°F}$ are the measured lengths of the specimen after conditioning in the 122ºF, 68ºF, and 50ºF environments, respectively, and $T_{122°F}$ and $T_{50°F}$ are the measured surface temperatures of the specimen (at time of length measurement) after conditioning in the 122ºF and 50ºF environments, respectively.

3.2.1.6.2 Thermal Conductivity

Thermal conductivity testing was performed using the TCi apparatus manufactured by C-Therm Technologies, shown in Figure 3-9. In this photograph, the transducer is shown on the table, to the left of the computer. This equipment measures the effusivity of a specimen. Using the measured effusivity, proprietary software computes the thermal conductivity of the specimen by utilizing a user-selected calibration curve that relates effusivity to thermal conductivity. The calibration curves are either supplied by the manufacturer or developed by the user with a material similar to the tested material.

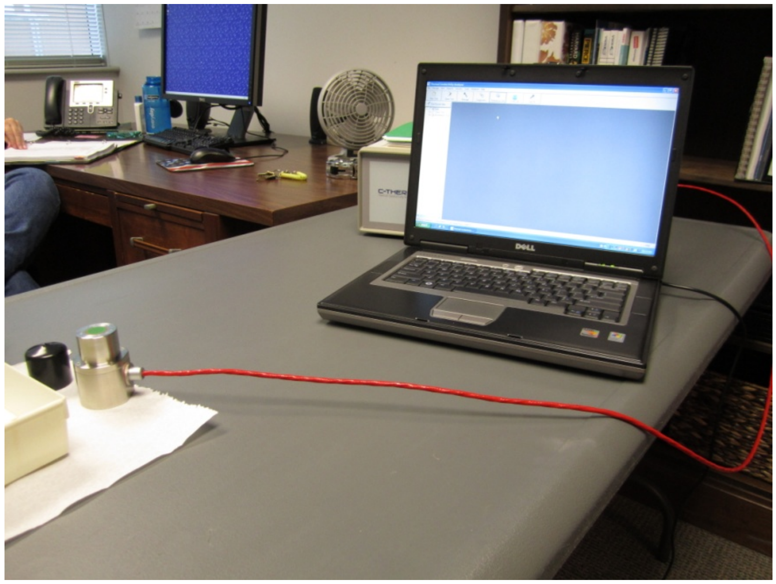

Figure 3-9: C-Therm Technologies TCi thermal conductivity testing apparatus.

One specimen of each material (brick, tile, clay, and mortar) was prepared, and tests were performed at multiple locations on each specimen. The specimens were sawcut or broken to a thickness of approximately ½ in (12.7 mm), and are shown in Figure A-3 in Appendix A. For all test specimens, thermal grease (Wakefield thermal joint compound 102, as recommended by the TCi apparatus manufacturer) was used as the contact solution between the test specimen and the glass-plated sensor, and a minimum of nine readings for each specimen were averaged to compute the thermal conductivity. The first reading was not used in the computations, as recommended by the TCi apparatus manufacturer.

3.2.1.6.3 Heat Capacity

The heat capacity of the brick, clay tile, and mortar material was determined using a thermogravimetric analyzer (TGA) apparatus, the SenSys Evo by Setaram. This calorimeter apparatus requires that the test specimen be ground to a powder and then compacted in a metal crucible, which is subsequently placed into the machine. Powder samples were prepared from whole clay brick, tile, and mortar specimens by crushing bits of each material with a modified proctor hammer, and then sieving the resulting material. Powder used for heat capacity testing was material passing a No. 60 sieve but retained on a No. 200 sieve.

Testing was performed at temperatures corresponding to warm service temperatures for concrete exposed to outdoor conditions, approximately 75°F to 130°F (24°C to 55°C). The carrier gas utilized for testing was nitrogen, and 0.000061 cubic inch (100 μL) aluminum crucibles were used. The samples were heated at a rate of 10 degrees K per minute. Two samples of each material (brick, clay tile, and mortar) were tested.

3.2.2 Experimental Results

Experimental results for testing of whole clay brick and tile are provided in the following sections. A summary of the results, particularly those useful in assessing concrete produced with the RBMA coarse aggregate (recycled brick masonry aggregate concrete, or RBMAC) is presented in Section 3.4, Summary and Conclusions.

3.2.2.1 Unit Weight

The average unit weight of brick specimens from Idlewild Elementary School is 111.6 pcf (1788 kg/m^3). The volume used to compute this unit weight includes the core volume of the bricks. If the core volume is subtracted from the gross volume, and only the clay brick material itself is considered, the average unit weight is 131.9 pcf (2113 kg/m^3).

The average unit weight of clay tile specimens from Idlewild Elementary School is 91.4 pcf (1464 kg/m^3). The volume used to compute this unit weight includes the core volume of the clay tiles. If the core volume is subtracted from the gross volume, and only the clay tile material itself is considered, the average unit weight is 168.6 pcf (2701 kg/m^3), which is considerably higher than that of the whole brick.

3.2.2.2 Absorption

The average absorption (24-hr soak method) of whole brick test specimens from Idlewild Elementary School is 8.5%. The average absorption (24-hr soak method) of whole clay tile specimens from Idlewild Elementary School is 4.0%. Supplemental data for absorption testing of bricks and clay tiles is provided in Appendix A, in Tables A-1 and A-2.

3.2.2.3 Suction

The average initial rate of absorption, or suction, of the whole brick test specimens from Idlewild Elementary School is 4 g, as corrected to a basis of 30 in^2. The average suction of the whole clay tile test specimens from Idlewild Elementary School is 0.9 g, as corrected to a basis of 30 in^2. These values are below the typical limitation of 30 g per 30 in^2 used as the threshold for pre-wetting brick prior to construction (Borchelt 2002). Additional information on suction tests for bricks and clay tiles is provided in Appendix A, in Tables A-3 and A-4.

3.2.2.4 Compressive Strength

The average compressive strength of the whole brick test specimens from Idlewild Elementary School is 9,752 psi (67.2 MPa). The average compressive strength of the whole clay tile test specimens is 11,805 psi (81.4 MPa). According to Borchelt

(2002), current-day bricks can have compressive strengths ranging from approximately 3,000 to 20,000 psi (20.7 to 137.9 MPa), with an average of about 10,000 psi (68.9 MPa).

A study of bricks from around the country by the Brick Institute of America (BIA) (2002) indicated that the mean unit compressive strength for extruded solid brick is 11,305 psi (77.9 MPa), while the mean unit compressive strength for structural clay tile with vertical coring is 10,057 psi (69.3 MPa). The value for Idlewild Elementary School brick and clay tile are within the expected range of compressive strengths. Supplemental data for compressive strength tests of bricks and clay tiles is provided in Appendix A, in Tables A-5 and A-6.

3.2.2.5 Modulus of Rupture

The average modulus of rupture of the whole brick test specimens from Idlewild Elementary School is 2,010 psi (13.9 MPa). The average modulus of rupture of the whole clay tile test specimens is 1,070 psi (7.4 MPa). According to Mamlouk and Zaniewski (2006), the typical range for modulus of rupture of most clay bricks is 500 to 3,800 psi (3.4 to 26.2 MPa). Therefore, the modulus of rupture of brick and clay tile from Idlewild Elementary School is within the expected range of values. Supplemental information for modulus of rupture tests on brick and clay tile is provided in Appendix A, in Tables A-7 and A-8.

3.2.2.6 Thermal Characteristics

Thermal characteristics of brick and clay tile from Idlewild Elementary School were obtained, and the test results are provided and discussed in the following sections.

3.2.2.6.1 Coefficient of Thermal Expansion

Difficulties were encountered during tests to determine the CTE of brick from Idlewild Elementary School. Since the bricks from the site had cores, a 5/8 in (15.9 mm) thick specimen sawcut along the length of the face of the brick was the thickest specimen that could be obtained. When the metal reference studs were mounted into the brick specimen, a crack formed adjacent to one of the reference studs. This crack likely occurred because the specimen was too thin. Despite the presence of the crack, the test was performed.

The testing procedure described in Section 3.2.1.6.1, Coefficient of Thermal expansion was initiated, and the specimen was conditioned to 68°F (20ºC), 122°F (50ºC), and 50°F (10ºC), in that order. Length measurements were obtained at each of these conditioning temperatures. Based upon measurements from the first round of tests, it appears that the brick expanded during conditioning from 68°F (20ºC) to 122°F (50ºC) as expected. However, due to the presence (or propagation) of the crack, when conditioned to 50°F (10ºC), the specimen did not shorten to a length less than the initial measured length at 68°F (20ºC). Therefore, Eq. 3-2 was modified to

$$CTE = \frac{\left(\frac{L_{122°F}-L_{68°F}}{L_{68°F}}\right)}{T_{122°F}-T_{68°F}} \tag{3-3}$$

where $L_{122ºF}$ and $L_{68ºF}$ are the measured lengths of the specimen after conditioning to 122°F (50ºC) and 68°F (20ºC), respectively, and $T_{122ºF}$ and $T_{68ºF}$ are the measured surface temperatures of the concrete specimen (at time of length measurement) after conditioning to 122°F (50ºC) and 68°F (20ºC), respectively.

Using the data obtained at the 122°F (50ºC) and 68°F (20ºC) temperatures, the CTE was determined to be 2.45×10^{-6} in/in/°F (4.41×10^{-6} m/m/°C). The CTE of clay brick is typically between 3×10^{-6} and 4×10^{-6} in/in/°F (5.4×10^{-6} and 7.2×10^{-6} m/m/°C) (Klingner 2010). The CTE obtained from measurement of the Idlewild Elementary School brick slightly lower than this expected range.

Additional tests resulted in measured values of CTE that are an order of magnitude greater than the first measured value (1.00×10^{-5} and 1.74×10^{-5} in/in/°F, or 1.8×10^{-5} and 3.13×10^{-5} m/m/°C), likely due to the influence of the crack. Further testing was not performed due to the fact that the geometry of the bricks from the subject site would not allow for thicker test specimen to be obtained. Data for the CTE tests on brick are shown in Appendix A, Table A-9.

Tests to determine the CTE of RBMAC (as described in Section 6.4.4.2.6.1, Coefficient of Thermal Expansion) were successful. The CTE for RBMAC is useful in further analysis of use of RBMAC properties in pavement design, as described in Chapter 7, Use of Recycled Brick Masonry Aggregate Concrete in Pavement Applications. However, in future work on RBMA, it is suggested that tests to determine the CTE be done in order to characterize the brick.

3.2.2.6.2 Thermal Conductivity

The average thermal conductivity of brick from Idlewild Elementary School was found to be 6.17 BTU•in/(hr•sf•°F), or 0.515 BTU/(hr•ft•°F) (0.891 W/(m•K)), and the average thermal conductivity of clay tile was found to be 10.13 BTU•in/(hr•sf•°F), or 0.844 BTU/(hr•ft•°F) (1.460 W/(m•K)). Thermal conductivity tests were also performed on samples of mortar from Idlewild Elementary School, and the average value was determined to be 1.18 BTU•in/(hr•sf•°F), or 0.098 BTU/(hr•ft•°F) (0.17 W/(m•K)). Typical thermal conductivity test results for brick, clay tile, and mortar are provided in Appendix A, Figures A-4 through A-6, respectively.

Typically, the thermal conductivity of brick is less than 1/6 that of concrete (Klingner 2010). ACI (2002) lists the thermal conductivity of fired clay bricks as ranging from 2.19 to 7.26 BTU•in/(hr•sf•°F) (0.316 to 1.05 W/(m•K)) for brick densities ranging from 70 to 150 pcf (1,121 to 2,403 kg/m^3), respectively. The measured value of thermal conductivity of brick from Idlewild Elementary School fell within this expected range.

3.2.2.6.3 Heat Capacity

The average heat capacity of brick from Idlewild Elementary School was found to be 1.13 BTU/(lb•°F) at 77°F (4,731 J/(kg•°C) at 25°C), and the average heat capacity of clay tile was found to be 2.05 BTU/(lb•°F) at 77°F (8,583 J/(kg•°C) at 25°C). Heat capacity tests were also performed on samples of mortar from Idlewild Elementary School, and the average value is 6.98 BTU/(lb•°F) at 77°F (29,223 J/(kg•°C) at 25°C 25°C). A photo of the samples of crushed brick, mortar, and clay tile used for heat capacity testing is provided in Appendix A (Figure A-7), along with a typical output spreadsheet from the TGA with associated heat capacity calculations (Figure A-8).

ACI (2002) reports that a typical value for the heat capacity of clay brick with a density of 135 pcf (2162 kg/m^3) is 0.20 BTU/(lb•°F) (34,164 J/(kg•°C)). A typical heat capacity value for mortar with a density of 120 pcf (1922 kg/m^3) reported by ACI (2002) is also is 0.20 BTU/(lb•°F) (34,164 J/(kg•°C)). The heat capacities obtained for both the Idlewild Elementary School brick and clay tile are higher than the heat capacity values reported by ACI as typical values for brick. The heat capacity of the mortar from

Idlewild Elementary School was found to be much higher than the typical value for the heat capacity of mortar reported by ACI (2002).

3.3 Recycled Brick Masonry Aggregate

RBMA was produced by crushing a portion of the demolished brick masonry walls at Idlewild Elementary School in Charlotte, North Carolina. The demolished brick masonry wall material used to create the RBMA was handled in a manner outlined in Section 3.1.2, and RBMA was produced as outlined in Section 3.1.4. Approximately two cubic yards of the RBMA material was shoveled from the stockpile at D.H. Griffin Crushing and Grading, placed into clean 55-gallon drums and 5-gallon buckets, and returned to UNC Charlotte's laboratories.

3.3.1 Experimental Procedures

Laboratory testing was performed on the RBMA in order to determine properties related to development of concrete mixture designs, as well as to provide characterization information useful in comparing the RBMA to other conventional (natural and manufactured lightweight) aggregates and recycled aggregates. Tests were performed as outlined in the following sections.

3.3.1.1 Composition by Weight and by Volume

Demolished brick masonry construction that is crushed to produce RBMA will consist of two main components: brick and mortar. In the case of the RBMA material produced from the Idlewild Elementary School case study site, the RBMA also contained some fraction of clay tile material. If demolished brick masonry material from different sites is used to create RBMA, the RBMA produced will have different compositions. It was of interest, therefore to characterize the composition of the RBMA from Idlewild Elementary School by weight and by volume.

Two representative samples of RBMA were obtained from randomly selected buckets containing the material. These RBMA samples were further reduced using a sample splitter until they were of sizes manageable for manual separation of the particles by type. One sample was approximately 0.9 lb (0.4 kg) and the other sample was approximately 1.5 lb (0.7 kg). After taking an initial weight and loose volume of each sample, the samples were poured onto a clean tabletop and manually separated into four piles: brick, mortar, tile, and other. After separation, the weight of each fraction of material was recorded, and the volume of each fraction of material was obtained by loosely pouring the material into a graduated cylinder and recording the volume.

3.3.1.2 Gradation

Although equipment at D.H. Griffin Crushing and Grading was configured in a manner capable of producing material that generally met specific AASHTO gradations for typical material processed at the facility, testing to obtain the gradation of the RBMA was performed at UNC Charlotte laboratories. Testing was performed in general accordance with ASTM C136, “Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates,” and three samples were used. Each sample exceeded the specified mass of 4.4 lb (2 kg), and was obtained from the mass stockpile of RBMA (stored in 5 gallon buckets) by using a sample splitter.

3.3.1.2 Particle Shape – Flat and Elongated Particles

The particle shapes of RBMA were characterized by testing in general accordance with ASTM D4791, “Standard Test Method for Flat Particles, Elongated Particles, or Flat and Elongated Particles in Coarse Aggregate." This testing was performed because visual observation of the RBMA material revealed that the crushed brick particles tended to have more flat and elongated particle shapes than the mortar particles. The crushed brick particles also appeared to be more flat and elongated than locally available natural aggregates.

This test requires selection of a ratio setting to be used on the proportional caliper device used for identifying flat, elongated, and flat and elongated particles. In keeping with statewide practices, it was of interest to utilize NCDOT guidelines, if available for this test. NCDOT currently performs flat and elongated particle testing to qualify coarse aggregates for use in asphaltic cement concrete mixtures, but not portland cement concrete mixtures. For asphaltic cement concrete, aggregates are tested using a 5:1 ratio on the proportional caliper device. In lieu of guidelines for use of this test method for concrete applications, it was decided to utilize the 5:1 ratio setting for flat and elongated testing as part of this work.

To determine the percentage of flat, elongated, and flat and elongated particles of the RBMA blend (i.e., including the brick, mortar, clay, and tile material in their naturally occurring proportions), a sample splitter was used to obtain three sets of 100 particles for testing using the proportional calipers. Additional testing was performed to determine the percentage of flat, elongated, and flat and elongated particles of the brick alone, the mortar alone, and the clay tile alone. To obtain these samples, a sample splitter was used to obtain three sets of 100 particles of brick and three sets of 100 particles of mortar. As clay tile was present in the aggregate RBMA material in a far smaller proportion, only one sample of 100 particles of clay tile was obtained by hand selection for testing.

3.3.1.4 Density, Specific Gravity, and Absorption

Testing to determine the density, specific gravity, and absorption of the RBMA was performed in general accordance with ASTM C127, “Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate.” Testing was performed on three samples of RBMA, each obtained from the mass stockpile of RBMA (stored in 5-gallon buckets) by using a sample splitter.

3.3.1.5 Bulk Density (Unit Weight)

The bulk density, or unit weight of the RBMA was determined in general accordance with ASTM C29, “Standard Test Method for Bulk Density (“Unit Weight”) and Voids in Aggregate.” Testing was performed following both the “Shoveling Procedure” and the “Rodding Procedure.” In the “Shoveling Procedure,” the measure is filled without tamping and then weighed. In the “Rodding Procedure,” the measure is filled in one-third layers and rodded after each of the three fillings. After rodding the last layer, the measure is weighed.

3.3.1.6 Abrasion Resistance

Abrasion resistance of the RBMA was evaluated by testing the material in accordance with ASTM C131, “Standard Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine.” Testing was performed on two samples of aggregate that met Gradation C (2,500±10 g of material passing the 3/8 in sieve and retained on the ¼ in sieve, and 2,500±10 g of material passing the ¼ in sieve and retained on the No. 4 sieve).

3.3.2 Experimental Results

Experimental results for tests performed on the RBMA are provided in the following sections. A summary of the results, particularly those useful in assessing RBMAC mixtures is presented in Section 3.4, Summary and Conclusions.

3.3.2.1 Composition by Weight and by Volume

The RBMA produced from the Idlewild Elementary School demolition waste was largely comprised of three components: clay brick, clay tile, and mortar. The results of sorting these components, and measuring the relative proportion of each by weight and by volume are shown in Table 3-1. A photograph of the RBMA is shown in Figure 3-10.

Table 3-1: Composition of the RBMA produced from Idlewild Elementary School.

Material	**% by weight**	**% by volume**
clay brick	64.5	63.9
clay tile	2.1	1.9
mortar	30.1	31.6
other (rock, porcelain, lightweight debris)	3.3	2.6

Figure 3-10: RBMA from Idlewild Elementary School.

Of particular interest is the amount of mortar present in the aggregate (almost 1/3 by weight and by volume), which while presumably influencing the performance of the RBMA in concrete mixtures, is largely ignored in virtually all published studies on this topic.

Although the “top-down” demolition sequence utilized at Idlewild Elementary School was largely successful in preventing most contaminants from being incorporated into the crushed brick masonry aggregate, it was evident from visual observations that some debris other than clay brick, clay tile, and mortar was present in the material; see Table 3-1.

3.3.2.2 Gradation

The sieve analysis test results of the AASHTO M43 #78 recycled brick masonry aggregate material are shown in Table 3-2, along with the acceptable range for AASHTO M43 #78 gradation material. The sieve analysis results of a recycled concrete aggregate

(Idlewild Elementary School source material) produced on the same day using the same equipment is also shown. It is noted that although both the recycled brick masonry aggregate and the recycled concrete aggregate met AASHTO M43 requirements for most sieve sizes, the percentage of material passing the 3/8 in sieve (retained on the No. 4 sieve) was slightly higher than the acceptable range. Discussions with D. H. Griffin Crushing and Grading personnel indicated that when they periodically discover that material is not meeting the required gradation, the settings within the mechanical equipment used in the crushing and/or grading process are modified slightly to bring the gradation of the material produced back into specification. More detailed information on the sieve analyses on the RBMA is provided in Appendix A, Table A-10.

Table 3-2: Results of ASTM C136 sieve analysis testing of RBMA and RCA produced from Idlewild Elementary School.

	% of Material Finer		
Sieve Opening (in)	**Recycled Brick Masonry Aggregate**	**Recycled Concrete Aggregate**	**AASHTO M43 #78 Acceptable Range**
3/4	100	100	100
1/2	99.8	100	90-100
3/8	85.1	85	40-75
No. 4	19.5	14	5-25
No. 8	0.8	3	0-10
Pan	0	0	---

3.3.2.3 Particle Shape – Flat and Elongated Particles

A summary of the results of testing performed on the RBMA to determine whether an excessive amount of flat, elongated, or flat and elongated particles were present in the material are shown in Table 3-3. Supplemental data from these tests is included in Table A-11 of Appendix A.

Table 3-3: Results of ASTM D4791 flat and elongated particle testing.

	Average % Flat and Elongated	
Material	**by particle count (%)**	**by mass (%)**
Recycled Brick Masonry Aggregate (Blend)	4.0	3.6
Brick only	9.0	6.7
Mortar only	0.7	0.5
Tile only	8.0	4.8

In this (ASTM D4791) test, the length to width, width to thickness, and length to thickness ratios are compared to a specified ratio. For this work, NCDOT specifications limiting the use of flat and elongated aggregates in asphalt pavement were used, as there are currently no NCDOT specifications limiting use of flat and elongated particles in portland cement concrete applications. According to Section 1012 of the NCDOT specifications, the maximum percentage of flat and elongated particles is 10% using the 5:1 ratio, tested on particles retained on the No. 4 sieve and larger. It is unclear whether the NCDOT limit is 10% by mass or by particle count. The RBMA did not contain greater than 10% by mass or by particle count, and therefore does not contain an excessive amount of flat and elongated particles when tested as a composite material nor when individual components (brick, mortar, tile) were tested alone, as shown in Table 3-3. The percentage of brick and tile particles that are flat and elongated, however, is significantly higher than the percentage of flat and elongated mortar particles.

3.3.2.4 Density, Specific Gravity, and Absorption

ASTM C127 testing was performed to determine the oven dried density, the saturated surface-dry density, and the apparent density of the RBMA. The oven dried density is 121.8 pcf (1951 kg/m^3), the saturated surface-dry (SSD) density is 136.6 pcf (2188 kg/m^3), and the apparent density is 159.8 pcf (2559.8 kg/m^3). ASTM C127 tests also indicate that the specific gravity of the RBMA is 2.19, and the absorption of the RBMA is 12.2%. Data obtained during these tests is provided in Appendix A in Table A-12.

3.3.2.5 Bulk Density (Unit Weight)

The bulk density of the RBMA was obtained using the “Shoveling Procedure” outlined in ASTM C127. Using the “Shoveling Procedure,” the loose bulk density of the recycled brick masonry aggregate is 60.9 pcf (976 kg/m^3). Using the “Rodding Procedure,” the dry rodded unit weight of the RBMA is 70.4 pcf (1128 kg/m^3). Data obtained during these tests is provided in Table A-13 of Appendix A.

3.3.2.6 Abrasion Resistance

Los Angeles Abrasion testing of the recycled brick masonry aggregate was performed in accordance with ASTM C131, on a sample meeting “Gradation C.” This testing indicated that the percent loss is 43.1%. Supporting data for abrasion resistance tests on RBMA are provided in Table A-14 of Appendix A.

According to Section 1014-2 of NCDOT specifications, “Crushed stone or gravel must have a percentage wear (loss) of not more than 55 percent. For concrete with a 28-day design strength greater than 6,000 psi (41.4 MPa), limit this wear to 40%.” Based on these limitations, the crushed brick masonry aggregate for this study could be used in

NCDOT pavement applications where the 28-day design strength is less than 6,000 psi (41.4 MPa).

3.4 Summary and Conclusions

The RBMA produced from the Idlewild Elementary School walls is composed of mostly brick (approximately 2/3 of the material, by weight and by volume), with trace amounts of clay tile. A summary of the properties of these whole materials is shown in Table 3-4. Future work utilizing RBMA from other sources should include testing to characterize these properties, at a minimum in order to facilitate comparison to this material.

Table 3-4: Properties of whole brick and clay tile from Idlewild Elementary School.

	Brick	**Clay Tile**
Compressive strength (psi)	9,752	11,805
Modulus of rupture (psi)	2,010	1,070
Absorption (%) (24-hr soak procedure)	8.5	4.0
Suction (g) (gain in weight corrected to basis of 30 in^2)	4.0	0.9
Solid unit weight (pcf)	131.9	168.6

Relevant ASTM standard test methods were used to determine properties of the RBMA, including: absorption, specific gravity, bulk density, gradation, and abrasion resistance. A summary of test results for the RBMA is shown in Table 3-5, along with test results for the recycled concrete aggregate (RCA) produced from a demolished slab-on-grade at Idlewild Elementary School. Published data for a locally quarried natural normalweight aggregate and for a locally manufactured lightweight aggregate are also shown for comparison.

Table 3-5: Properties of recycled aggregates from Idlewild Elementary School demolition waste, compared to properties of a locally manufactured lightweight aggregate and a local normalweight natural aggregate.

	Recycled Brick Masonry Aggregate (RBMA)	Locally Manufactured Lightweight Aggregate	Recycled Concrete Aggregate (RCA)	Locally Quarried Normalweight Natural Aggregate
	Idlewild Elementary School	Stalite	Idlewild Elementary School	Martin Marietta Quarry
Specific Gravity (at Saturated Surface Dry Condition)	2.19	1.53	N/A	2.84
Absorption	12.2	6.0	7.6	0.34
Abrasion (LA Abrasion testing, Gradation C)	43.1	25 to 28	N/A	17.2
Loose Bulk Density (Unit Weight, Dry) (pcf)	60.9	50	80.0	95.9

*Note: All data is for ½ in nominal maximum size material (AASHTO M43 #78 gradation)

It can be seen from Table 3-5 that the RBMA produced from the Idlewild Elementary School walls has a loose bulk density (ASTM C29 shoveling procedure) that is approximately 2/3 that of a locally quarried normalweight (natural) aggregate that is typically used in concrete. The absorption of the RBMA is far greater than that of the local normalweight natural aggregate and is almost twice that of the RCA produced from the Idlewild Elementary School concrete slab. The absorption of the RBMA is also roughly twice that of a locally manufactured lightweight aggregate (Stalite).

Properties of the RBMA were also compared to ASTM C330, “Standard Specification for Lightweight Aggregates for Structural Concrete” in order to select the proper mixture design procedure and to better understand the potential performance of this material in concrete mixtures. It is noted that the bulk density of the material is slightly higher than the upper limit for maximum dry loose bulk density specified by ASTM C330, which is 55 pcf (881 kg/m^3).

The abrasion resistance of brick aggregates is typically cited by researchers as a weakness. The measured abrasion loss of the Idlewild Elementary School RBMA is 43%, which would make it suitable for use in concrete mixtures with 28-day design strengths less than 6,000 psi, but not suitable for use in concrete mixtures with 28-day design strengths greater than 6,000 psi. Further testing of abrasion resistance of RBMAC was performed using the rotary cutter method of abrasion testing, which is discussed in Section 6.4.3.3.2, Abrasion Resistance (experimental procedure) and Section 6.4.4.3.2, Abrasion Resistance (experimental results).

CHAPTER 4: CONCRETE MATERIAL COMPONENTS AND PROPERTIES

4.1 Introduction

In this chapter, the materials utilized in this study are described. Information regarding the source, physical characteristics, and chemical characteristics of the materials is provided. For chemical admixtures, product information and recommended dosage rates are outlined.

4.2 Material Components

The material components used for this work include recycled brick masonry aggregate (RBMA), natural aggregate, portland cement, water, and chemical admixtures. The sources and characteristics of each of these materials are discussed in the following sections.

4.2.1 Recycled Brick Masonry Aggregate (Coarse Aggregate)

Concrete mixtures batched as part of this work incorporated RBMA. The source of the RBMA was demolished brick masonry obtained from a demolition site at Idlewild Elementary School in Charlotte, North Carolina. The recycled brick masonry used in this study was handled as outlined in Section 3.1.2, Demolition Sequence and Material Handling. The crushing, sieving, and processing of this material is discussed in Section 3.1.4, Obtaining Recycled Brick Masonry Aggregate.

Characterization of the RBMA, including composition by weight and by volume, gradation, particle shape, density, specific gravity, absorption, bulk density, and abrasion resistance, is discussed in Section 3.3, Recycled Brick Masonry Aggregate. Physical properties of the RBMA are also presented in Sections 3.3, Recycled Brick Masonry Aggregate and Section 3.4, Summary and Conclusions. To summarize, the absorption of the RBMA is 12.2%, the saturated surface-dry bulk specific gravity of the RBMA is 2.19, and the loose bulk density (dry unit weight) is 60.9 pcf (976 kg/m^3). No natural aggregate material was used as coarse aggregate in this study.

4.2.2 Natural Aggregate (Fine Aggregate)

The fine aggregate used for the concrete mixtures batched in this study is a natural pit silica sand from a deposit in Pageland, South Carolina. This sand meets ASTM C33 and AASHTO M6, and has a saturated surface-dry (SSD) bulk specific gravity of 2.60 and an absorption of 3.0%. The fineness modulus of this sand is 2.68. Prior to mixing each batch of concrete, the moisture content of the sand was calculated and water corrections were applied to the mixture proportions.

4.2.3 Portland Cement

The portland cement used in the concrete mixtures batched in this work is a Type I/II cement meeting ASTM C150 and AASHTO M85 requirements. The manufacturer of the cement is Old Castle. A standard mill analysis was not available, and testing to determine chemical composition was not performed as part of this work.

4.2.4 Water

Potable tap water at room temperature was used for mixing all concrete mixtures. The quality of the tap water supplied to the laboratories at UNC Charlotte is controlled and monitored by Charlotte-Mecklenburg Utilities.

4.2.5 Admixtures

Several commercially available liquid admixtures were utilized in the concrete mixtures. The commercial names of these admixtures, a brief description of their compositions, and the manufacturer's recommended dosage are provided in the subsequent sections.

4.2.5.1 Air-entraining Admixture

In all concrete mixtures batched as part of this study, an air entraining admixture was used. The air entraining admixture, Darex II manufactured by Grace Construction Products, is formulated using organic acid salts in an aqueous mixture. This admixture meets ASTM C260 requirements. The manufacturer recommended dosage for this admixture is 0.5 to 5 fluid oz per 100 lb of cement. The actual dosage used in each concrete mixture varied, but an air content range of 6 to 8% was targeted. Doses of the air-entraining admixture used in each mixture are provided in Chapter 5, Development of Concrete Mixture Proportions.

4.2.5.2 High-Range Water-Reducing Admixture

Water reducing admixtures were used to aid in obtaining the desired workability, as measured by slump tests. The high-range water-reducing admixture (superplasticizer), EXP 950 manufactured by Grace Construction Products, was used in some of the concrete mixtures in order to obtain an acceptable workability. This high-range water-reducing admixture is polycarboxylate-based, meeting ASTM C494 Type F and ASTM C1017 Type I requirements. The manufacturer recommended dosage for this admixture is 2 to 10 fluid oz per 100 lb of cementitious material. The actual dosage used in each concrete mixture varied (provided subsequently in Chapter 5, Development of Concrete Mixture Proportions), but a slump of 4 to 8 in (101.6 to 203.2 mm) was targeted.

4.2.5.3 Mid-Range Water-Reducing Admixture

For a few of the concrete mixtures batched as part of this work, the mid-range water-reducing admixture, WRDA 35 manufactured by Grace Construction Products, was used to obtain an acceptable workability. This mid-range water-reducing admixture is water-based, composed of organic compounds and a catalyst, and meets ASTM C494 Type A and Type D requirements. The manufacturer recommended dosage is 2 to 8 fluid oz per 100 lb of cementitious material. The actual dosage in each concrete mixture varied (provided subsequently in Chapter 5, Development of Concrete Mixture Proportions), but a slump of 4 to 8 in (101.6 to 203.2 mm) was targeted.

CHAPTER 5: DEVELOPMENT OF CONCRETE MIXTURE PROPORTIONS

5.1 Overview of Strategy

As discussed in Chapter 2, Literature Review, RBMAC has not been produced or studied in the United States. In order to establish the viability of RBMA from demolished brick masonry construction as material that can successfully be utilized in structural and pavement grade portland cement concrete, a series of concrete mixture designs were targeted, and trial batches were developed, batched, and tested. Many structural and pavement grade concrete mixtures have 28-day compressive strengths ranging from 4,000 to 6,000 psi (27.6 to 41.4 MPa). Therefore, it was desired to identify mixture proportions that would produce concrete that would be representative of commercially available 4,000 psi (27.6 MPa), 5,000 psi (34.5 MPa), and 6,000 psi (41.4 MPa) mixtures.

Due to the overdesign requirements outlined by the American Concrete Institute (ACI) in ACI 318 "Building Code Requirements for Structural Concrete," it was necessary to proportion these mixtures so that the concrete reached strengths sufficiently higher than the design strength. According to ACI 318, without a history of performance data, 4,000 psi (27.6 MPa), 5,000 psi (34.5 MPa), and 6,000 psi (41.4 MPa) mixtures must reach 28-day compressive strengths of 5,200 psi (35.9 MPa), 6,200 psi (42.7 MPa), and 7,300 psi (50.3 MPa), respectively. With these ultimate strength targets in mind, trial batches were developed and tested in an effort to identify a series of suitable mixtures (referred to as "baseline" mixtures from this point forward) that would be representative of commercially available 4,000 psi (27.6 MPa), 5,000 psi (34.5 MPa), and 6,000 psi (41.4 MPa) mixtures.

5.1.1 Anticipated Challenges

In developing RBMAC, several different types of challenges were anticipated. Some of the challenges were addressed prior to batching the trial mixtures; others needed to be studied and addressed as development of the trial mixtures progressed towards identification of the baseline mixtures. Several of the anticipated challenges are presented below, along with details regarding how they were addressed prior to (or during) development of trial batches of RBMAC.

The presence of contaminants in the RBMA was identified as a challenge early in the project. The "top-down demolition" sequence utilized by the demolition contractor, along with on-site material separation and careful handling procedures, helped to minimize the amount of contaminants included with the demolished brick masonry sent to the crusher. Incorporation of dirt or other contaminants into the RBMA at the crushing and grading facility was minimized by ensuring that the loading equipment did not scrape the ground beneath the piles of uncrushed brick masonry and RBMA. However, some fraction of contaminants present in the brick masonry wall would ultimately be present in the RBMA. The possibility of low strengths and/or wide variability in test results due to the inclusion of contaminants inherently exists when using RBMA in concrete.

In order for RBMA to be an economically viable product, it is likely necessary for it to be produced in a manner that does not involve modifications to existing processing procedures at the crushing and grading facility. Due to the lack of availability of aggregate washing equipment at the crushing and grading facility, it was not possible for the fine material clinging to the surface of the RBMA to be removed by washing prior to delivery to UNC Charlotte for laboratory testing. Therefore, measures were taken to develop a uniform washing protocol for the RBMA prior to use in laboratory tests or as a mixture component in RBMAC batches. This washing procedure is discussed in Section 5.1.3.1, Replacement of Natural Aggregate with Recycled Brick Masonry Aggregate.

In development of the RBMAC mixtures, it was anticipated that the high absorption and angularity of the brick aggregate could make achieving the desired workability (slump) problematic. Many previous researchers attributed difficulties encountered while trying to utilize RBMA and brick aggregate in concrete mixtures to these qualities, but did not attempt to address this issue by using commercially available water-reducing admixtures. During this work, it was found that by using commercially available mid-range and high-range water reducers, the desired workability could be achieved.

Ultimately, one of the biggest challenges encountered was due to the limited amount of material available for batching concrete mixtures. During demolition, a portion of one truckload of brick material was crushed, graded, and returned to UNC Charlotte for use in this study. A significant amount of material was utilized in the trial mixtures prior to identification of the baseline mixtures used in the full testing program. Additional RBMA would have been desirable, as tightened confidence intervals could have been achieved with more RBMAC test specimens. Additional information regarding experimental procedures for RBMAC, and the limitations in testing resulting from the exhaustible supply of RBMA, is presented in Section 6.4, Testing of Hardened Concrete Properties.

5.1.2 Design Variables and Constraints

In addition to the RBMA created using demolished brick masonry walls from Idlewild Elementary School in Charlotte, North Carolina, RBMAC mixtures developed as part of this work utilized only commercially available materials. These materials included Type I/II portland cement, a natural silica sand meeting ASTM C33, tap water, and commercially available liquid chemical admixtures. Information on the materials utilized in the RBMAC mixtures is presented in Chapter 4, Concrete Material Components and Properties.

Since RBMA exhibits characteristics that differ from those of conventional aggregates, it was expected that the RBMAC would exhibit performance different from conventional concrete in both fresh and hardened concrete tests. However, in development of the RBMAC, it was desirable that existing ASTM standards and ACI guidelines be followed as strictly as possible. Commercial viability of RBMA for use in concrete will strongly depend on the material conforming to existing standards and guidelines and being readily testable using existing standards.

5.1.3 Design Approach

In the United States, no published guidance for developing RBMAC mixture designs exists. Due to the relatively low unit weight and the high absorption of the brick masonry aggregate, mixture designs were developed in accordance with ACI 211.2, “Standard Practice for Selecting Proportions for Structural Lightweight Concrete.” This standard indicates that lightweight aggregates should conform to ASTM C330. It is noted that with a maximum dry loose bulk density of 60.9 pcf (976 kg/m^3), the RBMA created from the Idlewild Elementary School walls did not meet the maximum dry loose bulk density requirement for coarse aggregate of 55 pcf (881 kg/m^3) as outlined in ASTM C330. It was found, however, that this mixture design approach provided mixture proportions that produced suitable performance characteristics for both trial mixtures and baseline mixtures.

5.1.3.1 Replacement of Natural Aggregate with Recycled Brick Masonry Aggregate

Prior to performing tests with RBMA or using RBMA in concrete mixtures, the RBMA was washed in order to remove fine material (generated during the crushing process) that was on the surfaces of the aggregate particles. Even thin coatings of fine materials can affect hydration and bond of cement paste to the aggregates (Kosmatka et al. 2002). This is one reason why materials finer than the No. 200 (75 μm) sieve are limited in many specifications. The RBMA was washed in small quantities, approximately one 5-gallon bucketful at a time. The small quantity of RBMA was first poured over a mesh screen with openings significantly smaller than the smallest aggregate size. Water from a hose was sprayed on the RBMA, which was agitated with a rake to help facilitate wash-off of the fine materials.

RBMA was saturated for at least 24 hr prior to batching, and damp towels were used to bring the moisture content of the brick masonry aggregate to saturated surface dry (SSD) condition prior to batching.

RBMA was utilized as a 100% substitute for conventional natural coarse aggregate in the concrete mixtures. The fines generated during the manufacture of RBMA were not utilized in this study.

5.1.3.2 Aggregate Gradation

With the exception of being washed at UNC Charlotte, the RBMA was utilized in this study in the condition in which it was obtained from D.H. Griffin Crushing. The gradation of the RBMA was not modified prior to batching. Information regarding the gradation of the RBMA used as coarse aggregate in RBMAC mixtures is presented in Section 3.3.2.2, Gradation.

5.1.3.3 Cement Content

Portland cement is typically the most expensive component in concrete mixtures, thus concrete mixtures with lower cement contents are typically not as costly as concrete mixtures with higher cement contents. From a sustainability perspective, due to the energy demands and greenhouse gas emissions associated with cement production, an increased portland cement content is associated with a greater embodied energy of concrete and a larger carbon footprint (Schokker 2010). Therefore, it is desirable to minimize the cement content of concrete mixtures, including RBMAC mixtures, to the largest extent possible.

In initial trial batches of RBMAC, the cement contents were relatively high. As trial batches of RBMAC were batched and tested, and desirable compressive strengths were achieved, the cement content was reduced. For baseline mixtures, a range of cement contents capable of meeting the design strengths (with overdesign) was selected. However, the cement contents in each of the baseline mixtures is still within the range of cement contents expected for concrete mixtures using conventional aggregates utilized in structural and pavement applications.

5.1.3.4 Water/Cement Ratio

The water/cement (w/c) ratio of concrete is an important parameter in concrete mixture design. In general, higher water content in a concrete mixture is associated with a more permeable paste structure, weakened interfacial transition zones between paste and aggregate, and lower strengths. Therefore, it is desirable to obtain the desired workability at as low of a w/c ratio as possible. In developing and batching RBMAC mixtures, admixtures typically used by ready-mixed concrete suppliers were used to assist in obtaining the desired workability while simultaneously increasing strength by reducing water content. A mid-range water reducing admixture and a high-range water reducing admixture were utilized, and are described in Section 4.2.5, Admixtures.

5.1.3.5 Air Content

Concrete that includes a well-dispersed system entrained air voids exhibits improved durability performance in freeze-thaw exposures (Kosmatka 2002). For most exposure conditions, state and federal agencies recommend entrained air contents ranging from 5 to 8%. The target air content of RBMAC mixtures batched as part of this work was 6 to 8%.

5.1.3.6 Target Slump and Water-Reducing Admixture Usage

Adequate workability is required to ensure that concrete can be placed, consolidated, and finished properly. In addition to being workable, the components should not segregate during transport, handling, and placing of the concrete (Kosmatka 2002). Recommended slumps for various types of construction are presented in ACI 211.1. As outlined in ACI 211.1 and Kosmatka (2002), the target slump for many concrete mixtures is between 1 and 4 in (25.4 to 101.6 mm). However, in recent years, use of water reducers has allowed for much greater slumps at relatively low w/c ratios, and specifications for structural use often allow for slumps in the range of 4 to 8 in (101.6 to 203.2 mm) with an adequately low w/c ratio or adequate proof of performance in specified hardened concrete tests. Often for many structural applications, the desired slump depends on a number of factors including reinforcing steel spacing. For paving mixtures, however, slumps are much lower, often in the range of 1 to 3 in (25.4 to 76.2 mm).

For RBMAC mixtures batched as part of this work, it was desired to have workability suitable to accommodate proper consolidation of the concrete into the various forms used to cast test specimens. For this reason, the target slump for RBMAC mixtures batched as part of this study was 4 to 8 in (101.6 to 203.2 mm). In the first trial of mixture proportioning, developed using "Method 1: Weight Method" outlined in ACI 211.2, a target slump of 3 to 4 in (76.2 to 101.6 mm) was used in entering the tables that provide the recommended water content for the mixture. As trial mixtures were developed, water-reducing admixtures were utilized to increase the workability of the concrete while reducing the required water content. Using mid-range and high-range water reducers, slumps of most mixtures tended to be higher, falling in the range of 4 to 8 in (101.6 to 203.2 mm).

5.2 Mixture Proportions

As discussed previously, a series of trial RBMAC mixtures was batched prior to identifying the four baseline mixtures, which would be used for more extensive testing. The subsequent sections provide information regarding the strategy used in developing and testing trial mixtures, as well as information regarding the four baseline mixtures and their proportions.

5.2.1 Preliminary Mixture Proportions

A number of trial RBMAC mixtures exhibiting acceptable workability and compressive strength were successfully developed and tested. Information on trial mixtures, including batch proportions, slump and air content test results, and compressive strength test results, is presented in Table 5-1. Compressive strength test results for all specimens prepared from trial RBMAC mixtures are provided in Appendix B in Table B-1. These mixtures are identified as BAC 1.0, BAC 2.0, BAC 2.1, BAC 2.2, BAC 2.3, and BAC 3.0. These initial mixtures were batched in order to develop a mixing protocol that adequately addressed the need to utilize the RBMA in SSD condition, to assess the need for mid-range and high-range water reducing admixtures to obtain the desired workability, and to obtain preliminary compressive strength test results.

Table 5-1: Trial RBMAC mixture proportions and test results.

	BAC 1.0	BAC 2.0	BAC 2.1	BAC 2.2	BAC 2.3	BAC 3.0	BAC 4.0	BAC 4.1	BAC 4.2	BAC 4.3	BAC 4.4	BAC 4.5	BAC 4.6
Coarse Aggregate (pcy)	1,178.6	1,052.0	1,052.0	1,052.0	1,052.0	1,178.6	1,178.6	1,178.6	1,178.6	1,178.6	1,178.6	1,178.6	1,178.6
Sand (pcy)	1,149.4	1,281.0	1,281.0	1,281.0	1,281.0	1,417.0	1,284.0	1,284.0	1,284.0	1,355.7	1,427.6	1,254.0	1,185.0
Cement (pcy)	812.5	677.0	677.0	677.0	677.0	600.0	675.0	675.0	675.0	625.0	575.0	725.0	775.0
Water (pcy)	316.0	301.2	320.1	262.1	237.3	291.0	260.2	281.8	211.6	197.4	186.9	291.5	299.4
w/c ratio	0.39	0.44	0.47	0.39	0.35	0.49	0.39	0.42	0.31	0.32	0.33	0.40	0.39
Air Entraining Admixture (oz)	16.0	20.5	16.9	16.9	16.9	15.1	16.9	16.9	13.7	16.9	11.9	17.3	17.3
Mid-Range Water Reducing Admixture (oz)	0	0	0	65.7	0	0	0	0	0	0	0	0	0
High-Range Water Reducing Admixture (oz)	0	0	0	0	44.0	0	0	0	36.5	27.3	22.8	0	0
Slump (in)	7.0	3.5	7.5	5.0	10.0	4.5	8.0	3.5	7.0	7.0	6.0	5.0	4.5
Air content (%)	4.50	5.00	6.50	7.00	7.25	5.50	5.50	7.00	7.00	8.00	5.50	6.50	5.50
3-day compressive strength (psi)	3,895	2,668	3,097	1,804	5,337	2,378	2,200	3,491	5,722	4,084	2,642	2,872	2,821
7-day compressive strength (psi)	4,692	3,146	3,903	2,107	6,197	3,128	2,513	3,979	6,249	4,712	3,297	3,339	3,323
28-day compressive strength (psi)	6,067	4,117	4,726	2,480	6,962	3,642	3,515	5,167	7,858	5,304	3,484	4,518	4,306
90-day compressive strength (psi)	6,813	---	4,995	2,746	7,574	4,292	3,831	5,575	7,879	5,360	3,803	4,992	4,870

The compressive strengths of several of these early mixtures were higher than anticipated using the ACI 211.2 design procedures. Therefore, in keeping with the sustainability focus of this project, these mixture designs were subsequently modified with the goal of achieving the target strengths at lower cement contents (BAC 4.0 through BAC 4.6).

Acceptable workability was achieved with the water reducing admixtures, with the high-range water reducing admixture providing what was judged to be the best workability. After mixture BAC 2.2, only a high-range water reducing admixture was considered for further use in trial and baseline mixtures.

5.2.2 Final Mixture Proportions

Test data from the trial mixtures of RBMAC led to the identification of mixture proportions for the four baseline mixtures. These baseline mixtures, BAC 5.0, BAC 6.0, BAC 6.1, and BAC 6.2, are shown in Table 5-2, along with information on batch proportions, slump and air content test results, and compressive strength test results.

Table 5-2: Baseline RBMAC mixture proportions and test results.

	RBMAC Mixture			
	BAC 5.0	**BAC 6.0**	**BAC 6.1**	**BAC 6.2**
Coarse Aggregate (pcy)	1,178.6	1,178.6	1,178.6	1,178.6
Sand (pcy)	1,296.0	1,296.0	1,356.0	1,428.3
Cement (pcy)	675.0	675.0	625.0	575.0
Water (pcy)	292.0	216.0	200.0	183.6
w/c ratio	0.43	0.32	0.32	0.32
Air Entraining Admixture (oz)	13.7	16.4	13.7	13.7
Mid-Range Water Reducing Admixture (oz)	0	0	0	0
High-Range Water Reducing Admixture (oz)	0	36.5	29.2	29.2
Slump (in)	6.0	5.5	6.0	3.5
Air content (%)	5.50	7.50	8.00	6.50
3-day compressive strength (psi)	2,139	4,559	3,684	4,508
7-day compressive strength (psi)	2,858	6,182	4,074	5,283
28-day compressive strength (psi)	3,675	6,497	5,307	6,450
90-day compressive strength (psi)	3,872	6,903	5,362	7,343

As indicated earlier, the proportions of these mixtures were selected to produce 28-day compressive strengths which, including the overdesign requirements outlined by the American Concrete Institute (ACI) in ACI 318 “Building Code Requirements for Structural Concrete,” would be representative of commercially available 4,000 psi (27.6 MPa), 5,000 psi (34.5 MPa), and 6,000 psi (41.4 MPa) mixtures. Without a history of performance data, 4,000 psi (27.6 MPa), 5,000 psi (34.5 MPa), and 6,000 psi (41.4 MPa) mixtures must reach 28-day compressive strengths of 5,200 psi (35.9 MPa), 6,200 psi (42.7 MPa), and 7,300 psi (50.3 MPa).

Based on the cement contents used in these mixture designs and the compressive strength results obtained from the trial batches, it was anticipated that the overdesign strengths for commercially available 6,000 psi (41.4 MPa), 5,000 psi (34.5 MPa), and 4,000 psi (27.6 MPa) mixtures would be obtained by the baseline mixtures BAC 6.0, 6.1, and 6.2, respectively. However, the 28-day compressive strength tests on these mixtures were slightly lower than anticipated, and although exceeding the design strengths of 4,000 psi (27.6 MPa), 5,000 psi (34.5 MPa), and 6,000 psi (41.4 MPa), the mixtures did not meet the overdesign strengths. Therefore, when considering overdesign strength, mixture BAC 6.1 could be considered representative of a commercially available 4,000 psi (27.6 MPa) mixture, while BAC 6.0 and BAC 6.2 would both be representative of a commercially available 5,000 psi (34.5 MPa) mixture. Unfortunately, the supply of RBMA was exhausted after batching BAC 6.2, and no further mixtures could be prepared and tested.

Three of the four baseline mixtures included a high-range water reducing admixture (BAC 6.0, BAC 6.1, and BAC 6.2), while the fourth baseline mixture (BAC 5.0) did not. This latter mixture did, however, have the same cement content as one of the other three mixtures in order to identify the amount of water that would be needed to achieve similar workability without the aid of chemical admixtures.

5.3 Summary and Concluding Remarks

RBMAC has not previously been produced or studied in the United States. As part of this study, a number of RBMAC mixtures exhibiting satisfactory fresh and hardened properties were developed using mixture proportioning methodology outlined in ACI 211.2, Method 1: Weight Method. The most significant anticipated challenge, obtaining satisfactory concrete workability despite using RBMA (with a high absorption and relatively high particle angularity), was successfully addressed by utilizing water-reducing admixtures. Acceptable concrete strengths were obtained using cement contents that were within the range typically used in concrete mixtures used for structural and pavement applications.

Evaluation of the test results for a series of trial mixtures resulted in identification of four baseline mixtures that were batched and tested for fresh properties and, more extensively, hardened properties. Considering strength overdesign requirements outlined in ACI 318, these baseline mixtures are representative of those that would be commercially available as 4,000 psi (27.6 MPa) and 5,000 psi (34.5 MPa) concrete mixtures. Based upon the compressive strengths obtained in trial mixtures, higher strengths were anticipated from the baseline mixtures. The lower strengths could possibly be due to the variability in the RBMA. Additional discussion on this is presented in Chapter 6, Testing Program for Recycled Brick Masonry Aggregate Concrete.

CHAPTER 6: TESTING PROGRAM FOR RECYCLED BRICK MASONRY AGGREGATE CONCRETE

6.1 Introduction

In this chapter, the testing program utilized for RBMAC mixtures is presented. Information regarding the batching procedure and mixing method is provided, along with the procedures utilized for testing of fresh and hardened concrete properties. Specimen preparation procedures and information on curing and conditioning of test specimens are detailed. The experimental results for testing of fresh and hardened properties of RBMAC mixtures are presented. Fresh properties included in the test program were slump and entrained air content. Testing of hardened concrete specimens included tests for mechanical properties, thermal characteristics, and durability performance.

6.2 Batching Procedure and Mixing Method

Batching and mixing of RBMAC mixtures was performed in general accordance with ASTM C685, "Standard Specification for Concrete Made by Volumetric Batching and Continuous Mixing." Concrete mixtures were prepared in a 5 cf portable concrete drum mixer. Mixing was performed under standard laboratory conditions. In order to reduce variability, mixing procedures remained consistent for all mixes. The RBMA was saturated prior to testing, and damp towels were used to ensure that the RBMA was at saturated surface dry (SSD) condition when it was placed into the mixer. All batch components were stored under laboratory conditions for at least 24 hrs prior to mixing, so the temperature of all batches when mixed was approximately 72ºF (22.2ºC).

6.3 Testing of Fresh Concrete Properties

For each batch of concrete prepared as part of this work, testing was performed to obtain fresh concrete properties prior to preparation of other test specimens. The fresh concrete properties obtained included slump and entrained air content. Experimental procedures used to obtain the fresh concrete properties, as well as information pertaining to the test results, are presented in the subsequent sections.

6.3.1 Experimental Procedures

Testing to obtain fresh concrete properties was performed in accordance with applicable ASTM standards, as described in the subsequent sections. Variations from ASTM standards, if any, are noted.

6.3.1.1 Slump

The target slump for concrete mixtures prepared as part of this work is 4 to 8 in. Slump measurements were performed in general accordance with ASTM C43 "Standard Test Method for Slump of Hydraulic-Cement Concrete."

6.3.1.2 Entrained Air Content

The target entrained air content for concrete mixtures prepared as part of this work is 5 to 8%. Entrained air contents were obtained in general accordance with ASTM C138 "Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete." This test method was used due to the relatively high absorption of the RBMA, which indicates a relatively high porosity. Use of ASTM C231, "Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method" is recommended for relatively dense aggregates. This test procedure cannot be used for highly porous aggregates (such as RBMA), as the pressure method can give inaccurate values due to the voids in the aggregates.

6.3.2 Experimental Results

Results of tests of fresh concrete properties for each batch of RBMAC prepared as part of this work are discussed in the following sections. Additional information is presented in Appendix C, as noted.

6.3.2.1 Slump

Slumps obtained for the trial mixtures of RBMAC (mixtures BAC 1.0 through BAC 4.6) are presented in Table 5-1. The slumps for these batches ranged from 3½ to 10 in (88.9 to 254 mm), with some of the slumps outside the target range of 4 to 8 in (101.6 to 25.4 mm). The four trial batches had the lowest measured slump, at 3½ in (88.9 mm). Two of these four batches did not utilize a water reducing admixture. The mixture that had the 10 in (254 mm) slump was the first trial batch to utilize the high-range water reducing admixture. For subsequent mixtures utilizing the high-range water reducing admixture, the dosage was reduced. Slumps within the target range of 4 to 8 in (101.6 to 25.4 mm), were obtained.

Slumps obtained for the baseline mixtures of RBMAC (mixtures BAC 5.0, BAC 6.0, BAC 6.1, and BAC 6.2) ranged from 3½ to 6 in (88.9 to 152.4 mm), and are presented in Table 5-2. Although the 3½ in (88.9 mm) slump obtained for BAC 6.2 was outside of the target slump range, the mixture was judged to be workable, and the test specimens exhibited adequate compaction when demolded. It is likely that if a small additional amount of high-range water reducing admixture was added to the mixture, the slump would have fallen within the target range without the w/c ratio being adversely affected.

6.3.2.2 Entrained Air Content

The entrained air contents of the trial batches of RBMAC are shown in Table 5-1. The measured entrained air contents of the trial batches ranged from 4.5 to 8.0%, and were within the target range of 4 to 8%. The measured entrained air contents of the baseline RBMAC mixtures ranged from 5½ to 8%, and were also within the target range of 4 to 8%.

6.4 Testing of Hardened Concrete Properties

Information pertaining to preparation of test specimens, curing and conditioning of test specimens, and testing of hardened concrete properties is presented in the subsequent sections. Tests were performed on hardened RBMAC specimens to determine the mechanical properties of compressive strength, splitting tensile strength, flexural strength (modulus of rupture), modulus of elasticity, and drying shrinkage. Testing was also performed to determine thermal characteristics of RBMAC, including the coefficients of thermal expansion, thermal conductivity, and heat capacity. Additionally, tests to evaluate the durability performance of RBMAC, including air and water permeability, abrasion resistance, chloride resistance, and surface resistivity were performed.

6.4.1 Specimen Preparation Procedures

Specimens were prepared in general accordance with ASTM C192, “Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory.” For each test performed, preparation of test specimens was performed in general accordance with the ASTM standard(s) outlining the test procedure. Where ASTM standards were not used, a detailed description of specimen preparation is presented in the test procedures detailed in Section 6.4.3, Experimental Procedures.

6.4.2 Curing and Conditioning of Test Specimens

Most specimens were cured in general accordance with ASTM C192, "Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory." Where curing conditions differed from this method, the deviations are outlined in the test procedures outlined in Section 6.4.3, Experimental Procedures. For each test performed, conditioning of test specimens was performed in general accordance with the ASTM standard(s) outlining the test procedure. Where ASTM standards were not used, a detailed description of conditioning is presented in the test procedures detailed in Sections 6.4.3, Experimental Procedures.

6.4.3 Experimental Procedures

Test methods and procedures utilized for obtaining hardened concrete properties of RBMAC are discussed in the subsequent sections. Variations from ASTM standards, if any, are noted. Where ASTM standards were not used, a detailed description of the experimental procedure utilized is presented.

6.4.3.1 Equilibrium Density

The equilibrium density of the baseline RBMAC mixtures was determined in accordance with ASTM C567, "Standard Test Method for Determining Density of Structural Lightweight Concrete." The controlled humidity enclosure utilized for conditioning the specimens was a room controlled at 73.5 ± 5°F (23.1 ± 2.7°C) with a relative humidity of 50 ± 5%. For each baseline mixture, two test specimens were tested to determine the equilibrium density.

6.4.3.2 Mechanical Properties

The mechanical properties of the RBMAC mixtures were obtained in accordance with the procedures outlined below. For trial (preliminary) mixtures, testing was only performed to obtain the compressive strength of each mixture. For the baseline mixtures (BAC 5.0, BAC 6.0, BAC 6.1, and BAC 6.2), testing was performed to determine the splitting tensile strength, flexural strength (modulus of rupture), modulus of elasticity, and thermal characteristics of the RBMAC.

6.4.3.2.1 Compressive Strength

Compressive strength testing of RBMAC was performed in accordance with ASTM C39, "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens." Strength measurements were performed at 3, 14, 28, and 90 days after casting in order to generate strength-time relationships. For the preliminary mixtures, two or three 4 in by 8 in (101.6 mm by 203.2 mm) concrete cylinders were tested at each age. For the baseline mixtures, two or three 6 in by 12 in (152.4 mm by 304.8 mm) concrete cylinders were tested at each age. Compressive strength tests for the baseline mixtures were performed on the same cylinders used for testing to determine the modulus of elasticity, as outlined in Section 6.4.3.2.4, Modulus of Elasticity.

6.4.3.2.2 Splitting Tensile Strength

The splitting tensile strength test is commonly used to estimate the tensile strength of concrete. This is an indirect test performed on a standard cylinder that is turned on its side, with its axis horizontally placed between the platens of the testing machine.

Compression is applied along the length of the cylinder, and tensile stresses are distributed along the vertical diameter of the specimen (Mindess et al. 2003).

For the baseline mixtures, splitting tensile strength testing was determined in accordance with ASTM C496, “Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens.” Splitting tensile strength testing was performed at 28 days after casting. Two 6 in by 12 in (152.4 mm by 304.8 mm) concrete cylinders were tested for each baseline RBMAC mixture.

6.4.3.2.3 Flexural Strength (Modulus of Rupture)

Flexural strength testing to determine the modulus of rupture of the baseline RBMAC mixtures was performed in accordance with ASTM C78, “Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading).” Testing was performed at 7 days after casting. For each baseline RBMAC mixture, two 6 in by 6 in by 20 in (152.4 mm by 152.4 mm by 508 mm) beams were tested.

6.4.3.2.4 Modulus of Elasticity and Poisson’s Ratio

Modulus of elasticity testing of the baseline RBMAC mixtures was performed in accordance with ASTM C469, “Standard Test Method for Static Modulus of Elasticity and Poisson’s Ratio of Concrete in Compression.” Tests were performed at 3, 7, 28, and 90 days after casting. For each baseline RBMAC mixture, two or three 6 in by 12 in (152.4 mm by 304.8 mm) concrete cylinders were tested at each age.

6.4.3.2.5 Drying Shrinkage

Testing was performed in accordance with ASTM C157, “Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete” on the baseline

RBMAC mixtures to evaluate drying shrinkage. Two test specimens, each 3 in by 3 in in (76.2 mm by 76.2 mm) cross section with a length of 11¼ in (285.8 mm), were cast from each of the baseline mixtures. Each test specimen included a gage stud in each end. The demolding, curing, and initial measurement procedures outlined in ASTM C157 were also performed. The test specimens were then stored in water saturated with lime (at 73 ± 3ºF, or 22.9 ± 1.5ºC) for 28 days, and length measurements were again taken. Specimens were then placed into an environmental chamber where they were maintained at a temperature of 73 ± 3ºF (22.9 ± 1.5ºC) and a relative humidity of 50 ± 4% for the duration of the test.

6.4.3.2.6 Thermal Characteristics

Thermal characteristics, including CTE, thermal conductivity, and heat capacity, of the test specimens prepared from one baseline RBMAC mixture, BAC 6.2, were obtained. The test methods used to obtain these thermal characteristics are similar to those used to determine the thermal characteristics of representative brick, clay tile, and mortar specimens (described previously in Section 3.2.1.6, Thermal Characteristics of Chapter 3, Testing Program for Characterization of Recycled Materials). Differences are noted in the descriptions presented in the following sections.

6.4.3.2.6.1 Coefficient of Thermal Expansion

The currently accepted method for testing to determine the CTE for concrete is AASHTO T336, "Standard Method of Test for Coefficient of Thermal Expansion of Hydraulic Cement Concrete." Equipment required to perform AASHTO T336 is not currently available at UNC Charlotte. Therefore, testing to determine a reasonable value for the CTE of RBMAC was performed utilizing a methodology that used specimen conditioning and temperature ranges similar to those used in the AASHTO T336 method.

A portion of a four in diameter cylinder of BAC 6.2 was utilized for the test. Prior to being used for this testing, this cylinder was used for equilibrium density testing, and then subsequently stored in controlled laboratory conditions for approximately 21 months. Metal gage studs (to serve as reference points) were mounted in the cylinder at an approximate distance of 4 in (101.6 mm) from each other. A photograph of the cylinder with the metal gage studs is shown in Figure 6-1.

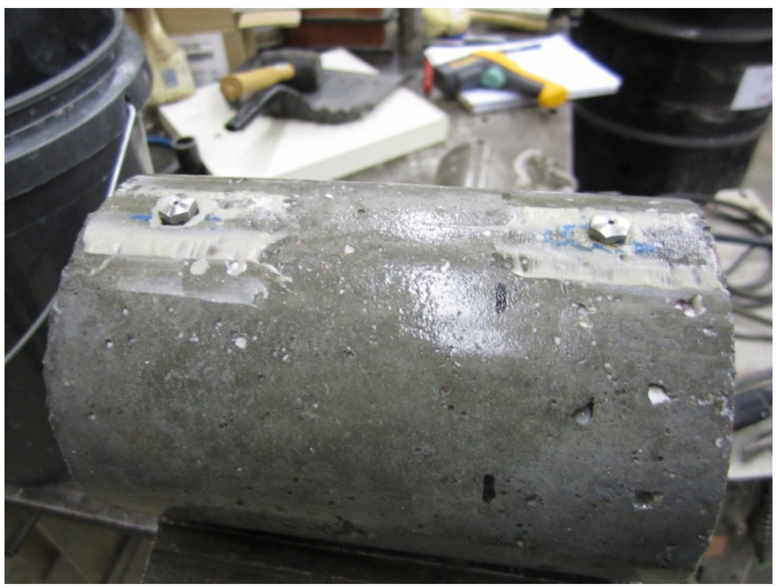

Figure 6-1: Reference studs mounted in cylinder used for coefficient of thermal expansion testing.

The test specimen was saturated in lime water at 68ºF (20°C) until a constant mass was achieved. An initial length measurement between the reference points was then obtained using a mechanical strain gage apparatus, shown in Figure 6-2. Prior to obtaining the initial measurement (and subsequent measurements), the mechanical strain gage was calibrated (zeroed at 4 in length) using the reference bar provided by the equipment manufacturer. This was done in lieu of an accepted standard or calibration specimen for this test method.

Figure 6-2: Mechanical strain gage used to measure length change of cylinder during testing to determine coefficient of thermal expansion.

The test specimen was then placed back in the saturated lime water, the container sealed, and placed in an environmental chamber. The environmental chamber was set to 50°F (10°C). Once a sensor in the water surrounding the test specimen registered 50°F (10°C), the specimen was allowed to remain in the lime water bath in the environmental chamber (at 50°F, or 10°C) overnight. The following morning, the specimen was removed from the lime water bath, and the mechanical strain gage was used to obtain a length measurement between the reference points. An infrared thermometer was used to obtain the temperature of the specimen at the time of measurement. The specimen was returned to the lime water bath, the container was sealed, and the container was placed back into the environmental chamber. The environmental chamber was then set to 122°F (50°C).

Once the sensor in the water surrounding the test specimen registered 122°F (50°C), the specimen was allowed to remain overnight in the lime water bath in the environmental chamber (at 122°F, or 50°C). The following morning, a mechanical strain gage was again used to obtain the length of the specimen between the reference points, and a temperature reading was taken with the infrared thermometer. The CTE was then computed using the formula

$$CTE = \frac{\left(\frac{\Delta L_a}{L_o}\right)}{\Delta T} \qquad (6\text{-}1)$$

where ΔL_a is the actual length change of the specimen during the temperature change (measured on concrete surface) ΔT and L_0 is the measured length of the specimen at room temperature. For the three conditioning temperatures used in this test (50ºF, 68ºF, and 122ºF) Eq. 6-1 is modified compute the CTE

$$CTE = \frac{\left(\frac{L_{122°F} - L_{50°F}}{L_{68°F}}\right)}{T_{122°F} - T_{50°F}} \qquad (6\text{-}2)$$

where $L_{122ºF}$, $L_{68ºF}$, and $L_{50ºF}$ are the measured lengths of the specimen after conditioning in the 122ºF, 68ºF, and 50ºF environments, respectively, and $T_{122ºF}$ and $T_{50ºF}$ are the measured surface temperatures of the concrete specimen (at time of length measurement) after conditioning in the 122ºF and 50ºF environments, respectively.

Due to the lack of specimens remaining at the time of testing, only one specimen (cast from BAC 6.2) was tested. The test was performed three times on the test specimen, with the results subsequently averaged.

6.4.3.2.6.2 Thermal Conductivity

As outlined in Section 3.2.1.6.2, Thermal Conductivity, in Chapter 3, Testing Program for Characterization of Recycled Materials, thermal conductivity testing was performed using the TCi apparatus manufactured by C-Therm Technologies. This equipment measures the effusivity of a specimen. Using the measured effusivity, proprietary software computes the thermal conductivity of the specimen by utilizing a user-selected calibration curve that relates effusivity to thermal conductivity. The calibration curves are either supplied by the manufacturer or developed by the user with a material similar to the tested material. For RBMAC, the calibration curves were either in the range of “polymer” or “ceramic” as supplied by the manufacturer, depending on the location tested.

Tests were performed on a specimen of RBMAC mixture BAC 6.2. The test specimen was a ½ in (12.7 mm) thick slice that was part of an equilibrium density test specimen of BAC 6.2 that had been stored in laboratory conditions for approximately 21 months. Readings were taken at 9 randomly selected locations on the sawcut slice, and then averaged in order to obtain a thermal conductivity value representative of this composite material. This test setup is shown in Figure 6-3.

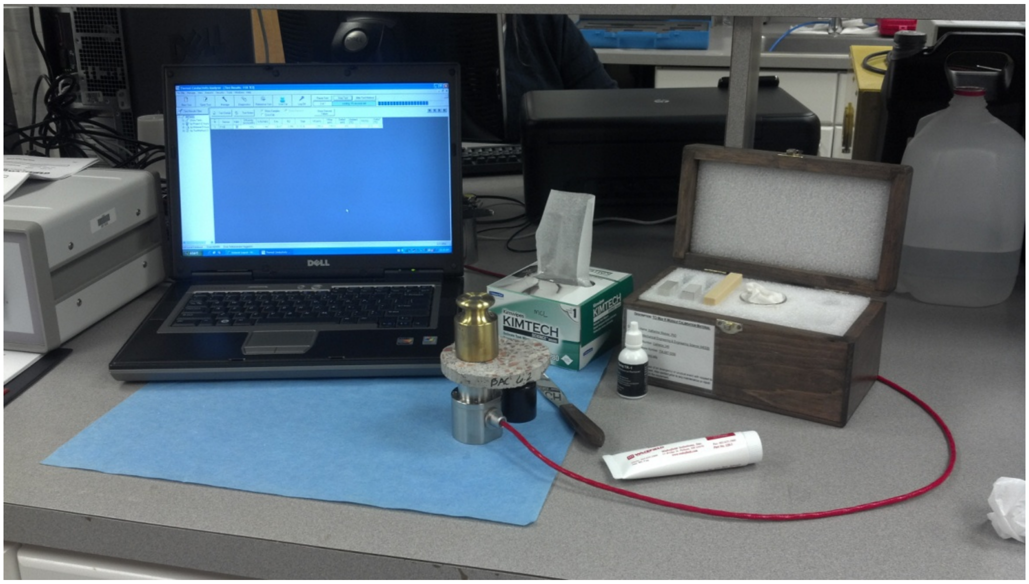

Figure 6-3 TCi apparatus used for determining thermal conductivity.

For all test locations, thermal grease (Wakefield thermal joint compound 102, recommended by the TCi apparatus manufacturer) was used as the contact agent between the test specimen and the glass-plated sensor, and a minimum of 9 readings were averaged to compute the thermal conductivity for the location. As recommended by the TCi apparatus manufacturer, the first reading was not used in computations.

6.4.3.2.6.3 Heat Capacity

The heat capacity of the RBMAC was determined using a thermogravimetric analyzer (TGA) apparatus, the SenSys Evo by Setaram, shown in Figure 6-4. This calorimeter apparatus requires that the test specimen be ground to a powder and then compacted into a metal crucible, which is subsequently placed into the machine. For this testing, powder samples were prepared from a piece of RBMAC concrete removed from a BAC 6.2 test cylinder. This test cylinder was originally used for equilibrium density testing, and prior to crushing, was stored in laboratory conditions for approximately 21 months. The powder sample was prepared by crushing pieces of the RBMAC with a modified proctor hammer, and then sieving the resulting material. Powder used for heat capacity testing was the material passing a No. 60 sieve but retained on a No. 200 sieve.

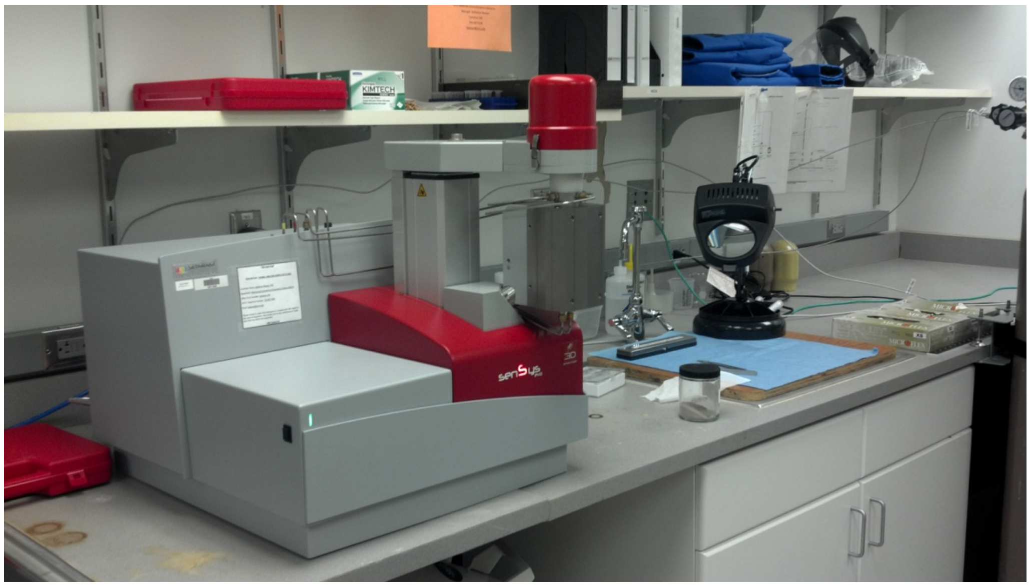

Figure 6-4: Thermogravimetric analyzer apparatus used for determining heat capacity.

Testing was performed at temperatures corresponding to warm service temperatures for concrete exposed to outdoor conditions, approximately 75°F to 130°F (24°C to 55°C). The carrier gas utilized for testing was nitrogen, and aluminum 0.000061 cubic in (100 μL) crucibles were used. The samples were heated at a rate of 10 degrees K per min. Four samples of crushed RBMAC (BAC 6.2) were tested.

6.4.3.3 Durability Performance Testing

Testing was performed on the baseline RBMAC mixtures to evaluate the potential durability performance of concrete made with RBMA as coarse aggregate. Tests related to durability performance that were included in this program include tests to determine air and water permeability, abrasion resistance, chloride ion permeability, and surface resistivity.

6.4.3.3.1 Air and Water Permeability

Tests to evaluate the air and water permeability of the baseline RBMAC mixtures were performed in general accordance with the Figg method as outlined in ACI 228.2R-98, “Nondestructive Test Methods for Evaluation of Concrete in Structures,” using the Poroscope Plus test equipment manufactured by NDT James Instruments. Due to the exhaustible supply of RBMA, only a limited number of test specimens could be batched from each of the baseline mixtures. Therefore, tests to determine the air and water permeability of the baseline RBMAC mixtures was performed on the sides of beam specimens used for modulus of rupture testing.

For each of the baseline mixtures, one beam specimen was tested. Four locations were tested per beam, and the results were averaged. Both air permeability and water permeability tests were performed at each test location, as outlined by the Poroscope Plus equipment manufacturer.

At each test location, a 0.394 in (10 mm) in diameter and 1.57 in (40 mm) deep hole was drilled, as specified by the manufacturer of the Poroscope Plus equipment. The four test holes drilled at the corners of a 3 in (76.2 mm) square. The configuration of the test holes in a beam specimen, as well as the Poroscope Plus equipment, is shown in Figure 6-5.

Figure 6-5: Air permeability testing using the Poroscope Plus equipment.

The holes were blown out with compressed air and the manufacturer-provided test plugs were inserted into each hole. A 0.394 in (10 mm) by 0.787 in (20 mm) void was left at the bottom of each hole. The test plugs were expanded into the side walls of the hole using plastic screws provided by the test equipment manufacturer. A lubricant was applied to the top of each test plug, and a hypodermic needle was inserted into each of the test plugs. A thin section of wire was subsequently inserted into each needle to ensure that it was clear. These hypodermic needles remained in each test plug for both air and water permeability testing.

At each test location, air permeability testing was performed first. The air tubing from the Poroscope Plus equipment was connected to a hypodermic needle in one of the test plugs, and a hand vacuum pump was used to evacuate the air from the void to a vacuum pressure less than 7.98 psi (55 kPa). The Poroscope Plus equipment was then used to measure the time for the pressure in the hole to increase from -7.98 psi (-55 kPa) to -7.25 psi (-50 kPa). At each test location, this procedure was repeated until the subsequent time readings stabilized to within approximately 2%.

After air permeability readings were taken on each of the four holes, water permeability testing was performed. The air tubing was removed from the Poroscope Plus equipment, and the water line was attached to the equipment. The water line was connected from the Poroscope Plus equipment to one of the hypodermic needles in the test plugs. This test setup is shown in Figure 6-6.

Figure 6-6: Water permeability testing using the Poroscope Plus equipment.

A large syringe was used to force distilled water through the Poroscope Plus equipment and the water line until the void in the concrete at the bottom of the test hole was full of water, and all air had been removed from both the test hole and the water line. For each test, the Poroscope Plus equipment measures the time it takes for a meniscus in the water tubing to move 1.97 in (50 mm), and this reading was recorded as the water permeability reading. At each test location, the procedure was repeated until readings stabilized within 2%.

6.4.3.3.2 Abrasion Resistance

Abrasion resistance of the baseline RBMAC mixtures was determined in accordance with ASTM C944, “Standard Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating-Cutter Method.” Due to the exhaustible supply of the RBMA, only a limited number of test specimens could be batched from each baseline mixture. Therefore, tests to determine the abrasion resistance of baseline RBMAC mixtures was performed on the sides of beam specimens used for modulus of rupture testing.

For each test specimen, three locations were tested. As specified by FHWA (2006), abrasion testing using the rotary cutter method was performed at each location with an applied force of 44 lb-f (196 N), for three 2 min abrasion periods (a total of 6 min of abrasion time per location). A calibrated load cell with a digital output was attached to the rotary drill press to assist in verifying the force applied to the rotary cutter device. A photograph of abrasion resistance testing by the rotary cutter method, including the calibrated load cell attached to the drill press, is shown in Figure 6-7.

Figure 6-7: Abrasion testing using the rotary cutter method.

The depth of abrasion was determined per ASTM C779, “Standard Test Method for Abrasion Resistance of Horizontal Concrete Surfaces,” Procedure B. The apparatus used to determine the depth of abrasion is a needle mounted onto a frame and stand. The needle is attached to a dial gage capable of measuring vertical movement of the needle with an accuracy of ±0.0039 in (±0.1 mm). This apparatus is shown in Figure 6-8.

Figure 6-8: Apparatus used to determine the depth of abrasion.

6.4.3.3.3 Chloride Ion Permeability

Resistance of the baseline RBMAC mixtures to chloride ion penetration was evaluated based on test measurements obtained using ASTM C1202, "Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration." In this test, an electrical potential is applied to a saturated concrete specimen that is in contact with sodium chloride and sodium hydroxide solutions (at negative and positive terminals, respectively). During the 6 hr test, chloride ions migrate through the concrete specimen from the negative terminal towards the positive terminal, and current readings are taken every 30 min. The total charge passed (in Coulombs) after 6 hr has been related to the susceptibility of concrete to chloride permeability.

Specimens used in this test were 2 in (50.8 mm) thick slices cut from concrete cylinders. The concrete cylinders used for these test specimens were 4 in (101.6 mm) in diameter and were cast from the baseline RBMAC mixtures, BAC 5.0, BAC 6.0, BAC

6.1, and BAC 6.2. These cylinders were originally used for equilibrium density testing, and had been stored at laboratory conditions for approximately 14 months prior to testing.

The circumferential sides of each test specimen were coated with a fast setting epoxy, which was allowed to cure overnight. Prior to testing for chloride ion permeability, test specimens were conditioned using the vacuum saturation procedure specified in ASTM C1202. The vacuum saturation apparatus is shown in Figure 6-9.

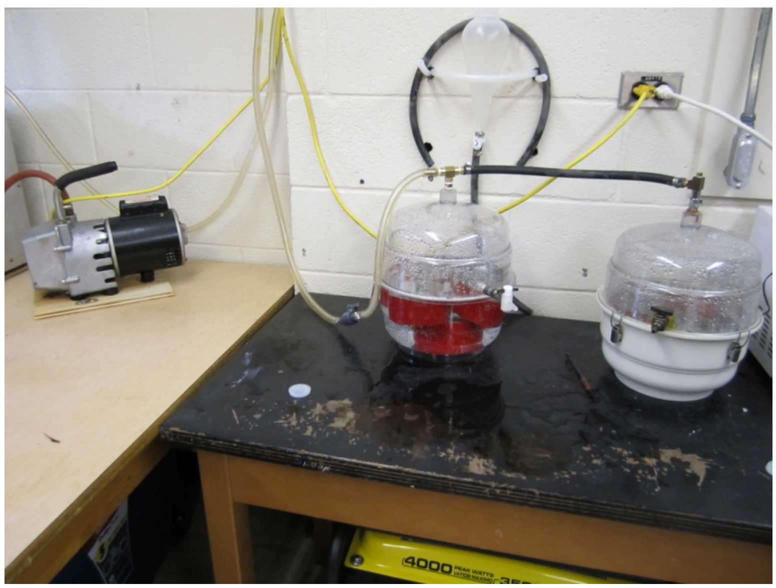

Figure 6-9: Vacuum saturation of chloride ion permeability test specimens.

Specimens were placed in a vacuum desiccator, and an attached vacuum pump was used to decrease the pressure in the desiccator to less than 50 mm Hg (6650 Pa). The specimens were allowed to remain under a vacuum for 3 hr. After 3 hr, de-aerated water was allowed to flow into the desiccator. While the de-aerated water was put into the desiccator, the vacuum pump remained on, and care was taken to not allow air to flow into the desiccator through the water tubing. The de-aired water was added to a depth that fully submerged the specimens, and the stopcock on the water feed line was closed (preventing air from flowing into the desiccator). The vacuum was allowed to run for one hour after the specimens were submerged. At the end of this hour, the vacuum pump was turned off, and air was allowed to flow into the desiccator. The specimens were then allowed to soak in the water for 18 ± 2 hr.

After vacuum saturation, each test specimen was placed between two test cells, and was sealed in place with silicone caulk. Test cells for this test were manufactured from a polymethylmethacrylate (e.g., Plexiglas) material. Each test cell had metal mesh mounted in front of a reservoir capable of holding a chemical reagent in contact with the concrete test specimens. The metal mesh in each test of the test cells facilitated application of a voltage potential across the concrete test specimen, via wires connected from banana plugs on the test cells to the RCPT equipment.

After the caulk had adequately cured, the test specimens, mounted in the test cells, were connected to the Rapid Chloride Permeability Test (RCPT) machine (manufactured by RLC Instrument Co.), with each cell on its own circuit. The test setup is shown in Figure 6-10.

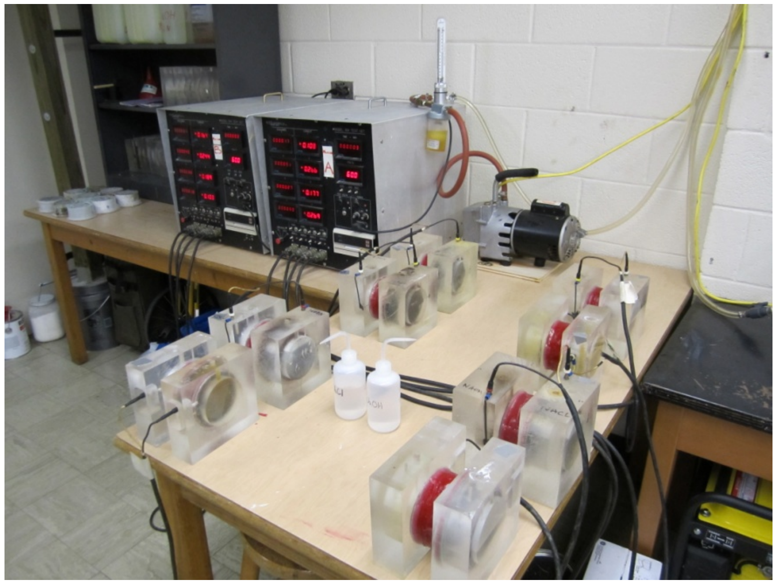

Figure 6-10: Rapid chloride permeability testing.

For each specimen, the reservoir of the test cell attached to the positive terminal was filled with a 0.3 normal sodium hydroxide solution, and the reservoir of the test cell attached to the negative terminal was filled with a 3% (by mass) sodium chloride solution. Using the RCPT equipment, a 60.0 volt potential was applied to each test cell. Instantaneous current readings and total charge passed readings are obtained at 30 min intervals for the duration of the six-hour test.

6.4.3.3.4 Surface Resistivity

Surface resistivity testing of the RBMAC was performed in general accordance with AASHTO Test Method T XXX-08, "Standard Method of Test for Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration." This test method was in draft form at the time this work was performed, and was recently adopted as AASHTO TP 95-2011. In this test method, the resistivity of 4 in by 8 in (101.6 mm by 203.2 mm) concrete cylinders is measured using test equipment that uses a 4-pin Wenner probe array. A voltage potential is applied to the concrete surface through the two outer pins of the probe, and the resultant potential difference between the inner pins is measured and subsequently converted to resistivity in kΩ•cm.

For each cylinder tested, the quarter points of the circumference (0°, 90°, 180°, and 270°) were marked on the ends of the specimen. The marks were extended onto the longitudinal side of the cylinder so that measurements could be made at four locations on each specimen. Surface resistivity readings were taken at each mark relative to mid depth of the cylinder. For each test, the cylinder was rotated twice, with readings taken at each quarter point, so that a total of eight readings were taken. The eight readings were then averaged to obtain the surface resistivity measurement. A photograph of surface resistivity testing is shown in Figure 6-11.

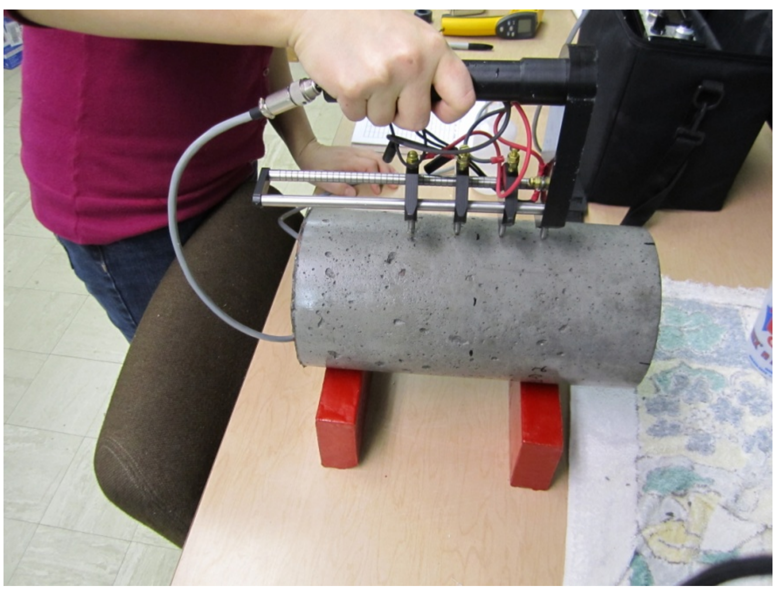

Figure 6-11: Surface resistivity testing.

Due to the sensitivity of surface resistivity to temperature, testing was performed at several temperatures ranging from approximately 35°F to 110°F (1.7°C to 43.3°C) to evaluate the resistivity of the RBMAC at temperatures typical of service temperatures of pavements and other structures. To condition the specimen for testing at each temperature, the specimen were placed in a water bath and allowed to remain at the test temperature for a minimum of one hr prior to testing. Heating and cooling was obtained via hot water and ice, as shown in Figure 6-12.

Figure 6-12: Conditioning of test specimens for surface resistivity testing at low temperatures.

6.4.4 Experimental Results

The results of tests on RBMAC, to determine hardened concrete properties and thermal characteristics, are outlined in the subsequent sections. The results are also discussed.

6.4.4.1 Equilibrium Density

The equilibrium density of the baseline RBMAC mixtures ranged from 111.8 to 128.2 pcf (1791 to 2054 kg/m^3), as shown in Table 6-1. Supporting data for this test are provided in Table C-1 in Appendix C.

Table 6-1: Equilbrium densities of the baseline RBMAC mixtures.

	RBMAC Mixture			
	BAC 5.0	**BAC 6.0**	**BAC 6.1**	**BAC 6.2**
Equilibrium density (pcf)	111.8	128.2	127.4	125.5

Each of the baseline mixtures have a coarse aggregate (RBMA) content of 1178.6 pcy (699 kg/m^3). The cement contents of both BAC 5.0 and BAC 6.0 are 675.0 pcy (400 kg/m^3), but the w/c ratio for BAC 5.0 is 0.43 and the w/c ratio for BAC 6.0 is 0.32. The void space left by the additional water contained in BAC 5.0 is likely responsible for the significantly lower equilibrium density. The w/c ratios of mixtures BAC 6.0, BAC 6.1, and BAC 6.2 are each 0.32, but the cement contents of each decreased. The cement contents of BAC 6.0, BAC 6.1, and BAC 6.2 are 675 pcy (400 kg/m^3), 625 pcy (625 kg/m^3), and 575 pcy (341 kg/m^3), respectively. Therefore, it appears that the equilibrium densities of these RBMAC mixtures are more sensitive to water content than cement content.

According to ACI 213, "Guide for Structural Lightweight-Aggregate Concrete," structural lightweight concrete mixtures are defined as follows:

"Structural lightweight-aggregate concrete made with structural lightweight aggregate as defined in ASTM C330. The concrete has a minimum 28-day compressive strength of 2500 psi (17 MPa), an equilibrium density between 70 and 120 lb/ft^3 (1120 and 1920 kg/m^3), and consists entirely of lightweight aggregate or a combination of lightweight and normal-density aggregate."

The RBMAC mixtures developed as part of this work utilize lightweight aggregate and normalweight fine aggregate. As discussed in Section 3.3.2.5, Bulk Density (Unit Weight), the RBMA produced from the Idlewild Elementary School demolition waste has a bulk density (60.9 pcf) that is slightly greater than the maximum bulk density for lightweight coarse aggregates specified in ASTM C330, which is 55 pcf, and therefore does not meet the requirements of a structural lightweight concrete as outlined in ACI 213.

As outlined in Table 6-1 above, the equilibrium densities of the baseline RBMAC mixtures ranged from 111.8 to 128.2 pcf (1791 to 2054 kg/m^3). Equilibrium densities for the RBMAC mixtures that utilized a high-range water reducing admixture (BAC 6.0, BAC 6.1, and BAC 6.2) were higher than the equilibrium density range presented in ACI 213's definition of structural lightweight concrete. Average 28-day compressive strengths for these three mixtures were higher than 2,500 psi (17.2 MPa), the minimum 28-day compressive strength included in ACI 213's definition of structural lightweight concrete, thus these RBMAC mixtures met this part of the definition.

The equilibrium density of BAC 5.0 does, however, meet the range of equilibrium densities for structural lightweight concrete as defined in ACI 213, and the average 28-day compressive strength of 3,675 psi also meets this definition. Therefore, if water reducers are not used, the RBMAC mixtures may fall within the ranges of equilibrium density and compressive strength outlined for structural lightweight concrete mixtures.

6.4.4.2 Mechanical Properties

In order to conserve the RBMA available for this work, only compressive strength tests were performed on the trial batches of RBMAC. The results of this compressive strength testing are shown in Table 5-1 and in Table B-1. For the baseline RBMAC mixtures, testing to determine other mechanical properties was performed. In order to facilitate comparison between the four baseline mixtures, a summary of the mechanical property test results is presented in Table 6-2. A discussion of the results for each of the mechanical property tests performed is presented in the subsequent sections.

Table 6-2: Mechanical properties of the baseline RBMAC mixtures.

	RBMAC Mixture			
	BAC 5.0	**BAC 6.0**	**BAC 6.1**	**BAC 6.2**
3-day compressive strength (psi)	2,139	4,559	3,684	4,508
7-day compressive strength (psi)	2,858	6,182	4,074	5,283
28-day compressive strength (psi)	3,675	6,497	5,307	6,450
90-day compressive strength (psi)	3,872	6,903	5,362	7,343
28-day splitting tensile strength (psi)	320	439	484	387
7-day flexural strength (modulus of rupture) (psi)	519	797	730	716
3-day modulus of elasticity (psi)	2,200,000	3,340,000	3,120,000	3,600,000
7-day modulus of elasticity (psi)	2,753,000	3,977,000	3,467,000	3,430,000
28-day modulus of elasticity (psi)	2,783,000	3,840,000	3,563,000	3,903,000
90-day modulus of elasticity (psi)	2,905,000	3,960,000	3,645,000	3,875,000
3-day Poisson's ratio	0.18	0.16	0.18	0.19
7-day Poisson's ratio	0.21	0.17	0.21	0.14
28-day Poisson's ratio	0.18	0.16	0.17	0.16
90-day Poisson's ratio	0.17	0.18	0.18	0.17

6.4.4.2.1 Compressive Strength

Average twenty-eight day compressive strengths for the baseline RBMAC mixtures ranged from 3,675 psi (25.3 MPa) (BAC 5.0) to 6,497 psi (44.8 MPa) (BAC 6.0). The average 90-day compressive strength of BAC 6.2 reached almost 7,350 psi (50.6 MPa) with a cement content of only 575 pcy (341 kg/m^3), which is within the range of typical cement contents used in commercially available concrete mixtures. As discussed in Section 5.2.2, Final Mixture Proportions, compressive strengths obtained from the baseline RBMAC mixtures were reasonable for commercially available 4,000 psi (27.6 MPa) and 5,000 psi (34.5 MPa) concrete. Mixtures of these strengths would be suitable for use in pavement and structural applications.

The 3-day, 7-day, 28-day, and 90-day compressive strengths in Tables 5-1, 5-2, and 6-2 are typically the average of three test cylinders. As would be expected, some variability was observed in the compressive strengths of the mixtures. Figures 6-13 to 6-16 are plots of the average compressive strength results and the range of the test results used to compute the averages for the baseline RBMAC mixtures (BAC 5.0, BAC 6.0, BAC 6.1, and BAC 6.2). Supporting data for Figures 6-13 to 6-16 are provided in Appendix C in Table C-2. Photographs of test specimens are shown in Figures C-1 to C-16.

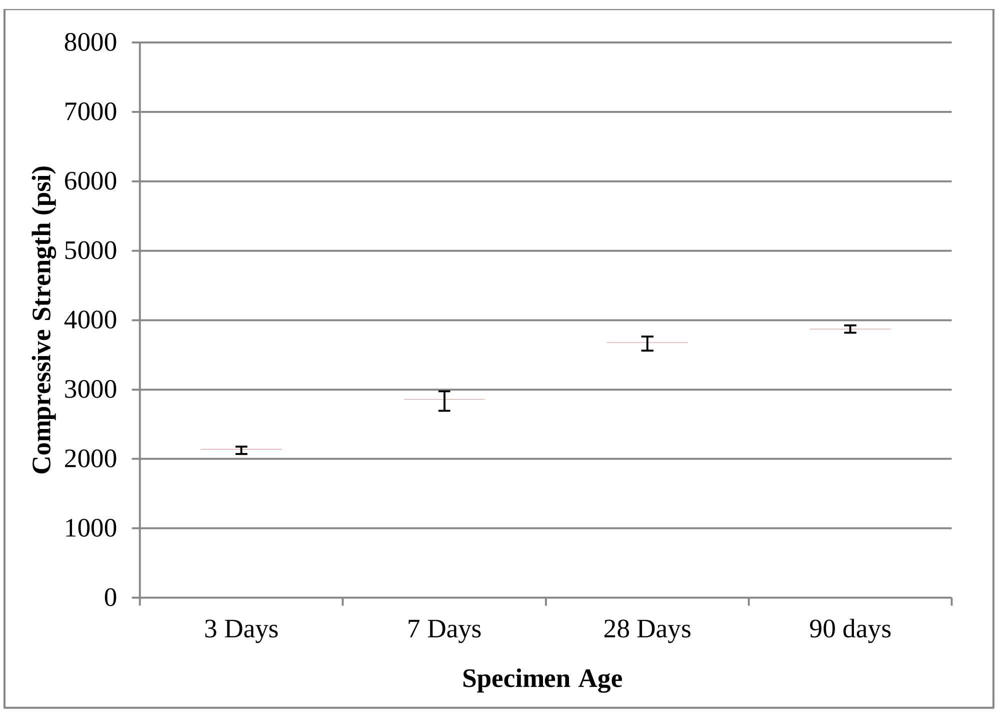

Figure 6-13: Average compressive strength results for BAC 5.0.

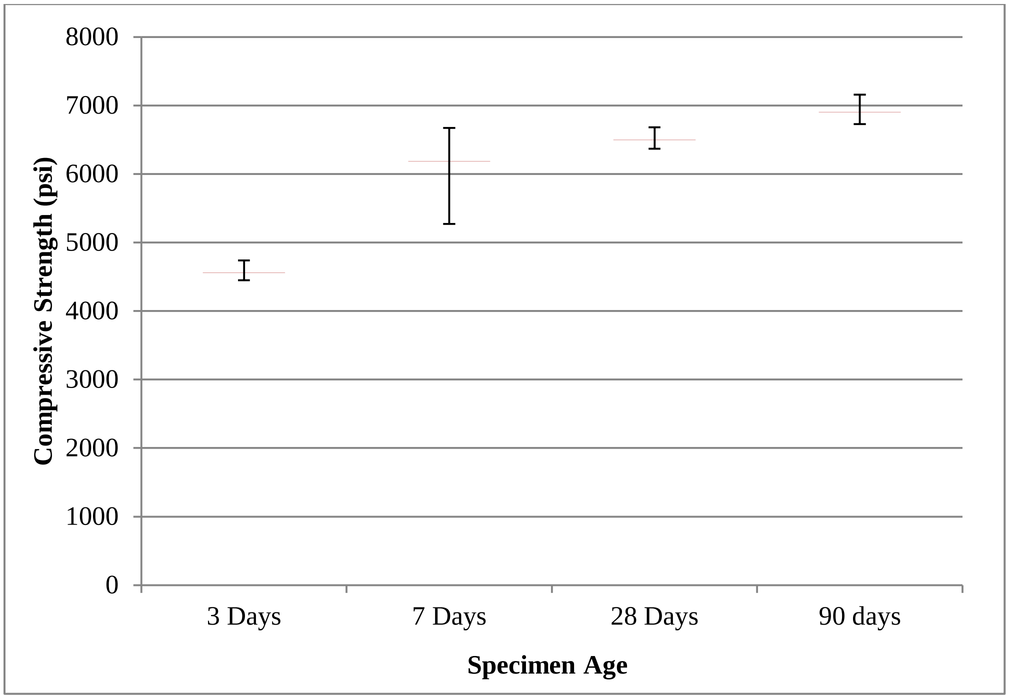

Figure 6-14: Average compressive strength results for BAC 6.0.

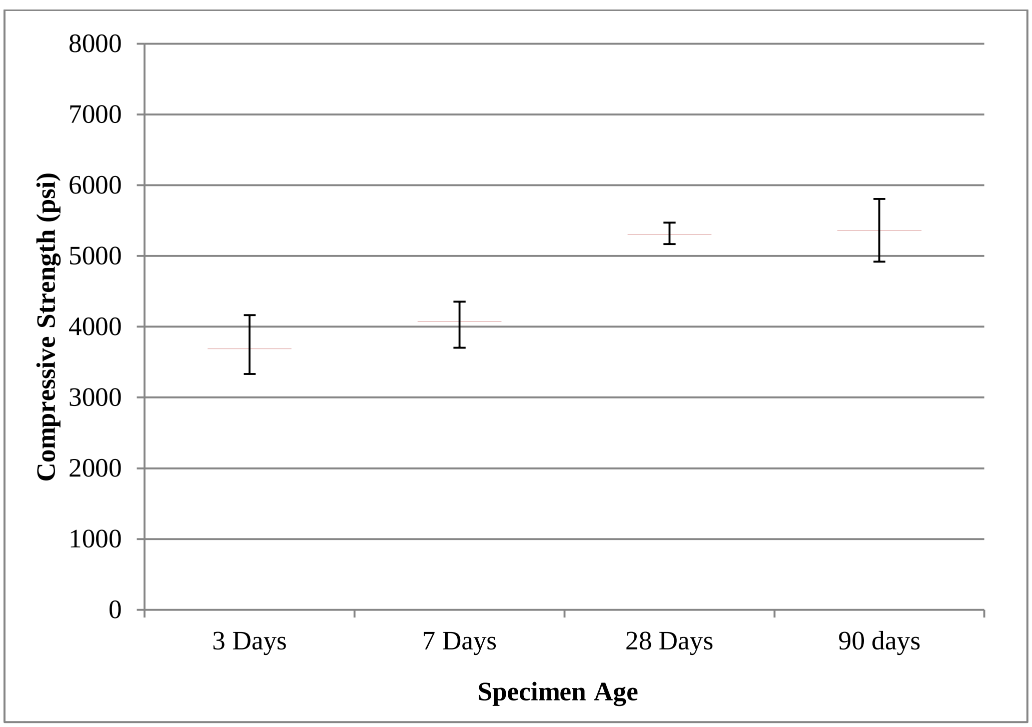

Figure 6-15: Average compressive strength results for BAC 6.1.

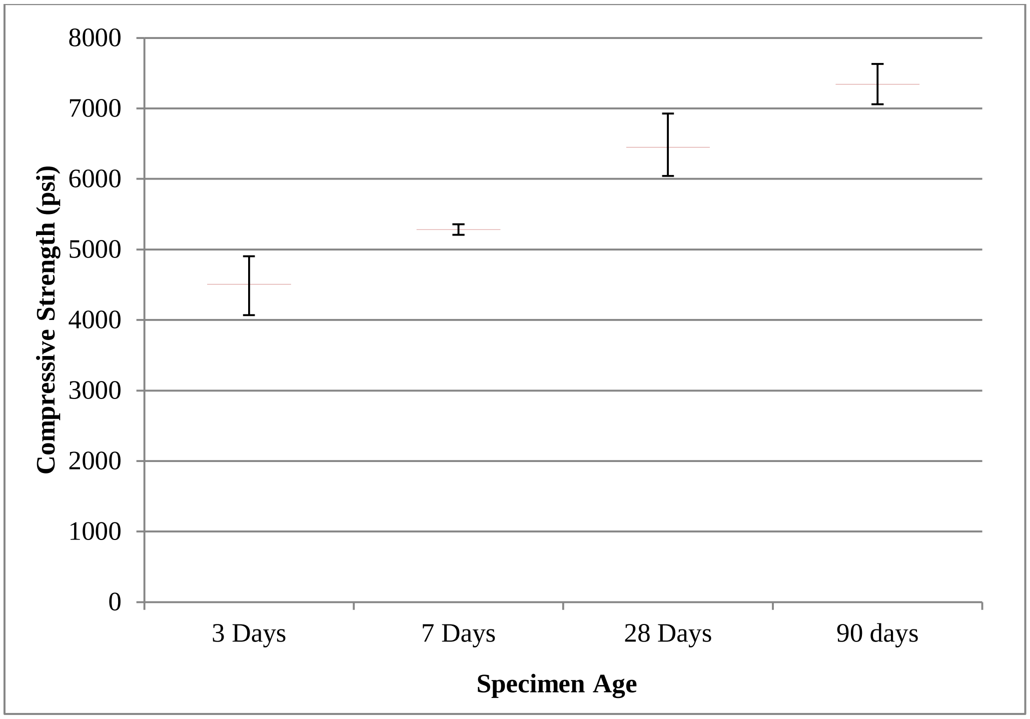

Figure 6-16: Average compressive strength results for BAC 6.2.

It can be observed from these plots that for a single average compressive strength result at a specific age, the range of test results could vary over several hundred psi to more than 1,000 psi. Commercial ready-mixed concrete suppliers are likely to have more confidence in mixtures that provide consistent compressive strength test results. The plots shown in Figures 6-13 through 6-16 indicate that, although multiple RBMAC cylinders tested at the same age can provide relatively consistent compressive strengths, there exists the potential for significant variability in the same-day compressive strengths.

One of the potential causes of this variability could be the inclusion of a small amount of contaminant waste, present in the initial demolition debris, in the RBMA. Contaminant particles could cause weak points within the concrete matrix, and provide locations for cracks to initiate (and propagate more easily) during compressive strength tests. This ultimately lowers the compressive strength of the test cylinder and could increase the variability for a set of cylinders tested at a certain age. Occasionally, contaminant particles were observed in one of the fractured surfaces of a compressive strength test cylinder, as shown in Figure 6-17. In this photograph, yellow arrows point towards the black particulate material that is not brick, mortar, or clay tile. It is not known exactly what this material is, but it is suspected that it is some component of the roof or an interior finish material.

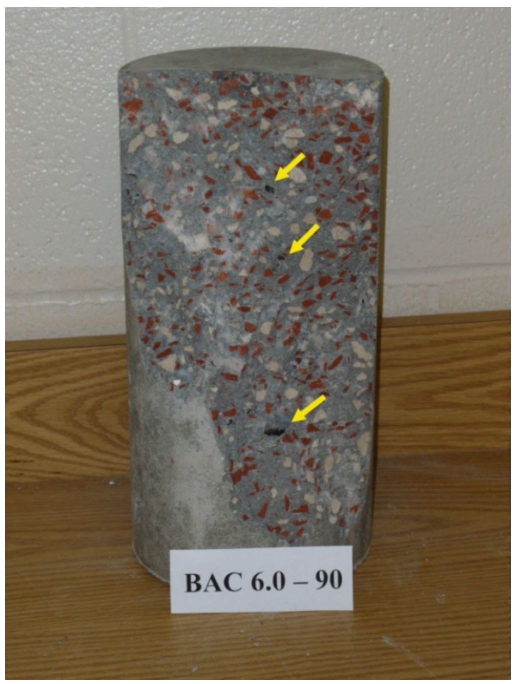

Figure 6-17: Contaminant particles are visible in the fractured surface of a compressive strength test cylinder.

6.4.4.2.2 Splitting Tensile Strength

Splitting tensile strength test results for the baseline RBMAC mixtures are presented in Table 6-2. Splitting tensile strength tests were performed at 28-days. Two cylinders were tested for each mixture, and the average splitting tensile strength is shown in Table 6-2. Supporting data are provided in Appendix C in Table C-3, and photographs of the tested specimens are provided in Figures C-17 and C-18. As outlined in Neville (1995), Oluokun (1991) suggested the relationship between the splitting tensile strength and the compressive strength of concrete

$$f_t = 1.4(f_c)^{0.7} \tag{6-3}$$

where f_t is the splitting tensile strength and f_c is the compressive strength of a cylinder, both in psi. This relationship was generated by fitting a curve to data on splitting tensile strength and compressive strength test results for normalweight concrete submitted by various investigators.

Using this Eq. 6-3, based on the 28-day average compressive strengths of the baseline RBMAC mixtures, the predicted 28-day splitting tensile strengths are given in Table 6-3 next to the actual splitting tensile strengths. The percent difference between actual and predicted splitting tensile strength was computed using

$$\%\, difference = 100\% \times \frac{f_{t,actual} - f_{t,predicted}}{f_{t,actual}} \tag{6-4}$$

where $f_{t,actual}$ is the actual measured splitting tensile strength and $f_{t,predicted}$ is the splitting tensile strength predicted by Eq. 6-4.

Table 6-3: Actual versus predicted splitting tensile strengths.

	RBMAC Mixture			
	BAC 5.0	**BAC 6.0**	**BAC 6.1**	**BAC 6.2**
28-day compressive strength (psi)	3,675	6,497	5,307	6,450
28-day splitting tensile strength, actual (psi)	320	439	484	387
28-day splitting tensile strength, predicted (psi)	438	653	567	650
% difference between actual and predicted splitting tensile strength	-37.0%	-48.8%	-17.1%	-67.9%

As shown in Table 6-3, the 28-day splitting tensile strengths of the baseline RBMAC mixtures were much lower than those predicted by Eq. 6-3. Although Oluokun's relationship (Eq. 6-3) was developed using normalweight concretes, Neville (1995) states that "lightweight concrete conforms broadly to the pattern of the relation between f_t and f_c for ordinary concrete." It is apparent, however, that the predicted splitting tensile strength of RBMAC is not accurately modeled by Oluokun's relationship (developed using test results for normalweight concrete).

6.4.4.2.3 Flexural Strength (Modulus of Rupture)

The results of flexural strength testing to determine the modulus of rupture for the baseline RBMAC mixtures are shown in Table 6-2. For each baseline RBMAC mixture, flexural strength tests were performed on two beams at 7 days of age. The average 7-day modulus of rupture for BAC 5.0 was 519 psi (3.58 MPa), while the average modulus of rupture values for BAC 6.0, BAC 6.1, and BAC 6.2 were all over 700 psi (over 4.83 MPa). The average modulus of rupture of BAC 6.0 was almost 800 psi (almost 5.52 MPa), the highest value obtained from the mixtures. Additional data from this testing is provided in Table C-4 in Appendix C. Typical test specimens are shown in Figures C-19 and C-20.

Typically, modulus of rupture testing is performed for concrete used in pavement applications. For many agencies, in order for the pavement to be open to traffic, a specified minimum modulus of rupture (or flexural strength) must be obtained. This specified minimum varies by agency and by type of pavement. Indiana DOT specifies that a flexural strength of at least 550 psi (3.79 MPa) must be obtained prior to opening the pavement (Olek et al. 2003). 2012 NCDOT Standard Specifications indicate that portland cement concrete for pavement applications must have a minimum flexural strength of 650 psi (4.48 MPa) at 28 days and a minimum compressive strength of 4,500 psi (31.0 MPa) at 28 days. Based on these criteria, mixtures BAC 6.0, BAC 6.1, BAC 6.2 would provide suitable flexural strengths for use in pavement applications, with the pavements able to be opened to traffic in fewer than 7 days.

6.4.4.2.4 Modulus of Elasticity and Poisson's Ratio

Testing to determine the modulus of elasticity of RBMAC was performed on the baseline mixtures at 3, 7, 28, and 90 days of age. For each age, the average modulus of elasticity, reported in Table 6-2, is the average of either two or three test cylinders. Supporting data for these tests are provided in Appendix C. A sample of data collected during a modulus of elasticity and Poisson's ratio test (and associated calculations) is shown in Table C-5. Typical plots of stress versus longitudinal strain and transverse strain versus longitudinal strain are shown in Figures C-21 and C-22, respectively. A summary of modulus of elasticity test results and Poisson's ratio test results are provided in Tables C-6 and C-7, respectively.

The moduli of elasticity for RBMAC at an early age (3 days) range from 2.2 million psi to 3.6 million psi (15.2 GPa to 24.8 GPa) for the four baseline mixtures. The mixtures that utilized a high-range water reducer (BAC 6.0, BAC 6.1, and BAC 6.2), and therefore had a lower w/c ratio (w/c = 0.32), had moduli of elasticity that were approximately 1 million psi (6.9 GPa) higher than the RBMAC mixture that did not utilize the admixture (BAC 5.0, w/c = 0.43). As expected, the moduli of elasticity generally increased, and at 90-days, ranged from 2.9 million psi (20.0 GPa) (BAC 5.0) to 4.0 million (27.6 GPa) (BAC 6.0).

Mindess et al. (2003) report typical values for moduli of elasticity of normalweight concrete and lightweight concrete as 2 to 6 million psi and 1.5 to 2.5 million psi, respectively. At later ages (28 and 90 days), the moduli of elasticity for RBMAC mixtures were typically higher than the published values for lightweight concrete, but within the range of published values for normalweight concrete. Since the modulus of elasticity of the coarse aggregate has a large influence on the modulus of elasticity of the concrete (Mindess et al. 2003), some observed variation in the measured values of moduli of elasticity could possibly be attributed to the relative proportion of brick, mortar, tile, and contaminants included in the RBMA. Other sources of variability are not readily apparent at this time.

The stress-strain behavior of concrete, and hence its modulus of elasticity, is highly dependent on the modulus of elasticity of the coarse aggregate used. As lightweight aggregate has a modulus of elasticity similar to that of the hardened cement paste, the modulus of elasticity of lightweight concrete is not as sensitive to mixture proportions as normalweight concrete (Neville 1995). Based upon the test results obtained in this work, it appears that the change in the modulus of elasticity of the RBMAC mixtures is sensitive to w/c ratio and cement content, as the RBMA content was kept constant for all of the baseline mixtures.

For concrete mixtures with unit weights (w_c) between 90 and 160 pcf, ACI 318-08 allows for the modulus of elasticity (E_c) of concrete (in psi) to be estimated using the compressive strength (f'_c) (in psi) as given by

$$E_c = {w_c}^{1.5} \times 33\sqrt{f'_c} \tag{6-5}$$

Using Eq. 6-5, based on the 28-day average compressive strengths of the baseline RBMAC mixtures and using the equilibrium density as the unit weight, the predicted 28-day modulus of elasticity as shown in Table 6-4. The percent difference between actual and predicted modulus of elasticity was computed using

$$\%\, difference = 100\% \times \frac{E_{c,actual} - E_{c,predicted}}{E_{c,actual}} \tag{6-6}$$

where $E_{c,actual}$ is the actual computed modulus of elasticity and $E_{c,predicted}$ is the modulus of elasticity predicted by Eq. 6-5.

Table 6-4: Actual versus predicted moduli of elasticity.

	RBMAC Mixture			
	BAC 5.0	**BAC 6.0**	**BAC 6.1**	**BAC 6.2**
Equilibrium density (pcf)	111.8	128.2	127.4	125.5
28-day compressive strength (psi)	3,675	6,497	5,307	6,450
28-day modulus of elasticity, actual (psi)	2,783,000	3,840,000	3,563,000	3,903,000
28-day modulus of elasticity, predicted (psi)	2,364,859	3,861,023	3,456,946	3,726,142
% difference between actual and predicted	15.0%	-0.5%	3.0%	4.5%

Using Eq. 6-5, the actual 28-day modulus of elasticity values for RBMAC were accurately predicted. The predicted values were within 5% of the actual value for the three baseline mixtures that utilized a water-reducing admixture (BAC 6.0, BAC 6.1, and BAC 6.2), and about 15% of the actual value for the baseline mixture that did not use a water-reducing admixture (BAC 5.0).

According to Mindess et al. (2003), Poisson's ratio for saturated concrete typically ranges from 0.2 to 0.3, and is typically slightly lower (about 0.18) when dried. For lightweight concrete, values of Poisson's ratio are quite dependent on the Poisson's ratio of the lightweight aggregate used, although Balendran (1995) reports values for concrete containing artificially manufactured lightweight aggregates that are similar to those of concrete containing with normalweight aggregates.

The Poisson's ratios for RBMAC mixtures tended to be slightly lower than 0.20. Similar to the modulus of elasticity, the Poisson's ratio of concrete is heavily dependent on the Poisson's ratio of the coarse aggregate. Some variation observed in this data could be due to the relative proportions of brick, mortar, clay tile, and contaminant material present in each specimen. Other sources of variability are not readily apparent at this time.

6.4.4.2.5 Drying Shrinkage

Tests to evaluate the drying shrinkage of RBMAC were initiated in accordance with ASTM C157, as outlined in Section 6.4.3.2.5, Drying Shrinkage. Photographs of typical test specimens are provided in Appendix C in Figures C-23 and C-24. Unfortunately, after the test specimens for the baseline RBMAC mixtures were placed in the conditioning chamber, an equipment malfunction occurred, and the relative humidity and temperature of the chamber was uncontrolled for several days. Therefore, results for this test are not reported.

6.4.4.2.6 Thermal Characteristics

Tests to determine the thermal characteristics of RBMAC were performed on test specimens created from BAC 6.2. BAC 6.2 was utilized because the relatively low cement content and the satisfactory compressive strength identified it as a suitable mixture for use in assessment of RBMAC in pavement applications (discussed in Chapter 7, Use of Recycled Brick Masonry Aggregate Concrete in Pavement Applications). As part of M-EPDG pavement design procedures, the thermal characteristics of concrete, particularly the CTE, are integral to the performance of concrete pavements. The results of the thermal characterization tests are presented in the following sections.

6.4.4.2.6.1 Coefficient of Thermal Expansion

The average CTE of BAC 6.2 was 5.51×10^{-6} in/in/°F (9.92×10^{-6} m/m/°C). As outlined in Section 6.4.3.2.6.1, Coefficient of Thermal Expansion, this result was obtained by averaging three readings on one test specimen. The values of the three tests were 4.40×10^{-6} in/in/°F (7.92×10^{-6} m/m/°C), 5.53×10^{-6} in/in/°F (9.95×10^{-6} m/m/°C), and 6.6×10^{-6} in/in/°F (11.88×10^{-6} m/m/°C) , which are fairly consistent given the test method utilized. Supporting test data are provided in Table C-8 in Appendix C.

The CTE of concrete is a function of the CTE of the cement paste and the CTE of the aggregate. According to Neville (1995), the CTE of hydrated cement paste varies from about 6×10^{-6} in/in/°F to about 11×10^{-6} in/in/°F (10.8×10^{-6} to about 19.8×10^{-6} m/m/°C). The CTEs for water-cured concretes of 1:6 mixture proportions containing a variety of aggregates is presented in Neville (1995) and is summarized in Table 6-5.

Table 6-5: Coefficient of thermal expansion of 1:6 concretes made with different aggregates (adapted from Neville 1995).

Type of Aggregate	**Linear Coefficient of Thermal Expansion for Water-Cured Concrete (10^{-6} in/in/°F)**
Granite	4.8
Quartzite	6.8
Dolerite	4.7
Sandstone	5.6
Limestone	3.4
Portland stone	3.4
Blastfurnace slag	5.1
Expanded slag	5.1

The value of the CTE obtained from testing of the RBMAC is comparable to the values presented in Table 6-5. The measured CTE of RBMAC is slightly lower than that of 1:6 concrete containing gravel and quartzite, but higher than that of mixtures utilizing some other aggregate materials. As discussed in Section 3.2.2.6.1, Coefficient of Thermal Expansion, the CTE of clay brick is typically between 3×10^{-6} and 4×10^{-6} in/in/°F (5.4×10^{-6} and 7.2×10^{-6} m/m/°C) (Klingner 2010). It is likely that higher brick content in the RBMA would result in a lower value of CTE for RBMAC.

The CTE of concrete has been found to be one of the more sensitive inputs in M-EPDG (Crawford et al. 2010), and lower values of the CTE have been shown to predict less cracking in pavements (Tanesi et al. 2007). Therefore, much research has been done recently to determine the CTE of concrete made with a variety of materials (mostly natural aggregates) located close to different agencies.

In “Part 2: Design Inputs” of the “Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures, Final Report (ARA 2004),” a table outlining typical ranges of CTEs for concrete (as well as common components of concrete). These data are shown in Table 6-6.

Table 6-6: Typical ranges for coefficients of thermal expansion for common components of concrete and of concrete made using these materials (from ARA 2004).

Material Type	**Coefficient of Thermal Expansion, 10^{-6} in/in/ºF**	**Concrete Coefficient of Thermal Expansion (made from this material), 10^{-6} in/in/ºF**
Aggregates		
Marbles	2.2 – 3.9	2.3
Limestones	2.0 – 3.6	3.4 – 5.1
Granites & Gneisses	3.2 – 5.3	3.8 – 5.3
Syenites, Diorites, Andesite, Basalt, Gabbros, Diabase	3.0 – 4.5	4.4 – 5.3
Dolomites	3.9 – 5.5	5.1 – 6.4
Blast Furnace Slag	Not reported	5.1 – 5.9
Sandstones	5.6 – 6.7	5.6 – 6.5
Quartz Sands & Gravels	5.5 – 7.1	6.0 – 8.7
Quartzite, Cherts	6.1 – 7.0	6.6 – 7.1
Cement Paste (saturated)		
w/c = 0.4 to 0.6	10-11	N/A
Concrete Cores		
Cores from Long Term Pavement Performance Program (LTPP)	N/A	4.0 (min), 5.5 (mean), 7.2 (max)

Sakyi-Bekoe (2008) performed testing to determine the CTE of concrete created with materials locally available in the state of Alabama. This research was performed utilizing the apparatus specified in AASTHTO TP-60. Based on results obtained in this study, the average CTE for concretes made with river gravel is 6.95×10^{-6} in/in/°F (12.51×10^{-6} m/m/°C), for concretes made with granite is 5.60×10^{-6} in/in/°F (10.1×10^{-6} m/m/°C), and for concretes made with dolomitic limestone is 5.52×10^{-6} in/in/°F (9.93×10^{-6} m/m/°C).

Wang et al. (2008) performed a similar study to determine the thermal characteristics of concrete made with materials local to Iowa. This study found that concrete made with Iowa materials had the following CTEs: 5.69×10^{-6} in/in/°F (10.24×10^{-6} m/m/°C) (limestone), 6.68×10^{-6} in/in/°F (12.02×10^{-6} m/m/°C) (dolomite), and 6.86×10^{-6} in/in/°F (12.15×10^{-6} m/m/°C) (quartzite). These were noted to all be higher than the default CTE of 5.5×10^{-6} in/in/°F (9.90×10^{-6} m/m/°C) used in M-EPDG.

6.4.4.2.6.2 Thermal Conductivity

Results of thermal conductivity testing of the test specimen prepared from BAC 6.2 are shown in Table 6-7. A photograph of the test specimen is shown in Appendix C (Figure C-25), along with typical test results from a thermal conductivity test (Figure C-26).

Table 6-7: Results of thermal conductivity testing of BAC 6.2.

	Effusivity ($W\bullet\sqrt{(s)}/(m^2\bullet K)$)	**Thermal Conductivity (W/(m•K))**	**Thermal Conductivity (BTU/(hr•ft•°F))**
BAC 6.2 location 1	1673	1.480	0.856
BAC 6.2 location 2	1118	0.770	0.445
BAC 6.2 location 3	1265	0.940	0.543
BAC 6.2 location 4	1342	1.040	0.601
BAC 6.2 location 5	754	0.400	0.231
BAC 6.2 location 6	1910	1.910	1.104
BAC 6.2 location 7	632	0.300	0.173
BAC 6.2 location 8	337	0.070	0.040
BAC 6.2 location 9	1617	1.380	0.798
BAC 6.2 Average	**1183**	**0.921**	**0.533**

The average thermal conductivity of the RBMAC test specimen prepared from BAC 6.2 is 0.533 BTU/(hr•ft•°F) (0.921 W/(m•K)). This value is somewhat lower than the typical range of values of thermal conductivity suggested for use in M-EPDG (ARA 2004) for conventional concrete, 1.0 to 1.5 BTU/(hr•ft•ºF) (1.77 to 2.60 W/(m•K)). For M-EPDG design, a typical value of thermal conductivity for conventional concrete is 1.25 BTU/hr•ft•°F (2.16 W/(m•K) is recommended (ARA 2004).

However, testing by Wang et al. (2008) using Iowa materials indicated that a thermal conductivity of 0.77 BTU/(hr•ft•°F) (1.33 W/(m•K)) may be more representative of Iowa concrete. Wang et al. (2008) indicated that further studies in this area were needed.

6.4.4.2.6.3 Heat Capacity

The results of testing to determine the heat capacity of RBMAC are shown in Table 6-8. Values of the heat capacity at 77ºF range from 0.100 BTU/(lb•ºF) to 0.209 BTU/(lb•ºF) (419 to 875 J/(kg•°C) at 25ºC). Photographs of the sample of RBMAC used for heat capacity tests are provided in Appendix C (Figures C-27 and C-28) along with a typical output spreadsheet of the TGA apparatus for heat capacity testing of RBMAC (Figure C-29).

Table 6-8: Heat capacity at 77ºF for BAC 6.2.

	Heat Capacity at 77ºF (BTU/(lb•ºF))
BAC 6.2 sample 1	0.100
BAC 6.2 sample 2	0.209
BAC 6.2 sample 3	0.146
BAC 6.2 sample 4	0.129
BAC 6.2 Average	**0.146**

The average heat capacity of RBMAC test specimens made from BAC 6.2 is measured to be 0.146 BTU/(lb•°F) at 77ºF (611 J/(kg•°C) at 25ºC). The heat capacity of concrete ranging from 130 to 140 pcf is 0.22 BTU/lb•°F (921 J/(kg•°C)) according to ACI (2002). Therefore, the heat capacity of BAC 6.2 is lower than that of concrete that uses conventional normalweight coarse aggregate.

The heat capacity of concrete is used as an input for M-EPDG. In lieu of test data for a particular concrete mixture, a reasonable range for heat capacity for conventional concrete for use in M-EPDG is 0.20 to 0.40 BTU/(lb•ºF) (837 to 1,675 J/(kg•°C)) (ARA 2004). For M-EPDG design, the value of 0.28 BTU/(lb•ºF) (1,172 J/(kg•°C)) is recommended for concrete (ARA 2004).

However, in their study of the thermal characteristics of Iowa materials for M-EPDG inputs, Wang et al. (2008) state that "the heat capacity of PCC is not a sensitive parameter for pavement design, and therefore it was proposed that it be studied in the future." Additional discussion regarding use of heat capacity in M-EPDG is presented in Chapter 7, Use of Recycled Brick Masonry Aggregate Concrete in Pavement Applications.

6.4.4.3 Durability Performance Testing

Specimens cast from the baseline RBMAC mixtures were subjected to several durability performance tests. A discussion of the results for each of the durability performance tests performed is presented in the subsequent sections. In order to conserve the RBMA available for this work, durability performance tests were only performed on the four baseline RBMAC mixtures. Durability performance tests were not performed on test specimens cast from trial batches of RBMAC.

6.4.4.3.1 Air and Water Permeability

Air and water permeability tests were performed using the Poroscope Plus test equipment. This equipment utilizes the Figg technique to determine the amount of time required for concrete to facilitate a given pressure increase under vacuum (air permeability) and to allow a given amount of water to penetrate into the concrete (water permeability). Additional information regarding the Figg technique is presented in Section 6.4.3.3.1, Air and Water Permeability.

Readings for air permeability and water permeability obtained from the Poroscope Plus Equipment are in units of seconds (sec). This test data is provided in Appendix C in Tables C-8 through C-12. Photographs of tested specimens are also provided in Appendix C in Figures C-30 and C-31. In literature provided with the Poroscope Plus, the manufacturer (NDT James Instruments) provides an equation for converting air permeability readings into an Air Exclusion Rating (AER) and an equation for converting the water permeability reading into a water absorption rate (WAR). The equations for computing AER and WAR are

$$AER = \frac{t}{\left[\frac{55V}{59} - V\right] \times \frac{55.5}{100}} = 19.05 \times \frac{t}{V} \tag{6-7}$$

$$WAR = \frac{t}{10} \times 10^3 \tag{6-8}$$

In Eq. 6-7 and Eq. 6-8, t is the measured time (sec) and V is the volume of the apparatus, including the test hole (units of mL). For the Poroscope equipment, V is 2.61 oz (77.1 mL), so AER = 0.247•t. The literature provided with the Poroscope equipment (NDT James Instruments 2007) "gives the tentative values for air and water permeability times and calculated AER ratings for concrete of varying protective quality for embedded reinforcement." A summary of information provided in this table is shown in Table 6-9. Figg (1989) provides a list of representative types of material for these "tentative ranges" of WAR and AER. The representative types of materials presented by Figg (1989) are also shown in Table 6-9.

Table 6-9: Values for air and water permeability times and calculated AER ratings for concrete of varying protective quality for embedded reinforcement (from NDT James Instruments 2007 and Figg 1989).

Concrete Category	**Protective Quality**	**Air Permeability**		**Water Permeability**	**Type of Material (from Figg 1989)**
		Time (sec)	**AER (sec/mL)**	**WAR (sec/ml)**	
0	Poor	<30	<8	<3	Porous mortar
1	Not very good	30-100	8-25	3-10	2,900 psi (20 MPa) concrete
2	Fair	100-300	25-75	10-30	4,350-7,250 psi (30-50 MPa) concrete
3	Good	300-1000	75-250	30-100	Densified, well-cured concrete
4	Excellent	>1000	>250	>100	Polymer-modified concrete

For each specimen of the baseline RBMAC mixtures tested, air permeability and water permeability tests were performed at four test locations. The test results for each test location were averaged. The average air permeability and the average water permeability for each RBMAC test specimen were then converted into average AER and average WAR values; see Table 6-10.

Table 6-10: Average Air Exclusion Rates (AER) and average Water Absorption Rates (WAR).

	RBMAC Mixture			
	BAC 5.0	**BAC 6.0**	**BAC 6.1**	**BAC 6.2**
Average air permeability reading (sec)	9.3	69.8	42.6	46.1
Average air exclusion rate, AER (sec/mL)	2.3	17.2	10.5	11.4
Average water permeability reading (sec)	3.3	71.7	19.7	29.5
Average water absorption rate, WAR (10^3 sec/mL)	0.3	7.2	2.0	3.0

Comparing the RBMAC test results in Table 6-10 to the information in Table 6-9, it can be seen that the air permeability test results for mixtures that utilized a water-reducing admixture (BAC 6.0, BAC 6.1, and BAC 6.2) resulted in average AER values that were within the "not very good" range for protective quality. According to Figg (1989), this protective quality rating would be typical of 20 MPa (2,900 psi) concrete. Two of the three RBMAC test mixtures that utilized a water reducing admixture (BAC 6.0 and BAC 6.2) resulted in average WAR values that were also within the "not very good" range for protective quality, while the third mixture had an average WAR value that fell within the "poor" range for protective quality. The RBMAC mixture that did not have a water reducer fell within the "poor" range for protective quality for both the average AER and average WAR values.

It is possible that, due to the high absorption of the RBMA, the values obtained for air and water permeability testing via the Figg Method gave a poor representation of the transport properties of the material. Bungey et al. (2006) caution that "aggregate characteristics have a profound effect on results, limiting the potential usage to comparative testing…" Based on these test results, it appears that in order for RBMAC to provide sufficiently low air and water permeability, a water-reducing admixture should be used to reduce the permeability of the paste fraction of the material.

6.4.4.3.2 Abrasion Resistance

Typically, for pavement applications, abrasion resistance is addressed by prequalification of aggregates by testing using the Los Angeles Abrasion method as outlined in ASTM C131. This test on coarse aggregate has proven useful in the prevention of unacceptable abrasion resistance of concrete pavements. However, in the ASTM C131 test method, the abrasion resistance of concrete is not tested, just one constituent component of the concrete (coarse aggregate). Recently, several different methods of testing the abrasion resistance of hardened concrete test specimen have generated increased interest from researchers. The ASTM C944 test method has recently been used by a number of agencies, including FHWA, in their evaluation of concrete abrasion resistance.

In their publication "High-Performance Concrete (HPC) Defined for Highway Structures," FHWA outlines the ASTM C944 abrasion resistance requirements for high-performance concrete used in highway structures such as bridges and pavements (Goodspeed et al. 2012). In an effort to better tailor concrete performance to expected service conditions, FHWA has developed performance grades for HPC. Relationships between project field conditions and the required resistance to exposure conditions can be established, and the proper performance grade for HPC can be identified for specifications. HPC performance grades are designated as 1 through 4, with the higher performance grade numbers corresponding to better performance in tests.

To use these performance grades in specifications, FHWA indicates that specifiers need to identify desired performance grades in relation to the desired field performance characteristics of their structures. For example, “A mixture for a bridge deck subjected to a high usage of deicing salts, high frequency of freezing and thawing cycles, and narrow beam spacing may be specified by a high grade to resist freezing and thawing distress, a medium to high grade to resist scaling, abrasion and chloride attack, and a low grade for strength and elasticity (Goodspeed et al. 2012).”

Table 6-11 outlines the ASTM C944 abrasion resistance test performance that corresponds to specific HPC performance grades, according to FHWA (Goodspeed et al. 2012).

Table 6-11: Abrasion resistance of HPC concrete mixtures (from Goodspeed et al. 2012).

	HPC Performance Grade			
	1	**2**	**3**	**4**
Abrasion resistance, *x*, ASTM C944 (average depth of wear in mm)	2.0□ *x* ≥1.0	1.0□ *x*≥0.5	0.5□ *x*	Not identified

With regards to testing, FHWA offers the following guidelines for abrasion testing in accordance with ASTM C944:

> *“Test areas shall receive a light trowel finish. Specimens shall be field cured for 56 days and air dried for 2 hours before testing. The tests shall be done on three different cylinders or at three different areas on the surface of a concrete structure. Each abrasion test shall be done using a*

196 N force for three 2 minute periods for a total of 6 minutes of abrasion testing; a wear depth is then measured (Goodspeed et al. 2012)."

Testing of RBMAC was performed in a manner that would allow comparison with FHWA's "High-Performance Concrete (HPC) Defined for Highway Structures" (Goodspeed et al. 2012). For each baseline RBMAC mixture tested, one test specimen was tested at three locations. For each test location, the depth of wear (for three subsequent 2-minute applications of the rotary cutting device at a load of 196 N) at circumference quarterpoints was averaged to get an average depth of wear. The results for the three test locations on each specimen were averaged to get an average depth of wear for the mixture. Average abrasion resistance test results for baseline RBMAC mixtures are presented in Table 6-12. Supporting test data are presented in Appendix C in Table C-13, and typical specimens after testing are shown in Figure C-29.

Table 6-12: Average abrasion resistance of baseline RBMAC mixtures using the rotary cutting device method (ASTM C944).

	RBMAC Mixture			
	BAC 5.0	**BAC 6.0**	**BAC 6.1**	**BAC 6.2**
Average total depth of wear after three 2-minute test periods, *x*, (mm)	0.99	0.16	0.40	0.41

As observed from Table 6-12, the RBMAC mixtures that utilized water-reducing admixtures (BAC 6.0, BAC 6.1, and BAC 6.2) each had average depths of wear less than 0.5 mm, which indicates that these mixtures exhibit abrasion resistance characteristics meeting FHWA HPC Performance Grade 3 (as shown in Table 6-11). The RBMAC mixture that did not utilize a water-reducing admixture, and therefore had a higher water/cement ratio (BAC 5.0), had an average depth of wear that was just under 1 mm, indicating that it exhibits abrasion resistance characteristics meeting FHWA HPC Performance Grade 2.

With regards to abrasion resistance, FHWA offers the following guidelines regarding the specification of specific performance grades for highway structures:

> *"Normal surface abrasion from rubber tires typically does not warrant an abrasion resistance consideration assuming well cured concrete of appropriate strength; the use of studded tires does. Thus a Grade 1 is recommended for less than a 50,000 average daily traffic count, Grade 2 for greater than 50,000 and less than 100,000, and Grade 3 for greater than 100,000 when steel studded tires are permitted. Similar estimates can be made by local engineers if the use of car chains is prevalent. Recommendations for other abrasion conditions such as stream flow laden with abrasive materials are the responsibility of the project engineer."*

Based on these guidelines and ASTM C944 abrasion resistance test results, RBMAC mixtures that utilize a water-reducing admixture (BAC 6.0, BAC 6.1, and BAC 6.2) are suitable for use in pavement applications for heavily trafficked roadways, where the average daily traffic is greater than 100,000. RBMAC mixtures that did not utilize a water-reducing admixture (BAC 5.0) would be suitable for a lesser trafficked roadway, where the average daily traffic is between 50,000 and 100,000.

6.4.4.3.3 Chloride Ion Permeability

The ASTM C1202 rapid chloride ion permeability test results for the four baseline RBMAC mixtures are presented in Table 6-13. As outlined in Section 6.4.3.3.3, Chloride Ion Permeability, for each RBMAC mixture, two specimens were tested. The results shown in the table are the average of the two specimens. Supporting test data are provided in Appendix C in Table C-14, and a summary of test results is provided in Table C-15. Photographs of typical specimens after testing are shown in Figure C-32.

Table 6-13: Results of rapid chloride ion permeability test.

	RBMAC Mixture			
	BAC 5.0	**BAC 6.0**	**BAC 6.1**	**BAC 6.2**
Average total charge passed (C)	8,379	982	1,599	3,127

In accordance with ASTM C1202, the susceptibility of concrete to chloride ion penetrability is assessed using Table 6-14. These ratings were originally outlined by Whiting (1981) in FHWA/RD-81/119, “Rapid Determination of the Chloride Permeability of Concrete.” Although several agencies have adjusted the performance classifications based on regional data, the ranges and permeability classifications shown in Table 6-14 are generally still accepted by many agencies.

Table 6-14: Chloride ion penetrability based on charge passed (according to ASTM C1202).

Charge passed (C)	**Chloride Ion Penetrability**
>4,000	High
2,000 to 4,000	Moderate
1,000 to 2,000	Low
100 to 1000	Very Low
<100	Negligible

Chloride ion penetrability test results indicate that the baseline RBMAC mixtures can exhibit reasonably good resistance to chloride ion ingress. The three baseline mixtures that utilized a high-range water reducing admixture performed relatively well, exhibiting “Very Low” to “Moderate” chloride ion penetrability. A water-reducing admixture was not used in BAC 5.0, which exhibited high chloride ion penetrability. This indicates that, similar to concrete made with other conventional aggregates, the chloride ion permeability of recycled brick masonry aggregate concrete is quite dependent on the quality of the paste.

Maximum permissible charge passed values specified by DOTs often differ by concrete use. Although current NCDOT guidelines for pavement mixtures do not provide ASTM C1202 performance requirements, in the “North Carolina Department of Transportation Partial and Full Depth Repair Manual,” NCDOT indicates that partial depth repair material should have ASTM C1202 charge passed values of 960-990 C.

Due to equipment availability, these tests were performed when the concrete was approximately 14 months old. Cylinders from which test specimens were removed had been used for equilibrium density testing. Prior to use for chloride ion penetrability testing, the cylinders had been wet-cured for 7 days and subsequently stored in a temperature and humidity controlled laboratory setting. It is quite possible that at earlier ages, such as 28-days, the charge passed values may have been lower. It is unclear, however, what the charge passed values for RBMAC would be if wet-curing conditioning (the typical procedure) was used for the test specimens. Testing on additional batches of RBMAC is needed.

6.4.4.3.4 Surface Resistivity

Surface resistivity testing is a relatively new method of assessing the corrosion potential of concrete, and has been correlated quite closely to chloride permeability by a number of researchers (Rupnow and Icenogle 2012). As resistivity is dependent on a number of factors, including geometry and temperature, a number of ongoing studies are being performed by others to fully understand results obtained from this testing and to correlate these results to field performance of concrete structures. Several agencies are investigating the possibility of replacing the labor-intensive ASTM C1202 rapid chloride ion permeability test with the much quicker surface resistivity test (Rupnow and Icenogle 2012). Results of this testing are provided for informational purposes only, as guidelines for use of surface resistivity measurements in evaluating concrete mixtures do not exist for many agencies, including NCDOT.

Surface resistivity testing was performed on the baseline RBMAC mixtures at a number of temperatures, and the data are provided in Table C-16 in Appendix C. A summary of results are shown in Figure 6-18. Again it is noted that, due to equipment availability, this testing was performed when the concrete was approximately 14 months old. Cylinders had been used for equilibrium density testing, with a wet-cure for 28 days and subsequent storage in a temperature and humidity controlled laboratory setting.

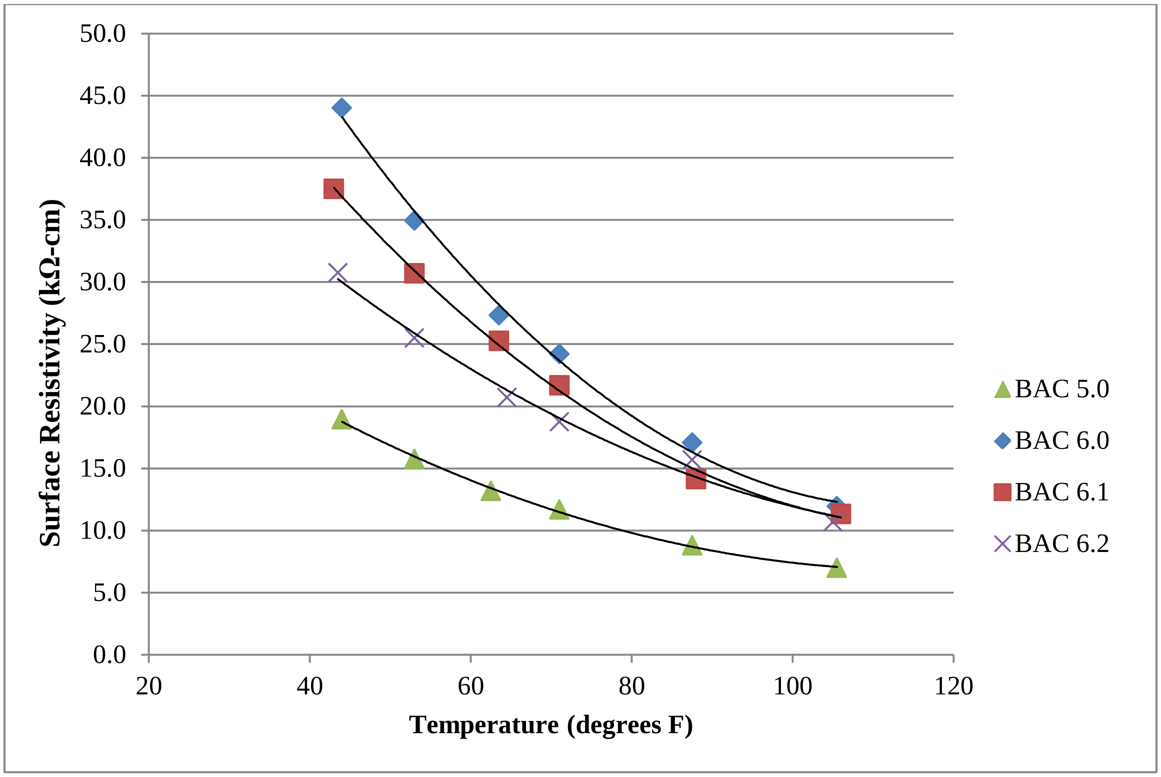

Figure 6-18: Surface Resistivity versus temperature for RBMAC.

6.5 Summary and Concluding Remarks

To date, RBMAC has not been produced or studied in the United States. In this study, RBMAC mixtures exhibiting acceptable performance in the fresh and hardened states were successfully batched and tested. A database of material properties of RBMAC was developed. In the future, this information can be used to assist designers interested in use of RBMA and RBMAC for sustainable design.

Potential issues with workability due to high absorption of the RBMA can be overcome with use of a commercially available high-range water reducer, and by batching with the RBMA at SSD conditions. Adequate performance in fresh property testing for air content and slump were obtained by using commercially available liquid admixtures at dosage rates within manufacturers' recommended ranges.

Concrete with acceptable compressive strengths can produced using RBMA as a 100% replacement for natural coarse aggregate. Although the top-down demolition sequence provided RBMA with very little contaminant material (see Table 3-1), a few contaminant pieces were observed in some fracture faces. It is possible that some variability evident in compressive strength test results is due to the inclusion of contaminants in the RBMA. It is also noted that the splitting tensile strength of RBMAC was lower than would be predicted using a published relationship, and these test results may also have been affected by contaminant materials.

Equilibrium density tests indicate that a significant weight reduction can be realized for RBMAC. At 111.8 pcf (1791 kg/m^3), the equilibrium density of the baseline RBMAC mixture produced without the use of a water-reducing admixture met the equilibrium density requirements of ACI 213, "Guide for Structural Lightweight-Aggregate Concrete." The equilibrium densities of the RBMAC mixtures produced using a water-reducing admixture ranged from 125.5 to 128.2 pcf (2010 to 2054 kg/m^3). Although these equilibrium densities were higher than those required in ACI 213, they were significantly lower than the unit weight of concrete using conventional normalweight aggregates (typically 140 to 145 pcf, or 2243 to 2323 kg/m^3). Therefore, use of RMBAC can provide the advantages of a reduction of deadload in structural applications and savings in hauling costs.

The modulus of elasticity of the baseline RBMAC mixtures was within the range of values expected for normalweight and lightweight concrete using conventional aggregate sources. An equation presented in ACI 318-08 relating compressive strength to modulus of elasticity (using the unit weight of concrete) can be used to reasonably predict the modulus of elasticity of RBMAC mixtures (made with the Idlewild Elementary School RBMA).

When compared to the guidelines of several agencies, the 7-day moduli of rupture obtained for the RBMAC mixtures (utilizing a water-reducing admixture) were sufficiently high for use in pavement applications. Values for thermal characteristics of RBMAC were within the expected range for concrete materials. As brick has a lower CTE than most conventional coarse aggregates, an increased brick content in the RBMA would result in a RBMAC with a lower CTE, and hence would lead to better predicted pavement performance in M-EPDG. As part of future testing, it is recommended that CTE testing of RBMAC be performed in accordance with AASHTO T336, with results compared to those using the appropriate calibration specimen. The thermal conductivity measured for RBMAC (BAC 6.2) was somewhat lower than the default value for concrete in M-EPDG. Future study of RBMAC could include batching of companion mixtures of conventional PCC to validate the thermal property test procedures and results. Additional discussion on this will be presented in Chapter 7, Use of Recycled Brick Masonry Aggregate Concrete in Pavement Applications.

Unfortunately, due to malfunctioning equipment in the conditioning chamber, drying shrinkage tests were not completed. It is recommended that this testing be performed as part of future studies with RBMAC.

Results for durability performance tests of RBMAC were mixed. Using the Figg method of testing, the air and water permeability results for all the baseline RBMAC mixtures were rather high. The baseline RBMAC mixtures did, however, exhibit fairly good results in the ASTM C1202 chloride ion permeability testing. Two of the RBMAC mixtures had ASTM C1202 charge passed values that indicated the potential for “very low” or “low” chloride penetrability.

The reason for these conflicting results may be explained by the means of air and water transport through the concrete matrix. Chloride ion permeability test results were sensitive to w/c ratio and cement content. Specimens with lower water cement ratios and higher cement contents exhibited much lower chloride ion permeability than those with higher w/c ratio and lower cement contents. This suggests that the primary mode of chloride transport is through the paste, and transport was not greatly influenced by the highly porous aggregates. It is possible that the voids within the RBMA are not as interconnected as those in the paste, although further study would be needed to confirm this.

Other researchers (Bungey et al. 2006) have suggested that due to the high absorption of the brick fraction of the aggregate, the Figg Method test may not be the most appropriate method of predicting durability performance. If the test hole is drilled directly into a highly porous coarse aggregate particle, air and water would easily be transported into the high porosity in the immediate vicinity of the test plug, giving results that do not represent the permeability of the bulk concrete. Additional testing may therefore be useful in assessing the air and water permeability of RBMAC. Other test methods, such as sorptivity using the capillary suction method (ASTM C1585) or the ponding method (as per Bentz et al. 2002) may be more appropriate tests for durability performance of RBMAC.

The possibility of replacing ASTM C1202 tests with surface resistivity measurements is currently being investigated by a number of agencies. Surface resistivity measurements were performed on the baseline RBMAC mixtures for informational purposes only, no preliminary conclusions regarding surface resistivity are made at this time.

CHAPTER 7: USE OF RECYCLED BRICK MASONRY AGGREGATE CONCRETE IN PAVEMENT APPLICATIONS

7.1 Introduction

RBMAC is currently not used in the United States for any type of construction. Testing performed as part of this work has indicated that pavement applications may be a viable use of RBMAC. In the following sections, the potential for RBMAC to be used in pavement applications is explored. Specific attention is given to current materials requirements for recycled aggregates and RAC in North Carolina, as specified in the North Carolina Department of Transportation (NCDOT) standards. Based on these specifications, challenges and barriers to using RBMAC in NCDOT pavement applications are discussed.

Many states, including North Carolina, are considering implementing the Mechanistic-Empirical Pavement Design Guide (M-EPDG) procedure for pavement design. The potential use of RBMAC in M-EPDG pavement design is explored. Pavement sections comprised of both RBMAC and concrete with locally available natural aggregate are designed for typical arterial and interstate pavements, and the performance is compared. Issues with use of RBMAC in the M-EPDG software are identified and discussed. Additionally, M-EPDG was used to design a proposed test pavement utilizing RBMAC, along with a companion test pavement comprised of conventional natural aggregate. Details regarding the proposed test pavements are provided, along with an instrumentation plan and a materials sampling and testing plan.

7.2 Potential Use of Recycled Brick Masonry Aggregate Concrete in Current North Carolina Department of Transportation (NCDOT) Pavement Applications

Projects designed and constructed for NCDOT must comply with "NCDOT Standard Specifications for Roads and Structures." At the time that most of the work for this project was performed, "NCDOT Standard Specifications for Roads and Structures," dated July 2006, was in effect. This version will subsequently be referred to as the 2006 NCDOT Standard Specifications. Currently, "NCDOT Standard Specifications for Roads and Structures," dated January 2012, is in effect and will be subsequently referred to as 2012 NCDOT Standard Specifications.

In both the 2006 NCDOT Standard Specifications and the 2012 NCDOT Standard Specifications, requirements for materials used in roads and structures are published in Division 10 – Materials. Requirements for portland cement concrete are provided in Section 1000, "Portland Cement Concrete Production and Delivery," and Section 1024, "Materials for Portland Cement Concrete." Requirements for aggregates are presented in several sections. The applicable sections for assessing the suitability of RBMA include Section 1005, "General Requirements for Aggregate," Section 1006, "Aggregate Quality Control / Quality Assurance," Section 1014, "Aggregate for Portland Cement Concrete," and Section 1043, "Aggregate from Crushed Concrete." It is noted that Section 1043, "Aggregate from Crushed Concrete" is new in the 2012 NCDOT Standard Specifications and was not included in the 2006 NCDOT Standard Specifications. Additionally, in order to evaluate the flatness and elongation parameters of RBMA, the requirements for aggregates used in asphalt pavement applications outlined in Section 1012, "Aggregate for Asphalt Pavements and Surface Treatments," are utilized.

7.2.1 Current NCDOT Requirements and Standards for Use of Recycled Aggregates

The 2006 NCDOT Standard Specifications do not contain provisions for use of recycled aggregates in any applications, including in portland cement concrete construction. The 2012 NCDOT Standard Specifications include a new section, Section 1043, "Aggregate from Crushed Concrete," which provides guidelines for use of RCA. As outlined in Section 1043-1, "Aggregate from crushed concrete is a recycled product made by crushing concrete obtained from concrete truck clean out, demolition of existing concrete structures or pavement, or similar sources and transported from a crushing facility (NCDOT 2012)." Crushed concrete aggregate must meet all approval requirements presented in Section 1005, "General Requirements for Aggregate," and Section 1006, "Aggregate Quality Control / Quality Assurance," although the limit on deleterious materials (as determined by the sodium sulfate test) is required to be less than or equal to 3%. Testing for aggregates, including crushed concrete aggregates includes the tests outlined in the Table 7-1.

Table 7-1: Testing of aggregates for use in portland cement concrete required by 2012 NCDOT Standard Specifications

Property	Test Method
Gradation	AASHTO T27, "Standard Method of Test for Sieve Analysis of Fine and Coarse Aggregates," and AASHTO T11, "Standard Method of Test for Materials Finer than No. 200 (75µm) Sieve in Mineral Aggregates by Washing"
Liquid limit	AASHTO T89, "Standard Method of Test for Determining the Liquid Limit of Soils" (NCDOT modified version)
Plasticity index	AASHTO T90, "Standard Method of Test for Determining the Plastic Limit and Plasticity Index of Soils"
Abrasion resistance	AASHTO T96, "Standard Method of Test for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine"
Soundness	AASHTO T104, "Standard Method of Test for Soundness of Aggregate by Use of Sodium Sulfate or Magnesium Sulfate" (using sodium sulfate)

Additionally, aggregates used for portland cement concrete have limitations on the amount of material passing the No. 200 sieve. At production, the material passing the No. 200 sieve shall be no more than 0.6%. Aggregates tested at the jobsite must have no more than 1.5% passing the No. 200 sieve. If aggregates tested at the jobsite are found to have more than 1.5% passing the No. 200 sieve, its use can be approved by the project engineer if the total percentage by weight of the combined coarse and fine aggregate does not exceed 2.0% and the water-cementitious ratio is not increased.

In Section 1043, the 2012 NCDOT Standard Specifications limits the uses of aggregate made from crushed concrete to Class B concrete mixes only. Class B concretes are lower-strength mixtures used for low-grade applications such as drainage appurtenances, endwalls, paved ditches, and curb and gutter. The minimum compressive strength at 28 days is 2,500 psi (17.2 MPa). The maximum allowable water-cementitious ratios for air-entrained concrete are limited to 0.469 for rounded aggregate and 0.545 for angular aggregate. For non-air entrained concrete, the maximum allowable water-cementitious ratios are 0.559 and 0.630, for rounded and angular aggregate, respectively. The maximum allowable slump for vibrated Class B concrete is 2.5 in and for non-vibrated Class B concrete is 4 in. Minimum to maximum cement contents for vibrated and non-vibrated Class B concrete mixtures are, respectively, 508 to 610 pcy (301 to 362 kg/m^3) and 545 to 654 pcy (323 to 388 kg/m^3).

7.2.2 Qualification of Recycled Brick Masonry Aggregate for Use in NCDOT Portland Cement Concrete Applications

As outlined in Section 7.2.1, Current NCDOT Requirements and Standards for Use of Recycled Aggregates, RBMA does not meet the definition of “Aggregate from Crushed Concrete,” in Section 1043 of the 2012 NCDOT Standard Specifications. At this time, NCDOT limits the material used as recycled aggregates to crushed concrete only. Requirements for coarse aggregate used in portland cement concrete are outlined in Section 1014-2 in the 2012 NCDOT Standard Specifications. Coarse aggregate material is to consist of “crushed stone, crushed or uncrushed gravel, crushed air-cooled blast furnace slag, or other inert materials that have similar characteristics (NCDOT 2012).” Although RBMA does not come from the sources listed, it may exhibit similar characteristics, and therefore could potentially be used in NCDOT portland cement concrete applications.

Section 1014-2 provides performance requirements for coarse aggregates used in portland cement concrete. A summary of pertinent characteristics and requirements is summarized in Table 7-2.

Table 7-2: Required characteristics of coarse aggregates used in portland cement concrete (from NCDOT 2012).

Characteristic	Requirement
Soundness	"When subjected to 5 cycles of the soundness test, the weighted average loss shall not exceed 15%. For concrete with a 28 day design compressive strength greater than 6,000 psi, the loss shall not exceed 8%."
Deleterious substances	"Determine the percentage of deleterious substances (clay lumps and friable particles) in accordance with AASHTO T112. The amount of deleterious substances shall not exceed 3.2% by weight."
Abrasion resistance	"The percentage of wear of crushed stone or gravel shall not exceed 55%. For concrete with a 28 day design strength greater than 6,000 psi, the wear shall not exceed 40%."
Gradation	For portland cement concrete pavement applications, use standard sizes No. 57, No. 67, or No. 78M unless otherwise specified by the engineer.

Results of laboratory tests performed to date indicate that RBMA can meet 2012 NCDOT Standard Specifications requirements for aggregates used in concrete, with the possible exception of abrasion resistance for concretes with specified 28-day design strengths greater than 6,000 psi (41.4 MPa). It is noted that RBMA from only one source has been tested. It would be worthwhile to perform LA abrasion testing on RBMA from a number of different sources to determine whether the LA abrasion value of 40% is generally representative of this material.

For this work, the RBMA material was produced with a target gradation of AASHTO M43 #78. Although the material did not quite meet the required percent composition range for the material passing the 3/8 in sieve (retained on the No. 4 sieve), personnel at the crushing and grading facility indicated that this could easily be achieved with a slight modification to the sieve apparatus used in the crushing/grading operation.

Aggregates are also limited to the amount of material passing the No. 200 sieve, no greater than 0.6% at production and no greater than 1.5% at the jobsite. Although a No. 200 sieve was not used in the sieve analyses of the RBMA produced as part of this work, the percent passing the No. 16 sieve was found to be 0.6%. Therefore, it is known that this material would have met this requirement, as even less material would have been able to pass the No. 200 sieve.

It is noted that soundness testing was not performed as part of this work. If RBMA was to be used in NCDOT construction, soundness testing would need to be performed and the results would need to be compared to NCDOT specifications. Sulfate soundness testing is a means of assessing the resistance of an aggregate to weathering. The test simulates freezing and thawing cycles by repeated immersion and removal of the aggregate from a sulfate solution that causes growth of salt crystals in the aggregate pores. Kosmatka et al. (2002) indicate that this test is "sometimes misleading" in its predictions of the freeze-thaw behavior of concrete produced from the aggregates, stating that "the test is most reliable for stratified rocks with porous layers or weak bedding planes." As brick and mortar comprising the RBMA are not stratified, this test may not provide a true indication of RBMAC performance, which may be better assessed through testing of RBMAC by ASTM C666, "Standard Test Method for Resistance of Concrete to Freezing and Thawing."

Testing to determine the deleterious substances (clay lumps and friable particles) was also not performed as part of this work. The applicability of this requirement, however, may be questionable due to the nature of the source material (fired clay) used to produce the brick in the RBMA. As outlined in Section 3.3.1.1 Composition by Weight and by Volume, the RBMA made from Idlewild Elementary School's demolition debris contained 3.3% of contaminant material by weight.

Upon initially observing the RBMA material shortly after it was produced at the crushing and grading facility, it was noted that the brick aggregates tended to be flatter and more elongated than the crushed mortar fraction. The brick aggregates were also noticeably flatter and more elongated than the local virgin aggregates and the RCA produced from concrete from the case study site. It was initially suspected that flat and elongated particles would be problematic, particularly with workability of the RBMA concrete in its fresh state. However, all of the RBMAC mixtures batched as part of this work were judged to exhibit adequate workability by the researcher. NCDOT requirements for flat and elongated particles are only applicable to aggregates used in asphalt cement concrete. However, testing in accordance with ASTM D4791 "Standard Test Method for Flat Particles, Elongated Particles, or Flat and Elongated Particles in Coarse Aggregate" was nonetheless performed to assess the RBMA. Results of this test indicated that the RBMA from the case study site meets the NCDOT requirements outlined in Section 1012 (NCDOT 2006). In order to place increased scrutiny on the brick particles, the components of the RBMA (brick, clay tile, and mortar) were segregated, and the ASTM D4791 procedure was repeated for each component separately. Tests run on the brick fraction of the RBMA indicated that the brick aggregate particles from the case study site still met NCDOT requirements, but were, indeed, flatter and more elongated when tested alone (as compared to the mortar and clay tile components of the RBMA).

7.2.3 Challenges and Barriers to Use of Recycled Brick Masonry Aggregate Concrete in NCDOT Pavement Applications

If RBMAC is to be used in NCDOT pavement applications, several existing barriers would need to be addressed. As discussed in Section 7.2.2, Qualification of Recycled Brick Masonry Aggregate for Use in NCDOT Portland Cement Concrete Applications, requirements outlined in the current NCDOT Standards and Specifications would need to be modified to allow the use of material other than crushed concrete for recycled aggregates. Additionally, to be used in pavements, NCDOT Standards and Specifications would need to allow recycled aggregates to be used in the "Pavement" class of concrete.

Requirements for pavement concrete are outlined in the 2012 NCDOT Standards and Specifications in Section 1000-3, "Portland Cement Concrete for Pavement." Pavement concrete must be air entrained, and the maximum w/c ratio is 0.559. The concrete must be vibrated, and the maximum slump requirements are 1.5 in (38.1 mm) for slip formed concrete and 3.0 in (76.2 mm) for hand-placed concrete. A minimum cement content of 526 pcy (312.1 kg/m^3) is specified. Strength requirements for concrete to be used in pavements include a minimum 28-day compressive strength of 4,500 psi (31.0 MPa) and a minimum 28-day flexural strength of 650 psi (4.48 MPa). The flexural strength requirement must be met in the design phase only (mixture proportions and test results submitted to NCDOT for approval), while the compressive strength requirement must be met in both the design phase and by test specimens prepared from the concrete placed in the field.

Each of the baseline RBMAC mixtures that contained a water-reducing admixture (BAC 6.0, BAC 6.1, and BAC 6.2) meets the current requirements for cement content, maximum w/c ratio, and minimum 28-day compressive strength outlined above. Flexural strength tests performed on baseline RBMAC mixtures as part of this work were performed at 7 days instead of 28 days. At 7 days of age, baseline mixtures BAC 6.0, BAC 6.1, and BAC 6.2 had 7-day flexural strengths of 797 psi (5.50 MPa), 730 psi (5.03 MPa), and 716 psi (4.94 MPa), respectively, and would therefore meet the NCDOT 28-day minimum flexural strength requirement of 650 psi (4.48 MPa). Slumps for these mixtures were higher than NCDOT's maximum slump requirements for pavement concrete, but it is likely that without the water-reducing admixture, the slumps would have been significantly lower and the requirements could have been met.

Although RBMAC can meet the performance requirements specified by NCDOT for pavement mixtures, NCDOT may be reluctant to use RBMA in pavement applications due to durability concerns. Abrasion resistance is a key performance requirement for pavement concrete. In addition to the abrasion test requirements for aggregates (performed using the Los Angeles abrasion method), NCDOT could specify that additional abrasion testing be performed for concrete mixtures (or just RBMAC mixtures) used in pavement applications. In addition to performing LA Abrasion testing on the RBMA used in this study, abrasion resistance of the RBMAC was evaluated using ASTM C944, "Standard Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating-Cutter Method." As outlined by FHWA (Goodspeed et al. 2011) and Naik et al. (1994), this test can serve as a useful tool for assessing concrete paving mixtures. Results of testing using the rotary cutter device indicated that the baseline

RBMAC mixtures that used a water-reducing admixture (BAC 6.0, BAC 6.1, and BAC 6.2) had abrasion resistance that met FHWA standards for Performance Grade 3 high-performance concrete (HPC) for use in highway structures (Goodspeed et al. 2011).

Since RBMA is produced from existing brick masonry construction, variability of material produced from different sources is a concern. This is, however, no different than the potential variability in RCA produced from different sources of waste concrete. Proponents of RCA implemented strategies that promoted understanding and control of the source material. These strategies typically include assessing potentially recyclable concrete for existing materials-related distress (such as ASR) as well as reuse of concrete in the same project being reconstructed (Taylor et al. 2006). Strategies for proper stockpile management also aid in ensuring consistency and minimal contamination (Taylor et al. 2006). Research and field implementation has shown that, with proper evaluation of the source concrete, RCA concrete exhibiting acceptable performance can be produced. Similar strategies could also be utilized in RBMA production and transport to help ensure adequate performance of RBMAC.

RCA concrete is gaining more widespread acceptance, and it is being successfully utilized in a number of applications throughout the country. Ultimately, it will be the burden of the recycled aggregate supplier to demonstrate that their RBMA product meets DOT requirements. Additionally, successful performance of RBMAC in both laboratory and field installations will be needed to provide NCDOT an adequate comfort level with this material.

In the case of D.H. Griffin Crushing and Grading, the company sees this RBMA material (and other aggregates they produce from reclaimed material) as a commodity, with enough potential value that they have invested significant resources to develop an on-site NCDOT certified laboratory. They believe in their process control so strongly that they are willing to construct a test pavement in cooperation with UNC Charlotte to evaluate the performance of RBMA in service applications. This proposed test pavement is discussed in Section 7.4, Proposed Test Pavement Utilizing Recycled Brick Masonry Aggregate Concrete in Charlotte, North Carolina. Prior to presenting the discussion on this test pavement, however, the use of RBMAC in M-EPDG pavement design is discussed in the next section.

7.3 Potential Use of Recycled Brick Masonry Aggregate Concrete in Mechanistic-Empirical Pavement Design

The Mechanistic-Empirical Pavement Design Guide (M-EPDG) is the "state-of-the-practice tool for the design and analysis of new and rehabilitated pavement structures, based on mechanistic-empirical (M-E) principles (AASHTO 2008)." At the time of this research, the M-EPDG software program was available for online trial, but is now only available commercially as the DARWin-ME software program. M-EPDG was determined to be a particularly useful tool for evaluating RBMAC pavements because the level of detail that can be incorporated into M-EPDG design and analysis is much greater than the level of detail allowed in the NCDOT Interim Design Guide procedure (currently used by NCDOT for pavement design) or the AASHTO 1993 Design Procedure.

The M-EPDG design process is a major change from previously utilized pavement design methods. Mechanical and statistical models incorporated into this software represent the work of leading researchers over the past two decades. Mechanistic analysis of pavement responses includes traffic and climatic data that are site specific. As part of the empirical analysis, results of the mechanistic analysis are compared to data on field-observed distresses of existing pavement sections in the Long-Term Pavement Performance (LTPP) database. Another significant characteristic of M-EPDG is that designers have the ability to analyze both flexible and rigid pavements using the same methodology, and rehabilitated pavement sections combining both rigid and flexible pavements can also be considered. Further discussion on M-EPDG and the implications of using RBMA in M-EPDG analysis is presented in the following sections, along with a comparison of pavement designs using both RBMAC and concrete utilizing locally available natural aggregates.

7.3.1 Mechanistic-Empirical Pavement Design Guide (M-EPDG) Overview

The M-EPDG process is an iterative approach to pavement design. The performance of trial pavement sections is compared to design performance criteria that are selected to "ensure that a pavement design will perform satisfactorily over its design life (AASHTO 2008)." Performance criteria for JPCP include joint faulting, transverse slab cracking, and smoothness, while different performance criteria are used for other types of pavement construction (such as hot mix asphalt pavements). Threshold values for performance criteria are selected by agencies based on a number of considerations, including pavement characteristics that trigger major rehabilitation efforts, impact safety, and require other maintenance.

Characteristics of a trial pavement section are input into the software program, along with site conditions including climate, traffic, and subgrade characteristics. Pavement responses such as stresses, strains, and deflections are then computed over the design life, along with incremental damage. Cumulative damage over the design life of the pavement is compared to empirical performance data collected on existing pavement sections. The trial pavement section is evaluated based upon the reliability values specified by the pavement designer based on their desired confidence levels. AASHTO (2008) defines design reliability as "the probability (P) that the predicted distress will be less than the critical level over the design period." The level of design reliability is selected based upon the "general consequence of reaching the terminal condition earlier than the design life (AASHTO 2008)." If the proposed design does not meet the desired performance criteria, it can be revised by the designer and the analysis rerun until an optimal design is identified (AASHTO 2008).

The Mechanistic-Empirical Pavement Design Guide: A Manual of Practice (AASHTO 2008) includes five basic steps for the design process. These steps are summarized below:

1. *Select a design strategy.*
2. *Select the appropriate performance indicator criteria (threshold value) and design reliability level for the project.*
3. *Obtain all inputs for the pavement trial design and consideration.*
4. *Run the M-EPDG software and examine the inputs and outputs for engineering reasonableness.*
5. *Revise the trial design, as needed.*

Most rigid (concrete) pavements constructed in North Carolina are jointed plain concrete pavement (JPCP). The three key performance criteria for JPCP are joint faulting, transverse slab cracking, and smoothness. Detailed information regarding the prediction models used for performance indicators is provided by AASHTO (2008), but a brief summary of each JPCP performance indicator and the methodologies used for prediction and assessment is presented here.

Joint faulting occurs when accumulated damage of the concrete at the joint and at load transfer devices (dowels) reduces the load transfer efficiency between adjacent slabs and leads to an elevation difference across a joint. Joint faulting also leads to severe roughness in the vicinity of joints (Huang 2004). Distress is measured in millimeters or inches (of elevation difference), and faulting is deemed positive if the surface of the approach slab is higher than the surface of the departure slab. Faulting is deemed negative if the surface of the departure slab is higher than the surface of the approach slab (Mallick and El-Korchi 2009). For faulting analysis in M-EPDG, all methods of load transfer are assessed, including dowels, aggregate interlock, and base or subgrade friction. Loss of shear capacity at the joint, damage at the dowel-concrete interface, and joint width changes due to temperature and moisture are considered. The cumulative faulting is compared to the allowable faulting at the end of the design life.

Transverse slab cracking (or simply transverse cracking) can occur from bottom-up or top-down. Both top-down and bottom-up transverse cracks occur "when a critical combination of loading and temperature curling creates summative stresses (Mallick and El-Korchi 2009)." Bottom-up cracks are caused when bending stresses at the bottom of a slab induced by traffic loads are added to stresses caused by a positive thermal gradient (top of the slab is warmer than the bottom of the slab) that produces downward curling (Mallick and El-Korchi 2009). Top-down transverse cracks are also caused by a combination of stresses from both traffic loading and temperature gradients, but the axle load placement is closer to a joint and the thermal gradient is negative (the top of the slab is cooler than the bottom of the slab), and upward curling has occurred (Mallick and El-Korchi 2009). Both the top-down and bottom-up modes of cracking are considered in the M-EPDG distress prediction equations, which are fatigue damage accumulations based on Miner's hypothesis. As described by AASHTO (2008), contact friction between the slab and base and load and temperature/moisture gradients are used to compute an equivalent section for the slab and base course. This equivalent section is used in the calculation of critical responses using site-specific climate data. The percentage of slabs in a given traffic lane predicted to exhibit transverse cracking at the end of the design life is compared to an acceptable threshold input by the designer.

Smoothness is assessed using the International Rideability Index (IRI), which is a measure of the cumulative roughness of the pavement in in per mile. The initial IRI is typically an estimated value based upon the smoothness of newly constructed pavements. The predicted IRI is computed using estimated increases in IRI due to cracking, spalling, and joint faulting occurring over the design life. Climatic effects are incorporated to allow the consideration of freeze-thaw cycles in a scaling factor and are also used to predict foundation movements contributing to increases in the IRI (AASHTO 2008).

In order to assess whether a pavement will provide satisfactory performance over its design life, threshold values for the performance criteria need to be selected. AASHTO (2008) provides guidance in identifying threshold values based on "experience, agency policies, and local needs." Often, agencies identify these threshold values based on maintenance/rehabilitation, safety, and economic considerations, as well as the desired reliability and consequences of premature failure. A discussion on the threshold values used by NCDOT will be presented in Section 7.3.3, Comparison of M-

EPDG Pavement Designs Using Recycled Brick Masonry Aggregate Concrete and Natural Aggregate Concrete.

To begin on M-EPDG analysis, the designer enters general project information into the software. These inputs include the design life, anticipated construction and traffic opening dates, and other information not used in the distress modeling and analysis such as the roadway name and its description. After this information is input, the designer begins inputting and selecting other traffic, climatic, and pavement section characteristics used in the distress models and analysis.

In order to best assess proposed pavement sections, site-specific values should be input into the M-EPDG software. However, due to the lack of availability of some site-specific data, as well as the cost associated with obtaining this data, M-EPDG allows inputs at several hierarchical levels. This allows users to input the site-specific data that is available, while utilizing reasonable default values for information that is not readily available. Level 1 inputs are site-specific data obtained by materials testing in the laboratory and field, and therefore theoretically allow for the highest level of accuracy in the predictions. Level 2 inputs are “estimated through correlations with other material properties that are commonly measured in the laboratory or field (AASHTO 2008).” Level 3 inputs are default values suggested by the M-EPDG software.

Traffic information used for M-EPDG consists only of truck traffic, Annual Average Daily Truck Traffic (AADTT). A significant amount of detailed traffic information can be entered into M-EPDG, if available. Conversely, some combination of available site-specific data (Level 1) and less-specific or default values (Levels 2 or 3) can be input. For example, if axle load spectra from weigh-in-motion scales is available, it can be input; otherwise reasonable default values suggested by the M-EPDG software can be used. Roadway-specific inputs include the initial two-way AADTT, percent trucks in the design lane, percent trucks in the design direction, operational speed, and predicted growth of truck traffic (AASHTO 2008). Inputs on axle-load spectra include axle load distributions (single, tandem, tridem, and/or quad axles), normalized truck volume distribution, axle spacing and wheelbase information, and monthly/hourly distributions.

Climate data for the project site can be downloaded from an extensive database of weather station data. If the user enters the latitude and longitude of the project location, the software will suggest several local weather stations. The designer can choose to use climatic data from one station or can create a custom "station" by combining the data available from multiple stations that are available in the online database. As outlined in AASHTO (2008), climate data includes hourly temperatures, precipitation, windspeed, relative humidity, and cloud cover. The depth of the water table is also a climate input and is selected by the designer.

Information on the subgrade soils obtained through laboratory and field testing can be input to M-EPDG. The most critical parameter for the subgrade is the resilient modulus. If resilient modulus testing in accordance with AASHTO T 307, "Determining the Resilient Modulus of Soils and Aggregate Materials" cannot be performed, this parameter can be estimated using information obtained through dynamic cone penetrometer (DCP) testing or through correlations with other physical properties of the soil.

Physical properties of the unbound aggregate base layers of a pavement in M-EPDG analysis include classification properties (gradation, Atterberg limits, dry density, moisture content, Poisson's ratio) and the key engineering performance property, resilient modulus. Other parameters, such as saturated hydraulic conductivity and information that can be used to predict moisture content changes, can also be input. If a rehabilitation project is being designed, information on the existing pavement and subgrade is entered.

For new portland cement concrete pavements and some PCC overlays, AASHTO recommends the Level 1 input parameters and test protocols shown in Table 7-3 (AASHTO 2008). If resources are not available to perform the testing necessary to obtain Level 1 input values, M-EPDG provides the recommended Level 2 and Level 3 input parameters and values presented in Table 7-4 (AASHTO 2008).

Table 7-3: M-EPDG Level 1 input parameters and test protocols for new and existing PCC pavements (from AASHTO 2008).

Measured Property	Source of Data		Recommended Test Protocol and/or Data Source
	Test	Estimate	
Elastic modulus	X		ASTM C469
Poisson's ratio	X		ASTM C469
Flexural strength	X		AASHTO T97
Indirect tensile strength (CRCP only)	X		AASHTO T198
Unit weight	X		AASHTO T121
Air content	X		AASHTO T152 or AASHTO T196
Coefficient of thermal expansion	X		AASHTO TP60 (currently AASHTO T336)
Surface shortwave absorptivity (correlates with amount of solar energy absorbed by pavement surface, (Wang et al. 2008)		X	National test protocol unavailable; use M-EPDG default value
Thermal conductivity	X		ASTM E1952
Heat capacity	X		ASTM D2766
PCC zero-stress temperature		X	National test protocol unavailable; estimate using agency historical data or select M-EPDG defaults.
Cement type		X	Select based on actual or expected cement source.
Cementitious material concrete		X	Select based on actual or expected concrete mix design.
Water-cement ratio		X	Select based on actual or expected concrete mix design.
Aggregate type		X	Select based on actual or expected aggregate source.
Curing method		X	Select based on agency recommendations and practices.
Ultimate shrinkage		X	Testing not practical. Estimate using prediction equation in M-EPDG.
Reversible shrinkage		X	Estimate using agency historical data or select M-EPDG defaults.
Time to develop 50% of ultimate shrinkage		X	Estimate using agency historical data or select M-EPDG defaults.

Table 7-4: M-EPDG Level 2 and 3 input parameters and test protocols for new and existing PCC pavements (from AASHTO 2008).

Measured Property	**Recommended Input Levels 2 and 3**
New PCC elastic modulus and flexural strength	28-day flexural strength AND 28-day PCC elastic modulus, or 28-day compressive strength AND 28-day PCC elastic modulus, or 28-day flexural strength ONLY, or 28-day compressive strength ONLY
Poisson's ratio	Poisson's ratio for new PCC typically ranges from 0.11 and 0.21, and values between 0.15 and 0.20 are typically assumed for PCC design.
Unit weight	Select agency historical data or from typical range for normal weight concrete: 140 to 160 pcf.

As many federal and state agencies move towards adopting M-EPDG for pavement design, guidelines for recommended inputs are being developed by FHWA, state DOTs and others. As part of FHWA's local calibration effort, North Carolina was selected as the pilot state to be included in their study "Local Calibration of the MEPDG Using Pavement Management Systems (FHWA 2010)." As part of this work, information from NCDOT's pavement management database and other sources was used to establish default (Level 2 and Level 3) values for M-EPDG. Information pertaining to these values is presented in the project report (FHWA 2010). Values used by NCDOT for the of new JPCP that are relevant to the work performed in this study are summarized in Table 7-5.

Table 7-5: Summary of M-EPDG default values for PCC pavement design used by NCDOT as part of FHWA local calibration study (FHWA 2010).

Description	Variable	PCC Input value
Project summary information	Design life (years)	30
Analysis parameters	Initial IRI (in/mi)	75
	Terminal IRI (in/mi)	170
	Transverse cracking (% slabs cracked)	10
	Mean joint faulting (in)	3/4
Traffic – Design properties	Initial two-way AADTT, trucks in the design direction, trucks in the design lane, operational speed	Pavement Management System data or project plans
Traffic volume and adjustment factors	Monthly adjustment factors, vehicle class distribution, truck hourly distribution factors, traffic growth factors	Traffic database
Axle load distribution factors	Axle load distribution factors by axle type	Traffic database
Climate	Surface shortwave absorptivity	M-EPDG default (0.85)
PCC design properties	Permanent curl/warp effective temperature difference (°F)	M-EPDG default (-10)
	Joint spacing (ft)	15
	Sealant type	Silicone
	Dowel diameter (in)	CD2
	Dowel bar spacing (in)	12
	Edge support – tied PCC	n/a
	Edge support – widened slab	n/a
	PCC-base interface	Full
	Base erodibility index	Resistant
	Loss of full friction (age in months)	360
PCC general properties	Unit weight (pcf)	150
	Poisson's ratio	0.20
PCC thermal properties	Coefficient of thermal expansion (per $°F \cdot 10^{-6}$)	Project files
	Thermal conductivity (BTU/(lf/°F)) and Heat capacity (BTU/(lf/°F))	Materials database developed by NC State University
PCC mixture properties	Cement type	Type II
	Cementitious material content (pcy)	526
	Water-cement ratio	0.559
	PCC zero-stress temperature (°F)	88
	ultimate shrinkage at 40% relative humidity (microstrain)	408
	reversible shrinkage (% of ultimate shrinkage)	50
	time to develop 50% of ultimate shrinkage	35
PCC strength properties	Compressive strength (psi)	4,500

Once threshold values for pavement performance, basic information on design life, and values for traffic, climate, and the pavement structure are input to the M-EPDG software, trial pavement sections can be analyzed. Suggested layer thicknesses and characteristics can be input and the analysis run iteratively in order to assess pavement performance and optimize the design pavement section.

7.3.2 Implications of Incorporating Recycled Brick Masonry Aggregate into M-EPDG

In Chapter 6, "Testing Program for Recycled Brick Masonry Aggregate Concrete," results of testing performed on RBMAC are compared to properties typical of concrete that uses natural coarse aggregate. Testing indicates that RBMAC exhibits several properties that differ from those of natural aggregate concrete, including unit weight and Poisson's ratio. Additionally, the thermal properties of RBMAC differ from those typically exhibited by concrete using natural coarse aggregates. Therefore, it is likely that the use of RBMAC in M-EPDG pavement design will result in design thicknesses that differ from those obtained using conventional concrete.

The M-EPDG inputs utilized by NCDOT for JPCP are shown in Table 7-5. A summary of M-EPDG input values that would differ between conventional PCC with the locally-available natural coarse aggregate (granite) and for RBMAC are shown in Table 7-6.

Table 7-6: M-EPDG input values for conventional PCC (using local granite aggregate) and for RBMAC

PCC Input	Default inputs (granite aggregate)	Value obtained for RBMAC
Aggregate type	granite	N/A
Unit weight (pcf)	150	125.5 to 128.2 (using water-reducing admixture)
Poisson's ratio	0.20	0.16 to 0.18 (at 28-days)
Coefficient of thermal expansion (in/in/°F)	5.58×10^{-6}	$4.4{\times}10^{-6}$ to $6.6{\times}10^{-6}$
Thermal conductivity (BTU/(hr•ft•°F))	1.25	0.533
Heat capacity (BTU/(ft•°F))	0.28	0.146

As shown in Table 7-6, the unit weight of RBMAC is over 20 pcf (320 kg/m^3) less than the NCDOT specified input for concrete containing natural aggregates available in North Carolina. Poisson's ratio of the RBMAC is slightly less than the default input specified by NCDOT for locally available materials (FHWA 2010).

Although guidelines are presented regarding NCDOT default inputs for some PCC properties, many other M-EPDG inputs, including those for thermal properties of PCC used in pavements, are not provided. AASHTO (2008) notes that "most agencies are not equipped with the testing facilities required to characterize the pavement materials" for portland cement concrete mixtures. NCDOT indicates that the input for CTE is to be obtained from project files (FHWA 2010), which is not applicable to this work. In lieu of material-specific testing, inputs for heat capacity and thermal conductivity are to be obtained from the MATS database developed by North Carolina State University (FHWA 2010), which was primarily developed for materials used in flexible pavement design (not readily available as part of this work). M-EPDG inputs recommended for the local natural aggregate type (granite), as shown in Table 7-6, were used. It can be seen in Table 7-6 that tests on RBMAC yielded values for thermal characteristics that differ significantly from the inputs recommended for use for concrete containing granite aggregate.

7.3.3 Comparison of M-EPDG Pavement Designs Using Recycled Brick Masonry Aggregate Concrete and Natural Aggregate Concrete

To assess the viability of RBMA for use in concrete pavement applications, it was desirable to compare JPCP pavements designed for similar traffic and exposure conditions that 1) use concrete containing a locally available natural coarse aggregate and 2) use concrete containing RBMA as coarse aggregate. M-EPDG software provides an excellent means for comparing the potential performance of RBMAC relative to concrete containing natural aggregate because concrete properties highly dependent on aggregate characteristics including the unit weight, CTE, thermal conductivity, and heat capacity are considered in the analysis models. For comparison purposes, traffic and climate information are held constant, along with some materials-related properties that can be controlled, such as: cement content, concrete strength, and water-cement ratio.

Truck traffic information typical of two different types of roadways was used in design of the comparison pavement sections. An AADTT value of 6000 was used to represent the typical truck traffic of an interstate, and an AADTT value of 600 was used to represent typical truck traffic of an arterial roadway pavement. These AADTT values are similar to those used for North Carolina roadways, including the FHWA local calibration of M-EPDG (FHWA 2010). Design speeds of 65 mph and 45 mph (104.6

km/hr and 72.4 km/hr) were used for the interstate and arterial roadway pavements, respectively. The lane width was set at 12 ft, with 2 lanes in the design direction for all analyses. The trucks in the design direction was input as 50%, with 95% trucks in the design lane. Traffic growth was input at 3% compound growth. Other M-EPDG input values associated with traffic, including the monthly and hourly volume adjustment factors, vehicle class distribution, mean wheel location, traffic wander standard deviation and axle configurations and load distributions were allowed to remain at the default settings. All inputs are shown in the M-EPDG analysis summaries provided in Appendix D.

Local climate data for Charlotte was downloaded from the M-EPDG online database. The depth to the water table was assumed to be 10 ft below grade. The Integrated Climatic Model (ICM) surface shortwave absorptivity, which correlates to the amount of solar energy absorbed by the pavement (Wang et al. 2008) was allowed to remain at the default value of 0.85 (unitless) for each analysis.

For the crushed stone and soil subgrade, Level 3 "best estimated" default values for materials typically used in the Charlotte, North Carolina area were used. Both the RBMAC pavements and conventional concrete pavements were designed on top of an 8 in thick layer of crushed stone placed on an A-6 subgrade soil material. The M-EPDG default values for the crushed stone base and A-6 subgrade material were used for each analysis and are shown in the M-EPDG analysis summaries provided in Appendix D.

Several structural design features of both the RBMAC pavement designs and the conventional concrete pavement designs were kept constant at values specified by NCDOT (FHWA 2010). These include the joint spacing (15 ft, or 4.57 m), sealant type (silicone), and dowel bar spacing (12 in, or 0.305 m). The base type was specified as "granular," with an erodibility index of "Erosion Resistant (3)." The PCC-Base Interface was input as "full friction contact," with a loss of full friction at 360 months. No edge support was utilized. Dowel bar diameter was 1.75 in (44.5 mm), as the software will not accept the 2 in (50.8 mm) dowel diameter specified by NCDOT for M-EPDG analysis (FHWA 2010).

To compare the required pavement thicknesses for RBMAC and conventional concrete pavement, only input values for properties that differed significantly between the two materials were changed for the M-EPDG analysis. These properties included unit weight, Poisson's ratio, CTE, thermal conductivity, and heat capacity. The values used for the performance comparisons are shown in Table 7-7.

Table 7-7: JPCP inputs varied in M-EPDG analysis of conventional PCC and RBMAC pavements.

Input	**Value used for conventional PCC (with granite aggregate)**	**Value used for RBMAC (with higher coefficient of thermal expansion)**	**Value used for RBMAC (with lower coefficient of thermal expansion)**
Unit weight (pcf)	150	130	130
Poisson's ratio	0.20	0.18	0.18
Coefficient of thermal expansion (in/in/°F)	5.6×10^{-6}	5.6×10^{-6}	4.4×10^{-6}
Thermal conductivity (BTU/(hr•ft•°F))	1.25	0.533	0.533
Heat capacity (BTU/(ft•°F))	0.28	0.20	0.20

The input values selected for the unit weight and Poisson's ratio of the conventional PCC with granite aggregate is the same as the default value used in M-

EPDG by NCDOT (FHWA 2010). The input values selected for the unit weight and Poisson's ratio of the RBMAC are the values obtained when testing BAC 6.2.

The value used of CTE of concrete using granite aggregate is the default value suggested by M-EPDG for concrete with granite aggregate, and it is also similar to the measured CTE for concrete containing granite coarse aggregate as determined by Sakyi-Bekoe (2008). Two values of the CTE for the RBMAC were selected for the comparative designs of the pavements. The average value of CTE obtained by testing the RBMAC was similar to that of the concrete containing granite aggregate, 5.6×10^{-6} in/in/°F (10.1×10^{-6} m/m/°C). Klingner (2010) indicates that the typical CTE of brick is between 3×10^{-6} and 4×10^{-6} in/in/°F (5.4×10^{-6} and 7.2×10^{-6} m/m/°C). As discussed in Section 6.4.3.2.6.1, "Coefficient of Thermal Expansion," equipment to perform the CTE test to an accuracy useful for M-EPDG analysis is not currently available at UNC Charlotte. It is suspected by the author that testing in accordance with AASHTO T336 will result in a value lower than 5.6×10^{-6} in/in/°F (10.1×10^{-6} m/m/°C). The lowest value of CTE determined by testing the RBMAC was 4.4×10^{-6} in/in/°F (7.92×10^{-6} m/m/°C). Therefore, in order to facilitate a comparison and a design sensitivity to CTE, a CTE of 4.4×10^{-6} in/in/°F (7.92×10^{-6} m/m/°C) was also used in some of the RBMAC pavement designs.

Testing of BAC 6.2 yielded an average heat capacity value of 0.146 BTU/(ft•°F) (611 J/(kg•°C)). This value is outside of the range of values considered reasonable for conventional concrete suggested for use in M-EPDG (AASHTO 2008), which is 0.22 to 0.40 BTU/(ft•°F) (921 to 1,675 J/(kg•°C)). As discussed in Section 6.4.4.2.6.3, Heat Capacity, the value suggested for use in design for conventional concrete is 0.28

BTU/(ft•°F) (1,172 J/(kg•°C)). Initial (trial) runs of M-EPDG to design RBMAC pavement utilizing a heat capacity of 0.146 BTU/(ft•°F) (611 J/(kg•°C)) yielded inconsistent results, with almost all slabs predicted to experience transverse cracking within the first few years of service. Kodide and Shin (2011) and Kodide (2010) report that use of heat capacity values lower than the minimum value of the suggested range can result in problems with the distress models. Therefore, a heat capacity that is closer to the lowest value of the suggested range (0.20 BTU/(ft•°F), or 837 J/(kg•°C)) was used in subsequent M-EPDG analysis. Reasonable predictions resulted when this value was used. The value for thermal conductivity used for the RBMAC was the measured value for BAC 6.2. It is noted that this value is significantly lower than the range suggested for use in M-EPDG, 1.0 to 1.5 BTU/(hr•ft•°F) (1.77 to 2.60 W/(m•K)). Use of this value did not appear to cause the M-EPDG software to produce unreasonable results. However, additional work could be performed in the future to confirm that these values did not affect the validity of the distress models.

Other values for both the RBMAC and conventional concrete pavement were kept constant to facilitate the comparison of the change in coarse aggregate type. These input values are listed in Table 7-8. Many of these values were those obtained through laboratory testing of the RBMAC, in particular, mixture BAC 6.2. In order to facilitate the comparison of RBMAC and conventional concrete made with local granite aggregates, it was assumed that the 28-day modulus of rupture obtained using BAC 6.2 could also be obtained by the conventional concrete using a similar cementitious content (575 pcy, or 341 kg/m^3) at a similar w/c ratio (0.32).

Table 7-8: JPCP inputs kept constant in M-EPDG analysis of conventional PCC and RBMAC pavement.

Input	Value used for both RBMAC and conventional concrete designs
Cement type	Type II
Cementitious material content (pcy)	575
Water/cement ratio	0.32
PCC zero-stress temperature (°F)	88
Ultimate shrinkage at 40% relative humidity (microstrain)	408
Reversible shrinkage (% of ultimate shrinkage)	50%
Time to develop 50% of ultimate shrinkage (days)	35
Curing method	Curing compound
28-day PCC modulus of rupture (psi)	716

M-EPDG requests that the user input a coarse aggregate type, which is input as a name only, and does not trigger values used in the distress modeling. The software will not run, however, without inputting the name of the coarse aggregate. Options include a number of conventional coarse aggregates including quartzite, limestone, dolomite, granite, rhyolite, basalt, synetite, gabbro, and chert. Since recycled brick masonry aggregate is not an option included in the software, effort was made to select an available aggregate type that had the closest composition (silicate and aluminate materials), along with a very fine grain size. As shown in Appendix D, it was decided to use “rhyolite,” as it is an igneous rock which has experienced temperatures similar to the firing temperatures of clay brick and also has a very fine grain structure.

For the M-EPDG analysis, distress model calibration settings for rigid pavement were maintained at the default values. These included:

- faulting coefficients and reliability model equations
- cracking and fatigue coefficients and reliability model equations, and
- ICM model coefficients.

Performance criteria, however, were set to the values used by NCDOT, as shown in Table 7-5. The initial IRI was set at 75 in/mile, while the terminal IRI was set to 170 in/mile. The limit on transverse cracking was set to 10% slabs cracked. The limit on mean joint faulting was set to 0.75 in (19.1 mm). For each of the performance criteria, the desired reliability was set at 90%.

For each type of pavement, M-EPDG analyses were run using a concrete thickness of 11 in (279.4 mm), a thickness which allowed all the types of concrete pavements to pass the failure criterion for transverse cracking, joint faulting, and roughness. Then an analysis of each pavement was run at successively thinner PCC thicknesses (in ¼ in increments) until one of the three failure criteria was met. The minimum required concrete thickness to pass all three failure criterion was recorded, along with the predicted mode of failure for the pavement if the concrete thickness is reduced by ¼ in from the minimum thickness passing all three failure criterion.

For interstate pavements, with the AADTT set to 6000 and vehicle speeds set to 65 mph, the minimum design thicknesses shown in Table 7-9 were obtained using M-EPDG. Similarly, arterial pavements, with the AADTT set to 600 and vehicle speeds set to 45 mph, the design thicknesses shown in Table 7-10 were obtained using M-EPDG. A typical M-EPDG input summary for RBMAC pavement is shown in Appendix D (Figure D-1), along with the corresponding output reliability summary (Figure D-2). A typical

M-EPDG input summary for conventional PCC pavement is also provided in Appendix D (Figure D-3), along with the corresponding output reliability summary (Figure D-4).

Table 7-9: Comparison of RBMAC and conventional PCC sections designed using M-EPDG for pavement with traffic typical of interstate roadways.

Type of pavement	Coefficent of thermal expansion (in/in/°F)	Thermal conductivity (BTU/(hr•ft•°F))	Heat capacity (BTU/(ft•°F))	Minimum required concrete thickness (in)	Mode of failure (below 90% reliability)
Conventional PCC with granite aggregate	5.6×10^{-6}	1.25	0.28	10	Transverse cracking (at 9.75 in)
RBMAC	4.4×10^{-6}	0.533	0.20	9.5	Transverse cracking (at 9.25 in)
	5.6×10^{-6}	0.533	0.20	10.75	Terminal IRI (at 10.5 in)

Table 7-10: Comparison of RBMAC and conventional PCC sections designed using M-EPDG for pavement with traffic typical of arterial roadways.

Type of pavement	Coefficent of thermal expansion (in/in/°F)	Thermal conductivity (BTU/(hr•ft•°F))	Heat capacity (BTU/(ft•°F))	Minimum required concrete thickness (in)	Mode of failure (below 90% reliability)
Conventional PCC with granite aggregate	5.6×10^{-6}	1.25	0.28	8.5	Transverse cracking (at 8.25 in)
RBMAC	4.4×10^{-6}	0.533	0.20	8.25	Transverse cracking (at 8 in)
	5.6×10^{-6}	0.533	0.20	8.75	Terminal IRI (at 8.5 in)

M-EPDG analyses of pavement sections designed using RBMAC versus conventional PCC indicate that RBMAC pavement could provide adequate performance at a thickness similar to, or slightly less than, PCC pavement. For both the interstate and arterial pavements, the thinnest concrete pavement section was obtained for the RBMAC with all thermal characteristic inputs lower than those of the conventional PCC. The differences in minimum required concrete thicknesses are more noticeable in the comparison of pavement sections for interstates than in the comparison of pavement sections for arterial roadways.

The sensitivity of the concrete's CTE on M-EPDG designs is evident as shown by the comparison pavement sections in Table 7-9 and Table 7-10; the variation of the CTE of the RBMAC results in a change in the minimum required thickness. This is in agreement with the findings of many recent studies that have shown that concrete pavement deterioration models in M-EPDG are very sensitive to aggregate characteristics, especially thermal behavior (Crawford et al. 2010 and Tanesi et al. 2010).

The CTE of concrete plays a large role in concrete pavement performance, as "the magnitudes of temperature-related pavement deformations are directly proportional to this value during early ages (i.e., within 72 h of paving) as well as during the pavement design life" (Mallela et al. 2005). These temperature-related deformations affect curling-induced stresses and axial stresses, which contribute to both top-down and bottom-up transverse cracking, as well as joint deterioration. As discussed by Mallela et al. (2005), higher values of concrete's CTE have been linked to:

- Early-age random cracking caused by excessive longitudinal slab movement on a highly-resistant base,
- Higher curling stresses, resulting in increased mid-slab transverse and longitudinal cracking,

- Larger amounts of slab support loss at early ages due to curling,
- Larger joint openings during cooler seasons,
- Greater magnitudes of corner deflections due to curling, and
- Excessive joint opening and closing, resulting in loss of joint sealant and subsequent faulting.

Several studies have been performed to evaluate the role of concrete's CTE on M-EPDG pavement distress predictions. Mallela et al. (2005) performed a sensitivity analysis on the CTE in M-EPDG and found that a higher value resulted in "increased top-down cracking damage in pavements." A similar sensitivity analysis by Tanesi et al. (2007) indicated that predictions for slab cracking were more sensitive to changes in the CTE at higher CTE values than at lower CTE values.

RBMA produced using the Idlewild Elementary School demolition waste contained approximately 1/3 mortar material by volume (and by mass) and also contained some clay tile material. Additionally, bricks from Idlewild Elementary School had cores (interior voids), and a significant amount of mortar was present within the cores. It is anticipated that a lower mortar fraction could be obtained from other sources of RBMA, particularly from bricks that do not have cores. The CTE of the RBMAC produced from this material would likely be closer to the published values of the CTE of brick, 3×10^{-6} and 4×10^{-6} in/in/°F (5.4×10^{-6} and 7.2×10^{-6} m/m/°C) (Klingner 2010). Therefore, M-EPDG would predict more favorable (i.e. thinner) sections using RBMAC.

It can be seen in Table 7-9 and Table 7-10 that, for RBMAC pavement sections, when the CTE is increased, the failure mode changes from transverse cracking to terminal IRI (increase in roughness to unacceptable levels). As discussed in Section

7.3.1, “Mechanistic-Empirical Pavement Design Guide (M-EPDG) Overview,” the deterioration model for smoothness includes an equation that considers the increase in the IRI value (in in/mi) by adding the increase in IRI attributable to slabs with transverse cracks, joints with spalling, and joint faulting. Therefore, the IRI failure mode includes the influences of the two other failure modes (transverse cracking and joint faulting) (Tanesi et al. 2007). It is possible that use of the larger input value for CTE, combined with the lower input values for heat capacity and thermal conductivity, resulted in the prediction of the IRI (roughness) mode of failure reaching the threshold reliability level (90%) prior to the transverse cracking failure mode. Mallela et al. (2005) found that although “the effect of CTE on IRI is more sensitive to concrete strength, a marginal increase in CTE even at lower values resulted in significantly more roughness.” Tanesi et al. (2005) found that “the higher the CTE, the greater the effect of δ_{CTE} [change in CTE] on the predicted IRI,” which may also support this finding.

It is noted that the testing to determine the heat capacity and thermal conductivity of the RBMAC were performed using materials characterization equipment readily available at UNC Charlotte. M-EPDG indicates that testing to determine the heat capacity of concrete be performed in accordance with ASTM D2766, “Standard Test Method for Heat capacity of Liquids and Solids (AASHTO 2008).” This test method uses a drop-method-of-mixtures calorimeter, which was not available at UNC Charlotte. The calorimeter used for this work (Sensys Evo TG-DSC) is a Calvet-type calorimeter. Both types of calorimeters measure changes in the voltage across a thermocouple with respect to changes in temperature. Further testing and evaluation would need to be performed to determine whether the type of calorimeter used for this work adversely affected the value of heat capacity obtained (in relation to typical values used in M-EPDG).

M-EPDG also indicates that thermal conductivity should be measured in accordance with ASTM E1952, “Standard Test Method for Thermal Conductivity and Thermal Diffusivity by Modulated Temperature Differential Scanning Calorimetry,” which utilizes the modulated temperature differential scanning method (mDSC) of calorimetry. This equipment is not currently available at UNC Charlotte, and therefore thermal conductivity testing was performed using the TCi technique, which computes thermal conductivity based on effusivity measurements. Again, further testing and evaluation would need to be performed to determine whether the type of instrument (and therefore method of computation of thermal conductivity) used for this work adversely affected the value of thermal conductivity obtained (in relation to typical values used in M-EPDG).

Section 7.3.4 Sensitivity of Slab Thickness to Thermal Property Inputs in M-EPDG for Recycled Brick Masonry Aggregate Concrete and Natural Aggregate Concrete

Analyses were performed to evaluate the sensitivity of pavement slab thickness in M-EPDG to the following thermal properties: coefficient of thermal expansion, thermal conductivity, and heat capacity. M-EPDG designs were performed in a manner similar to those in Section 7.3.3, “Comparison of M-EPDG Pavement Designs Using Recycled Brick Masonry Aggregate Concrete and Natural Aggregate Concrete,” except only thermal properties were varied, one property at a time, while all other properties were held constant at measured or default values. For these sensitivity analyses, the inputs listed below were held constant for all M-EPDG designs for both RBMAC and conventional PCC:

1. AADTT = 6000 trucks per day; design speed = 65 mph; lane width = 12 ft; 2 lanes in the design direction. Traffic growth = 3% compound growth.
2. Local climate data for Charlotte; depth to water table = 10 ft; surface shortwave absorptivity = 0.85.
3. 8 in crushed stone base; Level 3 default input values for stone base properties.
4. A-6 subgrade soil; Level 3 default inputs for soil properties with the exception of resilient modulus (6,000 psi).
5. Joint spacing = 15 ft; silicone joint sealant; 12 in dowel bar spacing; 1.75 in dowel bar diameter.
6. Granular-type base with erodibility index of “Erosion Resistant (3)”; PCC-Base interface = “full friction contact” with loss of full friction at 360 months. No edge support.
7. For the conventional PCC with granite aggregate, the unit weight = 150 pcf and Poisson’s ratio = 0.20; other values for conventional PCC inputs are those listed in Table 7-8.
8. For the RBMAC, the unit weight = 130 pcf and Poisson’s ratio = 0.18. Other values for RBMAC concrete inputs are those listed in Table 7-8.
9. Performance criteria values were set to those used by NCDOT, shown in Table 7-5, with the desired reliability set to 90% for each performance criteria.

The results of the sensitivity analyses for CTE are shown in Table 7-11 and in Figure 7-1. The RBMAC is more sensitive to changes in CTE at higher values.

However, these inputs would not be likely, due to the relatively low CTE of brick. At lower (more realistic) CTE values, the predicted minimum design thicknesses of RBMAC varied over a range of only approximately ¼ in. For RBMAC pavements, at lower CTE values, the predicted failure mode (for a 0.25 in thinner slab) was transverse slab cracking. However, at higher CTE values, the predicted mode of failure for RBMAC pavements was roughness (IRI).

Table 7-11: Sensitivity of slab thickness to CTE input in M-EPDG for RBMAC and conventional PCC pavements.

	Minimum slab thickness (in)	
Coefficient of Thermal Expansion ($\times 10^{-6}$ in/in/°F)	**Conventional PCC with granite aggregate**	**RBMAC**
	thermal conductivity = 1.25 BTU/(hr·ft·°F) **heat capacity = 0.28 BTU/(ft·°F)**	**thermal conductivity = 0.533 BTU/(hr·ft·°F)** **heat capacity = 0.20 BTU/(ft·°F)**
3.5	9.50	9.25
4.0	9.50	9.50
4.4	9.75	9.50
5.0	9.75	10.00
5.6	10.00	10.75
6.0	10.00	11.50

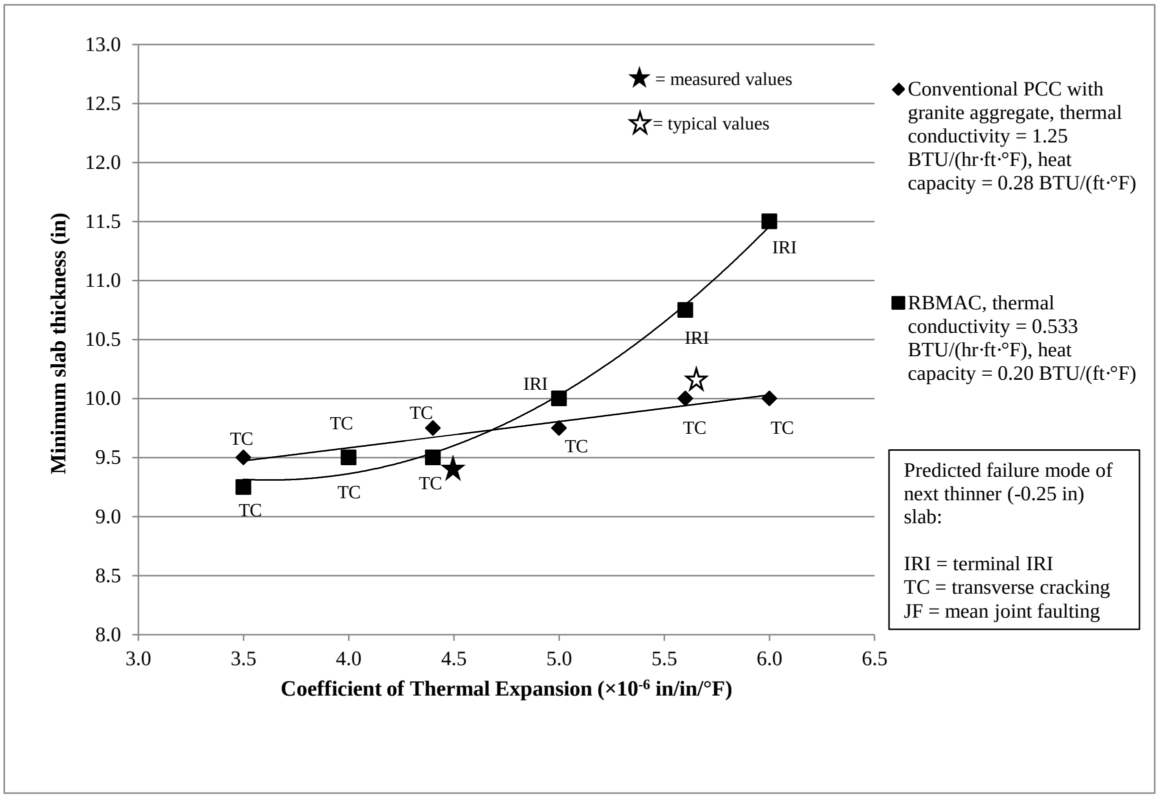

Figure 7-1: Sensitivity of slab thickness to CTE input in M-EPDG for RBMAC and conventional PCC pavements.

For conventional PCC slabs, the predicted minimum thickness increased nearly linearly with increasing CTE. Higher CTE values combined with low heat capacity and thermal conductivity values resulted in a non-linear relationship between predicted slab thickness and CTE for RBMAC. Several researchers (Tanesi et al. 2007, Crawford et al. 2010, and others) cite CTE as the key thermal property affecting concrete performance in M-EPDG. This sensitivity analyses supports this previous research.

The results of the sensitivity analyses for thermal conductivity are shown in Table 7-12 and in Figure 7-2. The predicted RBMAC slab thickness was not very sensitive to thermal conductivity, with the same minimum slab thickness required for thermal conductivities between the measured value of 0.533 BTU/(hr·ft·°F) and 1.25

BTU/(hr·ft·°F). If the thermal conductivity value was reduced to 0.5, a significant decrease in predicted minimum slab thickness occurs. The unrealistic results may be due to the thermal conductivity value being too far from the suggested input for design, 1.25 BTU/(hr·ft·°F).

Table 7-12: Sensitivity of slab thickness to thermal conductivity input in M-EPDG for RBMAC and conventional PCC pavements.

	Minimum slab thickness (in)	
Thermal Conductivity (BTU/(hr·ft·°F))	**Conventional PCC with granite aggregate**	**RBMAC**
	CTE = 5.6×10^{-6} in/in/°F heat capacity = 0.28 BTU/(ft·°F)	**CTE = 4.4×10^{-6} in/in/°F heat capacity = 0.20 BTU/(ft·°F)**
0.5	Not computed for this value	8.75
0.533	Not computed for this value	9.5
0.75	11.75	9.5
1	10.5	9.5
1.25	10	9.5
1.5	10	Failed ICM stability check, no result
1.75	10	Failed ICM stability check, no result

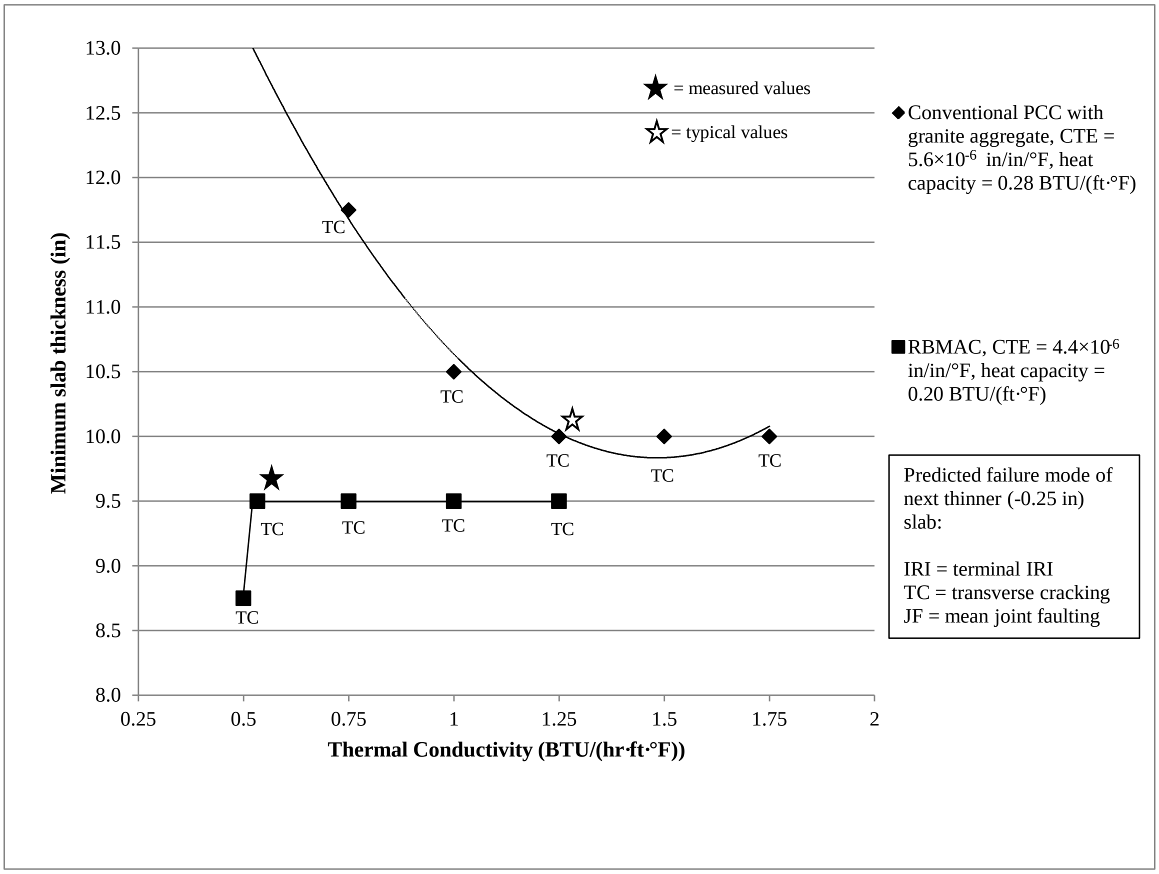

Figure 7-2: Sensitivity of slab thickness to thermal conductivity input in M-EPDG for RBMAC and conventional PCC pavements.

At values of thermal conductivity above the M-EPDG recommended input of 1.25 BTU/(hr·ft·°F), the software indicated an ICM stability failure, which is related to limitations in the models supporting the software. Kodide and Shin (2011) report similar ICM stability failure error messages for some combinations of thermal conductivity and heat capacity values. Kodide and Shin (2011) report that values of heat capacity and thermal conductivity that are below a line defined by

$$HC = 0.1386 \times TC - 0.0219 \qquad (7\text{-}1)$$

fall within a zone called the "ICM stability check failure zone" in M-EPDG, where *HC* is heat capacity and *TC* is thermal conductivity. It is noted that, for this work, the values of heat capacity and thermal conductivity resulting in ICM stability failure fall within the "ICM stability check failure zone" as defined by Kodide and Shin (2011) using Eq. 7-1.

Pavement designs using conventional PCC with granite aggregate were much more sensitive to thermal conductivity, particularly at low values of thermal conductivity. Above the suggested M-EPDG input of 1.25 BTU/(hr·ft·°F), predicted minimum slab thicknesses remained constant at 10 in. However, predictions indicated that successively thicker slabs were required at lower values of thermal conductivity. When thermal conductivity was reduced to 0.75 BTU/(hr·ft·°F), M-EPDG predicted that the pavement needed to be 1.75 in thicker than the minimum thickness predicted when thermal conductivity is input at the recommended 1.25 BTU/(hr·ft·°F). This may indicate that M-EPDG is providing unrealistic results, possibly because minimum slab thicknesses were not identified for conventional PCC pavements with thermal conductivities below 0.75 BTU/(hr·ft·°F).

The results of the sensitivity analysis for heat capacity are shown in Table 7-13 and in Figure 7-3. As indicated in Table 7-13, unreliable results were obtained for the conventional PCC pavement when the heat capacity was input as 0.16 BTU/(ft·°F). At this input value, contrary to expectations, increasing slab thicknesses above 10.25 in resulted in M-EPDG predicting failures much earlier than the 30 year design life. These results were considered inconsistent at this heat capacity value and therefore no minimum slab thickness is reported.

Table 7-13: Sensitivity of slab thickness to heat capacity input in M-EPDG for RBMAC and conventional PCC pavements.

Heat Capacity (BTU/(ft·°F))	Minimum slab thickness (in)	
	Conventional PCC with granite aggregate	**RBMAC**
	CTE = 5.6×10^{-6} in/in/°F thermal conductivity = 1.25 BTU/(hr·ft·°F)	**CTE = 4.4×10^{-6} in/in/°F thermal conductivity = 0.533 BTU/(hr·ft·°F)**
0.16	Inconsistent results*	9.75
0.2	10.25	9.5
0.24	10	9.25
0.28	10	9.25
0.32	10	9.25
0.36	9.75	9
0.4	9.75	9

*At heat capacity = 0.16 BTU/(ft·°F), PCC with granite aggregate began to show successively earlier failure (% of slabs showing transverse cracks) for increasing thicknesses (greater than 10.25 in.), and results were therefore deemed inconsistent.

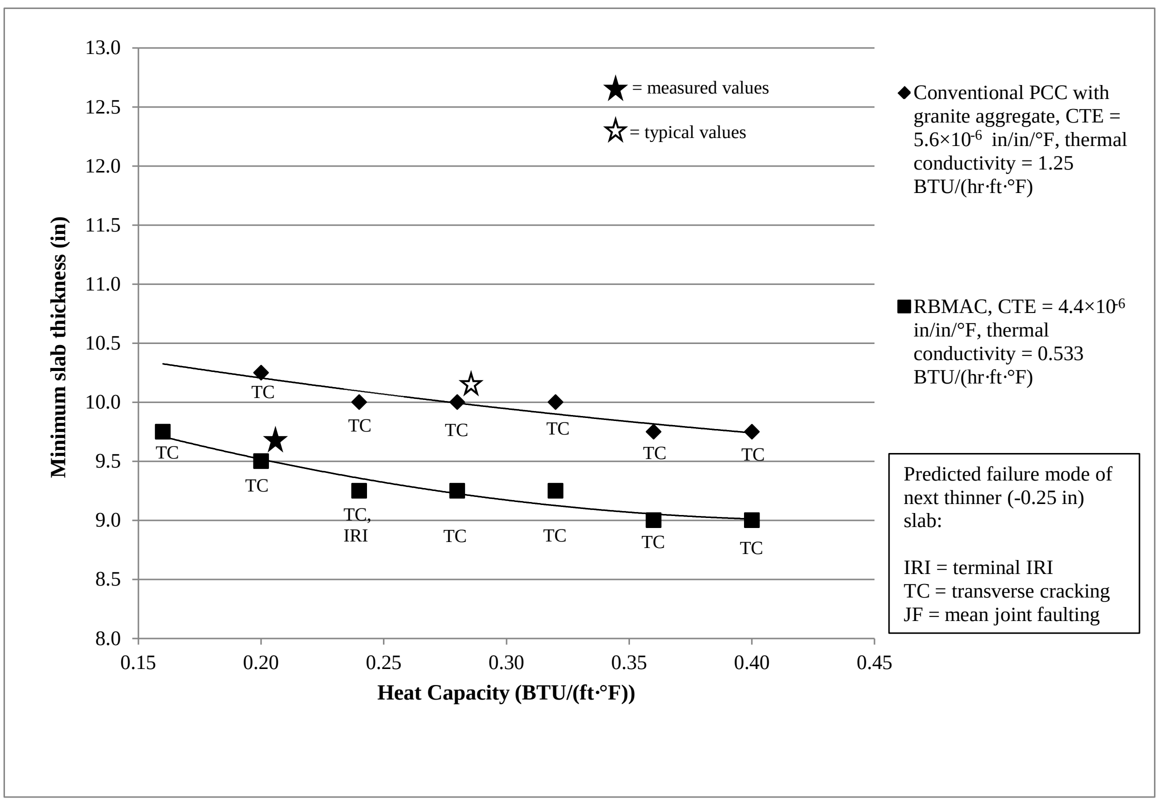

Figure 7-3: Sensitivity of slab thickness to heat capacity input in M-EPDG for RBMAC and conventional PCC pavements.

As discussed previously in section 6.4.4.2.6.3, "Heat Capacity," Wang et al. (2008) state that "the heat capacity of PCC is not a sensitive parameter for pavement design." As can be seen in Table 7-13 and Figure 7-3, M-EPDG predicts that thicker slabs are required at lower heat capacity inputs. The slope of best-fit lines for predicted slab thickness for RBMAC and conventional PCC pavements is similar, indicating a similar sensitivity to heat capacity. Based on the results observed in this sensitivity study, it can be stated that the M-EPDG predicted minimum slab thicknesses are less sensitive to heat capacity than to the coefficient of thermal expansion.

7.4 Proposed Test Pavement Utilizing Recycled Brick Masonry Aggregate Concrete in Charlotte, North Carolina

No RBMAC construction, including RBMAC pavement, exists in the United States. Although laboratory testing indicates that RBMAC can exhibit acceptable mechanical properties and durability performance for use in pavement applications, field studies are necessary to confirm that RBMAC exhibits suitable performance under actual service conditions. As part of this work, a test pavement is proposed. Details on the design of the test pavement, its predicted performance, an instrumentation plan, and a materials sampling and testing plan, are provided in this section.

7.4.1 Overview of Proposed Project

It is planned that the test pavement will be constructed at D.H. Griffin Crushing and Grading in Charlotte, North Carolina. The proposed pavement will be approximately 60 ft (18.3 m) wide by approximately 200 ft (61.0 m) long. Half of this test pavement will be constructed of RBMAC, while the other half will be constructed using a similar conventional concrete mixture (using locally available natural aggregate). Both sections will be constructed in the travel lanes, in line with the weigh scales that serve the crushing and grading facility. A photograph of the site is shown in Figure 7-4.

Figure 7-4: Overview of the proposed test pavement site.

Currently, a deteriorated undoweled JPCP of varying thicknesses and composition is present at the site. The existing pavement is severely distressed, exhibiting extensive cracking and deflection at its joints. Moisture ingress into the subgrade has likely resulted in its substantial weakening. It is proposed that the existing concrete pavement will be removed and the subgrade prepared as required to provide suitable support for the new pavement. This may include removal and replacement of the existing subgrade material to a depth that will ensure that sound foundation soils can be backfilled.

7.4.2 Design of Recycled Brick Masonry Aggregate Concrete Test Pavement and Control Pavement

The test pavement and the control pavement were designed using M-EPDG. Level 1 input values (site specific) were utilized wherever possible, including the input values for the RBMAC. Level 2 input values (correlated data) then Level 3 inputs (default values) were used when Level 1 input values were not available. When appropriate, the M-EPDG input values used by NCDOT (as shown in Table 7-5) were used in the design. The M-EPDG input data used for the RBMAC test pavement, along with the reliability summary (output), are shown in Appendix D in Figures D-5 and D-6. The M-EPDG input data used for the conventional PCC control pavement, along with the reliability summary (output), are shown in Appendix D in Figures D-7 and D-8.

A design life of 30 years was selected for the test pavement. D.H. Griffin personnel provided information to be used in the pavement design, including truck weights, axle configurations, and trip counts. For design purposes, pavement construction was assumed to begin in October 2011, with the pavement opening to traffic in January 2012. NCDOT performance criteria for concrete pavements were used as limits and reliability levels for IRI, transverse cracking, and mean joint faulting. These limits and reliability levels are shown in Table 7-14.

Table 7-14: Performance criteria used in M-EPDG design of test pavement and selected limits and reliability levels.

Performance Criteria	**Limit**	**Reliability**
Initial IRI (in/mi)	63	
Terminal IRI (in/mi)	170	90%
Transverse cracking (% slabs cracked)	1	90%
Mean joint faulting (in)	0.75	90%

Trucks entering the facility carry loads of demolition rubble headed to the crushing and grading operations. Trucks leaving the facility typically contain crushed, graded recycled aggregate material or are empty. D.H. Griffin personnel indicated that the one-day maximum traffic loading experienced by the entrance drive where the proposed test pavement will be constructed is 293 tri-axle trucks at approximately 78,060 lb each.

For this analysis, the initial one-way AADTT was rounded up to 300. D.H. Griffin personnel indicated that they did not anticipate any growth at the facility over the design lifetime. However, to be conservative, a compound growth factor of 2% was utilized. In order to account for the directionality of the provided traffic information, M-EPDG inputs included 100% trucks in the design direction and 100% of the trucks in the design lane. Monthly and hourly adjustment factors were left as default values, as were vehicle-specific inputs such as axle configuration and spacing, tire spacing, wheelbase spacing, and tire pressure. These default values are shown in Appendix D in Figures D-5 and D-7.

Traffic information provided by D.H. Griffin did not include information on weight distribution between the front single axle and the rear triple axles of the trucks. Therefore, the websites of several truck manufacturers were reviewed in order to determine the typical maximum axle loads for similar tri-axle trucks, along with typical spacing between axles. Based upon information posted on the manufacturers' websites, it was decided that truck traffic at the subject site should be classified as Class 7 vehicles for the M-EPDG designs. Since the test pavement will experience loads from almost exclusively tri-axle trucks, M-EPDG inputs were adjusted to allow for 100% heavy vehicles, Class 7. Default load equivalency factor (LEF) values for Class 7 vehicles are 1 single axle, 0.26 tandem axle and 0.83 tridem axle. Other pavement-specific inputs also included with the traffic information in this M-EPDG analysis included the design lane width (assumed to be 12 ft, or 3.66 m), traffic wander (10 in, default value) and mean wheel location (18 in from the lane marking, default value).

Joint spacing for the test pavement is proposed at 12 ft (3.66 m), with 1.75 in (44.5 mm) diameter dowel bars spaced at 12 in (304.8 mm) on-center. No sealant is planned for the joints. It is anticipated that the test pavement will be included in a larger concrete pavement comprising several acres, which will be much wider than the drive lanes. Therefore, the edge support was modeled as tied shoulders with widened slabs. It is assumed that the area of concrete pavement not included in the test pavement will contain natural aggregates.

For the subject site, climatic data for the Charlotte-Douglas airport was downloaded from the M-EPDG website and utilized in the analysis. The depth to the water table was assumed to be 10 ft (3.05 m). In order to design both the RBMAC test pavement and the conventional concrete control section, all inputs other than those listed in Table 7-15 were held constant. Inputs used in the analysis, including the default values, are included in Figures D-5 and D-7.

Table 7-15: M-EPDG Inputs for the RBMAC test pavement and conventional concrete control pavement.

PCC Input Value	Value used for control section (PCC with natural aggregate)	Value used for RBMAC test section
Aggregate type	granite	rhyolite
Unit weight (pcf)	150	130
Poisson's ratio	0.20	0.18
Coefficient of thermal expansion (in/in/°F)	5.6×10^{-6}	4.4×10^{-6}
Thermal conductivity (BTU/(hr•ft•°F))	1.25	0.533
Heat capacity (BTU/(ft•°F))	0.28	0.20

The proposed RBMAC test pavement and the control pavement were designed using an unbound crushed stone base, 12 in (0.305 m) thick, with an elastic modulus of 30,000 psi (206.8 MPa). Poisson's ratio was specified as 0.35, with the coefficient of lateral pressure allowed to remain at the default value of 0.5. The gradation and plastisticity index values were left at the default values (shown in Appendix D). The ICM included in the software was used to calculate or derive other parameters for the crushed stone base, including the maximum dry unit weight (127.2 pcf, or 2038 kg/m^3), specific gravity of solids (2.70), and moisture-related parameters, such as: saturated hydraulic conductivity, optimum gravimetric water content, and calculated degree of saturation.

Information on the characteristics of the soils underlying the subject site was obtained from the United States Department of Agriculture (USDA) Natural Resources Conservation Service Web Soil Survey (WSS). The WSS contains soils information obtained by the National Cooperative Soil Survey. This website allows the user to define an area of interest (in this case, by address) and provides soils information including the USDA classification. Additional information on some soil properties is also available for some locations. According to the WSS, the soils underlying the subject area (where the test pavement is to be constructed) are sandy clay loam, CeB2 or CeD2.

Using the grain size distribution presented in the USDA textural classification triangle, the soil is likely comprised of approximately 60% sand, 30% clay, and 10% silt. With this grain size distribution, the soil can be classified in the Unified Classification System as an SC (clayey sand). Due to the likelihood of a low liquid limit, the soil could be considered either an A-4 or an A-6 by the AASHTO Classification System. In order to better classify the soil within the AASHTO system (either A-4 or A-6), Atterburg limits testing would need to be performed to determine whether the plasticity index (PI) is greater than 10 (A-6) or less than 10 (A-4) (Das 1994).

For an A-6 soil, M-EPDG indicates that a resilient modulus value within the range of 12,000 to 24,000 psi (82.7 to 165.5 MPa) should be used, with 14,000 psi (110.3 MPa) recommended. For an A-4 soil, a resilient modulus value within the range of 13,000 to 29,000 psi (89.6 to 200.0 MPa) is suggested, with 15,000 psi (103.4 MPa) recommended. Based on experience with local soils, these values are quite high and could result in an unconservative pavement section, falsely indicating successful performance against the M-EPDG performance criteria. It is also unclear whether the existing pavement is sitting on fill or residual soils. Without having specific information obtained from on-site borings and soil tests, it was decided that a more conservative (lower) value of resilient modulus should be used in these designs. A resilient modulus of 6,000 psi was thus used for the subgrade resilient modulus input.

The Gradation and Plasticity Index values were allowed to remain the default values. Using the ICM data and modeling, additional soil properties were computed and/or derived by the M-EPDG software. These included maximum dry unit weight (118.4 pcf), specific gravity of solids (2.70), and moisture-related parameters such as: saturated hydraulic conductivity, optimum gravimetric water content, and calculated degree of saturation. These values are shown in Appendix D in Figures D-5 and D-7.

7.4.3 M-EPDG Predicted Performance of Proposed Test Pavement and Control Pavement

Based upon the input values and assumptions previously described, M-EPDG analyses indicated that the proposed RBMAC and the conventional PCC (control) pavement sections, summarized in Table 7-16, should perform satisfactorily over the 30 year design life. The predicted reliabilities for the proposed pavement sections are shown in Table 7-17. It is noted that the required thickness of the control pavement section, which will be comprised of concrete with natural aggregates, needs to be slightly thicker than the RBMAC pavement in order to provide a similar reliability in M-EPDG distress modeling. However, it is likely that both pavement sections would be constructed to the same thickness (9.5 in, or 241.3 mm) for constructability reasons. As indicated previously, output from the M-EPDG software for the proposed test pavement and the control pavement sections are included in Appendix D in Figures D-6 and D-8.

Table 7-16: Layer thicknesses for proposed RBMAC test pavement and control pavement.

Layer	Control pavement (PCC with natural aggregate)		RBMAC test pavement	
JPCP	PCC with locally available natural aggregate (granite)	10.5 in	RBMAC	9.25 in
Base	Crushed stone base	12 in	Crushed stone base	12 in
Subgrade	Subgrade soils, A-4, with 6,000 psi resilient modulus	Infinite	Subgrade soils, A-4, with 6,000 psi resilient modulus	Infinite

Table 7-17: Predicted reliabilities for the proposed RBMAC test pavement and control pavement.

Performance criteria	Distress target	Reliability target	Distress predicted for control pavement	Reliability predicted for control pavement	Distress predicted for RBMAC test section	Reliability predicted for RBMAC test section	Accept-ability
Terminal IRI (in/mi)	170	90%	69.9	99.999%	72.4	99.99%	Pass
Transverse cracking (% slabs cracked)	15	90%	3.5	97.03%	4.2	95.51%	Pass
Mean joint faulting (in)	0.75	90%	0.002	99.999%	0.004	99.999%	Pass

7.4.4 Instrumentation Plan

As part of the construction of the test pavement and control section, instrumentation will be placed in the pavement and subgrade to allow data collection to assist in performance evaluation of the RBMAC test pavement. The proposed instrumentation plan focuses on monitoring strains in the pavement section and stresses in the underlying soil. A detailed sampling plan, including the location and number of sensors to be installed, will be developed once a budget is established and funding for the test pavement is obtained.

Stresses experienced by subgrade soils underlying the test pavement can be obtained using soil pressure cells. These pressure cells will be installed on the prepared subgrade soil prior to placement of the stone base layer. These sensors typically consist of a liquid-filled cell and a pressure transducer. Semi-conductor transducers are utilized in these cells in order to allow them to measure dynamic pressures, which cannot be as accurately measured using traditional vibrating wire strain gage technology. Thermistors included in many models of these sensors can provide temperature readings at the subgrade/base interface.

In order to assess the strains experienced by the pavement, rebar strainmeters will be installed prior to construction of the pavement. The test pavement is designed to be unreinforced (JPCP), but lengths of rebar could be placed at strategic locations and depths in order to utilized rebar strainmeters to obtain strain data. These rebar strainmeters, or “sister bars” consist of a length of reinforcing steel (rebar) on which a vibrating wire strain gage has been mounted. Good contact between the rebar/strainmeter and the concrete ensures that the strains measured by the strainmeter are the same as the strains in the surrounding concrete. Thermistors are also included in many models of these transducers, which allow temperature readings of the surrounding concrete to be recorded.

Dial gauges may also be utilized to measure curling deflections (Yu et al. 1998) as well as joint deflections. Additional temperature sensors can be installed at different depths (such as top, middle, bottom, and ¼-depth points) in order to collect data to produce temperature gradients. Temperature moisture sensors have also been successfully utilized in other similar projects (Weyer and Ogunro 2011). Temperature and moisture data will be useful in assessing pavement responses to dimensional changes in the concrete pavement due to temperature and moisture gradients.

Data from the embedded soil pressure cells, rebar strainmeters, and other sensors will be collected and stored using a high-speed data acquisition unit. High-speed data acquisition units, suitable for recording the response of pavement layers to dynamic traffic loads, need to be capable of obtaining a large number of samples over a short time period, typically greater than 100,000 samples per second (Weyer and Ogunro 2011).

7.4.5 Materials Sampling and Testing Plan

Evaluation of the proposed RBMAC test pavement will also include tests of the pavement materials (soil subgrade, aggregate base, and concrete) prior to and after construction of the pavement. Soils test reports for the subject site are not presently available. Therefore, as part of the fieldwork prior to construction of the test pavement, testing of the subgrade soils will be performed to verify that the parameters used in the M-EPDG designs of the test pavement reasonably reflect actual site conditions. Due to the importance of the resilient modulus as an input, testing following AASHTO T307, “Standard Method of Test for Determining the Resilient Modulus of Soils and Aggregate Materials” will be performed, if possible.

If undisturbed samples (for resilient modulus or other testing) cannot be obtained, alternate field tests will be performed in order to assess the in-place properties of the subgrade soil. A series of test borings will be performed along the proposed roadbed to depths of at least five ft below the planned elevation of the roadbed. Testing may include using a dynamic cone penetrometer (DCP) in accordance with ASTM D6951, “Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications” or in-place stiffness testing to determine the California Bearing Ratio (CBR) in accordance with AASHTO T193, “Standard Method of Test for the California Bearing Ratio.” This information can be used to provide a reasonable estimate of the soil’s resilient modulus of the soil for use in M-EPDG analyses.

For the proposed M-EPDG pavement design, soils information was obtained from the USDA Natural Resources Conservation Service Web Soil Survey. To verify soil properties used in the M-EPDG design, bulk samples of subgrade material should be removed for laboratory testing and analysis prior to beginning construction. Tests that should be performed are outlined in Table 7-18.

Table 7-18: Subgrade soil testing to be performed prior to construction of the proposed test pavement.

Property	**ASTM or AASHTO Standards**
Classification tests	ASTM D2487, “Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)” or AASHTO M145, “Standard Specification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes”
Atterberg limits	ASTM D4318, “Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils”
Moisture content	ASTM D 2216, “Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass” or AASHTO T265, “Standard Method of Test for Laboratory Determination of Moisture Content of Soils”

As part of this work, the supply of RBMA material created from the Idlewild Elementary School demolition site was exhausted. A new source of RBMA will need to be identified for use in the test pavement. Prior to construction of the test pavement, testing will need to be performed to confirm that the properties of the RBMA obtained from the new source will allow it to be used in RBMAC mixtures that are acceptably similar to those used in the M-EPDG design of the test pavement. Trial mixtures of the concrete utilizing the new RBMA will be batched and tested in order to confirm that the mechanical properties of the RBMAC used in the M-EPDG design can be obtained with the new RBMA. Tests that should be performed on the RBMA should include those outlined in Chapter 3, “Testing Program for Characterization of Recycled Materials,” and tests that should be performed on the RBMAC include those outlined in Chapter 6, “Testing Program for Recycled Brick Masonry Aggregate Concrete.” If the properties of the RBMA and the RBMAC are not similar to those already used in the M-EPDG designs, new designs should be performed using the new properties in order to revise the test pavement section thicknesses, if necessary.

During construction of the test pavement, testing should be performed in order to assess the fresh and hardened properties of the RBMAC and the conventional concrete used in the control section. These tests should include those listed in Tables 7-19 and 7-20. Additionally, testing to evaluate the hardened concrete characteristics typically linked to durability performance should be performed. These tests are listed in Table 7-21.

Table 7-19: Fresh property tests to be performed during placement of RBMAC and conventional concrete.

Property	ASTM or AASHTO Standards
Slump	ASTM C43, "Standard Test Method for Slump of Hydraulic-Cement Concrete."
Air content, unit weight, and yield	ASTM C138, "Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete."

Table 7-20: Hardened property tests to be performed on specimens cast from RBMAC and conventional concrete.

Property	ASTM or AASHTO Standards
Compressive strength	ASTM C39, "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens."
Modulus of elasticity and Poisson's ratio	ASTM C138, "Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression."
Splitting tensile strength	ASTM C496, "Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens."
Flexural strength (modulus of rupture)	ASTM C78, "Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)."
Drying shrinkage	ASTM C531, "Standard Test Method for Linear Shrinkage and Coefficient of Thermal Expansion of Chemical-Resistant Mortars, Grouts, Monolithic Surfacings, and Polymer Concretes."
Thermal expansion	AASHTO TP-60, "Coefficient of Thermal Expansion of Hydraulic Cement Concrete."
Thermal conductivity	ASTM E 1952, "Standard Test Method for Thermal Conductivity and Thermal Diffusivity by Modulated Temperature Differential Scanning Calorimetry."
Heat capacity	ASTM D 2766, "Standard Test Method for Heat capacity of Liquids and Solids."

Table 7-21: Durability tests to be performed on specimens cast from RBMAC and conventional concrete.

Property	**ASTM or AASHTO Standards**
Permeability and water absorption	Figg method outlined in ACI 228.2R-98, "Nondestructive Test Methods for Evaluation of Concrete in Structures."
Freeze-thaw durability	ASTM C666, "Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing."
Chloride resistance	ASTM C1202, "Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration."
Surface resistivity	AASHTO TP95-2011, "Standard Method of Test for Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration."

To assess the performance of the test pavement over time, core samples will be periodically removed from the test pavement and petrographic analysis and laboratory testing will be performed. It is anticipated that sampling and testing will be performed when the pavement has reached the age of six months, one year, two years, three years, five years, and subsequently thereafter for an undetermined amount of time. Several cores will be obtained immediately after construction of the pavement in order to prepare reference specimens for petrographic comparison. Cores will be removed from areas of the test pavement that do not interfere with sensors collecting other data. Petrographic analysis will be performed in general accordance with ASTM C856, "Petrographic Examination of Hardened Concrete." Microscopic observations will include:

- Observation of the general characteristics of the concrete, including the paste, aggregates, and air void system.
- Observation of microcracks present within the paste and/or aggregates.
- Observation of secondary deposits evident within voids and/or microcracks, or around aggregate perimeters.

Additionally, the air contents of samples will be quantitatively determined using the procedure outlined in ASTM C457, “Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete.” This information will be useful in assessing the freeze-thaw performance of the RBMAC.

7.5 Summary and Concluding Remarks

RBMAC is currently not used in construction applications in the United States, and tests to characterize this material have only been performed in studies abroad. In this study, data on RBMA and RBMAC produced using local RBMA was obtained. Testing of RBMAC mechanical properties, as well as several durability performance characteristics, indicates that RBMAC can likely provide acceptable performance in pavement applications. Currently, the 2012 NCDOT Standard Specifications do not allow RBMAC to be used in pavement concrete, as the use of recycled aggregates is limited to crushed concrete. Recycled aggregates are also limited to Class B (lower grade) uses. A review of currently applicable NCDOT specifications for aggregates used in concrete, as well as specifications for concrete used in pavement applications, indicates that RBMAC mixtures prepared as part of this work could qualify for use in pavement applications based upon performance in laboratory tests and other criteria based on mixture proportions.

Many transportation agencies, including NCDOT, are considering implementation of the M-EPDG method of pavement design. M-EPDG allows the designer to input information for more material properties and performance characteristics than previous pavement design methodologies, and distress prediction models are very advanced. A RBMAC pavement and a conventional PCC pavement (containing locally available aggregates) were designed for both arterial and interstate traffic loading, and similar minimum pavement thicknesses were obtained. Due to the brick content of RBMAC, the CTE, heat capacity, and thermal conductivity can be lower than that of conventional PCC. Using these lower values for the thermal property inputs, M-EPDG predicts a design thickness of the RBMAC pavement that is thinner than the conventional concrete pavement.

Additional, possibly more accurate, testing of thermal properties is suggested in order to confirm the thermal input properties used in M-EPDG designs for both the RBMAC and the conventional concrete. Specifically, CTE testing of RBMAC in accordance with AASHTO T336 should be performed, with the test results compared to those using the appropriate calibration specimen.

As discussed previously in Section 7.3.3, Comparison of M-EPDG Pavement Designs Using Recycled Brick Masonry Aggregate Concrete and Natural Aggregate Concrete, values for unit weight, heat capacity, and thermal conductivity of RBMAC are outside the suggested range for M-EPDG. It is possible that these values affect the validity of the distress model predictions. If RBMAC is to be used in M-EPDG design, additional study on the influence of these parameters on the M-EPDG models should be performed.

Testing and evaluation of the test pavement will provide valuable insight into the suitability of RBMAC for concrete pavement applications. In addition to simply being a "new," untried aggregate for concrete pavements, RBMA has a relatively higher LA Abrasion value than conventional aggregates. In-place assessment of the performance of a test pavement section will allow entities such as NCDOT to determine whether this waste material could be used as a cost-effective component of concrete pavements that provide satisfactory field performance. Even if a slightly thicker pavement section is required, the savings realized may warrant the use of RBMAC for economic reasons.

The transportation industry has shown increasing interest in identifying uses for marginal aggregates (both virgin aggregates and other materials), which do not meet the standards and specifications for a particular application. Marginal aggregates have successfully been used in the lower layer of two-lift PCC pavements (Hooker 2011) where abrasion resistance is not critical, and in concrete used on lower traffic areas such as roadway shoulders (Haque and Ward 1981). Similarly, NCDOT could also use RBMAC in the lower-lift of two-lift PCC pavements, in roadway shoulders, or in Class B concrete.

Although it does not currently meet NCDOT standards, RBMAC could provide satisfactory service in private roadways and other privately owned pavements, including both drive lanes and parking lots. Due to the increase in haul costs over the past years, RBMA may be a more economical choice of aggregate for roadways or other paved areas in the vicinity of an existing brick masonry structure being demolished. The demolished brick masonry could be crushed on-site and reused in concrete as aggregate if batching operations are reasonably close in proximity. RBMAC pavement could also prove to be of interest to those involved in green building, as it could be counted in a green building rating system as reuse of a recycled material.

CHAPTER 8: SUMMARY AND CONCLUSIONS

8.1 Findings and Conclusions

Use of recycled aggregates in portland cement concrete can offer benefits associated with both economy and sustainability. Significant research has been performed on use of RCA in concrete elements, but use of RBMA has not been studied in the United States. In this study, RBMA was shown to be a viable material for use in structural and pavement grade concrete mixtures, possessing mechanical properties similar to those of PCC with conventional aggregates. Data collected on RBMA and RBMAC as part of this work can be used by designers interested in using this material in sustainable construction. Information on the properties of RBMA and RBMAC produced in the United States did not exist prior to this study.

Many concerns related to use of any recycled aggregate, including RBMA, include those related to the potential for contaminants to be introduced into the aggregate. At this case study site, it was found that top-down demolition techniques paired with on-site source separation of demolition debris can facilitate the generation of RBMA that is relatively "clean" and free of debris. The potential for variation in quality of the source brick and masonry mortar included in RBMA exists, and prior to use of this material, testing should be performed on whole brick as well as samples of crushed RBMA.

Testing to characterize the RBMA indicated that values for properties such as absorption, specific gravity, bulk density, and abrasion resistance differ from those typical of RCA and locally available natural aggregate. RBMA produced from the subject site exhibits an absorption value (12.2%) approximately twice that of a locally manufactured lightweight aggregate. The loose bulk density of the AASHTO M43 #78 RBMA from the subject site is 60.9 pcf (976 kg/m^3). Compared to other aggregates of the same gradation, this loose bulk density is approximately 10 pcf (160 kg/m^3) greater than that of a locally manufactured lightweight aggregate, yet 20 pcf (320 kg/m^3) and 30 pcf (481 kg/m^3) less than that of RCA from the subject site and locally quarried granite aggregate, respectively. The RBMA produced from the case study site does not meet ASTM C330, "Standard Specification for Lightweight Aggregates for Structural Concrete," in which the maximum dry loose bulk density of lightweight aggregate is 55 pcf (881 kg/m^3).

Ultimately, for RBMA to be considered as a viable aggregate source, guidelines for tests to confirm acceptable material properties and performance characteristics will need to be developed. Test methods performed in this study, as detailed in Chapter 3, Testing Program for Characterization of Recycled Materials, could be included in a framework of selection guidelines used by agencies to assess the suitability of RBMA produced from different sources. Other tests should be those required by the agency for acceptance of other aggregates, with performance criteria modified as appropriate.

RBMAC mixtures utilizing RBMA as a 100% replacement for natural coarse aggregate were developed. Due to the high absorption of the RBMA and its relatively low bulk density, RBMAC mixture proportions were developed using ACI 211.2, "Standard Practice for Selecting Proportions for Structural Lightweight Concrete." This approach provided mixture proportions that produced RBMAC exhibiting suitable performance in both the fresh and hardened state.

Workability issues arising due to the high absorption of RBMA can be addressed by using a commercially available water-reducing admixture. Air contents typically specified for concrete with outdoor exposure were obtained using a commercially available air-entraining admixture. Compressive strengths of trial batches were higher than anticipated using the ACI 211.2 mixture proportioning procedure, and it was found that RBMAC exhibiting compressive strengths that met the target strengths could be developed using cement contents typical of conventional pavement and structural grade concrete mixtures.

Mixtures meeting ACI 318 strength overdesign requirements for commercially available 4,000 psi (27.6 MPa) and 5,000 psi (34.5 MPa) mixtures were developed and tested for mechanical properties and durability performance. RBMAC mixtures with a water-reducing admixture (w/c = 0.32) had equilibrium densities ranging from 125.5 pcf to 128.2 pcf (2054 kg/m^3). Although higher than the equilibrium density requirements for lightweight concrete in ACI 213, this is significantly lower than the unit weight of conventional normalweight concrete, indicating that RBMAC could offer advantages of reduction of deadload in structural applications, as well as cost-savings related to transport. RBMAC that did not utilize a water-reducing admixture (w/c = 0.43) had an equilibrium density of 111.8 pcf (1791 kg/m^3), which does meet ACI 213 requirements for lightweight concrete. The modulus of elasticity of RBMAC is within the range of expected values for normalweight and lightweight concrete containing aggregate from conventional sources. Flexural strength testing at 7 days indicate that RBMAC mixtures can exhibit moduli of rupture suitable for use in pavement applications.

Several of the RBMAC mixtures produced for this study that had a relatively low w/c ratio (0.32) exhibited acceptable resistance to chloride ion penetration. Unsatisfactory resistance to chloride ion penetration was exhibited by the RBMAC mixture that had a relatively higher w/c ratio (0.43). Air and water permeability test results indicated that the RBMAC mixtures are more permeable than similar concrete mixtures incorporating normalweight or conventional lightweight aggregates used in North Carolina. This testing was performed using the Figg Method. Due to the limitations of this method for use with highly porous aggregates, other methods of testing such as the capillary suction method (ASTM C1585) or ponding method (Bentz et al. 2002) may be more appropriate for routine testing.

Abrasion resistance is a key characteristic of concrete used in transportation applications. The abrasion resistance of the RBMA used in this study met the requirements of 2012 NCDOT Standard Specifications for some concrete uses, but not for concrete with a 28-day design strength greater than 6,000 psi (41.4 MPa). Additional laboratory abrasion testing of the RBMAC in accordance with ASTM C944, “Standard Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating-Cutter Method” indicated that RBMAC can exhibit adequate abrasion resistance according to FHWA HPC standards (Goodspeed et al. 2011).

RBMA does not meet the 2012 NCDOT Standard Specifications for recycled aggregates (which limit recycled aggregates to crushed concrete only), and NCDOT currently only allows use of RCA in low-grade concrete applications. However, testing of RBMA and RBMAC performed as part of this work indicates that it may provide acceptable performance in pavement applications based on the 2012 NCDOT Standard Specifications for aggregates used in concrete and for concrete used in pavement applications.

M-EPDG, which is currently available commercially as DARWin-ME, is the state-of-the-art software program currently available for pavement design. M-EPDG offers pavement designers the ability to account for the properties of RBMAC that differ from conventional PCC (such as unit weight, Poisson's ratio, and thermal characteristics) in pavement design. A comparison of M-EPDG pavement designs using RBMAC and PCC pavements incorporating locally available natural aggregate was performed as part of this work. For the traffic, climate, and materials characteristics input in the comparison pavement designs, the design thicknesses of RBMAC pavements are similar to those of conventional PCC with granite aggregate. Laboratory testing indicates that RBMAC can exhibit favorable thermal characteristics, including a CTE lower than that of conventional concrete containing some natural aggregates. Using the most favorable thermal characteristics obtained during testing, M-EPDG predicted adequate performance of RBMAC pavements with section thicknesses less than those using conventional concrete. RBMAC could possibly be used in the lower lift of two-lift pavements, if an agency does not have an adequate comfort level with it serving as the wearing surface of a pavement.

RBMAC is currently not used in any type of construction in the United States. In addition to use in public transportation applications, it is envisioned that with the kinds of performance data provided here, use of RBMAC likely will increase as sustainable building practices become the norm. Rating systems such as LEED offer points for reuse of building materials (particularly on-site) and use of recycled materials. If renovations at an existing facility call for the demolition of existing brick masonry constructions, the rubble could be included as RBMA in new concrete pavement, sidewalks, or curb and gutter. The material could be transported to a crushing and grading facility if one is nearby, or, if a suitable processing facility is not nearby, mobile crushers could be brought on-site to process the material minimizing the transportation costs of the heavier aggregate. The material could be transported to a local ready-mix concrete plant, or for smaller applications, it could be kept on site and batched there in small ready-mix concrete trucks capable of concrete batching from jobsite stockpiles. As reuse applications of this nature become more widely used, it is likely that specialized equipment will be developed that will bring costs down further.

Other potential uses for RBMAC could include those in the precast concrete industry, particularly in architectural precast concrete applications. In addition to providing acceptable strength and economy, the color of RBMA could be an attractive component of architectural precast concrete panels or other façade components.

8.2 Recommendations for Future Work

Work performed as part of this study provides designers a first look at the material properties of RBMA and RBMAC produced in the United States. Testing performed to date indicates that RBMA can be used as a 100% replacement for conventional coarse aggregate in concrete that exhibits acceptable mechanical properties for use in structural and pavement elements, including satisfactory performance in some durability tests.

For RBMA and RBMAC to be viewed as viable construction materials, additional studies will need to be performed. The thermal properties of RBMAC obtained in this study resulted in M-EPDG predicting thinner RBMAC pavement sections than conventional PCC pavement for some traffic and climatic conditions. To confirm these findings, it is recommended that CTE testing of RBMAC be performed in accordance with AASHTO T336, with the test results compared to those using the appropriate calibration specimen. Future study of RBMAC could include batching of companion mixtures of conventional PCC to validate the thermal property test procedures and results.

Use of RBMA as a replacement for fine aggregate and as a partial replacement of coarse aggregate was not included in the scope of this study, and should be explored in future work. Studies to evaluate the effects of optimizing RBMA gradation on RBMAC could also be performed. Incorporation of supplementary cementitious materials (such as fly ash or slag cement) in RBMAC to replace portland cement would provide RBMAC with a higher recycled materials content, and hence it could be viewed as an even more sustainable concrete.

Use of RBMA produced from brick masonry with a lower mortar content would likely produce better-performing RBMAC. A greater proportion of brick in the RBMA would enhance the favorable thermal characteristics exhibited by RBMAC and potentially increase its desirability for use in pavement applications. Conversely, a higher proportion of mortar used in some masonry construction could cause RBMA produced from this material to have properties closer to RCA (i.e. higher unit weight, lower absorption, etc.). Future work could include a study on the effects of mortar content of RBMA on RBMAC performance, as well as a study on the influence of contaminant materials on the properties of RBMAC. Recommendations regarding procedures that would allow an engineer to rapidly assess the suitability of in-place masonry construction for use as RBMA (or of an RBMA stockpile for use in PCC) do not currently exist, and should be developed.

The high absorption of RBMA indicates that internal curing benefits may be provided, with the saturated highly-porous RBMA particles providing additional water to facilitate hydration after conventional curing provisions have been removed or halted. Testing of the shrinkage and creep characteristics of RBMAC would help to evaluate the possible internal curing benefits of RBMA. Additional durability testing should also be performed on RBMAC mixtures, including tests related to freeze-thaw durability, sulfate attack, and alkali-aggregate reactivity.

Batching and testing of mortar mixtures using recycled brick masonry fine aggregate (sand) produced from the case study site was not performed as part of this work. Previous studies by others have shown that brick material may exhibit pozzolanic activity, and this should be further investigated. Microstructural characterization of RBMAC, including an evaluation of the paste-aggregate bond (interfacial transition zone), would help to provide insight into possible pozzolanic benefits that could be provided by RBMA.

Although RBMAC has been untested in field applications, results of laboratory studies performed to date indicate that this material shows promise for use in pavement and structural applications. As part of this study, a test pavement incorporating RBMAC has been designed, and construction is planned for the near future. The test pavement will help provide insight into the long-term performance of this material in a service environment. Future testing of RBMAC in both laboratory and field settings will allow stakeholders to gain a comfort level with its properties, identify specific potential uses, and establish guidelines that will assist in ensuring acceptable service life performance.

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APPENDIX A: SUPPLEMENTAL INFORMATION FOR CHAPTER 3

Figure A-1: Whole brick specimens.

Figure A-2: Whole clay tile specimens.

Figure A-3: Test specimens used for thermal conductivity testing.

Table A-1: Absorption test results for bricks.

Specimen ID	Oven dried wt (g)	5-hr soak wet wt (g)	24-hr soak wet wt (g)	5-hr boil wt (g)	Absorption (5-hr soak) (%)	Absorption (24-hr soak) (%)	Absorption (5-hr boil) (%)	Saturation Coefficient
4	2469.2	2716.4	2726	2793.6	10.0	10.4	13.1	0.79
13	2696	2938.5	2945.3	3017.3	9.0	9.2	11.9	0.78
21	2627.9	2826.9	2831.5	2881.2	7.6	7.7	9.6	0.80
24	2549.4	2757.5	2764.1	2822.6	8.2	8.4	10.7	0.79
25	2591.4	2763.4	2766.4	2816.9	6.6	6.8	8.7	0.78
Average					8.3	8.5	10.8	0.8

Table A-2: Absorption test results for clay tiles.

Specimen ID	Oven dried wt (g)	5-hr soak wet wt (g)	24-hr soak wet wt (g)	5-hr boil wt (g)	Absorption (5-hr soak) (g)	Absorption (24-hr soak) (%)	Absorption (5-hr boil) (%)	Saturation Coefficient
3	4931	5096.7	5115	5126.6	3.4	3.7	4.0	0.94
4	3568.6	3708.8	3715.2	3744.8	3.9	4.1	4.9	0.83
6	4732.9	4929.1	4934.7	4976.7	4.1	4.3	5.2	0.83
Average					3.8	4.0	4.7	0.9

Table A-3: Suction test results for bricks.

Specimen ID	Initial wt (g)	Final wt (g)	Weight gain, W (g)	length (cm)	Width (cm)	Area of cores (cm^2)	Net Area (cm^2)	Wt gain corrected to 30 in^2 (g per 30 in^2)
3	2650	2667.6	17.6	26.67	8.57	38.4	190.2	2.8
5	2471.5	2494.3	22.8	26.67	8.57	38.4	190.2	3.6
6	2567.1	2596.3	29.2	26.67	8.57	38.4	190.2	4.6
16	2462.2	2486.5	24.3	26.67	8.57	38.4	190.2	3.8
18	2476.4	2505.4	29	26.67	8.57	38.4	190.2	4.6
27	2551.6	2579	27.4	26.67	8.57	38.4	190.2	4.3
Average								4.0

Table A-4: Suction test results for clay tiles.

Specimen ID	Initial wt (g)	Final wt (g)	Weight gain, W (g)	length (cm)	Width (cm)	Area of cores (cm^2)	Net Area (cm^2)	Wt gain corrected to 30 in^2 (g per 30 in^2)
1	5117.2	5122	4.8	29.845	9.2075	122.78	152.02	0.9
3	4931.6	4935.8	4.2	29.845	9.2075	122.78	152.02	0.8
Average								0.9

Table A-5: Compressive strength test results for bricks.

Specimen ID	Length (in)	Width (in)	Area of cores (in^2)	Net Area (in^2)	Load (lb)	Compressive Strength (psi)
1A	5.75	3.375	2.975	16.43	110,115	6,702
1B	5.75	3.375	2.975	16.43	156,245	9,509
2A	5.75	3.375	2.975	16.43	120,110	7,310
2B	5.75	3.375	2.975	16.43	167,880	10,217
19A	5.75	3.375	2.975	16.43	169,030	10,287
19B	5.75	3.375	2.975	16.43	219,025	13,330
22A	5.75	3.375	2.975	16.43	181,725	11,060
23A	5.75	3.375	2.975	16.43	131,265	7,989
23B	5.75	3.375	2.975	16.43	208,450	12,686
31A	5.75	3.375	2.975	16.43	148,680	9,049
31B	5.75	3.375	2.975	16.43	150,055	9,132
Average						9,752

Table A-6: Compressive strength test results for clay tiles.

Specimen ID	Length (in)	Width (in)	Area of cores (in^2)	Net Area (in^2)	Load (lb)	Compressive Strength (psi)
2	5.875	3.625	9.75	11.55	181,710	15,737
8A	5.875	3.625	9.281	12.02	104,345	8,684
8B	5.875	3.625	9.75	11.55	126,955	10,995
Average						11,805

Table A-7: Modulus of rupture test results for bricks.

Specimen ID	Length (in)	Load (lb)	Width (in)	Height (in)	Avg. distance from midspan to plane of failure, X (in)	Modulus of Rupture (psi)
9	10.5	1545	2.1875	2.25	0	2,197
10	10.5	1125	2.125	2.25	0.0625	1,627
11	10.5	1265	2.1875	2.25	0.125	1,756
15	10.5	1450	2.25	2.1875	0	2,121
20	10.5	1740	2.25	2.25	0.125	2,349
Average						2,010

Table A-8: Modulus of rupture test results for tiles.

Specimen ID	Length (in)	Load (lb)	Width (in)	Height (in)	Avg. distance from midspan to plane of failure, X (in)	Modulus of Rupture (psi)
1	10.5	2245	1.25	5	0.125	1,105
7	10.5	2155	1.1875	5.125	0.25	1,036
Average						1,070

Table A-9: Coefficient of thermal expansion test results for brick.

	Initial Temp. (°F)	Length at Initial Temp. (in)	High Temp. (°F)	Length at High Temp. (in.)	Low Temp. (°F)	Length at Low Temp. (in) *	CTE (in/in/°F) **
Test 1	70.2	4.0074	111.0	4.0078	55.2	4.0078	2.45×10^{-6}
Test 2	70.5	4.0075	109.2	4.0102	57.4	4.0089	1.74×10^{-5}
Test 3	72.7	4.0094	112.5	4.0110	56.6	4.0089	1.00×10^{-5}
Average							N/A

* crack in brick appears to have caused these values to be questionable, Test 2 and Test 3 are deemed invalid.

** CTE computed using only initial and high temps only.

Test Report
Report Generated on: 06-Jan-2012 9:51:18

Test TCI-114
Instrument: TH89-05-00129
Test Method: Polymers
Project: Cavalline
Material Idlewild brick
Material Lot:

Software Version: 2.3.3954
Test started on: 06-Jan-2012
Performed by: Administrator
User ID: ADMIN

#	Repea	Sensor ID	Start Time	Effusivity W*√(s)/(m^2)*K	Conductivity (W/mK)	Ambient T	DeltaT (°C)	V0
1	1	T132	9:40:08	1,243	0.910	21.60	0.62	2,355.90
2	1	T132	9:41:14	1,218	0.880	20.92	0.63	2,357.32
3	1	T132	9:42:20	1,226	0.890	20.82	0.63	2,356.98
4	1	T132	9:43:25	1,229	0.900	21.80	0.62	2,354.43
5	1	T132	9:44:31	1,232	0.900	21.76	0.62	2,355.29
6	1	T132	9:45:37	1,224	0.890	20.93	0.63	2,357.56
7	1	T132	9:46:42	1,222	0.890	20.93	0.64	2,356.95
8	1	T132	9:47:48	1,231	0.900	21.87	0.62	2,354.60
9	1	T132	9:48:54	1,229	0.900	22.00	0.62	2,354.08
10	1	T132	9:49:59	1,231	0.900	21.92	0.63	2,354.95

Figure A-4: Thermal conductivity test results for brick.

Test Report
Report Generated on: 06-Jan-2012 10:06:25

Test TCI-115
Instrument: TH89-05-00129
Test Method: Ceramics
Project: Cavalline
Material Idlewild tile
Material Lot:

Software Version: 2.3.3954
Test started on: 06-Jan-2012
Performed by: Administrator
User ID: ADMIN

#	Repea t	Sensor ID	Start Time	Effusivity W*√(s)/(m^2)*K	Conductivity (W/mK)	Ambient T (°C)	DeltaT (°C)	V0 (mV)
1	1	T132	9:55:31	1,693	1.520	21.53	0.54	2,357.30
2	1	T132	9:56:37	1,639	1.420	22.06	0.55	2,354.72
3	1	T132	9:57:43	1,647	1.440	22.17	0.55	2,355.02
4	1	T132	9:58:48	1,649	1.440	21.09	0.54	2,357.81
5	1	T132	9:59:54	1,661	1.460	21.53	0.55	2,356.00
6	1	T132	10:01:00	1,655	1.450	22.16	0.55	2,354.39
7	1	T132	10:02:05	1,657	1.460	22.13	0.55	2,354.54
8	1	T132	10:03:11	1,672	1.480	22.13	0.54	2,354.61
9	1	T132	10:04:17	1,675	1.490	22.22	0.55	2,354.76
10	1	T132	10:05:22	1,659	1.460	21.71	0.55	2,356.29

Figure A-5: Thermal conductivity test results for clay tile.

C-THERM TCi™ Thermal Conductivity Analyzer

Test Report

Report Generated on: 06-Jan-2012 16:23:25

Test TCI-117
Instrument: TH89-05-00129
Test Method: Polymers

Project: Cavalline

Material Idlewild mortar
Material Lot:

Software Version: 2.3.3954
Test started on: 06-Jan-2012
Performed by: Administrator
User ID: ADMIN

#	Repeat	Sensor ID	Start Time	Effusivity W*√(s)/(m^2)*K	Conductivity (W/mK)	Ambient T (°C)	DeltaT (°C)	V0 (mV)
1	1	T132	10:15:54	483	0.180	22.12	0.91	2,356.11
2	1	T132	10:17:00	479	0.170	21.15	0.91	2,358.09
3	1	T132	10:18:06	477	0.170	22.22	0.90	2,355.20
4	1	T132	10:19:11	479	0.170	22.14	0.90	2,355.48
5	1	T132	10:20:17	479	0.170	22.38	0.90	2,354.84
6	1	T132	10:21:23	479	0.170	22.27	0.91	2,355.74
7	1	T132	10:22:28	479	0.170	21.33	0.91	2,357.70
8	1	T132	10:23:34	476	0.170	22.34	0.91	2,355.05
9	1	T132	10:24:40	477	0.170	22.27	0.90	2,355.23
10	1	T132	10:25:45	474	0.170	22.16	0.90	2,355.60

Figure A-6: Thermal conductivity test results for mortar.

Figure A-7: Samples of crushed brick, mortar, and clay tile, for heat capacity testing.

TLC Brick Test 1 - 11/30/2011
Creation Date : 11/30/2011 1:36:42 PM
User : admin

TG :
Initial Mass : 83.34 mg
Molar Mass : N/A

TG |-b [TLC RBMAC Blank 1 - 11/30/2011 / 2 Heating Zone / TG] :
Initial Mass : 83.34 mg
Molar Mass : N/A

HeatFlow :
Initial Mass : 83.34 mg
Molar Mass : N/A

HeatFlow |-b [TLC RBMAC Blank 1 - 11/30/2011 / 2 Heating Zone / HeatFlow] :
Initial Mass : 83.34 mg
Molar Mass : N/A

Index	Time (s)	Furnace Temp. (°C)	Sample Temp. (°C)	TG (mg)	TG (minus blank crucible) (mg)	HeatFlow (mW)	HeatFlow (minus blank crucible) (mW)	Heat Capacity (J/(mg•°C))	Heat Capacity (J/(g•°C))	Heat Capacity (BTU/(lb•°F)
1	0	24.996487	24.88892	83.34187	83.32444	0.493721	0.01840	0.00022	0.22083	0.92457
2	0.1	24.996508	24.88891	83.34177	83.324356	0.493727	0.01842	0.00022	0.22101	0.92533
3	0.2	24.996529	24.88891	83.34168	83.324261	0.493732	0.01844	0.00022	0.22120	0.92613
4	0.3	24.99655	24.88891	83.34158	83.324169	0.493742	0.01846	0.00022	0.22145	0.92718
5	0.4	24.996571	24.8889	83.34148	83.32407	0.493753	0.01849	0.00022	0.22185	0.92884
6	0.5	24.996594	24.8889	83.34139	83.323978	0.49376	0.01853	0.00022	0.22231	0.93075
7	0.6	24.996615	24.88889	83.3413	83.323879	0.49376	0.01856	0.00022	0.22265	0.93221
8	0.7	24.996635	24.88888	83.34123	83.323791	0.49376	0.01858	0.00022	0.22297	0.93351
9	0.8	24.996643	24.88886	83.34114	83.323689	0.493772	0.01862	0.00022	0.22345	0.93552
10	0.9	24.996651	24.88885	83.34107	83.323589	0.493779	0.01866	0.00022	0.22384	0.93718

Figure A-8: Typical output spreadsheet of TGA, with associated heat capacity calculations.

Table A-10: Sieve analyses of RBMA.

Sample 1				
Sieve No.	Weight retained (g)	% retained	Cumulative % retained	% passing
3/4	0.0	0.0	0.0	100.0
1/2	3.2	0.1	0.1	99.9
3/8	454.4	13.2	13.3	86.7
No. 4	2262.7	65.6	78.9	21.1
No. 8	681.4	19.8	98.6	1.4
No. 16	27.5	0.8	99.4	0.6
pan	19.4	0.6	100.0	0.0

Total Wt (g)	3448.6

Sample 2				
Sieve No.	Weight retained (g)	% retained	Cumulative % retained	% passing
3/4	0.0	0.0	0.0	100.0
1/2	4.2	0.1	0.1	99.9
3/8	419.4	12.4	12.5	87.5
No. 4	2237.4	66.1	78.7	21.3
No. 8	669.1	19.8	98.5	1.5
No. 16	28.1	0.8	99.3	0.7
pan	24.2	0.7	100.0	0.0

Total Wt (g)	3382.4

Sample 3				
Sieve No.	Weight retained (g)	% retained	Cumulative % retained	% passing
3/4	0.0	0.0	0.0	100.0
1/2	7.9	0.2	0.2	99.8
3/8	626.9	18.7	19.0	81.0
No. 4	2179.9	65.1	84.0	16.0
No. 8	474.8	14.2	98.2	1.8
No. 16	19.6	0.6	98.8	1.2
pan	40.0	1.2	100.0	0.0

Total Wt (g)	3349.1

Average

Sieve No.	% passing
3/4	100.0
1/2	99.8
3/8	85.1
No. 4	19.5
No. 8	1.6
No. 16	0.8
pan	0.0

Table A-11: Particle shape test results for RBMA.

							Flat Particles				Elongated Particles			
							Not flat		Flat		Not elongated		Elongated	
Material	Set ID	Total number of particles	Total Mass (g)	Type	Count	Mass (g)	count	mass (g)	count	mass (g)	count	mass (g)	count	mass (g)
Blend	1	100	78.7	brick	72	55.8	66	52.3	6	3.5	72	55.8	0	0.0
				mortar	28	22.9	28	22.9	0	0.0	28	22.9	0	0.0
				tile	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
Blend	2	100	47.7	brick	73	33.3	71	32.6	2	0.7	73	33.3	0	0.0
				mortar	23	12.0	23	12.0	0	0.0	23	12.0	0	0.0
				tile	4	2.4	4	2.4	0	0.0	4	2.4	0	0.0
Blend	3	100	42.5	brick	62	26.5	61	26.0	1	0.5	62	26.5	0	0.0
				mortar	36	13.9	36	13.9	0	0.0	36	13.9	0	0.0
				tile	2	2.1	2	2.1	0	0.0	2	2.1	0	0.0
Brick	4	100	37.3	brick	100	37.3	98	37.0	2	0.3	100	37.3	0	0.0
Brick	5	100	39.3	brick	100	39.3	95	38.6	5	0.7	100	39.3	0	0.0
Brick	6	100	40.3	brick	100	40.3	99	40.2	1	0.1	100	40.3	0	0.0
Mortar	7	100	43.0	mortar	100	43.0	100	43.0	0	0.0	100	43.0	0	0.0
Mortar	8	100	43.8	mortar	100	43.8	100	43.8	0	0.0	100	43.8	0	0.0
Mortar	9	100	45.0	mortar	100	45.0	100	45.0	0	0.0	100	45.0	0	0.0
Tile	10	100	52.2	tile	100	52.2	99	52.0	1	0.2	100	52.2	0	0.0

Table A-11: Particle shape test results for RBMA.

					Flat and Elongated Particles				% Flat and Elongated		% Flat and Elongated Averages	
					Not flat and elongated		Flat and elongated					
Material	Set ID	Total number of particles	Total Mass (g)	Type	count	mass (g)	count	mass (g)	by count (%)	by mass (%)	by count (%)	by mass (%)
Blend	1	100	78.7	brick	69	54.4	3	1.4	3.0	1.8	4.0	3.6
				mortar	28	22.9	0	0.0				
				tile	0	0.0	0	0.0				
Blend	2	100	47.7	brick	66	29.9	7	3.4	7.0	7.1		
				mortar	0	0.0	0	0.0				
				tile	0	0.0	0	0.0				
Blend	3	100	42.5	brick	60	25.7	2	0.8	2.0	1.9		
				mortar	36	13.9	0	0.0				
				tile	2	2.1	0	0.0				
Brick	4	100	37.3	brick	93	35.1	7	2.2	7.0	5.9	9.0	6.7
Brick	5	100	39.3	brick	89	37.4	11	2.9	11.0	7.4		
Brick	6	100	40.3	brick	91	36.6	9	2.7	9.0	6.7		
Mortar	7	100	43.0	mortar	99	42.6	1	0.4	1.0	0.9	0.7	0.5
Mortar	8	100	43.8	mortar	100	43.8	0	0.0	0.0	0.0		
Mortar	9	100	45.0	mortar	99	44.8	1	0.2	1.0	0.4		
Tile	10	100	52.2	tile	92	49.7	8	2.5	8.0	4.8	8.0	4.8

Table A-12: Density, specific gravity, and absorption test results for RBMA.

Sample	SSD wt (g)	Submerged wt (g)	Oven dried wt (g)	Specific Gravity (Oven Dried)	Specific Gravity at SSD	Apparent Specific Gravity	Density (Oven Dried) (lb/ft^3)	Density (SSD) (lb/ft^3)	Apparent Density (lb/ft^3)	Absorption (%)
1	3489.5	1899.8	3107.6	1.95	2.20	2.57	121.7	136.7	160.2	12.3
2	3350.6	1824.9	2982.5	1.95	2.20	2.58	121.7	136.8	160.4	12.3
3	3296.6	1791.5	2947.4	1.96	2.19	2.55	121.9	136.4	158.8	11.8
Average				1.96	2.19	2.57	121.8	136.6	159.8	12.2

Table A-13: Bulk density (unit weight) test results for RBMA, shoveling procedure and rodding procedure.

Shoveling Procedure (lightweight)

Sample	Wt of aggregate (g)	Wt of aggregate (lb)	Bulk density (pcf)
1	3195	7.04	61.5
2	3140	6.92	60.5
3	3150	6.95	60.7
Average			60.9

Rodding Procedure (normalweight)

Sample	Wt of aggregate (g)	Wt of aggregate (lb)	Bulk density (pcf)
1	3680	8.11	70.9
2	3625	7.99	69.8
3	3655	8.06	70.4
Average			70.4

Table A-14: Los Angeles abrasion resistance test results for RBMA.

		Sample 1	Sample 2
Initial sample wt (g)	retained on 1/4 in sieve	2500.0	2500.0
	retained on No. 4 in sieve	2500.0	2500.0
Total initial sample wt (g)		5000.0	5000.0
Final sample wt (g) (retained on No. 12 sieve)		2775.4	2907.8
% loss		44.4%	41.8%

APPENDIX B: SUPPLEMENTAL INFORMATION FOR CHAPTER 5

Table B-1: Compressive strength test results for trial RBMAC mixtures.

	RBMAC Mixture								
	BAC 1.0			BAC 2.0			BAC 2.1		
	Specimen	**Comp. Str. (psi)**	**Average Comp. Str. (psi)**	**Specimen**	**Comp. Str. (psi)**	**Average Comp. Str. (psi)**	**Specimen**	**Comp. Str. (psi)**	**Average Comp. Str. (psi)**
3-day	BAC1.0-3a	4090		BAC2.0-3a	2706		BAC2.1-3a	3004	
	BAC1.0-3b	3776	3895	BAC2.0-3b	2719	2667.67	BAC2.1-3b	3193	3097.33
	BAC1.0-3c	3818		BAC2.0-3c	2578		BAC2.1-3c	3095	
7-day	BAC1.0-7a	4680		BAC2.0-7a	3164.0		BAC2.1-7a	3865.0	
	BAC1.0-7b	4414	4692	BAC2.0-7b	3086.0	3146	BAC2.1-7b	3941.0	3903
	BAC1.0-7c	4982		BAC2.0-7c	3188.0				
28-day	BAC1.0-28a	6221		BAC2.0-28a	4287.0		BAC2.1-28a	4606.0	
	BAC1.0-28b	5821	6067	BAC2.0-28b	3873.0	4117	BAC2.1-28b	4795.0	4726
	BAC1.0-28c	6158		BAC2.0-28c	4191.0		BAC2.1-28c	4776.0	
90-day	BAC1.0-90a			BAC2.0-90a			BAC2.1-90a	4999.0	
	BAC1.0-90b			BAC2.0-90b			BAC2.1-90b	4991.0	4995
	BAC1.0-90c			BAC2.0-90c					

Table B-1 (con't): Compressive strength test results for trial RBMAC mixtures.

	RBMAC Mixture								
	BAC 2.2			BAC 2.3			BAC 3.0		
	Specimen	**Comp. Str. (psi)**	**Average Comp. Str. (psi)**	**Specimen**	**Comp. Str. (psi)**	**Average Comp. Str. (psi)**	**Specimen**	**Comp. Str. (psi)**	**Average Comp. Str. (psi)**
3-day	BAC2.2-3a	1850		BAC2.3-3a	5117		BAC3.0-3a	2174	
	BAC2.2-3b	1753	1804.00	BAC2.3-3b	5603	5337.33	BAC3.0-3b	2483	2378.00
	BAC2.2-3c	1809		BAC2.3-3c	5292		BAC3.0-3c	2477	
7-day	BAC2.2-7a	2091.0		BAC2.3-7a	6197.0		BAC3.0-7a	3108.0	
	BAC2.2-7b	2123.0	2107	BAC2.3-7b	5213.0	6197	BAC3.0-7b	3198.0	3128
					7b bad break		BAC3.0-7c	3077.0	
28-day	BAC2.2-28a	2349.0		BAC2.3-28a	6639.0		BAC3.0-28a	3699.0	
	BAC2.2-28b	2520.0	2480	BAC2.3-28b	7177.0	6962	BAC3.0-28b	3509.0	3642
	BAC2.2-28c	2571.0		BAC2.3-28c	7070.0		BAC3.0-28c	3717.0	
90-day	BAC2.2-90a	2746.0		BAC2.3-90a	7542.0		BAC3.0-90a	4280.0	
	BAC2.2-90b		2746	BAC2.3-90b	7605.0	7574	BAC3.0-90b	4304.0	4292

Table B-1 (con't): Compressive strength test results for trial RBMAC mixtures.

	RBMAC Mixture								
	BAC 4.0			BAC 4.1			BAC 4.2		
	Specimen	**Comp. Str. (psi)**	**Average Comp. Str. (psi)**	**Specimen**	**Comp. Str. (psi)**	**Average Comp. Str. (psi)**	**Specimen**	**Comp. Str. (psi)**	**Average Comp. Str. (psi)**
3-day	BAC4.0-3a	2231	2200	BAC4.1-3a	3444	3491	BAC4.2-3a	5814	5722
	BAC4.0-3b	2178		BAC4.1-3b	3505		BAC4.2-3b	5604	
	BAC4.0-3c	2191		BAC4.1-3c	3525		BAC4.2-3c	5749	
7-day	BAC4.0-7a	2386	2513	BAC4.1-7a	3925	3979	BAC4.2-7a	6222	6249
	BAC4.0-7b	2577		BAC4.1-7b	4032		BAC4.2-7b	6289	
	BAC4.0-7c	2575		BAC4.1-7c			BAC4.2-7c	6236	
28-day	BAC4.0-28a	3522	3515	BAC4.1-28a	5073	5167	BAC4.2-28a	7574	7858
	BAC4.0-28b	3639		BAC4.1-28b	5096		BAC4.2-28b	8266	
	BAC4.0-28c	3383		BAC4.1-28c	5333		BAC4.2-28c	7733	
90-day	BAC4.0-90a	3810	3831	BAC4.1-90a	5481	5575	BAC4.2-90a	7843	7879
	BAC4.0-90b	3855		BAC4.1-90b	5618		BAC4.2-90b	7883	
	BAC4.0-90c	3828		BAC4.1-90c	5625		BAC4.2-90c	7911	

Table B-1 (con't): Compressive strength test results for trial RBMAC mixtures.

	RBMAC Mixture								
	BAC 4.3			BAC 4.4			BAC 4.5		
	Specimen	**Comp. Str. (psi)**	**Average Comp. Str. (psi)**	**Specimen**	**Comp. Str. (psi)**	**Average Comp. Str. (psi)**	**Specimen**	**Comp. Str. (psi)**	**Average Comp. Str. (psi)**
3-day	BAC4.3-3a	4203	4084	BAC4.4-3a	2585	2642	BAC4.5-3a	2856	2872
	BAC4.3-3b	4049		BAC4.4-3b	2821		BAC4.5-3b	2770	
	BAC4.3-3c	3999		BAC4.4-3c	2519		BAC4.5-3c	2989	
7-day	BAC4.3-7a	4787	4712	BAC4.4-7a	3383	3297	BAC4.5-7a	3345	3339
	BAC4.3-7b	4550		BAC4.4-7b	3239		BAC4.5-7b	3293	
	BAC4.3-7c	4799		BAC4.4-7c	3270		BAC4.5-7c	3379	
28-day	BAC4.3-28a	5465	5304	BAC4.4-28a	3484	3484	BAC4.5-28a	4740	4518
	BAC4.3-28b	5143		BAC4.4-28b			BAC4.5-28b	4077	
	BAC4.3-28c			BAC4.4-28c			BAC4.5-28c	4738	
90-day	BAC4.3-90a	5385	5360	BAC4.4-90a	3897	3803	BAC4.5-90a	4991	4992
	BAC4.3-90b	5223		BAC4.4-90b	3888		BAC4.5-90b	4785	
	BAC4.3-90c	5472		BAC4.4-90c	3625		BAC4.5-90c	5200	

Table B-1 (con't): Compressive strength test results for trial RBMAC mixtures.

	RBMAC Mixture		
	BAC 4.6		
	Specimen	**Comp. Str. (psi)**	**Average Comp. Str. (psi)**
3-day	BAC4.6-3a	2561	2821
	BAC4.6-3b	2918	
	BAC4.6-3c	2984	
7-day	BAC4.6-7a	3015	3323
	BAC4.6-7b	3299	
	BAC4.6-7c	3655	
28-day	BAC4.6-28a	4488	4306
	BAC4.6-28b	4023	
	BAC4.6-28c	4406	
90-day	BAC4.6-90a	4951	4870
	BAC4.6-90b	4871	
	BAC4.6-90c	4788	

APPENDIX C: SUPPLEMENTAL INFORMATION FOR CHAPTER 6

Table C-1: Equilibrium density test results for RBMAC.

Specimen ID	Diam (in)	Length (in)	Vol. (cf)	Submerged wt (lb)	SSD wt (lb)	Equilibrium density wt (lb)	Equilibrium density (pcf)	Average equilibrium density (pcf)
BAC 5.0-a	6	12	0.196	11.139	23.624	22.410	111.8	111.8
BAC 5.0-b	6	12	0.196	11.150	23.675	22.475	111.7	
BAC 6.0-a	6	12	0.196	14.164	26.800	25.950	127.9	128.2
BAC 6.0-b	6	12	0.196	14.272	26.900	26.055	128.5	
BAC 6.1-a	4	8	0.058	4.214	7.919	7.635	128.4	127.3
BAC 6.1-b	6	12	0.196	13.743	26.310	25.480	126.3	
BAC 6.2-a	4	8	0.058	4.222	7.900	7.550	127.8	125.5
BAC 6.2-b	4	8	0.058	4.132	7.899	7.450	123.2	

Table C-2: Compressive strength test results for baseline RBMAC mixtures.

	RBMAC Mixture					
	BAC 5.0			BAC 6.0		
	Specimen	Comp. Str. (psi)	Average Comp. Str. (psi)	Specimen	Comp. Str. (psi)	Average Comp. Str. (psi)
3-day	BAC5.0-3a	2179	2139	BAC6.0-3a	4449	4559
	BAC5.0-3b	2069		BAC6.0-3b	4487	
	BAC5.0-3c	2168		BAC6.0-3c	4741	
7-day	BAC5.0-7a	2976	2858	BAC6.0-7a	6605	6182
	BAC5.0-7b	2692		BAC6.0-7b	6671	
	BAC5.0-7c	2905		BAC6.0-7c	5271	
28-day	BAC5.0-28a	3560	3675	BAC6.0-28a	6441	6497
	BAC5.0-28b	3761		BAC6.0-28b	6680	
	BAC5.0-28c	3704		BAC6.0-28c	6370	
90-day	BAC5.0-90a	3818	3872	BAC6.0-90a	7158	6903
	BAC5.0-90b	3925		BAC6.0-90b	6727	
	BAC5.0-90c			BAC6.0-90c	6824	

	RBMAC Mixture					
	BAC 6.1			BAC 6.2		
	Specimen	Comp. Str. (psi)	Average Comp. Str. (psi)	Specimen	Comp. Str. (psi)	Average Comp. Str. (psi)
3-day	BAC6.1-3a	3332	3684	BAC6.2-3a	4902	4508
	BAC6.1-3b	3555		BAC6.2-3b	4550	
	BAC6.1-3c	4165		BAC6.2-3c	4071	
7-day	BAC6.1-7a	3704	4074	BAC6.2-7a	5359	5283
	BAC6.1-7b	4161		BAC6.2-7b	5206	
	BAC6.1-7c	4356		BAC6.2-7c		
28-day	BAC6.1-28a	5472	5307	BAC6.2-28a	6929	6450
	BAC6.1-28b	5170		BAC6.2-28b	6042	
	BAC6.1-28c	5279		BAC6.2-28c	6378	
90-day	BAC6.1-90a	4918	5362	BAC6.2-90a	7629	7343
	BAC6.1-90b	5805		BAC6.2-90b	7057	
	BAC6.1-90c			BAC6.2-90c		

Figure C-1: Test specimen used for compressive strength, modulus of elasticity, and Poisson’s ratio testing, RBMAC mixture BAC 5.0, 3 day tests.

Figure C-2: Test specimen used for compressive strength, modulus of elasticity, and Poisson’s ratio testing, RBMAC mixture BAC 5.0, 7 day tests.

Figure C-3: Test specimen used for compressive strength, modulus of elasticity, and Poisson's ratio testing, RBMAC mixture BAC 5.0, 28 day tests.

Figure C-4: Test specimen used for compressive strength, modulus of elasticity, and Poisson's ratio testing, RBMAC mixture BAC 5.0, 90 day tests.

Figure C-5: Test specimen used for compressive strength, modulus of elasticity, and Poisson’s ratio testing, RBMAC mixture BAC 6.0, 3 day tests.

Figure C-6: Test specimen used for compressive strength, modulus of elasticity, and Poisson’s ratio testing, RBMAC mixture BAC 5.0, 3 day tests.

Figure C-7: Test specimen used for compressive strength, modulus of elasticity, and Poisson's ratio testing, RBMAC mixture BAC 5.0, 3 day tests.

Figure C-8: Test specimen used for compressive strength, modulus of elasticity, and Poisson's ratio testing, RBMAC mixture BAC 5.0, 3 day tests.

Figure C-9: Test specimen used for compressive strength, modulus of elasticity, and Poisson's ratio testing, RBMAC mixture BAC 6.1, 3 day tests.

Figure C-10: Test specimen used for compressive strength, modulus of elasticity, and Poisson's ratio testing, RBMAC mixture BAC 6.1, 7 day tests.

Figure C-11: Test specimen used for compressive strength, modulus of elasticity, and Poisson's ratio testing, RBMAC mixture BAC 6.1, 28 day tests.

Figure C-12: Test specimen used for compressive strength, modulus of elasticity, and Poisson's ratio testing, RBMAC mixture BAC 6.1, 90 day tests.

Figure C-13: Test specimen used for compressive strength, modulus of elasticity, and Poisson's ratio testing, RBMAC mixture BAC 6.2, 3 day tests.

Figure C-14: Test specimen used for compressive strength, modulus of elasticity, and Poisson's ratio testing, RBMAC mixture BAC 6.2, 7 day tests.

Figure C-15: Test specimen used for compressive strength, modulus of elasticity, and Poisson’s ratio testing, RBMAC mixture BAC 6.2, 28 day tests.

Figure C-16: Test specimen used for compressive strength, modulus of elasticity, and Poisson’s ratio testing, RBMAC mixture BAC 6.2, 90 day tests.

Table C-3: Splitting tensile strength test data for RBMAC.

RBMAC Mixture	Specimen ID	Load (lb)	Splitting tensile strength (psi)	Average splitting tensile strength (psi)
BAC 5.0	BAC 5.0-28a	33,851	299	320
	BAC 5.0-28b	38,631	342	
BAC 6.0	BAC 6.0-28a	53,831	476	439
	BAC 6.0-28b	45,405	401	
BAC 6.1	BAC 6.1-28a	61,248	542	484
	BAC 6.1-28b	48,119	425	
BAC 6.2	BAC 6.2-28a	52,325	463	387
	BAC 6.2-28b	35,272	312	

Figure C-17: Typical splitting tensile strength test specimens.

Figure C-18: Several splitting tensile strength test specimens.

Table C-4: Modulus of rupture test data for RBMAC.

RBMAC Mixture	Specimen ID	Load at rupture (lb)	Modulus of rupture (psi)	Average modulus of rupture (psi)
BAC 5.0	BAC 5.0-7a	5,996	500	519
	BAC 5.0-7b	6,471	539	
BAC 6.0	BAC 6.0-7a	11,118	927	797
	BAC 6.0-7b	8,017	668	
BAC 6.1	BAC 6.1-7a	7,151	596	730
	BAC 6.1-7b	10,372	864	
BAC 6.2	BAC 6.2-7a	9,192	766	717
	BAC 6.2-7b	8,006	667	

Figure C-19: Typical test specimens used for modulus of rupture testing, and subsequently, abrasion resistance testing.

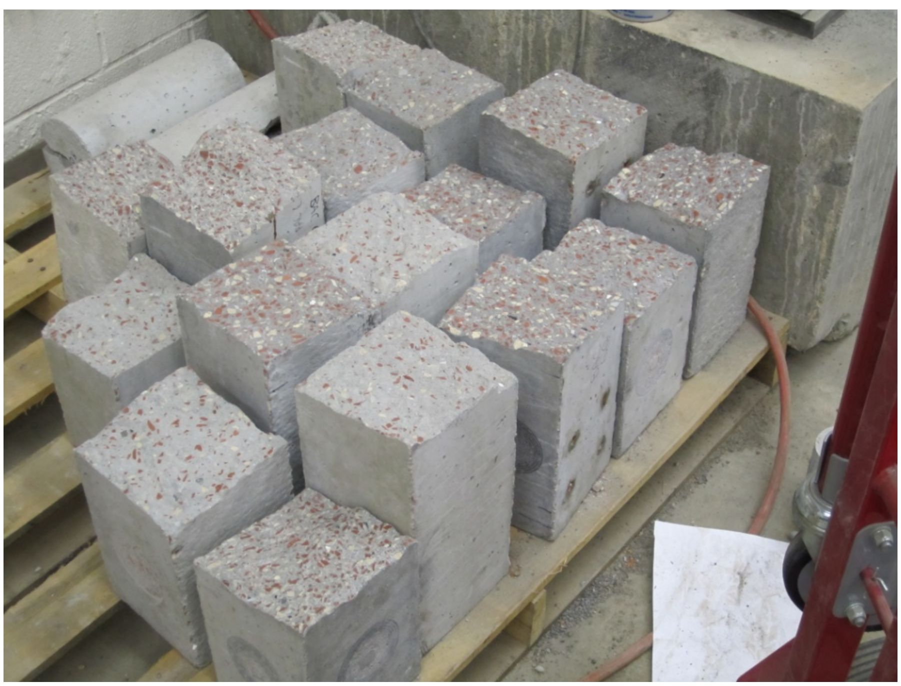

Figure C-20: Typical test specimens used for modulus of rupture testing, and subsequently, abrasion resistance testing and air and water permeability testing.

Table C-5: Typical data collected from modulus of elasticity and Poisson's ratio test.

Specimen	BAC6.2-3a	
Raw Data		
Load (lb)	Longitudinal dial gage reading (in)	Transverse dial gage reading (in)
0	0	0
5800	0.0008	0.0000
10000	0.0014	0.0002
15000	0.0021	0.0003
20000	0.0028	0.0004
25000	0.0037	0.0005
30000	0.0045	0.0006

Stress (psi)	Longitudinal strain (in/in)	Transverse strain (in/in)
0	0.000000	0.000000
205	0.000050	0.000000
354	0.000088	0.000017
531	0.000131	0.000025
707	0.000175	0.000033
884	0.000231	0.000042
1061	0.000281	0.000050

ulttimate load (lb)
138570

	Stress (psi)	Longitudinal strain (in/in)	Transverse strain (in/in)
at 40% of ultimate load	1961	0.000507	0.000091
at ultimate load	4902	0.001266	0.000228

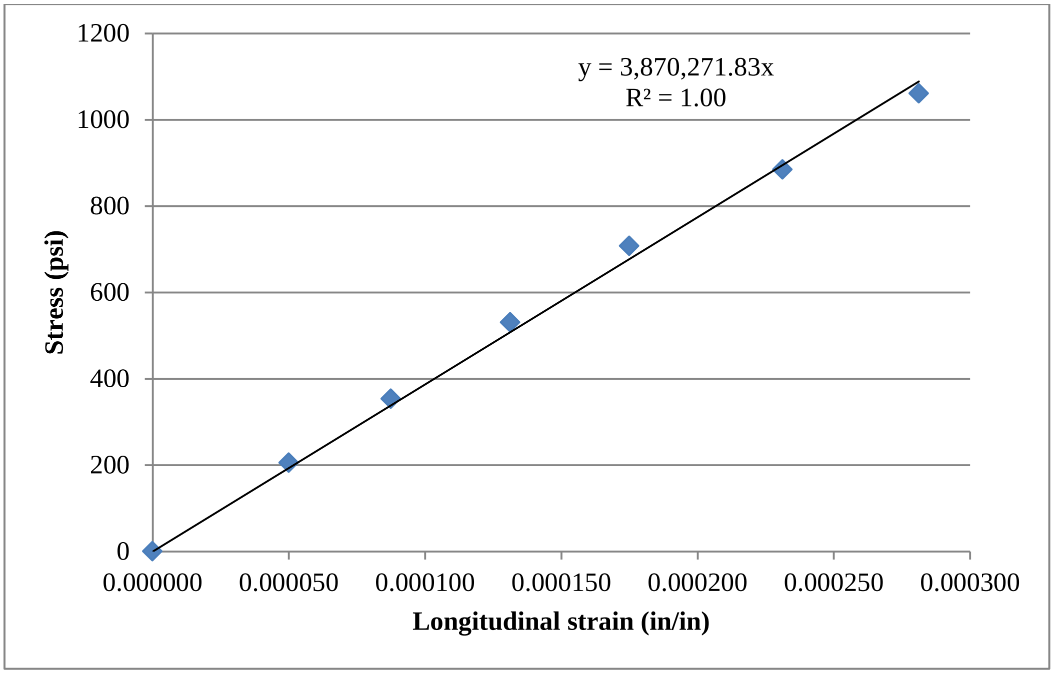

Figure C-21: Typical stress versus longitudinal strain plot from modulus of elasticity test.

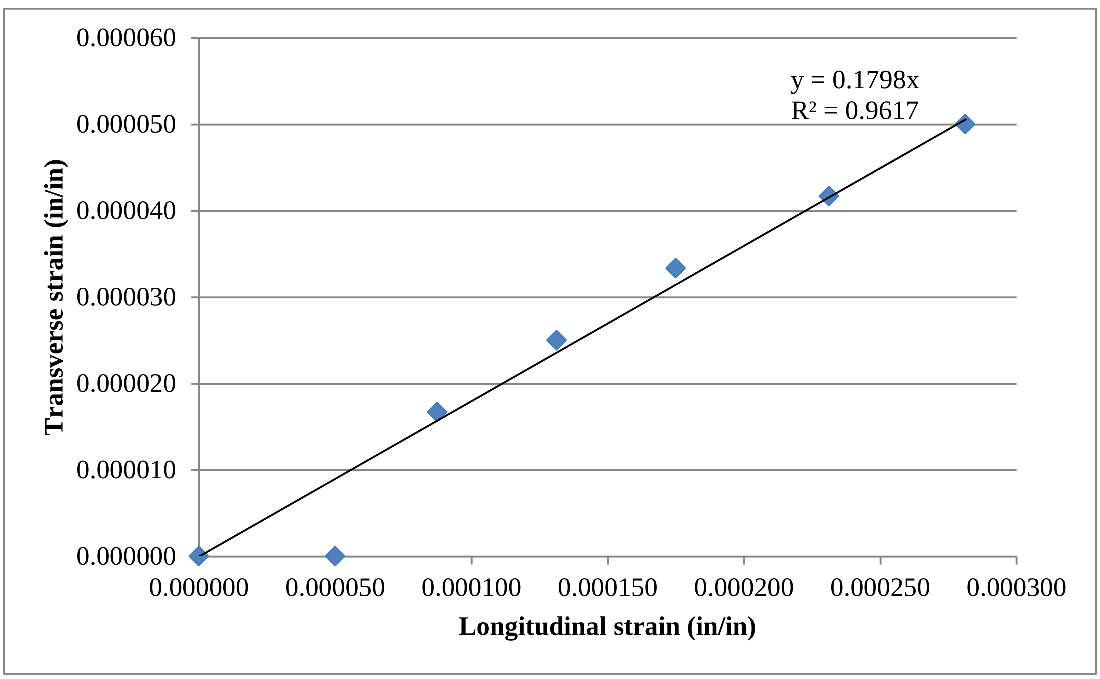

Figure C-22: Typical transverse strain versus longitudinal strain plot for Poisson’s ratio test.

Table C-6: Summary of modulus of elasticity test results.

	3-day			7-day			28-day			90-day		
	Test specimen			Test specimen			Test specimen			Test specimen		
RBMAC Mixture	a	b	c	a	b	c	a	b	c	a	b	c
BAC 5.0	2,180,000	2,160,000	2,260,000	2,690,000	2,740,000	2,830,000	2,650,000	2,740,000	2,960,000	2,830,000	2,980,000	---
BAC 6.0	3,390,000	3,260,000	3,370,000	4,050,000	4,010,000	3,870,000	4,010,000	3,870,000	3,640,000	3,940,000	3,940,000	4,000,000
BAC 6.1	3,190,000	3,000,000	3,170,000	3,290,000	3,250,000	3,860,000	3,620,000	3,590,000	3,480,000	3,510,000	3,780,000	---
BAC 6.2	3,840,000	3,660,000	3,300,000	3,530,000	3,330,000	---	4,050,000	3,720,000	3,940,000	4,070,000	3,680,000	---

Table C-7: Summary of Poisson's ratio test results.

	3-day			7-day			28-day			90-day		
	Test specimen			Test specimen			Test specimen			Test specimen		
RBMAC Mixture	**a**	**b**	**c**	**a**	**b**	**c**	**a**	**b**	**c**	**a**	**b**	**c**
BAC 5.0	0.18	0.18	0.17	0.21	0.21	0.21	0.16	0.19	0.20	0.14	0.2	---
BAC 6.0	0.13	0.17	0.18	0.16	0.17	0.17	0.18	0.15	0.15	0.19	0.18	0.18
BAC 6.1	0.18	0.18	0.19	0.2	0.2	0.22	0.17	0.17	0.16	0.18	0.18	---
BAC 6.2	0.2	0.2	0.16	0.13	0.14	---	0.15	0.16	0.17	0.16	0.18	---

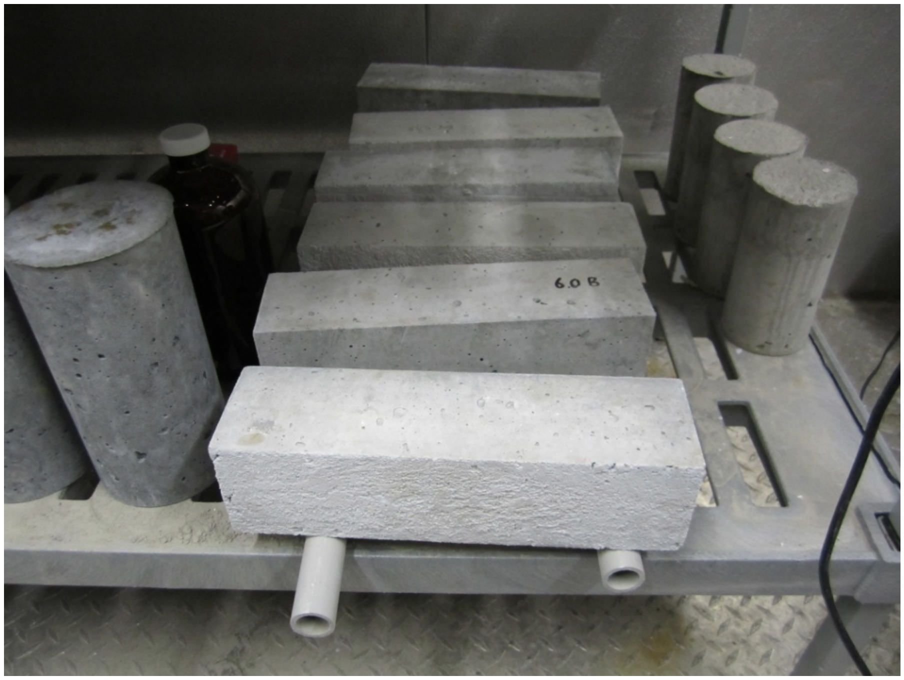

Figure C-23: Drying shrinkage test specimens (beams).

Figure C-24: Length measurement of drying shrinkage test specimen.

Table C-8: Coefficient of thermal expansion test results for RBMAC.

	Initial Temp. (°F)	Length at Initial Temp. (in)	High Temp. (°F)	Length at High Temp. (in.)	Low Temp. (°F)	Length at Low Temp. (in)	CTE (in/in/°F)
Test 1	71.5	4.0091	114.7	4.0098	52.4	4.0087	4.40×10^{-6}
Test 2	71.5	4.0049	115.3	4.0057	54.8	4.0041	6.60×10^{-6}
Test 3	70.5	4.0049	116.9	4.0058	53.7	4.0044	5.53×10^{-6}
Average							5.51×10^{-6}

Figure C-25: Test specimen prepared from BAC 6.2 used for thermal conductivity testing.

C-THERM TCi™ Thermal Conductivity Analyzer

Test Report

Report Generated on: 06-Jan-2012 11:15:18

Test TCI-122
Instrument: TH89-05-00129
Test Method: Polymers

Project: Cavalline

Material Idlewild RBMAC
Material Lot:

Software Version: 2.3.3954
Test started on: 06-Jan-2012
Performed by: Administrator
User ID: ADMIN

#	Repeat	Sensor ID	Start Time	Effusivity W*√(s)/(m^2)*K	Conductivity (W/mK)	Ambient T (°C)	DeltaT (°C)	V0 (mV)
1	1	T132	11:04:33	1,256	0.930	22.58	0.62	2,357.04
2	1	T132	11:05:39	1,275	0.950	22.78	0.62	2,356.47
3	1	T132	11:06:44	1,261	0.940	22.76	0.61	2,356.64
4	1	T132	11:07:50	1,266	0.940	22.66	0.62	2,357.50
5	1	T132	11:08:56	1,253	0.930	21.67	0.62	2,359.59
6	1	T132	11:10:01	1,269	0.950	22.86	0.63	2,356.36
7	1	T132	11:11:07	1,277	0.950	22.61	0.63	2,357.81
8	1	T132	11:12:13	1,263	0.940	21.81	0.61	2,359.94
9	1	T132	11:13:18	1,272	0.950	21.78	0.63	2,360.07
10	1	T132	11:14:24	1,252	0.920	21.76	0.62	2,360.09

Figure C-26: Typical thermal conductivity test results for RBMAC mixture BAC 6.2.

Figure C-27: Sample of crushed RBMAC (mixture BAC 6.2) used for heat capacity testing.

Figure C-28: Samples of crushed brick, mortar, clay tile, and RBMAC used for heat capacity testing.

TLC RBMAC Test 4 - 1/6/2012
Creation Date : 1/6/2012 11:47:03 AM
User : admin

TG :
Initial Mass : 101.52 mg
Molar Mass : N/A

TG |-b [TLC RBMAC Blank 1 - 1/6/2012 / 2 Heating Zone / TG] :
Initial Mass : 101.52 mg
Molar Mass : N/A

HeatFlow :
Initial Mass : 101.52 mg
Molar Mass : N/A

HeatFlow |-b [TLC RBMAC Blank 1 - 1/6/2012 / 2 Heating Zone / HeatFlow] :
Initial Mass : 101.52 mg
Molar Mass : N/A

Index	Time (s)	Furnace Temp. (°C)	Sample Temp. (°C)	TG (mg)	TG (minus blank crucible) (mg)	HeatFlow (mW)	HeatFlow (minus blank crucible) (mW)	Heat Flow (J/°C)	Heat Capacity (J/(mg•°C))	Heat Capacity (J/(g•°C))	Heat Capacity (BTU/(lb•°F)
1	0	24.903164	24.80681	42.23283	101.19931	0.134925	-0.610845	-0.00366507	0.0000361	0.0361020	0.1511516
2	0.1	24.903212	24.80685	42.2327	101.199184	0.134943	-0.61084	-0.00366504	0.0000361	0.0361017	0.1511504
3	0.2	24.903254	24.8069	42.23257	101.19907	0.13496	-0.610835	-0.00366501	0.0000361	0.0361014	0.1511492
4	0.3	24.903303	24.80694	42.23245	101.198948	0.134977	-0.61083	-0.00366498	0.0000361	0.0361011	0.1511479
5	0.4	24.903353	24.80698	42.23233	101.198845	0.134994	-0.610821	-0.003664926	0.0000361	0.0361005	0.1511457
6	0.5	24.903395	24.80703	42.23221	101.198734	0.135012	-0.610812	-0.003664872	0.0000361	0.0361000	0.1511435
7	0.6	24.903446	24.80707	42.2321	101.198631	0.13505	-0.610785	-0.00366471	0.0000361	0.0360984	0.1511368
8	0.7	24.903496	24.80711	42.23199	101.198528	0.135088	-0.61076	-0.00366456	0.0000361	0.0360969	0.1511306
9	0.8	24.903545	24.80715	42.2319	101.198441	0.135124	-0.610738	-0.003664428	0.0000361	0.0360956	0.1511252
10	0.9	24.903595	24.80719	42.2318	101.198349	0.135157	-0.610719	-0.003664314	0.0000361	0.0360945	0.1511205
11	1	24.903645	24.80723	42.23171	101.198273	0.13519	-0.610702	-0.003664212	0.0000361	0.0360935	0.1511163

Figure C-29: Typical output spreadsheet of TGA for RBMAC, with associated heat capacity calculations.

Table C-9: Air and water permeability test results for RBMAC mixture BAC 5.0.

Air and Water Permeability	Mixture	BAC 5.0
	Date	6/7/2011

Test Location 1

A	Air		Water	
	Min.	Sec.	Min.	Sec.
1	0	10	0	3
2	0	10	0	3
3	0	10	0	3
4				
5				

B	Air		Water	
	Min.	Sec.	Min.	Sec.
1	0	6	0	3
2	0	7	0	3
3	0	8	0	3
4	0	8		
5				

D	Air		Water	
	Min.	Sec.	Min.	Sec.
1	0	10	0	4
2	0	11	0	4
3	0	12	0	4
4				
5				

C	Air		Water	
	Min.	Sec.	Min.	Sec.
1	0	8	0	3
2	0	9	0	3
3	0	10	0	4
4				
5				

Location 1 Air (sec)						
Test Plug	1	2	3	4	5	Average
A	10	10	10			10.0
B	6	7	8	8		7.3
C	8	9	10			9.0
D	10	11	12			11.0
Average for location						9.3

Location 1 Water (sec)						
Test Plug	1	2	3	4	5	Average
A	3	3	3			3.0
B	3	3	3			3.0
C	3	3	4			3.3
D	4	4	4			4.0
Average for location						3.3

Table C-10: Air and water permeability test results for RBMAC mixture BAC 6.0.

Air and Water Permeability	Mixture	BAC 6.0
	Date	6/3/2011

Test Location 1

A	Air		Water	
	Min.	Sec.	Min.	Sec.
1	1	45	0	20
2	1	53	0	20
3	1	52		
4				
5				

B	Air		Water	
	Min.	Sec.	Min.	Sec.
1	0	15	1	28
2	0	34	1	44
3	0	34	2	51
4			3	39
5				

D	Air		Water	
	Min.	Sec.	Min.	Sec.
1	0	48	0	32
2	1	3	0	44
3	1	8	0	52
4	1	8		
5				

C	Air		Water	
	Min.	Sec.	Min.	Sec.
1	1	12	1	15
2	1	23	1	22
3	1	24		
4				
5				

Location 1 Air (sec)						
Test Plug	1	2	3	4	5	Average
A	105	113	112			110.0
B	15	34	34			27.7
C	72	83	84			79.7
D	48	63	68	68		61.8
Average for location						69.8

Location 1 Water (sec)						
Test Plug	1	2	3	4	5	Average
A	20	20				20.0
B	88	104	171	219		145.5
C	75	82				78.5
D	32	44	52			42.7
Average for location						71.7

Table C-11: Air and water permeability test results for RBMAC mixture BAC 6.1.

Air and Water Permeability	Mixture	BAC 6.1
	Date	6/8/2011

Test Location 1

A	Air		Water	
	Min.	Sec.	Min.	Sec.
1	0	35	0	20
2	0	40	0	21
3	0	44		
4	0	47		
5	0	48		

B	Air		Water	
	Min.	Sec.	Min.	Sec.
1	0	39	0	33
2	0	49	0	35
3	0	50	0	35
4				
5				

D	Air		Water	
	Min.	Sec.	Min.	Sec.
1	0	26	0	14
2	0	34	0	13
3	0	35		
4				
5				

C	Air		Water	
	Min.	Sec.	Min.	Sec.
1	0	43	0	10
2	0	53	0	11
3	0	54		
4				
5				

Location 1 Air (sec)						
Test Plug	1	2	3	4	5	Average
A	35	40	44	47	48	42.8
B	39	49	50			46.0
C	43	53	54			50.0
D	26	34	35			31.7
Average for location						42.6

Location 1 Water (sec)						
Test Plug	1	2	3	4	5	Average
A	20	21				20.5
B	33	35	35			34.3
C	10	11				10.5
D	14	13				13.5
Average for location						19.7

Table C-12: Air and water permeability test results for RBMAC mixture BAC 6.2.

Air and Water Permeability	Mixture	BAC 6.2
	Date	6/7/2011

Test Location 1

A	Air		Water	
	Min.	Sec.	Min.	Sec.
1	0	41	0	10
2	0	52	0	13
3	0	53	0	14
4			0	16
5				

B	Air		Water	
	Min.	Sec.	Min.	Sec.
1	0	41	0	20
2	0	53	0	12
3	0	58	0	23
4	1	0	0	24
5				

D	Air		Water	
	Min.	Sec.	Min.	Sec.
1	0	36	0	10
2	0	41	0	9
3	0	41	0	11
4			0	13
5				

C	Air		Water	
	Min.	Sec.	Min.	Sec.
1	0	38	2	6
2	0	43	0	57
3	0	45	0	56
4	0	47	0	58
5				

Location 1 Air (sec)						
Test Plug	1	2	3	4	5	Average
A	41	52	53			48.7
B	41	53	58	60		53.0
C	38	43	45	47		43.3
D	36	41	41			39.3
Average for location						46.1

Location 1 Water (sec)						
Test Plug	1	2	3	4	5	Average
A	10	13	14	16		13.3
B	20	12	23	24		19.8
C	126	57	56	58		74.3
D	10	9	11	13		10.8
Average for location						29.5

Figure C-30: Typical air and water permeability test specimens after testing.

Figure C-31: Typical test specimens used for abrasion resistance testing (three specimens in front of photo) and air and water permeability testing (two specimens in rear of photo).

Table C-13: Abrasion resistance test results for RBMAC.

RBMAC Mixture	Location	Time (min)	Depth of wear at each circumference quarter point (mm)				Average depth of wear (mm)	Average depth of wear for each location after three 2-minute test periods (mm)	Average total depth of wear after three 2-minute test periods (mm)
BAC 5.0	1	0-2	1.10	1.70	2.00	1.50	1.58	2.21	0.99
		2-4	1.60	2.00	2.60	1.80	2.00		
		4-6	1.70	2.30	2.60	2.25	2.21		
	2	0-2	0.35	0.30	0.20	0.10	0.24	0.64	
		2-4	0.45	0.55	0.40	0.05	0.36		
		4-6	0.75	0.80	0.40	0.60	0.64		
	3	0-2	0.10	0.15	0.25	0.10	0.15	1.04	
		2-4	0.85	0.95	0.10	0.50	0.60		
		4-6	0.85	1.10	1.10	1.10	1.04		
BAC 6.0	1	0-2	0.10	0.10	0.10	0.05	0.09	0.09	0.16
		2-4	0.05	0.05	0.20	0.15	0.11		
		4-6	0.00	0.05	0.10	0.20	0.09		
	2	0-2	0.10	0.05	0.10	0.10	0.09	0.03	
		2-4	0.00	0.05	0.00	0.00	0.01		
		4-6	0.00	0.05	0.00	0.05	0.03		
	3	0-2	0.00	0.05	0.10	0.05	0.05	0.33	
		2-4	0.50	0.15	0.10	0.10	0.21		
		4-6	0.10	0.10	0.50	0.60	0.33		
BAC 6.1	1	0-2	0.30	0.00	0.00	0.00	0.08	0.19	0.40
		2-4	0.15	0.20	0.10	0.05	0.13		
		4-6	0.10	0.05	0.40	0.20	0.19		
	2	0-2	0.30	0.30	0.50	0.50	0.40	0.45	
		2-4	0.30	0.60	0.90	0.60	0.60		
		4-6	0.50	0.40	0.40	0.50	0.45		
	3	0-2	0.40	0.45	0.25	0.70	0.45	0.91	
		2-4	0.75	0.85	0.80	0.65	0.76		
		4-6	1.00	0.90	0.65	1.10	0.91		
BAC 6.2	1	0-2	0.00	0.10	0.15	0.10	0.09	0.05	0.41
		2-4	0.25	0.05	0.10	0.10	0.13		
		4-6	0.10	0.10	0.00	0.00	0.05		
	2	0-2	0.00	0.00	0.20	0.00	0.05	0.53	
		2-4	0.70	0.70	0.30	0.40	0.53		
		4-6	0.60	0.50	0.40	0.60	0.53		
	3	0-2	0.30	0.25	0.40	0.40	0.34	0.65	
		2-4	0.80	0.65	0.80	0.80	0.76		
		4-6	0.70	0.90	0.11	0.90	0.65		

Table C-14: Rapid chloride ion permeability test results for RBMAC.

Rapid Chloride Permeabilty Test Datasheet

Date	**3/30/2011**	**Specimen BAC 5.0 EQ-D2**		**Specimen BAC 6.0 EQ-D2**		**Specimen BAC 6.0 EQ-D1**		**Specimen BAC 6.2 EQ-D2**	
		Machine	Position	Machine	Position	Machine	Position	Machine	Position
		A	1	A	2	A	3	A	4
Duration (min)	**Time**	Charge Passed (Coulombs)	Current (amperes)	Charge Passed (Coulombs)	Current (amperes)	Charge Passed (Coulombs)	Current (amperes)	Charge Passed (Coulombs)	Current (amperes)
0	8:30	0	0.211	0	0.042	0	0.041	0	0.095
30	9:00	421	0.264	77	0.043	78	0.044	184	0.106
60	9:30	955	0.313	158	0.044	158	0.042	385	0.114
90	10:00	1561	0.360	236	0.044	237	0.044	597	0.121
120	10:30	2256	0.413	315	0.045	316	0.045	819	0.130
150	11:00	3026	0.444	396	0.045	397	0.045	1059	0.137
180	11:30	3840	0.464	476	0.046	477	0.045	1310	0.145
210	12:00	4706	0.484	561	0.047	560	0.046	1580	0.152
240	12:30	5571	0.497	643	0.047	642	0.047	1854	0.158
270	1:00	6475	0.514	728	0.048	726	0.047	2142	0.164
300	1:30	7405	0.546	811	0.048	809	0.048	2432	0.169
330	2:00	8442	0.568	901	0.049	898	0.048	2748	0.172
360	2:30	9417	0.576	983	0.049	980	0.049	3242	0.174

Table C-14 (con't): Rapid chloride ion permeability test results for RBMAC.

Rapid Chloride Permeabilty Test Datasheet

Date	**3/30/2011**	**Specimen BAC 5.0 EQ-D1**		**Specimen BAC 6.1 EQ-D2**		**Specimen BAC 6.1 EQ-D1**		**Specimen BAC 6.2 EQ-D1**	
		Machine	Position	Machine	Position	Machine	Position	Machine	Position
		B	1	B	2	B	3	B	4
Duration (min)	**Time**	Charge Passed (Coulombs)	Current (amperes)	Charge Passed (Coulombs)	Current (amperes)	Charge Passed (Coulombs)	Current (amperes)	Charge Passed (Coulombs)	Current (amperes)
0	8:30	0	0.183	0	0.066	0	0.055	0	0.096
30	9:00	351	0.217	124	0.072	104	0.059	181	0.105
60	9:30	786	0.249	262	0.075	215	0.060	383	0.111
90	10:00	1263	0.277	400	0.077	325	0.061	590	0.119
120	10:30	1780	0.307	541	0.080	436	0.062	809	0.127
150	11:00	2352	0.336	686	0.082	550	0.064	1044	0.133
180	11:30	2977	0.361	835	0.084	665	0.065	1290	0.141
210	12:00	3650	0.381	989	0.086	785	0.066	1555	0.149
240	12:30	4342	0.395	1144	0.088	902	0.067	1824	0.155
270	1:00	5053	0.407	1303	0.090	1021	0.067	2106	0.161
300	1:30	5784	0.417	1465	0.092	1141	0.068	2393	0.166
330	2:00	6569	0.426	1638	0.094	1268	0.069	2707	0.171
360	2:30	7340	0.435	1806	0.095	1391	0.070	3011	0.174

Table C-15: Summary of rapid chloride ion permeability test results for RBMAC.

RBMAC Mixture	Test Specimen	ASTM C1202 Rapid chloride ion permeability, total charge passed (Coulombs) Specimen	Average
BAC 5.0	BAC 5.0 EQ-D1	7340	8379
	BAC 5.0 EQ-D2	9417	
BAC 6.0	BAC 6.0 EQ-D1	980	982
	BAC 6.0 EQ-D2	983	
BAC 6.1	BAC 6.1 EQ-D1	1391	1599
	BAC 6.1 EQ-D2	1806	
BAC 6.2	BAC 6.2 EQ-D1	3011	3127
	BAC 6.2 EQ-D2	3242	

Figure C-32: Typical test specimens after rapid chloride ion permeability testing.

Table C-16: Surface resistivity test results for RBMAC.

			Surface Resistivity Readings (kΩ•cm)											
Specimen ID	Diam (in)	Length (in)	0°	90°	180°	270°	0°	90°	180°	270°	Average	Temp	Average surface resistivity (kΩ•cm)	Average temp (°F)
BAC 5.0 EQ-D 1	6	12	19	20	18	19	19	20	18	18	18.9	44	18.9	44
BAC 5.0 EQ-D 2	6	12	20	19	18	20	18	18	19	20	19.0	44		
BAC 6.0 EQ-D 1	6	12	45	43	45	43	44	43	45	43	43.9	44	42.9	44
BAC 6.0 EQ-D 2	6	12	45	43	41	39	44	41	39	44	42.0	44		
BAC 6.1 EQ-D 1	6	12	35	32	36	34	31	32	36	36	34.0	44	37.5	43
BAC 6.1 EQ-D 2	4	8	40	41	43	43	37	40	42	42	41.0	42		
BAC 6.2 EQ-D 1	4	8	29	29	31	31	29	29	30	31	29.9	45	30.8	43.5
BAC 6.2 EQ-D 2	4	8	33	31	31	32	30	32	31	33	31.6	42		
BAC 5.0 EQ-D 1	6	12	15	17	16	16	15	16	15	16	15.8	53	15.8	53
BAC 5.0 EQ-D 2	6	12	16	16	15	17	15	16	15	16	15.8	53		
BAC 6.0 EQ-D 1	6	12	36	34	35	35	36	35	35	35	35.1	53	34.9	53
BAC 6.0 EQ-D 2	6	12	34	37	33	34	36	37	33	34	34.8	53		
BAC 6.1 EQ-D 1	6	12	28	27	29	28	28	28	30	28	28.3	53	30.7	53
BAC 6.1 EQ-D 2	4	8	30	33	35	34	31	33	35	34	33.1	53		
BAC 6.2 EQ-D 1	4	8	26	25	23	24	25	26	25	26	25.0	53	25.5	53
BAC 6.2 EQ-D 2	4	8	27	26	25	26	27	25	25	27	26.0	53		
BAC 5.0 EQ-D 1	6	12	15	13	13	14	13	14	13	13	13.5	63	13.2	62.5
BAC 5.0 EQ-D 2	6	12	13	13	12	13	13	13	13	13	12.9	62		
BAC 6.0 EQ-D 1	6	12	28	26	29	27	27	27	28	27	27.4	63	27.3	63.5
BAC 6.0 EQ-D 2	6	12	26	29	27	25	27	29	28	27	27.3	64		
BAC 6.1 EQ-D 1	6	12	24	24	23	23	25	23	23	22	23.4	64	25.3	63.5
BAC 6.1 EQ-D 2	4	8	24	28	29	28	27	27	26	28	27.1	63		
BAC 6.2 EQ-D 1	4	8	22	22	19	20	20	21	20	21	20.6	64	20.7	64.5
BAC 6.2 EQ-D 2	4	8	22	20	20	21	21	21	20	21	20.8	65		
BAC 5.0 EQ-D 1	6	12	11	12	11	11	12	12	11	12	11.5	71	11.7	71
BAC 5.0 EQ-D 2	6	12	12	12	12	12	12	12	12	11	11.9	71		
BAC 6.0 EQ-D 1	6	12	25	23	24	24	25	26	24	25	24.5	71	24.2	71
BAC 6.0 EQ-D 2	6	12	24	24	24	23	24	24	25	23	23.9	71		
BAC 6.1 EQ-D 1	6	12	20	18	21	20	20	18	21	21	19.9	71	21.7	71
BAC 6.1 EQ-D 2	4	8	23	24	24	24	21	23	25	24	23.5	71		
BAC 6.2 EQ-D 1	4	8	18	18	18	18	19	19	19	18	18.4	71	18.8	71
BAC 6.2 EQ-D 2	4	8	20	18	19	20	19	19	19	19	19.1	71		
BAC 5.0 EQ-D 1	6	12	9	9	9	8	8	9	9	8	8.6	87	8.8	87.5
BAC 5.0 EQ-D 2	6	12	9	9	9	9	9	9	9	9	9.0	88		
BAC 6.0 EQ-D 1	6	12	17	17	17	17	18	17	17	17	17.1	88	17.1	87.5
BAC 6.0 EQ-D 2	6	12	17	18	16	17	16	18	18	16	17.0	87		
BAC 6.1 EQ-D 1	6	12	14	13	15	13	14	14	15	14	14.0	88	14.1	88
BAC 6.1 EQ-D 2	4	8	15	14	14	14	14	14	14	15	14.3	88		
BAC 6.2 EQ-D 1	4	8	14	14	14	14	14	15	14	14	14.1	87	15.7	87.5
BAC 6.2 EQ-D 2	4	8	16	17	18	17	17	17	18	18	17.3	88		
BAC 5.0 EQ-D 1	6	12	7	7	7	7	7	7	7	7	7.0	105	7.0	105.5
BAC 5.0 EQ-D 2	6	12	7	7	7	7	7	7	7	7	7.0	106		
BAC 6.0 EQ-D 1	6	12	12	12	12	12	12	12	12	12	12.0	105	11.9	105.5
BAC 6.0 EQ-D 2	6	12	11	11	13	13	11	11	12	13	11.9	106		
BAC 6.1 EQ-D 1	6	12	10	10	11	9	10	11	11	10	10.3	105	11.3	106
BAC 6.1 EQ-D 2	4	8	12	12	13	13	12	12	13	12	12.4	107		
BAC 6.2 EQ-D 1	4	8	10	11	11	11	11	10	10	11	10.6	105	10.7	105
BAC 6.2 EQ-D 2	4	8	11	11	11	11	11	11	10	10	10.8	105		

APPENDIX D: SUPPLEMENTAL INFORMATION FOR CHAPTER 7

Project: RBMAC pavement analysis.dgp

General Information

Design Life	30 years
Pavement construction:	October, 2011
Traffic open:	January, 2012
Type of design	JPCP

Description:
RBMAC pavement, 6000 AADTT, 65mph design speed, CTE = 4.4×10-6 in/in/°F

Analysis Parameters

Performance Criteria

	Limit	Reliability
Initial IRI (in/mi)	75	
Terminal IRI (in/mi)	170	90
Transverse Cracking (% slabs cracked)	10	90
Mean Joint Faulting (in)	0.75	90

Location:	Charlotte, North Carolina	
Project ID:	RBMAC pavement	
Section ID:		
Date:	1/9/2012	
Station/milepost format:	Feet: 00 + 00	
Station/milepost begin:		0
Station/milepost end:		1000
Traffic direction:	East bound	

Default Input Level

Default input level — Level 3, Default and historical agency values.

Traffic

Initial two-way AADTT:	6000
Number of lanes in design direction:	2
Percent of trucks in design direction (%):	50
Percent of trucks in design lane (%):	95
Operational speed (mph):	65

Traffic -- Volume Adjustment Factors

Monthly Adjustment Factors (Level 3, Default MAF)

	Vehicle Class									
Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
February	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
March	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
April	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
May	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
June	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
July	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
August	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
September	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
October	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
November	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Figure D-1: Typical M-EPDG input summary for RBMAC pavement.

December	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Vehicle Class Distribution
(Level 3, Default Distribution)

AADTT distribution by vehicle class

Class 4	1.8%
Class 5	24.6%
Class 6	7.6%
Class 7	0.5%
Class 8	5.0%
Class 9	31.3%
Class 10	9.8%
Class 11	0.8%
Class 12	3.3%
Class 13	15.3%

Hourly truck traffic distribution
by period beginning:

Midnight	2.3%	Noon	5.9%
1:00 am	2.3%	1:00 pm	5.9%
2:00 am	2.3%	2:00 pm	5.9%
3:00 am	2.3%	3:00 pm	5.9%
4:00 am	2.3%	4:00 pm	4.6%
5:00 am	2.3%	5:00 pm	4.6%
6:00 am	5.0%	6:00 pm	4.6%
7:00 am	5.0%	7:00 pm	4.6%
8:00 am	5.0%	8:00 pm	3.1%
9:00 am	5.0%	9:00 pm	3.1%
10:00 am	5.9%	10:00 pm	3.1%
11:00 am	5.9%	11:00 pm	3.1%

Traffic Growth Factor

Vehicle Class	Growth Rate	Growth Function
Class 4	3.0%	Compound
Class 5	3.0%	Compound
Class 6	3.0%	Compound
Class 7	3.0%	Compound
Class 8	3.0%	Compound
Class 9	3.0%	Compound
Class 10	3.0%	Compound
Class 11	3.0%	Compound
Class 12	3.0%	Compound
Class 13	3.0%	Compound

Traffic -- Axle Load Distribution Factors
Level 3: Default

Traffic -- General Traffic Inputs

Mean wheel location (inches from the lane marking):	18
Traffic wander standard deviation (in):	10
Design lane width (ft):	12

Number of Axles per Truck

Vehicle Class	Single Axle	Tandem Axle	Tridem Axle	Quad Axle
Class 4	1.62	0.39	0.00	0.00
Class 5	2.00	0.00	0.00	0.00
Class 6	1.02	0.99	0.00	0.00
Class 7	1.00	0.26	0.83	0.00
Class 8	2.38	0.67	0.00	0.00
Class 9	1.13	1.93	0.00	0.00
Class 10	1.19	1.09	0.89	0.00
Class 11	4.29	0.26	0.06	0.00
Class 12	3.52	1.14	0.06	0.00
Class 13	2.15	2.13	0.35	0.00

Figure D-1 (con’t): Typical M-EPDG input summary for RBMAC pavement.

Axle Configuration

Average axle width (edge-to-edge) outside dimensions,ft): 8.5
Dual tire spacing (in): 12

Axle Configuration

Tire Pressure (psi) : 120

Average Axle Spacing

Tandem axle(psi): 51.6
Tridem axle(psi): 49.2
Quad axle(psi): 49.2

Wheelbase Truck Tractor

	Short	Medium	Long
Average Axle Spacing (ft)	12	15	18
Percent of trucks	33%	33%	34%

Climate

icm file: C:\DG2002\Projects\Charlotte-Now.icm
Latitude (degrees.minutes) 35.13
Longitude (degrees.minutes) -80.56
Elevation (ft) 724
Depth of water table (ft) 10

Structure--Design Features

Permanent curl/warp effective temperature difference (°F): -10

Joint Design

Joint spacing (ft): 15
Sealant type: Silicone
Dowel diameter (in): 1.75
Dowel bar spacing (in): 12

Edge Support None

Long-term LTE(%): n/a
Widened Slab (ft): n/a

Base Properties

Base type: Granular
Erodibility index: Erosion Resistant (3)
PCC-Base Interface Full friction contact
Loss of full friction (age in months): 360

Structure--ICM Properties

Surface shortwave absorptivity: 0.85

Structure--Layers

Layer 1 -- JPCP

General Properties

PCC material JPCP
Layer thickness (in): 9.5
Unit weight (pcf): 130

Figure D-1 (con't): Typical M-EPDG input summary for RBMAC pavement.

Poisson's ratio	0.18

Thermal Properties

Coefficient of thermal expansion (per F° x 10- 6):	4.4
Thermal conductivity (BTU/hr-ft-F°) :	0.533
Heat capacity (BTU/lb-F°):	0.2

Mix Properties

Cement type:	Type II
Cementitious material content (lb/yd^3):	575
Water/cement ratio:	0.32
Aggregate type:	Rhyolite
PCC zero-stress temperature (F°)	Derived
Ultimate shrinkage at 40% R.H (microstrain)	Derived
Reversible shrinkage (% of ultimate shrinkage):	50
Time to develop 50% of ultimate shrinkage (days):	35
Curing method:	Curing compound

Strength Properties

Input level:	Level 3
28-day PCC modulus of rupture (psi):	716
28-day PCC compressive strength (psi):	n/a

Layer 2 -- Crushed stone

Unbound Material:	Crushed stone
Thickness(in):	8

Strength Properties

Input Level:	Level 3
Analysis Type:	ICM inputs (ICM Calculated Modulus)
Poisson's ratio:	0.35
Coefficient of lateral pressure,Ko:	0.5
Modulus (input) (psi):	30000

ICM Inputs

Gradation and Plasticity Index

Plasticity Index, PI:	1
Liquid Limit (LL)	6
Compacted Layer	No
Passing #200 sieve (%):	8.7
Passing #40	82.4
Passing #4 sieve (%):	44.7
D10(mm)	0.1035
D20(mm)	0.425
D30(mm)	1.306
D60(mm)	10.82
D90(mm)	46.19

Sieve	Percent Passing
0.001mm	
0.002mm	
0.020mm	
#200	8.7
#100	
#80	12.9
#60	

Figure D-1 (con't): Typical M-EPDG input summary for RBMAC pavement.

#50	
#40	20
#30	
#20	
#16	
#10	33.8
#8	
#4	44.7
3/8"	57.2
1/2"	63.1
3/4"	72.7
1"	78.8
1 1/2"	85.8
2"	91.6
2 1/2"	
3"	
3 1/2"	97.6
4"	97.6

Calculated/Derived Parameters

Maximum dry unit weight (pcf):	127.2 (derived)
Specific gravity of solids, Gs:	2.70 (derived)
Saturated hydraulic conductivity (ft/hr):	0.05054 (derived)
Optimum gravimetric water content (%):	7.4 (derived)
Calculated degree of saturation (%):	61.2 (calculated)

Soil water characteristic curve parameters: Default values

Parameters	Value
a	7.2555
b	1.3328
c	0.82422
Hr.	117.4

Layer 3 -- A-6

Unbound Material:	A-6
Thickness(in):	Semi-infinite

Strength Properties

Input Level:	Level 3
Analysis Type:	ICM inputs (ICM Calculated Modulus)
Poisson's ratio:	0.35
Coefficient of lateral pressure,Ko:	0.5
Modulus (input) (psi):	6000

ICM Inputs

Gradation and Plasticity Index

Plasticity Index, PI:	16
Liquid Limit (LL)	33
Compacted Layer	No
Passing #200 sieve (%):	63.2
Passing #40	20
Passing #4 sieve (%):	93.5
D10(mm)	0.000285
D20(mm)	0.0008125

Figure D-1 (con't): Typical M-EPDG input summary for RBMAC pavement.

D30(mm)	0.002316
D60(mm)	0.05364
D90(mm)	1.922

Sieve	Percent Passing
0.001mm	
0.002mm	
0.020mm	
#200	63.2
#100	
#80	73.5
#60	
#50	
#40	82.4
#30	
#20	
#16	
#10	90.2
#8	
#4	93.5
3/8"	96.4
1/2"	97.4
3/4"	98.4
1"	99
1 1/2"	99.5
2"	99.8
2 1/2"	
3"	
3 1/2"	100
4"	100

Calculated/Derived Parameters

Maximum dry unit weight (pcf):	107.9 (derived)
Specific gravity of solids, Gs:	2.70 (derived)
Saturated hydraulic conductivity (ft/hr):	1.95e-005 (derived)
Optimum gravimetric water content (%):	17.1 (derived)
Calculated degree of saturation (%):	82.1 (calculated)

Soil water characteristic curve parameters: Default values

Parameters	Value
a	108.41
b	0.68007
c	0.21612
Hr.	500

Distress Model Calibration Settings - Rigid (new)

Faulting

Faulting Coefficients

C1	1.0184
C2	0.91656
C3	0.002185
C4	0.000884
C5	250
C6	0.4

Figure D-1 (con't): Typical M-EPDG input summary for RBMAC pavement.

C7	1.83312
C8	400

Reliability (FAULT)

Std. Dev.	POWER(0.0097*FAULT,0.5178)+0.014

Cracking

Fatigue Coefficients

C1	2
C2	1.22

Cracking Coefficients

C4	1
C5	-1.98

Reliability (CRACK)

Std. Dev.	POWER(5.3116*CRACK,0.3903) + 2.99

IRI(jpcp)

C1	0.8203
C2	0.4417
C3	20.37
C4	1.4929
C5	25.24
Standard deviation in initial IRI (in/mile):	5.4

Figure D-1 (con't): Typical M-EPDG input summary for RBMAC pavement.

Reliability Summary: Project RBMAC pavement analysis.dgp
2/13/2012 6:53 PM

1 of 1

Project: RBMAC pavement analysis.dgp
Reliability Summary

Performance Criteria	Distress Target	Reliability Target	Distress Predicted	Reliability Predicted	Acceptable
Terminal IRI (in/mi)	170	90	111.4	93.52	Pass
Transverse Cracking (% slabs cracked)	10	90	1.6	94.33	Pass
Mean Joint Faulting (in)	0.75	90	0.061	99.999	Pass

Figure D-2: Typical M-EPDG reliability summary for RBMAC pavement.

Project: RBMAC pavement analysis.dgp

General Information

Design Life	30 years
Pavement construction:	October, 2011
Traffic open:	January, 2012
Type of design	JPCP

Description:
Conventional PCC pavement with granite aggregate, 6000 AADTT, 65mph design speed

Analysis Parameters

Performance Criteria

	Limit	Reliability
Initial IRI (in/mi)	75	
Terminal IRI (in/mi)	170	90
Transverse Cracking (% slabs cracked)	10	90
Mean Joint Faulting (in)	0.75	90

Location:	Charlotte, North Carolina	
Project ID:	RBMAC pavement	
Section ID:		
Date:	1/9/2012	
Station/milepost format:	Feet: 00 + 00	
Station/milepost begin:		0
Station/milepost end:		1000
Traffic direction:	East bound	

Default Input Level

Default input level — Level 3, Default and historical agency values.

Traffic

Initial two-way AADTT:	6000
Number of lanes in design direction:	2
Percent of trucks in design direction (%):	50
Percent of trucks in design lane (%):	95
Operational speed (mph):	65

Traffic -- Volume Adjustment Factors

Monthly Adjustment Factors (Level 3, Default MAF)

	Vehicle Class									
Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
February	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
March	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
April	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
May	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
June	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
July	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
August	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
September	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
October	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
November	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Figure D-3: Typical M-EPDG input summary for conventional PCC pavement.

December	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Vehicle Class Distribution
(Level 3, Default Distribution)

AADTT distribution by vehicle class

Class 4	1.8%
Class 5	24.6%
Class 6	7.6%
Class 7	0.5%
Class 8	5.0%
Class 9	31.3%
Class 10	9.8%
Class 11	0.8%
Class 12	3.3%
Class 13	15.3%

Hourly truck traffic distribution
by period beginning:

Midnight	2.3%	Noon	5.9%
1:00 am	2.3%	1:00 pm	5.9%
2:00 am	2.3%	2:00 pm	5.9%
3:00 am	2.3%	3:00 pm	5.9%
4:00 am	2.3%	4:00 pm	4.6%
5:00 am	2.3%	5:00 pm	4.6%
6:00 am	5.0%	6:00 pm	4.6%
7:00 am	5.0%	7:00 pm	4.6%
8:00 am	5.0%	8:00 pm	3.1%
9:00 am	5.0%	9:00 pm	3.1%
10:00 am	5.9%	10:00 pm	3.1%
11:00 am	5.9%	11:00 pm	3.1%

Traffic Growth Factor

Vehicle Class	Growth Rate	Growth Function
Class 4	3.0%	Compound
Class 5	3.0%	Compound
Class 6	3.0%	Compound
Class 7	3.0%	Compound
Class 8	3.0%	Compound
Class 9	3.0%	Compound
Class 10	3.0%	Compound
Class 11	3.0%	Compound
Class 12	3.0%	Compound
Class 13	3.0%	Compound

Traffic -- Axle Load Distribution Factors
Level 3: Default

Traffic -- General Traffic Inputs

Mean wheel location (inches from the lane marking):	18
Traffic wander standard deviation (in):	10
Design lane width (ft):	12

Number of Axles per Truck

Vehicle Class	Single Axle	Tandem Axle	Tridem Axle	Quad Axle
Class 4	1.62	0.39	0.00	0.00
Class 5	2.00	0.00	0.00	0.00
Class 6	1.02	0.99	0.00	0.00
Class 7	1.00	0.26	0.83	0.00
Class 8	2.38	0.67	0.00	0.00
Class 9	1.13	1.93	0.00	0.00
Class 10	1.19	1.09	0.89	0.00
Class 11	4.29	0.26	0.06	0.00
Class 12	3.52	1.14	0.06	0.00
Class 13	2.15	2.13	0.35	0.00

Figure D-3 (con’t): Typical M-EPDG input summary for conventional PCC pavement.

Axle Configuration

Average axle width (edge-to-edge) outside dimensions,ft):	8.5
Dual tire spacing (in):	12

Axle Configuration

Tire Pressure (psi) :	120

Average Axle Spacing

Tandem axle(psi):	51.6
Tridem axle(psi):	49.2
Quad axle(psi):	49.2

Wheelbase Truck Tractor

	Short	Medium	Long
Average Axle Spacing (ft)	12	15	18
Percent of trucks	33%	33%	34%

Climate

icm file:	C:\DG2002\Projects\Charlotte-Now.icm
Latitude (degrees.minutes)	35.13
Longitude (degrees.minutes)	-80.56
Elevation (ft)	724
Depth of water table (ft)	10

Structure--Design Features

Permanent curl/warp effective temperature difference (°F):	-10

Joint Design

Joint spacing (ft):	15
Sealant type:	Silicone
Dowel diameter (in):	1.75
Dowel bar spacing (in):	12

Edge Support	None
Long-term LTE(%):	n/a
Widened Slab (ft):	n/a

Base Properties

Base type:	Granular
Erodibility index:	Erosion Resistant (3)
PCC-Base Interface	Full friction contact
Loss of full friction (age in months):	360

Structure--ICM Properties

Surface shortwave absorptivity:	0.85

Structure--Layers

Layer 1 -- JPCP

General Properties

PCC material	JPCP
Layer thickness (in):	10
Unit weight (pcf):	150

Figure D-3 (con't): Typical M-EPDG input summary for conventional PCC pavement.

Poisson's ratio	0.2

Thermal Properties

Coefficient of thermal expansion (per F° x 10- 6):	5.6
Thermal conductivity (BTU/hr-ft-F°) :	1.25
Heat capacity (BTU/lb-F°):	0.28

Mix Properties

Cement type:	Type II
Cementitious material content (lb/yd^3):	575
Water/cement ratio:	0.32
Aggregate type:	Granite
PCC zero-stress temperature (F°)	Derived
Ultimate shrinkage at 40% R.H (microstrain)	Derived
Reversible shrinkage (% of ultimate shrinkage):	50
Time to develop 50% of ultimate shrinkage (days):	35
Curing method:	Curing compound

Strength Properties

Input level:	Level 3
28-day PCC modulus of rupture (psi):	716
28-day PCC compressive strength (psi):	n/a

Layer 2 -- Crushed stone

Unbound Material:	Crushed stone
Thickness(in):	8

Strength Properties

Input Level:	Level 3
Analysis Type:	ICM inputs (ICM Calculated Modulus)
Poisson's ratio:	0.35
Coefficient of lateral pressure,Ko:	0.5
Modulus (input) (psi):	30000

ICM Inputs

Gradation and Plasticity Index

Plasticity Index, PI:	1
Liquid Limit (LL)	6
Compacted Layer	No
Passing #200 sieve (%):	8.7
Passing #40	20
Passing #4 sieve (%):	44.7
D10(mm)	0.1035
D20(mm)	0.425
D30(mm)	1.306
D60(mm)	10.82
D90(mm)	46.19

Sieve	Percent Passing
0.001mm	
0.002mm	
0.020mm	
#200	8.7
#100	
#80	12.9
#60	

Figure D-3 (con’t): Typical M-EPDG input summary for conventional PCC pavement.

#50	
#40	20
#30	
#20	
#16	
#10	33.8
#8	
#4	44.7
3/8"	57.2
1/2"	63.1
3/4"	72.7
1"	78.8
1 1/2"	85.8
2"	91.6
2 1/2"	
3"	
3 1/2"	97.6
4"	97.6

Calculated/Derived Parameters

Maximum dry unit weight (pcf):	127.2 (derived)
Specific gravity of solids, Gs:	2.70 (derived)
Saturated hydraulic conductivity (ft/hr):	0.05054 (derived)
Optimum gravimetric water content (%):	7.4 (derived)
Calculated degree of saturation (%):	61.2 (calculated)

Soil water characteristic curve parameters: Default values

Parameters	Value
a	7.2555
b	1.3328
c	0.82422
Hr.	117.4

Layer 3 -- A-6

Unbound Material:	A-6
Thickness(in):	Semi-infinite

Strength Properties

Input Level:	Level 3
Analysis Type:	ICM inputs (ICM Calculated Modulus)
Poisson's ratio:	0.35
Coefficient of lateral pressure,Ko:	0.5
Modulus (input) (psi):	6000

ICM Inputs

Gradation and Plasticity Index

Plasticity Index, PI:	16
Liquid Limit (LL)	33
Compacted Layer	No
Passing #200 sieve (%):	63.2
Passing #40	82.4
Passing #4 sieve (%):	93.5
D10(mm)	0.000285
D20(mm)	0.0008125

Figure D-3 (con't): Typical M-EPDG input summary for conventional PCC pavement.

D30(mm)	0.002316
D60(mm)	0.05364
D90(mm)	1.922

Sieve	Percent Passing
0.001mm	
0.002mm	
0.020mm	
#200	63.2
#100	
#80	73.5
#60	
#50	
#40	82.4
#30	
#20	
#16	
#10	90.2
#8	
#4	93.5
3/8"	96.4
1/2"	97.4
3/4"	98.4
1"	99
1 1/2"	99.5
2"	99.8
2 1/2"	
3"	
3 1/2"	100
4"	100

Calculated/Derived Parameters

Maximum dry unit weight (pcf):	107.9 (derived)
Specific gravity of solids, Gs:	2.70 (derived)
Saturated hydraulic conductivity (ft/hr):	1.95e-005 (derived)
Optimum gravimetric water content (%):	17.1 (derived)
Calculated degree of saturation (%):	82.1 (calculated)

Soil water characteristic curve parameters: Default values

Parameters	Value
a	108.41
b	0.68007
c	0.21612
Hr.	500

Distress Model Calibration Settings - Rigid (new)

Faulting

Faulting Coefficients

C1	1.0184
C2	0.91656
C3	0.002185
C4	0.000884
C5	250
C6	0.4

Figure D-3 (con't): Typical M-EPDG input summary for conventional PCC pavement.

C7	1.83312
C8	400

Reliability (FAULT)

Std. Dev.	POWER(0.0097*FAULT,0.5178)+0.014

Cracking

Fatigue Coefficients

C1	2
C2	1.22

Cracking Coefficients

C4	1
C5	-1.98

Reliability (CRACK)

Std. Dev.	POWER(5.3116*CRACK,0.3903) + 2.99

IRI(jpcp)

C1	0.8203
C2	0.4417
C3	20.37
C4	1.4929
C5	25.24
Standard deviation in initial IRI (in/mile):	5.4

Figure D-3 (con't): Typical M-EPDG input summary for conventional PCC pavement.

Reliability Summary: Project RBMAC pavement analysis.dgp
2/13/2012 6:57 PM

Project: RBMAC pavement analysis.dgp
Reliability Summary

Performance Criteria	Distress Target	Reliability Target	Distress Predicted	Reliability Predicted	Acceptable
Terminal IRI (in/mi)	170	90	101	97.49	Pass
Transverse Cracking (% slabs cracked)	10	90	1.5	94.69	Pass
Mean Joint Faulting (in)	0.75	90	0.042	99.999	Pass

Figure D-4: Typical M-EPDG reliability summary for conventional PCC pavement.

Project: RBMAC test pavement 2012.dgp

General Information

Design Life	30 years
Pavement construction:	October, 2011
Traffic open:	January, 2012
Type of design	JPCP

Description:
RBMAC test pavement

Analysis Parameters

Performance Criteria

	Limit	Reliability
Initial IRI (in/mi)	63	
Terminal IRI (in/mi)	170	90
Transverse Cracking (% slabs cracked)	15	90
Mean Joint Faulting (in)	0.75	90

Location:	Charlotte, North Carolina	
Project ID:	RBMAC pavement	
Section ID:		
Date:	1/9/2012	
Station/milepost format:	Feet: 00 + 00	
Station/milepost begin:		0
Station/milepost end:		1000
Traffic direction:	East bound	

Default Input Level

Default input level — Level 3, Default and historical agency values.

Traffic

Initial two-way AADTT:	300
Number of lanes in design direction:	1
Percent of trucks in design direction (%):	100
Percent of trucks in design lane (%):	100
Operational speed (mph):	30

Traffic -- Volume Adjustment Factors

Monthly Adjustment Factors (Level 3, Default MAF)

	Vehicle Class									
Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
February	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
March	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
April	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
May	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
June	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
July	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
August	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
September	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
October	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
November	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Figure D-5: M-EPDG input summary for proposed RBMAC test pavement.

December	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Vehicle Class Distribution
(Level 3, Default Distribution)

AADTT distribution by vehicle class

Class 4	0.0%
Class 5	0.0%
Class 6	0.0%
Class 7	100.0%
Class 8	0.0%
Class 9	0.0%
Class 10	0.0%
Class 11	0.0%
Class 12	0.0%
Class 13	0.0%

Hourly truck traffic distribution
by period beginning:

Midnight	2.3%	Noon	5.9%
1:00 am	2.3%	1:00 pm	5.9%
2:00 am	2.3%	2:00 pm	5.9%
3:00 am	2.3%	3:00 pm	5.9%
4:00 am	2.3%	4:00 pm	4.6%
5:00 am	2.3%	5:00 pm	4.6%
6:00 am	5.0%	6:00 pm	4.6%
7:00 am	5.0%	7:00 pm	4.6%
8:00 am	5.0%	8:00 pm	3.1%
9:00 am	5.0%	9:00 pm	3.1%
10:00 am	5.9%	10:00 pm	3.1%
11:00 am	5.9%	11:00 pm	3.1%

Traffic Growth Factor

Vehicle Class	Growth Rate	Growth Function
Class 4	2.0%	Compound
Class 5	2.0%	Compound
Class 6	2.0%	Compound
Class 7	2.0%	Compound
Class 8	2.0%	Compound
Class 9	2.0%	Compound
Class 10	2.0%	Compound
Class 11	2.0%	Compound
Class 12	2.0%	Compound
Class 13	2.0%	Compound

Traffic -- Axle Load Distribution Factors
Level 3: Default

Traffic -- General Traffic Inputs

Mean wheel location (inches from the lane marking):	18
Traffic wander standard deviation (in):	10
Design lane width (ft):	12

Number of Axles per Truck

Vehicle Class	Single Axle	Tandem Axle	Tridem Axle	Quad Axle
Class 4	1.62	0.39	0.00	0.00
Class 5	2.00	0.00	0.00	0.00
Class 6	1.02	0.99	0.00	0.00
Class 7	1.00	0.26	0.83	0.00
Class 8	2.38	0.67	0.00	0.00
Class 9	1.13	1.93	0.00	0.00
Class 10	1.19	1.09	0.89	0.00
Class 11	4.29	0.26	0.06	0.00
Class 12	3.52	1.14	0.06	0.00
Class 13	2.15	2.13	0.35	0.00

Figure D-5 (con’t): M-EPDG input summary for proposed RBMAC test pavement.

Axle Configuration

Average axle width (edge-to-edge) outside dimensions,ft):	8.5
Dual tire spacing (in):	12

Axle Configuration

Tire Pressure (psi) :	120

Average Axle Spacing

Tandem axle(psi):	51.6
Tridem axle(psi):	49.2
Quad axle(psi):	49.2

Wheelbase Truck Tractor

	Short	Medium	Long
Average Axle Spacing (ft)	12	15	18
Percent of trucks	33%	33%	34%

Climate

icm file:	C:\DG2002\Projects\Charlotte-Now.icm
Latitude (degrees.minutes)	35.13
Longitude (degrees.minutes)	-80.56
Elevation (ft)	724
Depth of water table (ft)	10

Structure--Design Features

Permanent curl/warp effective temperature difference (°F):	-10

Joint Design

Joint spacing (ft):	12
Sealant type:	None
Dowel diameter (in):	1.75
Dowel bar spacing (in):	12

Edge Support

Edge Support	Tied PCC shoulder, Widened slab
Long-term LTE(%):	50
Widened Slab (ft):	12

Base Properties

Base type:	Granular
Erodibility index:	Erosion Resistant (3)
PCC-Base Interface	Full friction contact
Loss of full friction (age in months):	360

Structure--ICM Properties

Surface shortwave absorptivity:	0.85

Structure--Layers

Layer 1 -- JPCP

General Properties

PCC material	JPCP
Layer thickness (in):	9.25
Unit weight (pcf):	130

Figure D-5 (con't): M-EPDG input summary for proposed RBMAC test pavement.

Poisson's ratio	0.18

Thermal Properties

Coefficient of thermal expansion (per F° x 10- 6):	4.4
Thermal conductivity (BTU/hr-ft-F°) :	0.533
Heat capacity (BTU/lb-F°):	0.2

Mix Properties

Cement type:	Type II
Cementitious material content (lb/yd^3):	575
Water/cement ratio:	0.32
Aggregate type:	Rhyolite
PCC zero-stress temperature (F°)	Derived
Ultimate shrinkage at 40% R.H (microstrain)	Derived
Reversible shrinkage (% of ultimate shrinkage):	50
Time to develop 50% of ultimate shrinkage (days):	35
Curing method:	Curing compound

Strength Properties

Input level:	Level 3
28-day PCC modulus of rupture (psi):	716
28-day PCC compressive strength (psi):	n/a

Layer 2 -- Crushed stone

Unbound Material:	Crushed stone
Thickness(in):	12

Strength Properties

Input Level:	Level 3
Analysis Type:	ICM inputs (ICM Calculated Modulus)
Poisson's ratio:	0.35
Coefficient of lateral pressure,Ko:	0.5
Modulus (input) (psi):	30000

ICM Inputs

Gradation and Plasticity Index

Plasticity Index, PI:	1
Liquid Limit (LL)	6
Compacted Layer	No
Passing #200 sieve (%):	8.7
Passing #40	20
Passing #4 sieve (%):	44.7
D10(mm)	0.1035
D20(mm)	0.425
D30(mm)	1.306
D60(mm)	10.82
D90(mm)	46.19

Sieve	Percent Passing
0.001mm	
0.002mm	
0.020mm	
#200	8.7
#100	
#80	12.9
#60	

Figure D-5 (con't): M-EPDG input summary for proposed RBMAC test pavement.

#50	
#40	20
#30	
#20	
#16	
#10	33.8
#8	
#4	44.7
3/8"	57.2
1/2"	63.1
3/4"	72.7
1"	78.8
1 1/2"	85.8
2"	91.6
2 1/2"	
3"	
3 1/2"	97.6
4"	97.6

Calculated/Derived Parameters

Maximum dry unit weight (pcf):	127.2 (derived)
Specific gravity of solids, Gs:	2.70 (derived)
Saturated hydraulic conductivity (ft/hr):	0.05054 (derived)
Optimum gravimetric water content (%):	7.4 (derived)
Calculated degree of saturation (%):	61.2 (calculated)

Soil water characteristic curve parameters: Default values

Parameters	Value
a	7.2555
b	1.3328
c	0.82422
Hr.	117.4

Layer 3 -- A-4

Unbound Material:	A-4
Thickness(in):	Semi-infinite

Strength Properties

Input Level:	Level 3
Analysis Type:	ICM inputs (ICM Calculated Modulus)
Poisson's ratio:	0.35
Coefficient of lateral pressure,Ko:	0.5
Modulus (input) (psi):	6000

ICM Inputs

Gradation and Plasticity Index

Plasticity Index, PI:	5
Liquid Limit (LL)	21
Compacted Layer	No
Passing #200 sieve (%):	60.6
Passing #40	82.7
Passing #4 sieve (%):	93
D10(mm)	0.0002981
D20(mm)	0.0008889

Figure D-5 (con't): M-EPDG input summary for proposed RBMAC test pavement.

D30(mm)	0.00265
D60(mm)	0.07024
D90(mm)	2.057

Sieve	Percent Passing
0.001mm	
0.002mm	
0.020mm	
#200	60.6
#100	
#80	73.9
#60	
#50	
#40	82.7
#30	
#20	
#16	
#10	89.9
#8	
#4	93
3/8"	95.6
1/2"	96.7
3/4"	98
1"	98.7
1 1/2"	99.4
2"	99.6
2 1/2"	
3"	
3 1/2"	99.8
4"	99.8

Calculated/Derived Parameters

Maximum dry unit weight (pcf):	118.4 (derived)
Specific gravity of solids, Gs:	2.70 (derived)
Saturated hydraulic conductivity (ft/hr):	8.325e-006 (derived)
Optimum gravimetric water content (%):	11.8 (derived)
Calculated degree of saturation (%):	75.3 (calculated)

Soil water characteristic curve parameters: Default values

Parameters	Value
a	68.838
b	0.99829
c	0.47572
Hr.	500

Distress Model Calibration Settings - Rigid (new)

Faulting

Faulting Coefficients

C1	1.0184
C2	0.91656
C3	0.002185
C4	0.000884
C5	250
C6	0.4

Figure D-5 (con't): M-EPDG input summary for proposed RBMAC test pavement.

C7	1.83312
C8	400

Reliability (FAULT)

Std. Dev.	POWER(0.0097*FAULT,0.5178)+0.014

Cracking

Fatigue Coefficients

C1	2
C2	1.22

Cracking Coefficients

C4	1
C5	-1.98

Reliability (CRACK)

Std. Dev.	POWER(5.3116*CRACK,0.3903) + 2.99

IRI(jpcp)

C1	0.8203
C2	0.4417
C3	20.37
C4	1.4929
C5	25.24
Standard deviation in initial IRI (in/mile):	5.4

Figure D-5 (con't): M-EPDG input summary for proposed RBMAC test pavement.

Reliability Summary: Project RBMAC test pavement 2012.dgp
2/13/2012 7:02 PM

Project: RBMAC test pavement 2012.dgp Reliability Summary

Performance Criteria	Distress Target	Reliability Target	Distress Predicted	Reliability Predicted	Acceptable
Terminal IRI (in/mi)	170	90	72.4	99.99	Pass
Transverse Cracking (% slabs cracked)	15	90	4.2	95.51	Pass
Mean Joint Faulting (in)	0.75	90	0.004	99.999	Pass

Figure D-6: M-EPDG reliability summary for proposed RBMAC test pavement.

Input Summary: Project RBMAC test pavement 2012.dgp
2/13/2012 7:04 PM

1 of 7

Project: RBMAC test pavement 2012.dgp

General Information

Design Life	30 years
Pavement construction:	October, 2011
Traffic open:	January, 2012
Type of design	JPCP

Description:
RBMAC test pavement - control section with granite aggregate

Analysis Parameters

Performance Criteria

	Limit	Reliability
Initial IRI (in/mi)	63	
Terminal IRI (in/mi)	170	90
Transverse Cracking (% slabs cracked)	15	90
Mean Joint Faulting (in)	0.75	90

Location:	Charlotte, North Carolina
Project ID:	RBMAC pavement
Section ID:	
Date:	1/9/2012
Station/milepost format:	Feet: 00 + 00
Station/milepost begin:	0
Station/milepost end:	1000
Traffic direction:	East bound

Default Input Level

Default input level	Level 3, Default and historical agency values.

Traffic

Initial two-way AADTT:	300
Number of lanes in design direction:	1
Percent of trucks in design direction (%):	100
Percent of trucks in design lane (%):	100
Operational speed (mph):	30

Traffic -- Volume Adjustment Factors

Monthly Adjustment Factors (Level 3, Default MAF)

Month	Vehicle Class									
	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
February	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
March	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
April	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
May	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
June	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
July	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
August	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
September	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
October	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
November	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Figure D-7: M-EPDG input summary for proposed conventional PCC (control) test pavement.

December	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Vehicle Class Distribution
(Level 3, Default Distribution)

AADTT distribution by vehicle class

Class 4	0.0%
Class 5	0.0%
Class 6	0.0%
Class 7	100.0%
Class 8	0.0%
Class 9	0.0%
Class 10	0.0%
Class 11	0.0%
Class 12	0.0%
Class 13	0.0%

Hourly truck traffic distribution
by period beginning:

Midnight	2.3%	Noon	5.9%
1:00 am	2.3%	1:00 pm	5.9%
2:00 am	2.3%	2:00 pm	5.9%
3:00 am	2.3%	3:00 pm	5.9%
4:00 am	2.3%	4:00 pm	4.6%
5:00 am	2.3%	5:00 pm	4.6%
6:00 am	5.0%	6:00 pm	4.6%
7:00 am	5.0%	7:00 pm	4.6%
8:00 am	5.0%	8:00 pm	3.1%
9:00 am	5.0%	9:00 pm	3.1%
10:00 am	5.9%	10:00 pm	3.1%
11:00 am	5.9%	11:00 pm	3.1%

Traffic Growth Factor

Vehicle Class	Growth Rate	Growth Function
Class 4	2.0%	Compound
Class 5	2.0%	Compound
Class 6	2.0%	Compound
Class 7	2.0%	Compound
Class 8	2.0%	Compound
Class 9	2.0%	Compound
Class 10	2.0%	Compound
Class 11	2.0%	Compound
Class 12	2.0%	Compound
Class 13	2.0%	Compound

Traffic -- Axle Load Distribution Factors
Level 3: Default

Traffic -- General Traffic Inputs

Mean wheel location (inches from the lane marking):	18
Traffic wander standard deviation (in):	10
Design lane width (ft):	12

Number of Axles per Truck

Vehicle Class	Single Axle	Tandem Axle	Tridem Axle	Quad Axle
Class 4	1.62	0.39	0.00	0.00
Class 5	2.00	0.00	0.00	0.00
Class 6	1.02	0.99	0.00	0.00
Class 7	1.00	0.26	0.83	0.00
Class 8	2.38	0.67	0.00	0.00
Class 9	1.13	1.93	0.00	0.00
Class 10	1.19	1.09	0.89	0.00
Class 11	4.29	0.26	0.06	0.00
Class 12	3.52	1.14	0.06	0.00
Class 13	2.15	2.13	0.35	0.00

Figure D-7 (con’t): M-EPDG input summary for proposed conventional PCC (control) test pavement.

Axle Configuration

Average axle width (edge-to-edge) outside dimensions,ft): 8.5
Dual tire spacing (in): 12

Axle Configuration

Tire Pressure (psi) : 120

Average Axle Spacing

Tandem axle(psi): 51.6
Tridem axle(psi): 49.2
Quad axle(psi): 49.2

Wheelbase Truck Tractor

	Short	Medium	Long
Average Axle Spacing (ft)	12	15	18
Percent of trucks	33%	33%	34%

Climate

icm file: C:\DG2002\Projects\Charlotte-Now.icm
Latitude (degrees.minutes) 35.13
Longitude (degrees.minutes) -80.56
Elevation (ft) 724
Depth of water table (ft) 10

Structure--Design Features

Permanent curl/warp effective temperature difference (°F): -10

Joint Design

Joint spacing (ft): 12
Sealant type: None
Dowel diameter (in): 1.75
Dowel bar spacing (in): 12

Edge Support Tied PCC shoulder, Widened slab

Long-term LTE(%): 50
Widened Slab (ft): 12

Base Properties

Base type: Granular
Erodibility index: Erosion Resistant (3)
PCC-Base Interface Full friction contact
Loss of full friction (age in months): 360

Structure--ICM Properties

Surface shortwave absorptivity: 0.85

Structure--Layers

Layer 1 -- JPCP

General Properties

PCC material JPCP
Layer thickness (in): 10.5
Unit weight (pcf): 150

Figure D-7 (con’t): M-EPDG input summary for proposed conventional PCC (control) test pavement.

Poisson's ratio	0.2

Thermal Properties

Coefficient of thermal expansion (per F° x 10- 6):	5.6
Thermal conductivity (BTU/hr-ft-F°) :	1.25
Heat capacity (BTU/lb-F°):	0.28

Mix Properties

Cement type:	Type II
Cementitious material content (lb/yd^3):	575
Water/cement ratio:	0.32
Aggregate type:	Granite
PCC zero-stress temperature (F°)	Derived
Ultimate shrinkage at 40% R.H (microstrain)	Derived
Reversible shrinkage (% of ultimate shrinkage):	50
Time to develop 50% of ultimate shrinkage (days):	35
Curing method:	Curing compound

Strength Properties

Input level:	Level 3
28-day PCC modulus of rupture (psi):	716
28-day PCC compressive strength (psi):	n/a

Layer 2 -- Crushed stone

Unbound Material:	Crushed stone
Thickness(in):	12

Strength Properties

Input Level:	Level 3
Analysis Type:	ICM inputs (ICM Calculated Modulus)
Poisson's ratio:	0.35
Coefficient of lateral pressure,Ko:	0.5
Modulus (input) (psi):	30000

ICM Inputs

Gradation and Plasticity Index

Plasticity Index, PI:	1
Liquid Limit (LL)	6
Compacted Layer	No
Passing #200 sieve (%):	8.7
Passing #40	20
Passing #4 sieve (%):	44.7
D10(mm)	0.1035
D20(mm)	0.425
D30(mm)	1.306
D60(mm)	10.82
D90(mm)	46.19

Sieve	Percent Passing
0.001mm	
0.002mm	
0.020mm	
#200	8.7
#100	
#80	12.9
#60	

Figure D-7 (con't): M-EPDG input summary for proposed conventional PCC (control) test pavement.

#50	
#40	20
#30	
#20	
#16	
#10	33.8
#8	
#4	44.7
3/8"	57.2
1/2"	63.1
3/4"	72.7
1"	78.8
1 1/2"	85.8
2"	91.6
2 1/2"	
3"	
3 1/2"	97.6
4"	97.6

Calculated/Derived Parameters

Maximum dry unit weight (pcf): 127.2 (derived)
Specific gravity of solids, Gs: 2.70 (derived)
Saturated hydraulic conductivity (ft/hr): 0.05054 (derived)
Optimum gravimetric water content (%): 7.4 (derived)
Calculated degree of saturation (%): 61.2 (calculated)

Soil water characteristic curve parameters: Default values

Parameters	Value
a	7.2555
b	1.3328
c	0.82422
Hr.	117.4

Layer 3 -- A-4

Unbound Material: A-4
Thickness(in): Semi-infinite

Strength Properties

Input Level: Level 3
Analysis Type: ICM inputs (ICM Calculated Modulus)
Poisson's ratio: 0.35
Coefficient of lateral pressure,Ko: 0.5
Modulus (input) (psi): 6000

ICM Inputs

Gradation and Plasticity Index

Plasticity Index, PI: 5
Liquid Limit (LL) 21
Compacted Layer No
Passing #200 sieve (%): 60.6
Passing #40 82.7
Passing #4 sieve (%): 93
D10(mm) 0.0002981
D20(mm) 0.0008889

Figure D-7 (con't): M-EPDG input summary for proposed conventional PCC (control) test pavement.

D30(mm)	0.00265
D60(mm)	0.07024
D90(mm)	2.057

Sieve	Percent Passing
0.001mm	
0.002mm	
0.020mm	
#200	60.6
#100	
#80	73.9
#60	
#50	
#40	82.7
#30	
#20	
#16	
#10	89.9
#8	
#4	93
3/8"	95.6
1/2"	96.7
3/4"	98
1"	98.7
1 1/2"	99.4
2"	99.6
2 1/2"	
3"	
3 1/2"	99.8
4"	99.8

Calculated/Derived Parameters

Maximum dry unit weight (pcf):	118.4 (derived)
Specific gravity of solids, Gs:	2.70 (derived)
Saturated hydraulic conductivity (ft/hr):	8.325e-006 (derived)
Optimum gravimetric water content (%):	11.8 (derived)
Calculated degree of saturation (%):	75.3 (calculated)

Soil water characteristic curve parameters: Default values

Parameters	Value
a	68.838
b	0.99829
c	0.47572
Hr.	500

Distress Model Calibration Settings - Rigid (new)

Faulting

Faulting Coefficients

C1	1.0184
C2	0.91656
C3	0.002185
C4	0.000884
C5	250
C6	0.4

Figure D-7 (con't): M-EPDG input summary for proposed conventional PCC (control) test pavement.

C7	1.83312
C8	400
Reliability (FAULT)	
Std. Dev.	POWER(0.0097*FAULT,0.5178)+0.014
Cracking	
Fatigue Coefficients	
C1	2
C2	1.22
Cracking Coefficients	
C4	1
C5	-1.98
Reliability (CRACK)	
Std. Dev.	POWER(5.3116*CRACK,0.3903) + 2.99
IRI(jpcp)	
C1	0.8203
C2	0.4417
C3	20.37
C4	1.4929
C5	25.24
Standard deviation in initial IRI (in/mile):	5.4

Figure D-7 (con't): M-EPDG input summary for proposed conventional PCC (control) test pavement.

Reliability Summary: Project RBMAC test pavement 2012.dgp
2/13/2012 7:04 PM

Project: RBMAC test pavement 2012.dgp
Reliability Summary

Performance Criteria	Distress Target	Reliability Target	Distress Predicted	Reliability Predicted	Acceptable
Terminal IRI (in/mi)	170	90	69.9	99.999	Pass
Transverse Cracking (% slabs cracked)	15	90	3.5	97.03	Pass
Mean Joint Faulting (in)	0.75	90	0.002	99.999	Pass

Figure D-8: M-EPDG reliability summary for proposed conventional PCC (control) test pavement.