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Efficacy of a Ternary Blend in Improving Bridge Deck Concrete Durability in Tennessee

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Dedication

I dedicate this thesis to my parents, Bob and Cindy Nidiffer, who continue to inspire and support me in all of my endeavors. This work would not be possible without their positive influence on my life. I also dedicate this thesis to Kiersten, my sister Abby, and my grandparents – Robert Nidiffer, Mary Ellen Nidiffer, and Barbara Clark – for offering relentless support and encouragement throughout my life. Additionally, I dedicate this thesis in memory of my grandfather, Bill Clark, who had always hoped that I would become a civil engineer.

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Abstract

A study related to the durability of bridge deck concrete in the state of Tennessee has been ongoing at the University of Tennessee (UT) for the past decade. The most recent phase of this research was begun in the fall of 2009 with a focus on developing assessment criteria and methodology to assess the durability of bridge deck concrete in the state of Tennessee. The methodology that was used to assess the durability of Tennessee bridge deck concrete was to determine the concrete's resistance to chloride ion penetration by way of two test methods, the Surface Resistivity (SR) test and the Rapid Chloride Ion Penetration (RCP) test. Current guidelines set forth by the Tennessee Department of Transportation (TDOT) require that the "Class D" concrete mixture be placed on all bridge deck applications in the state of Tennessee.   SR and RCP tests have been performed on "Class D" concrete cylinders from various bridge deck placements across the state for the past two and a half years. Results from the tests indicate that the current "Class D" concrete mixture is not adequate in resisting chloride ion penetration at satisfactory levels. Thus, it was decided to propose a ternary blended concrete mixture for use on Tennessee bridge decks in order to better resist chloride ion penetration and, as a result, improve the durability of bridge deck concrete in Tennessee. Ternary concrete mixtures have been found to offer many benefits to both the strength and durability properties of concrete. A concrete mixture is classified as "ternary" when it contains three different types of cementitious materials. While TDOT's current specifications do not specifically prohibit the use of a ternary "Class D" mixture, the "Class D" concrete mixtures currently being placed on bridge decks are

typically 100% portland cement mixtures or are binary mixtures containing mostly portland cement with a relatively small amount of fly ash. Ternary blended laboratory samples were created to compare SR and RCP values to the typical "Class D" mixtures. Results from the tests, as well as results reported in technical literature, indicate that ternary mixtures have significantly better resistance to chloride ion penetration as compared to the typical "Class D" mixtures.

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1.0   Introduction

As the United States maintains a steadfast emphasis on improving the national infrastructure, the condition of bridges across the country continues to present major concerns to both the government and general public. According to ASCE's 2009 Report Card for America's Infrastructure, "More than 26%, or one in four, of the nation's bridges are either structurally deficient or functionally obsolete" (17). While this statistic can be attributed to a variety of factors, the durability of concrete bridge decks is often a key determinant in the overall condition and longevity of a bridge. More durable concrete bridge decks will help to provide longer service lives for bridges, reduced maintenance costs, and fewer traffic delays due to construction on the bridge deck.

On September 1, 2009, the University of Tennessee (UT) began a research project in conjunction with the Tennessee Department of Transportation (TDOT) assessing the durability of concrete bridge decks across the state of Tennessee. Beginning on February 22, 2010, through the present, UT has received concrete cylinders from numerous bridge decks cast in all four of TDOT's regions. The essential scope of the testing that has been performed by the UT research team relates to the concrete's ability to resist chloride ion penetration. The two tests that have been performed on each set of samples are the Rapid Chloride Ion Penetration (RCP) test and the Surface Resistivity (SR) test. The RCP test is a well-established procedure and is described in detail in both ASTM C1202 and AASHTO T277. This test provides a rapid indication of the concrete's resistance to the penetration of chloride ions by determining the electrical conductance of the concrete (16). According to the ASTM

standard, this test is intended to correlate strongly with the widely accepted ponding test. Although the RCP test is much quicker to perform than the ponding test, it should by no means be considered rapid. The RCP test takes almost twenty-four hours for preparation of the sample and then another six hours to run the actual test. The Surface Resistivity (SR) test, on the other hand, is a much quicker and easier test to perform. On average, it takes a total of about fifteen minutes to perform. However, the SR test is not as well established as the RCP test. The SR test does not conform to an ASTM standard, but it is recognized by the American Association of State Highway and Transportation Officials (AASHTO) and adheres to the AASHTO Provisional Standard TP 95-11. The SR test provides an indication of the concrete's permeability by measuring the electrical resistivity of the concrete (8).

In order to better understand the relationship between the RCP and SR tests, it is important to understand the relationship between the results that are produced from each test. The RCP test produces values that represent the electrical conductance of the test specimens in terms of Coulombs passed through in a 6-hour period. The SR test produces values that represent the resistivity of the test cylinders in terms of kilo ohm centimeters, or kohm-cm. A simple way to at least partially explain the relationship between these two tests is to consider the basic principle of Ohm's Law which states that current is inversely related to resistance. A thorough explanation of the relationship between RCP and SR test results is presented in an M.S. thesis by Eric Ryan which was done as a part of the first phase of this research (59).

Results for the RCP and SR tests through May 2012 have shown strong correlation with one another. The main research concern in UT's study of bridge deck concrete durability is now beginning to shift from a focus on the correlation between these two tests to an emphasis on what can be done to improve the concrete placed on bridge decks across the state of Tennessee. The current TDOT specification for concrete placed on bridge decks in Tennessee requires a "Class D" mixture. The "Class D" concrete mixture prescribed by TDOT currently contains a general guideline for compressive strength, water-to-cementitious materials (w/cm) ratio, air content, and slump, along with prescribing a minimum cementitious materials content of 620 lbs/yd^3, but it gives no guidance on chloride ion penetration limits. The results from the RCP and SR tests thus far in the research have shown values that leave much to be desired in terms of producing low permeability concrete with high resistance to chloride ion penetration.

A growing trend in the construction industry is the use of ternary blended concrete mixtures. The American Concrete Institute (ACI) simply defines a ternary concrete mixture as concrete containing three cementitious materials. Ternary mixtures typically contain cement and two other supplementary cementitious materials (SCMs). Common SCMs used in concrete mixtures are fly ash, ground granulated blast furnace slag (GGBFS), and silica fume. GGBFS is often referred to as slag cement, or simply slag, in the concrete industry. Ternary blended concrete mixtures offer many advantages in terms of both strength and durability. This thesis investigates the efficacy

of using a ternary blend mixture and proposes an alternative to the currently used

"Class D" mixture.

2.0   Literature Review

This chapter provides a literature review of concrete durability and the factors that affect concrete durability. A strong emphasis is placed on the corrosion of steel in concrete and the effects of chloride ion penetration on that corrosion and on concrete durability in general. A brief discussion of the surface resistivity (SR) and rapid chloride ion penetration (RCP) tests is also included in this chapter. In addition, the effects of supplementary cementitious materials (SCMs), specifically slag cement, and ternary concrete mixtures on concrete durability and chloride ion penetration in concrete are discussed in this chapter.

2.1   Concrete Durability

ACI Committee 201 defines durability of concrete as the concrete's ability to resist weathering action, chemical attack, abrasion, and any other process of deterioration. "Durable concrete will retain its original form, quality, and serviceability when exposed to its environment" (2). It is widely discussed among people in the concrete industry about what is the most important factor that affects concrete durability. It was a common belief for many years that concrete strength is a direct indicator of concrete durability. "Historically, the perception that there is a direct relation between the strength of concrete and durability has been at the heart of most of the approaches that have been made toward proportioning of durable concrete mixtures" (44). This pattern of thinking is no longer prevalent among many concrete researchers, and a stronger emphasis is being placed on other factors such as low permeability concrete with minimal cracks. "An ideal durable structure needs to have a low permeability

concrete with a proper air-void system, no cracks, and not be subject to deleterious chemical reactions" (53). Durable concrete on bridge decks is important because it can increase the bridge's service life as well as decrease the bridge's life cycle costs due to decreased maintenance, repair, and rehabilitation costs over its period of service.

2.2   Factors Affecting Concrete Durability

A variety of factors can produce harmful effects on concrete durability. Several important factors that affect concrete durability include, but are not limited to, corrosion of steel, chemical attack, physical deterioration, and concrete cracking, specifically related to shrinkage, and freezing and thawing of the concrete. An extensive study of current literature suggests that these factors are of utmost importance in terms of concrete durability.

2.2.1   Corrosion of Steel

The condition of concrete bridge decks across the United States is continuing to deteriorate in large part due to the corrosion of the steel reinforcement embedded in the concrete. "Generally, corrosion related costs are estimated to be in the range of 3 to 5% of the Gross National Product (GNP) in the U.S., Western Europe, and Japan, 25% of which can be ascribed to corrosion of steel reinforcement in concrete structures" (20). In order to understand the process of steel corrosion in concrete structures, it is important to understand how concrete and steel interact with one another before being exposed to harmful chemicals. Metals are known to corrode when exposed to acids; however, concrete is a material that is highly alkaline, the opposite of acidic. Therefore,

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when steel and concrete exist solely with one another, corrosion is not an issue. In fact, the relationship between the steel and concrete creates a "passive" layer around the steel that protects the steel from harmful substances. Unfortunately, when reinforced concrete is subjected to extreme conditions, such as saltwater from the ocean or deicing salts, the "passive layer" is not capable of withstanding the corrosive attack. A serious problem for bridge deck concrete durability is the spalling of the concrete. Once the steel begins to corrode, the corrosion process produces an expansive force that causes the concrete to spall above the steel. Adequate cover distances and the use of low-permeability concrete are beneficial in ensuring concrete durability in these cases (2). Two main causes of corrosion in steel exist in concrete: carbonation and chloride ion penetration (21). These two corrosion mechanisms are unusual in the sense that they attack the integrity of the reinforcing steel, not the concrete, by passing aggressive chemicals directly through the pores in the concrete to reach the steel. Other types of chemical attacks such as sulfate attack and alkali-silica reaction (ASR) as discussed in Section 2.2.2 are harmful to the concrete before they are harmful to the steel.

Carbonation

Carbonation results from carbon dioxide gas, either in the atmosphere or in water, interacting with the alkaline hydroxides in the concrete. The reaction that takes place is accompanied by shrinkage (2). As stated by ACI Committee 201, carbonation can be either beneficial or harmful depending on the time, rate, and extent to which they occur. Carbonation can be beneficial when it occurs intentionally during the production process causing improvement in the strength, hardness, and dimensional stability of the

products in the concrete (2). However, carbonation is often negative in a sense that it can result in deterioration and a decrease in the pH of the cement paste leading to corrosion of the steel reinforcement (2). The method of transport for carbonation through concrete is typically through the process of diffusion. Damage caused by carbonation is most common in areas with little concrete cover over the reinforcing steel. This mechanism of steel corrosion is typically not of highest concern on concrete bridge decks. A review of literature concludes that carbonation is rare on modern highway bridges and other civil engineering structures due to low water cement ratios, high cement contents with good compaction and curing, and enough cover to prevent the carbonation from advancing into the concrete to any significant extent (21). The w/cm ratio appears to be the key factor in carbonation damage. While the rate of carbonation will increase with a decrease in total cement content, the main variation in carbonation rates is due to change in the w/cm ratio (19). At the w/cm ratio of 0.40 that is currently used on Tennessee bridge decks, it is reasonable to suggest that, with sufficient concrete cover, a total cement content of 500 to 600 lbs/yd^3 would provide adequate resistance to carbonation (75).

Chloride Ion Penetration

Chloride ion penetration, also known as chloride ingress, is a key factor in the corrosion of steel on concrete bridge decks. It is believed to be the most widely experienced distress in concrete structures (53). Chloride ingress is an important issue for Tennessee bridge decks due to the state's relatively cold winters that require deicing salts to be applied to the icy bridge decks in order to improve driving conditions.

Several mechanisms of chloride transport exist that allow the chloride ions to penetrate through the concrete and reach the reinforcing steel. The three main processes that allow chloride ions to penetrate through concrete are diffusion, hydrostatic pressure, and capillary absorption (60). Diffusion occurs when the transport of chloride is driven by the difference of the concentration of chloride in various zones. The chloride will always diffuse into zones with smaller chloride concentration (56). A chloride ion concentration gradient must exist and the concrete must have a continuous liquid phase in order for diffusion to occur. Diffusion is the principal method that brings chloride ions into the concrete to the level of the reinforcing steel. A second mechanism that can cause chloride ingress in concrete is hydrostatic pressure, also known as permeation. This can occur when an applied hydraulic head, such as the saltwater from the ocean, exists on one face of the concrete and chlorides are present. This hydraulic head will allow the chlorides to permeate into the concrete. A situation where a hydraulic head occurs on a highway structure is not common (60). Lastly, a third mechanism of chloride ingress in concrete is capillary absorption. This process occurs due to the wetting and drying cycles that the concrete experiences when exposed to the environment. When water comes into contact with a dry surface, it is drawn into the pore structure through capillary suction. In the case of chloride ingress, water can sometimes contain chlorides which will penetrate into the concrete through capillary absorption. The depth that capillary absorption occurs in concrete is relatively small; therefore, the chloride ions will not typically reach to the level of the reinforcing steel on their own unless the concrete is of extremely poor quality or the cover of the steel is

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shallow. Capillary absorption does, however, bring the chloride ions to some depth in the concrete causing the distance that they must diffuse to reach the reinforcing steel to decrease (67). This process is more common than hydrostatic pressure on bridges, but diffusion is clearly the principal mechanism of chloride ingress in concrete.

2.2.2   Chemical Attack

Concrete is exposed to a variety of both internal and external chemical attacks throughout its service life. Sulfate attack is a type of chemical attack that is typically considered to originate from an external source but can also exist internally in concrete. This type of attack is a common concern in concrete durability. Naturally occurring sulfates, such as sodium, potassium, calcium, or magnesium, can attack hardened concrete and cause problems for the concrete in the future (2). These sulfates can often be found in soil or dissolved in groundwater adjacent to concrete structures which would cause the attack to originate externally. Internal sulfate attacks occur when the source of sulfate ions is located within the concrete such as in the case of delayed ettringite formation (DEF) (46). Other internal chemical attacks, such as chemical reactions of aggregates, are also a hindrance towards maintaining durable concrete. Alkali-silica reaction (ASR) is arguably the most well-known internal chemical attack that concrete is exposed to.

Sulfate Attack

Sulfate attack is often considered to be the distress caused by a chemical reaction between sulfate ions and the hydration products of portland cement, with consequential damage by ettringite and gypsum (34). Ettringite (calcium aluminate

trisulfate 32-hydrate) and gypsum (calcium sulfate dehydrate) are the two best known chemical consequences of sulfate attack. The formation of ettringite can result in an increase of concrete volume which can lead to expansion and cracking. The formation of gypsum often leads to softening of the concrete and loss of strength (2). Delayed ettringite formation (DEF) is an example of sulfate attack that originates internally in the concrete. Unlike external sulfate attacks that often come from sulfates in soil or groundwater, DEF occurs when either a gypsum-contaminated aggregate or a type of cement containing unusually large sulfate content is used in the concrete production (46). The "delayed" term in delayed ettringite formation refers to the fact that this type of formation does not occur until later on during the service life of the structure. DEF is not likely to occur unless the concrete is exposed to high temperatures during curing that would cause the sulfate ions to react internally with the pore solution in the hardened concrete (28).

Alkali-Silica Reaction

Two main types of alkali-aggregate reactions can typically exist in concrete: alkali-silica reaction (ASR) and alkali-carbonate reaction (ACR). Unlike ASR, concrete durability problems related to ACR are restricted to a few isolated locations around the world (54). It is primarily for this reason that ASR is the main concern in terms of alkali-aggregate reactions as related to concrete durability. Alkali-silica reaction is a chemical reaction between the hydroxyl ions located within the concrete's pore water solution and certain silica minerals located within the aggregate. Abnormal concrete expansion is a common result from this type of reaction which can lead to excessive cracking. "Such

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cracking is known to be highly detrimental to the serviceability behavior of concrete structures leading to loss of tensile strength, and, in exceptional circumstances, to failure" (63). The effects of ASR on the engineering properties of concrete can be highly destructive. The compressive strength, flexural strength, and dynamic modulus of concrete can be greatly reduced when ASR expansion occurs in concrete. Studies have shown that, at as little as 0.1 percent expansion, the loss in the concrete's flexural strength can be as high 50 percent, and the dynamic modulus of the concrete can be reduced by as much as 20 percent. According to these same studies, an expansion of about 0.6 percent after one year would cause reductions in compressive strength, flexural strength, and dynamic modulus by values of 40 percent, 75 percent, and 60 percent, respectively (62). Practical ways to prevent ASR expansion in concrete are to avoid the use of reactive aggregates, limit the amount of alkali in the cement, or to use pozzolanic materials such as slag cement or fly ash in the concrete mixture (74). Although ASR is not a widespread problem in the construction industry, it is still common enough, even in Tennessee, to take the proper precautions in order to prevent it (64).

2.2.3   Physical Deterioration

Physical deterioration of concrete can exist in a variety of forms. A few examples of physical deterioration on concrete structures include abrasion, erosion, and scaling. ACI Committee 116 defines abrasion as the wearing away of a surface by a rubbing or friction process. This form of physical deterioration is of high concern on highway bridge decks due to the large amount of vehicular traffic that the decks are exposed to

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on regular occasion. Research shows that a strong relationship exists between the compressive strength of concrete and the concrete's resistance to abrasion, with the resistance to abrasion increasing as compressive strength increases (74).

Erosion can be described as concrete deterioration brought about by the abrasive action of fluids or solids in motion. Erosion can also be caused by cavitation or chemical attack. ACI Committee 210 defines cavitation as the formation of bubbles or cavities in a liquid. In hydraulic structures, the liquid applied to the concrete is water, and the cavities are filled with water vapor and air. Pitting of concrete is a common result of erosion caused by cavitation. The mechanisms of chemical attacks and their effects on concrete erosion can become an issue when the concrete is highly permeable or when the concrete is subjected to highly acidic environments (3). The resistance of concrete to erosion is important particularly in hydraulic structures in which concrete is subjected to moving water carrying solid particles (74).

ACI Committee 116 defines concrete scaling as local flaking or peeling away of the near-surface portion of hardened concrete or mortar (1). The concern of concrete scaling continues to be a serious issue on concrete bridge decks. "Scaling of concrete pavements, sidewalks, driveways, decks, and other slabs is a common problem in outdoor construction exposed to severe winter weather and deicing salts" (37). Scaling problems with concrete are often attributed to the quality of the concrete provided by the supplier, construction practices, or the use of deicing salts for snow removal. Of these three factors, the use of deicing salts is the most common culprit.

2.2.4   Cracking

Concrete cracking can be attributed to a variety of factors and can critically affect the durability of bridge decks. "Cracks in bridge decks provide the principal path for deicing salts to reach reinforcing steel and may extend through the deck and accelerate corrosion of the supporting girders" (41). The ability of concrete to resist chloride ion penetration is insignificant once the concrete begins to crack. Once this occurs, the chloride ions are provided with a direct path to reach the reinforcing steel.   Crack formations in concrete can be due to volumetric changes, freezing and thawing effects, thermal changes, chemical attack, and structural stress. This section primarily focuses on volumetric changes, freezing and thawing, and thermal changes in concrete and their effects on concrete cracking.

Volumetric Changes

Volumetric changes in concrete are typically due to shrinkage. Concrete shrinkage is defined by ACI Committee 116 as a decrease in either the concrete's length or volume (1). Shrinkage can generally be attributed to moisture evaporation in the concrete or chemical reactions between varying components of the concrete. Volumetric changes are often considered to occur during two stages of the concrete's life, the plastic state and hardened state. The plastic state refers to the freshly mixed condition of the concrete immediately after mixing. The hardened state refers to the concrete's condition after the plastic state has ended and the concrete has developed sufficient strength. Two types of volume change can occur during the concrete's plastic state: plastic shrinkage and settlement cracking. The main source of volume change for

concrete in its hardened state is drying shrinkage. Autogenous shrinkage is another source of volume change in concrete; it is not, however, specifically related to the plastic or hardened state of the concrete.

Plastic shrinkage is the most common type of volume change for concrete in its plastic state (73). Plastic shrinkage cracks on bridge decks occur when the surface evaporation rate exceeds the rate at which bleed water rises to the concrete surface. Cracks caused by plastic shrinkage are typically shallow in nature and will usually appear in random patterns. High evaporation rates at the concrete surface can be attributed to a variety of factors including high concrete temperatures, high ambient air temperatures, low humidity, and high wind speeds (18). Proper curing and placement of concrete at lower temperatures are examples of good precautions that can be taken to prevent plastic shrinkage cracking. Settlement cracking is another type of volume change associated with the plastic state of concrete. This type of cracking occurs when settlement of fresh concrete is inhibited due to some type of restraining element, such as reinforcing steel. If the reinforcing steel were not present, in this case, the concrete would still experience a reduction in depth and volume due to the settlement, but no cracking would occur because of the outside restraining element being absent. Crack formations formed by settlement may develop at regular spacing intervals reflecting the steel reinforcement layout (25). As the depth of concrete cover over the steel reinforcement increases, the likelihood of settlement cracking decreases.

Drying shrinkage of hardened concrete is an important factor in terms of durability. As the name infers, drying shrinkage is the reduction in concrete volume

caused by the loss of water (4). This type of shrinkage is arguably the most common cause of cracking on bridge decks. After the concrete is cured and is exposed to the environment, concrete bridge decks will lose some of the original mixing water to the atmosphere and begin to shrink. Throughout this process, the longitudinal beams under the deck will restrain the shrinkage and cause transverse cracking in the deck (18). The American Concrete Institute's committee on "Control of Cracking in Concrete Structures" suggests that the major factors that contribute to the ultimate drying shrinkage of concrete are relative humidity, aggregate type and content, water content, and the water to cementitious materials (w/cm) ratio (4). As reported by Tinsley, higher cement and water contents have been shown to increase the drying shrinkage in concrete (70).

Autogenous shrinkage is a type of concrete shrinkage that results from chemical reactions occurring internally within the concrete. This type of shrinkage is the change in concrete volume that occurs without a loss of moisture. As stated by Holt, it is a widely accepted belief that "autogenous shrinkage cannot be prevented by casting, placing, or curing methods, but must be addressed when proportioning the concrete mixture" (36). Research has shown that the chemical components existing internally within the concrete mixture have the greatest influence on autogenous shrinkage. Mokarem reports that autogenous shrinkage depends on the hydration of C_3A and C_4AF in the cement paste, and it increases as these compounds increase (48).

Freezing and Thawing

A major concern for bridge decks, particularly in colder climates, is the freezing and thawing cycles that often cause durability problems for the concrete. Freezing and thawing damage can affect concrete through two main mechanisms: hydraulic pressure and ice accretion (26). Freeze-thaw damage by hydraulic pressure will occur when water in the capillary pores of the cement paste expands upon freezing. When the amount of space needed to accommodate this increase in volume is not sufficient, the excess water will be forced out by the pressure of expansion. The magnitude of this hydraulic pressure depends on a variety of factors including the permeability of the cement paste, the distance to the nearest unfilled void, degree of saturation, and the rate of freezing (26). Once the hydraulic pressure produced by this volume expansion exceeds the tensile strength of the cement paste, local cracking will occur. When concrete is exposed to repeated cycles of freezing and thawing, water will enter these local cracks during the thawing segment of the cycle only to freeze again at a later time. Deterioration to the bridge deck concrete will only worsen over time as the cycles are repeated through various weather changes.

Ice accretion is also a significant concern for freeze-thaw damage in concrete. Ice accretion in concrete can be simply described as ice accumulation on or within the concrete structure. In terms of freeze-thaw damage, pressure can build up due to ice accumulation in the capillary pores even if the hydraulic pressure is not strong enough to damage the cement paste. Ice accretion in concrete tends to progress with time and

is more common in areas where the concrete remains frozen for extended periods of time (26). Hydraulic pressures will tend to be greatest when the rate of freezing is rapid.

It is widely believed that freeze-thaw damage in concrete can be prevented by use of entrained air. ACI Committee 201 states, "there is general agreement that cement paste of adequate strength and maturity can be made completely immune to damage from freezing by means of entrained air, unless unusual exposure conditions result in filling of the air voids" (2). Although the use of air-entrained concrete is effective in controlling freeze-thaw damage in the cement paste, it will not, however, prevent freeze-thaw damage as related to the aggregate particles. A number of properties related to the pore structure within the aggregate particles can be indicators of possible durability problems when the coarse aggregate particles are used in concrete that is exposed to water and will freeze in service. A few of these indicators that can affect the aggregate's ability to resist freeze-thaw action include the aggregate's absorption, porosity, pore size, and permeability properties (2). The tacit assumption is made in Tennessee that properly air-entrained concrete obviates a freeze-thaw problem.

Thermal Changes

Thermal changes in concrete can play a key role in cracking of bridge decks. The temperature of concrete rises during the curing process due to the heat of hydration process of the cement paste. The heat of hydration is the exothermic reaction that occurs between the cement and water during curing. By the time the concrete reaches its peak temperature, it will have already reached its hardened state. Following this

heating process, the hardened concrete will begin to cool down and the temperature will begin to reach the ambient temperature (18). This change in temperature will cause shrinkage in the bridge deck. The longitudinal beams underlying the deck will restrain the deck shrinkage, eventually causing cracking in the bridge deck. The intensity of thermal cracking in bridge decks depends on the magnitude of the restrained thermal shrinkage. The amount of restrained thermal shrinkage in the bridge deck depends on the difference between the peak concrete temperature and the temperature of the longitudinal beams supporting the deck at the time of peak temperature, which is typically the ambient temperature (18). A couple of procedures that can be followed to control thermal shrinkage in bridge decks include minimizing the total cementitious materials contents of mixtures and using SCMs such as slag cement as a partial substitute for portland cement. Using lean concrete mixtures, or mixtures with low cement contents, will generate less heat of hydration which will help to control the peak temperatures reached during this process. The use of SCMs such as fly ash or slag cement as partial substitutes for portland cement will lower the heat of hydration of the cement paste.

2.3   Ternary Concrete Mixtures

In an era where increased durability and lifespan of concrete structures is becoming progressively more important, ternary blended concrete mixtures seem to be the most viable option in terms of simply improving the concrete as a material. Ternary concrete mixtures contain three different types of cementitious materials, typically cement and two other supplementary cementitious materials (SCM). As discussed in

Chapter 1, common supplementary cementitious materials include fly ash, ground granulated blast furnace slag (GGBFS), and silica fume. The typical Class D concrete mixture that is produced and placed on Tennessee bridge decks is a binary mixture which contains two different types of cementitious materials. The two materials used in these mixtures are, typically, mostly cement (75%-80%) and a small portion of fly ash (20%-25%) (65). It should be noted that although a binary mixture is typical for TDOT mixtures, it is not required, and there are occasional mixtures that only contain cement. The unfortunate truth in regard to the current status of bridge deck concrete in Tennessee is that the durability of bridge decks is lacking, and a significant portion of the blame can be attributed to poor concrete with high permeability. The RCP and SR results for the field samples collected from across the state over the last two and a half years support this statement. Thus, a strong argument can be made that a ternary concrete mixture needs to be implemented as part of TDOT's specification for bridge deck concrete. This section discusses the advantages, disadvantages, availability, and costs of ternary blend concrete mixtures. A particular emphasis is placed on slag cement as this will be the third SCM used to create the proposed ternary blend mixture.

2.3.1 Advantages

Ternary concrete mixtures can offer a variety of advantages to a concrete structure. As Rupnow states, there is general agreement that the use of SCMs has the following positive effects on concrete (57):

1. Improved workability and finishability.

2. Strength gain; despite early strength reduction, beyond 7 days concrete containing SCMs will tend to show increased strengths over portland cement concrete.

3. The use of SCMs has been proven to reduce the early rate of heat generation in mass concrete applications.

4. Permeability is reduced in mature concrete, and resistance to sulfate and chloride attack is improved.

5. Freeze-thaw resistance, modulus of elasticity, and resistance to de-icing salts are all about the same as ordinary portland cement concrete.

6. Resistance to corrosion of reinforcing steel; the use of SCMs in concrete helps to reduce permeability and leads to the reduction of chloride ion penetration.

In addition to the physical advantages that ternary concrete mixtures create, they are also considered to have a positive impact on sustainability. The use of supplementary cementitious materials in concrete is widely considered to be less harmful on the environment than portland cement. As Mehta mentions, about 7% of the world's carbon dioxide emission is attributable to the portland cement industry (45). SCMs such as slag cement and fly ash are byproducts of iron production and coal burning, respectively, and are making use of industrial wastes while portland cement is doing the opposite.

2.3.2 Disadvantages

Despite the numerous advantages created by ternary blend concrete mixtures, a few disadvantages exist. Rupnow reports that an increased time of setting and an

unpredictable change in time between initial and final set can be expected when dealing with a ternary blend mixture (57). This can be of significant concern for saw cutting operations. The strength characteristics of a ternary mixture will sometimes discourage people in the concrete industry from using it due to the slower strength gain. This issue is discussed in Section 2.4.2. The use of a ternary blend in extremely cold temperatures is discouraged due to prolonged curing times. While the slower hydration rate that comes from use of a ternary blend can be advantageous in hot temperatures, this is not always the case when temperatures drop to lower levels. The availability of slag cement and the capability of concrete plants to store it is often a concern for DOT officials when discussing possible implementation of a ternary mixture. These issues are discussed in Section 2.3.4.

2.3.3 Costs

A ternary concrete mixture is typically less expensive than a portland cement concrete mixture. The cost of a ternary blend concrete mixture will vary based on the type of SCM used and the location of the project (30). Rupnow performed a cost-benefit analysis in cooperation with the Louisiana DOT to determine the monetary implications of a ternary concrete mixture (57). For his study, he made the assumption that the difference in delivered cost of Class C and Class F fly ash and the difference in delivered cost of Grades 100 and 120 slag cement were negligible. The input costs of all three cementitious materials used in his study are shown in Table 1.

Table 1: Average Costs of SCMs Per Ton (57)

Cementitious Material	Cost per Ton ($)
Portland Cement	$100.00
Fly Ash	$50.00
Slag	$90.00

The cost-benefit analysis was performed during the Louisiana DOT bid years 2007 and 2008. The monetary values in Table 2 show the estimated total cost for cementitious materials and potential cost savings if each mixture were used on all concrete paving projects that occurred during this two year span. The three mixtures shown in the table are as follows: (1) 80% Type I Cement, 20% Class C Fly Ash; (2) 40% Type I Cement, 30% Grade 100 Slag Cement, 30% Class C Fly Ash; and (3) 30% Type I Cement, 35% Class C Fly Ash, 35% Class F Fly Ash. As can be seen in Table 2, the binary mixture containing cement and fly ash was significantly more expensive than the two ternary mixtures. The mixture with cement, slag, and fly ash would have saved around 20% of the total cost, and the mixture containing both classes of fly ash and portland cement would have saved even more with almost 28% savings (57).

Table 2: Total Costs of Louisiana DOT Cost-Benefit Analysis (58)

Mixture Design	Cementitious Materials Cost	Potential Savings ($)	Potential Savings (%)
80TI-20C	$17,298,106	N/A	N/A
40TI-30G100S-30C	$13,838,523	$3,459,583	20.0
30TI-35C-35F	$12,493,119	$4,807,515	27.8

1. *80TI - 20C = 80% Type 1 Cement, 20% Class C Fly Ash*
2. *40TI – 30G100S – 30C = 40% Type I Cement, 30% GGBFS (Slag Cement), and 30% Class C Fly Ash*
3. *30TI – 35C – 35F = 30% Type I Cement, 35% Class C Fly Ash, 35% Class F Fly Ash*

Although the monetary costs of supplementary cementitious materials are ever-changing, a survey of various sources at ready mix plants validates these pricing trends discussed by Rupnow and demonstrates that prices of this nature can generally be expected for ternary concrete mixtures.

2.3.4 Availability

Before a ternary mixture is implemented into a concrete specification, one of the most important concerns that will have to be addressed is in regard to the availability of the supplementary cementitious materials. Both Class C and Class F fly ash are readily abundant around the world. Fly ash first became available in quantity in the United States in the 1930s from coal-burning electric power plants (5). Class F fly ash normally

results from the burning of anthracite or bituminous coal and is typically low in calcium

oxide, while Class C fly ash normally results from the burning of lignite or subbituminous

coal and typically has a high calcium oxide content. Class C fly ash is commonly

produced from coal found in the Western parts of the United States. Both types of fly

ash are readily available and are often used in concrete mixtures around the world.

Although slag cement is not as readily available as fly ash, the availability of this

material is sufficient enough to implement its use in ternary concrete mixtures. The

United States Geological Survey (USGS) estimated that 3.6 million metric tons of slag

cement was produced in the United States in 2003 (55). Review of various sources of

literature indicates an increasing trend in the amount produced in the United States as

time progresses. ACI Committee 233 reports that five companies provide slag cement

in the United States. It is important to remember the difference between slag cement

and typical blast furnace slag when determining the amount of slag cement produced in

the United States. Blast furnace slag is not considered slag cement until it is passed

through a granulator and ground up, giving it the name ground granulated blast furnace

slag. According to the 1991 Bureau of Mines Annual Report, 13,293,000 metric tons of

blast furnace slag were either sold or used during that year (6). The likely reason that

this value is significantly higher than the 3.6 million metric tons produced in 2003 can be

attributed to the lack of granulating facilities at all furnace facilities. It is reasonable to

believe that more slag cement would be available in the United States if a higher

emphasis were placed on its use in concrete mixtures.

In reality, the larger concern related to the use of slag cement in concrete should be placed on the storage capabilities of ready mix plants, not the general availability of slag cement. Cement and other SCMs are typically stored in storage silos at the ready mix plant. Almost all plants will have an extra storage silo for an additional SCM such as fly ash (30). The addition of a third SCM such as slag cement could cause problems initially if a ready mix plant were not capable of properly storing the material. Barriers to implementing a ternary mixture, such as storage issues, can easily be overcome and should not be considered a reason to forgo implementation of a ternary concrete mixture. In fact, the larger ready mix plants will most likely already have the proper storage facilities on site.

2.4   Effects of SCMs on Concrete Durability

This section discusses the effects of supplementary cementitious materials (SCMs) on the engineering properties of concrete as related to concrete durability. The effects of SCMs on all of the concrete durability factors mentioned in Section 2.2 are addressed. The two SCMs considered are fly ash and slag cement. Both of these materials are finer grained than portland cement; in order of descending grain size, there is portland cement, slag cement, and fly ash.

2.4.1   Description of Material

Fly Ash

Fly ash is defined by ASTM C618 as "the finely divided residue that results from the combustion of ground or powdered coal and that is transported by flue gasses" (15).

Two types of fly ash are commonly used in concrete mixtures: Class C and Class F. ASTM states that "Class F fly ash is typically produced from burning anthracite or bituminous coal, but may also be produced from subbituminous coal and from lignite" (15). Class C fly ash is often produced from burning lignite or subbituminous coal, but similar to Class F, may also be produced from anthracite or bituminous coal. Class C fly ashes typically have higher calcium contents than Class F. Also, Class C fly ash has some cementitious properties in addition to having pozzolanic properties. Class F only has pozzolanic properties (15).

Ground Granulated Blast-Furnace Slag

Ground granulated blast furnace slag, better known as slag cement, is defined by ACI Committee 233 as granulated blast-furnace slag that has been finely ground and that is a hydraulic cement (7). Blast-furnace slag is a by-product of iron production, and, when passed through a granulator and ground up, can be used as a supplementary cementitious material in concrete. Slag cement is classified into three different grades as specified by ASTM C989: Grade 80, Grade 100, and Grade 120 (14). The slag cement is specified its grade by its slag activity index, which is the compressive strength ratio at 28 days of a 50% slag cement and 50% portland cement mortar cube divided by a 100% portland cement reference mortar cube. The ratio is expressed as a percentage. If the percentage at 28 days is 80%, then it is considered Grade 80 slag cement; if it is 100%, then it is Grade 100; lastly, if it is 120% then it is Grade 120. It is most common for Grades 100 and 120 to be used in concrete mixtures due in large part to the slow strength gain that the Grade 80 slag cement will exhibit.

Grade 80 slag cement is sometimes used in mass concrete applications where a slower heat of hydration is beneficial.

2.4.2   Strength

Regardless of the supplementary cementitious material being used, it is a widely accepted belief that, while a number of factors play a role in determining concrete strength, the main factor that affects concrete's compressive strength is the water/cementitious materials (w/cm) ratio. The effects of fly ash and slag cement on compressive strength are discussed herein.

Fly Ash

As discussed by Obla, concrete containing fly ash will generally have a slower rate of strength development and will often result in a higher ultimate strength than portland cement concrete (52). Concrete mixtures containing high calcium, or Class C, fly ash will typically have higher early strengths than concrete containing the low calcium, or Class F, fly ash due to the pozzolanic properties of Class F fly ash. Malhotra discusses a study conducted by Gebler and Klieger (29) in 1986 on the effects of Class C and Class F fly ashes from 10 different sources on the compressive strength development of concrete mixtures subjected to various curing conditions. This study included the effects of low temperature and moisture availability on the concretes. Results of the study indicated that concrete containing fly ash had the potential to produce satisfactory compressive strength development (29,42). The influence of the class of fly ash on the long-term, ultimate compressive strength of the concrete was insignificant. Gebler and Klieger concluded that the Class F fly ash was more likely to

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be influenced by low curing temperatures than Class C fly ash, indicating the importance of proper curing for fly ash concrete.

Ground Granulated Blast-Furnace Slag

Similar to concrete containing fly ash, slag cement concrete will typically exhibit slower strength gains than portland cement concrete. The rate of strength gain varies significantly depending on the grade of slag cement used. ACI Committee 233 suggests that Grade 120 slag cements will cause reduced strength at early ages (one to three days) compared to a portland cement control mixture but will produce increased strength at later ages (seven days and beyond). Grade 100 slag cements will also experience lower strengths at early ages, even up to around 21 days after concrete mixing. However, it will typically have equal or greater strength to portland cement concrete at 28 days and later. Grade 80 slag cement concrete mixtures will essentially always have lower early strengths than a portland cement control mixture but will occasionally have 28-day strengths equivalent to a portland cement mixture (7).

2.4.3   Corrosion of Steel

As discussed by Haque et al., one of the main causes of steel corrosion in concrete structures is the loss of efficiency of the concrete cover, which is known to protect the steel and keep it in a passive state (33). Deterioration of the passive barrier that the concrete creates to protect the reinforcing steel can either come by carbonation, which reduces the level of pH adjacent to the steel to values that can no longer maintain a passive state, or by the presence of large amounts of chloride ions in the concrete that will cause severe attacks on the steel reinforcement (33).

Fly Ash

It is a generally accepted belief that the use of fly ash, regardless of type, in concrete mixtures can have positive effects on permeability. Thus, it is logical to believe that this will improve the concrete's resistance to chloride ion penetration. Although this is a factual statement regarding fly ash's effect on concrete permeability, the permeability of fly ash concrete is "directly related to the quantity of hydrated cementitious material at any given time" (42). In summary, curing time is a key factor in terms of the permeability of fly ash concrete due to its slower rate of hydration as compared to portland cement. The proper amount of curing time should be allowed for fly ash concrete to reach desired permeability levels.

General disagreement seems to exist on the effects of fly ash on carbonation in concrete. While some researchers have reported that fly ash concretes carbonate faster than concrete mixtures without fly ash, others claim that their research has shown no indication of increased carbonation in equal strength concretes with and without fly ash (33). Haque et al. concludes from a carbonation study performed on fly ash concrete during a four year testing period that fly ash concretes achieved a significantly higher depth of carbonation than the portland cement control mixtures, although the maximum depth of the fly ash mixtures was not significant enough to consider a durability issue (33). Proper curing practices are essential in protecting fly ash concrete from carbonation issues. The ability of a concrete mixture to resist carbonation and chloride ion penetration is directly related to its permeability (42).

Ground Granulated Blast-Furnace Slag

The use of slag cement as a partial replacement for portland cement in concrete mixtures has been proven to decrease the concrete's permeability. ACI Committee 233 explains that the microstructure of the cementitious matrix is changed through the reaction of the slag cement with the calcium hydroxide and alkalis released during the hydration process (7). The cement paste experiences a reduction in pore size due to the addition of slag cement, causing a decrease in the permeability. This decrease in permeability will significantly reduce the penetration of chloride ions within the concrete, thus preventing corrosion of the resisting steel (7).

Similar to fly ash, general disagreement seems to exist on the effects of slag cement on carbonation in concrete. Hamada and Meyer suggest that carbonation occurs more rapidly in concretes incorporating slag cement than in ordinary portland cement mixtures (31,42,47). Sulapha et al. conclude that concrete mixtures containing low fineness slag cement and fly ash had higher rates of carbonation than ordinary portland cement mixtures, whereas concrete mixtures containing slag cements of higher fineness and fly ash actually exhibited carbonation rates that were lower than that of ordinary portland cement mixtures (61). Despite the positive impact that slag cement with higher fineness will have on concrete's ability to resist carbonation, it is neither practical nor significant enough to specify fineness requirements other than the typical No. 325 sieve testing as required by TDOT's construction specification (65). Sulapha et al. also recommend that concretes containing slag cement, fly ash, or silica fume should

experience extended curing periods as compared to ordinary portland cement mixtures in order to improve their resistance to carbonation (61).

2.4.4   Chemical Attack

As discussed in Section 2.2.2, sulfate attack and alkali-silica reaction (ASR) are two of the most common types of chemical attacks that attack concrete. The effects of fly ash and slag cement on these mechanisms are addressed in this section.

Fly Ash

It is commonly known that most fly ashes will have positive effects on concrete's resistance to sulfate attack. As Tikalsky and Carrasquillo explain, sulfate attack most often occurs when the cementitious matrix of the paste in concrete is corroded through the formation of the crystalline ettringite and gypsum (68). Common methods to prevent sulfate attack are to change from a Type I portland cement that contains higher amounts of crystalline tricalcium aluminate, or C_3A, to a Type II or Type V portland cement, which contains lower amounts of C_3A, or to introduce a pozzolan such as fly ash to the mixture (68). Tricalcium aluminate is the reactive aluminate compound known to act as one of the chief contributors to the expansive reactions associated with ettringite formation. Tikalsky and Carrasquillo attribute two mechanisms in explaining the effect of fly ash on the sulfate resistance of concrete. First, the replacement of a percentage of the portland cement with fly ash decreases the total amount of C_3A in the concrete. This is known as the "dilution effect" (68). The second mechanism, known as the "pozzolanic effect," is related to the refined calcium silicate hydrate binder matrix that is formed from the pozzolanic reaction between the fly ash and portland cement.

This new binder matrix causes the concrete to become less permeable, and the excess calcium is consumed and made unavailable to expansive ettringite and gypsum formations (68). It can be generally concluded from the review of literature that fly ashes with high calcium oxide (CaO) contents are more susceptible to sulfate attack than are fly ashes with low CaO contents, such as Class F fly ash (5,68,69). It is for this reason that Class F fly ash is most often recommended for sulfate resistance due to its low CaO content.

The use of fly ash in concrete has been considered a viable method to mitigate ASR expansion in concrete structures for many years. As ACI Committee 232 explains, "the reaction between the siliceous glass in fly ash and alkali hydroxides in the portland cement paste consumes alkalies, which reduces their availability for expansive reactions" (5). Class F fly ash is traditionally thought of as the best option to reduce ASR expansion in concrete due to its low calcium content. Touma et al. performed a study on several reactive aggregates using various mixtures consisting of either Class C or Class F fly ash. Results of this study showed that a concrete mixture incorporating 25% Class F fly ash replacement resulted in less ASR expansion than a mixture containing 35% Class C fly ash replacement (43,72). Malvar recommends that Class F fly ash (not Class C) be substituted into concrete mixtures at somewhere between 25% to 40% replacement for ASR prevention (43). Although Class F fly ash is commonly recommended for use in ASR mitigation, Class C fly ash of good quality can also prove to be beneficial if used at the proper replacement levels. Replacement levels for Class

C fly ash must be somewhat higher than replacement levels for Class F fly ash due to the inefficiency of Class C fly ash in preventing ASR expansion.

Ground Granulated Blast-Furnace Slag

The sulfate resistance of concrete can also be improved by the use of slag cement. As discussed by ACI Committee 233, several changes occur within a concrete mixture when slag cement is used in sufficient quantities (7). A few of these include as follows:

1. The C_3A content of the mixture is proportionally reduced based on the percentage of slag cement used. (7) However, as noted by Lea, increased sulfate resistance in concrete is not only dependent on the C_3A content, but it is also dependent on the Al_2O_3 content of the slag cement (7,40,42). Lea reports that when the alumina content is less than 11%, increased sulfate resistance will exist in the concrete (40).

2. The environment for the formation of calcium sulfoaluminate, the main cause of deterioration caused by sulfate attack, is reduced through the reduction of soluble calcium hydroxide caused by the hydration of slag cement (7)

3. The permeability of the cementitious paste is reduced preventing the intrusion of destructive sulfates (7)

Hogan and Meusel report that high resistance to sulfate attack has been found when the slag cement replacement percentage exceeds 50% of the total cementitious material and Type II cements were used (35). Other tests have shown that concrete mixtures containing 50% slag replacement with Type I portland cement have achieved

sulfate resistance equivalent to that of Type V cement (7). In summary, the cement replacement percentages necessary for slag cement concrete to attain adequate sulfate resistance are moderately higher than those necessary for concrete containing fly ash.

The use of slag cement as a partial replacement for portland cement in concrete mixtures has also been found to reduce ASR expansion. As Thomas and Innis report from their study on the effects of slag on ASR expansion, partial replacement of portland cement with slag reduces the expansion of concrete prisms and mortar bars made with alkali-silica reactive aggregates, with the slag cement's effectiveness increasing as the amount of slag was increased (66). Review of literature suggests that the amount of slag required to mitigate ASR expansion in concrete is dependent upon the nature of the slag cement, the reactivity of the aggregate, and the alkali content of the portland cement (7,66). Malvar recommends that slag cement be used in concrete mixtures at 40% to 50% replacement of portland cement for ASR mitigation. In regard to ternary blends containing slag cement and fly ash, Malvar states that a combination of his recommended replacement percentages for fly ash (25%-40%) and slag cement (40%-50%) should be sufficient for resistance to ASR expansion (43). It can be concluded that a combination of fly ash and slag cement, i.e. ternary blend, will only further enhance the concrete's resistance to ASR expansion.

2.4.5   Physical Deterioration

Fly Ash

Deterioration of concrete surfaces can be caused by a variety of mechanisms such as erosion, cavitation, and abrasion (50). Of these mechanisms that cause

physical deterioration to concrete surfaces, abrasion is arguably the most important. Abrasion wear can occur due to rubbing, scraping, skidding, or sliding of objects on a concrete surface (50). This can be of particular importance on highway bridge decks due to the constant rubbing and scraping of tires on the concrete surface. As stated by Naik, Singh, and Hossain, the characteristics of the concrete surface layer, including abrasion resistance, are affected by the cement content, water-cement ratio, slump, air content, type of finish, and curing (50). While some scholars would argue that the aggregate toughness is of utmost importance in abrasion resistance, numerous studies have shown that compressive strength is the most important factor controlling the abrasion resistance of concrete (27,50). Naik et al. concluded from their study on the abrasion resistance of fly ash concrete that the concrete's resistance to abrasion was strongly affected by its compressive strength, regardless of fly ash content (50). His study also reported that the compressive strengths were considered satisfactory for fly ash concretes containing up to 50% cement replacement. Concrete containing up to 30% fly ash replacement exhibited abrasion resistance that was closely related to concrete mixtures without fly ash at ages of 28, 91, and 365 days (50). Thus, the similar compressive strengths help to conclude that concrete containing fly ash at reasonable cement replacement levels will exhibit adequate abrasion resistance, but that fly ash does not enhance abrasion resistance.

Ground Granulated Blast Furnace Slag

The abrasion resistance of concrete containing slag cement is similar to that of fly ash. The characteristics of the concrete surface layer in slag cement concrete are

affected by the same factors that affect fly ash concrete as described by Naik et al. (50).

Furthermore, the concrete's compressive strength is still the governing factor in terms of

the abrasion resistance of a concrete mixture containing slag cement. It is reasonable

to believe that, with sufficient curing, adequate abrasion resistance could be attained for

both slag cement and fly ash concrete mixtures.

2.4.6   Cracking

As discussed in Section 2.2.4, cracking of bridge deck concrete is a significant

concern in terms of maintaining a durable structure and protecting the reinforcing steel

from corrosion. Several factors that are known to cause concrete to crack on bridge

decks are volumetric change, freezing and thawing effects, and thermal changes within

the concrete. The effects of fly ash and slag cement on these factors are discussed in

this section.

Fly Ash

The primary concern related to volumetric change of concrete on bridge decks is

drying shrinkage. Although some scholars may disagree, the use of fly ash in concrete

mixtures is generally considered to have positive effects on the drying shrinkage of

concrete. Davis et al. performed a study on different concrete mixtures containing fly

ash and cement and found no apparent differences in the drying shrinkage between

concrete with up to 20% fly ash and typical portland cement concrete (5,24). Other

studies have shown that increased fly ash content, even beyond 20% replacement,

resulted in slightly less drying shrinkage (5).

The resistance of concrete containing fly ash to freezing and thawing has been studied for many years. As stated by ACI Committee 232, "the resistance to damage from freezing and thawing of concrete made with or without fly ash depends upon the adequacy of the air-void system, the soundness of the aggregates, age, maturity of the cement paste, and moisture condition of the concrete (5). It is a widely accepted belief that proper air-entrainment of the concrete is the most important factor in resisting freezing and thawing effects. In properly air-entrained concrete, research has shown that no significant difference exists in the resistance to freezing and thawing of concretes with and without fly ash (5).

The use of fly ash can be beneficial in preventing thermal shrinkage cracking of concrete on bridge decks. Thermal shrinkage cracking can occur when the concrete reaches high internal temperatures due to the heat of hydration reaction of the cement paste. Portland cement has a higher heat of hydration than fly ash. Concrete containing fly ash as a partial replacement for portland cement will generally have lower heats of hydration due to the reduction of the portland cement in the mixture (5). Thus, the temperature rise of concrete can be controlled by the use of fly ash as a partial replacement for portland cement.

Ground Granulated Blast-Furnace Slag

A review of literature on drying shrinkage of slag cement concrete produces somewhat conflicting results. Despite the critical viewpoint that some researchers may have on the effects of slag cement on concrete shrinkage, it is a widely held belief that drying shrinkage of slag cement concrete is similar to that of ordinary portland cement

concrete (7). Research performed by the Ohio Department of Transportation reported from their study on high performance concrete mixtures on bridge decks that concrete containing 30% slag cement showed less drying shrinkage than a 100% portland cement mixture (7,39). Mokarem concluded from his study of concrete shrinkage that mixtures containing fly ash displayed greater drying shrinkage than those containing slag cement (48). There is not complete consensus on the effect of either fly ash or slag cement on shrinkage, but from a practical standpoint, the effect, either slightly positive or slightly negative, may be considered negligible.

The resistance of slag cement concrete to freezing and thawing has also been a topic of numerous research studies during the past century. Similar to fly ash, studies have generally indicated that the freezing and thawing resistance of concrete containing slag cement is essentially the same to that of concrete made with Type I or Type II portland cements (7). The most important factor to assure that slag cement concrete will resist freezing and thawing is for the mixture to have the proper air content and air-void system. It is the opinion of some researchers that the addition of supplementary cementitious materials will improve concrete's resistance to freezing and thawing because of the decrease in permeability that materials such as fly ash and slag cement will produce when substituted for portland cement, but this viewpoint is widely debated and cannot be guaranteed to be true due to conflicting research studies (23).

The use of slag cement can also be beneficial in preventing thermal shrinkage cracking of concrete on bridge decks. Similar to fly ash, portland cement also has a higher heat of hydration than slag cement. Concrete containing slag cement as a partial

replacement for portland cement will generally have lower heats of hydration due to the reduction of the portland cement in the mixture (7). Thus, the temperature rise of concrete can be controlled by the use of slag cement as a partial replacement for portland cement.

2.5   Test Methods

Establishing consistent, reliable test methods to predict the durability properties of concrete can prove to be a difficult task. The main purpose of this research is to test the concrete's ability to resist chloride ion penetration as well as to propose new ideas on how to improve the concrete's resistance to chloride attack. A variety of test methods exist that can provide an indication of the concrete's ability to resist chloride ion penetration. Some existing long-term procedures used for this purpose are the chloride ponding test (AASHTO T259) and the Nordtest bulk diffusion test. The chloride ponding test is, perhaps, the most well-known and well-accepted test that is used to determine the actual resistance of concrete to chloride ion penetration in concrete (32). The duration of the chloride ponding test is approximately three months once the samples are fully prepared. The lengthiness of this test is one of the primary reasons that short-term testing is desired in the concrete industry. Several short-term tests for determining concrete's resistance to chloride ion penetration are available and are used in practice. A few of these tests include the Rapid Chloride Ion Penetration (RCP) test, electrical migration techniques, resistivity tests, and pressure penetration techniques (60). The tests that were performed for this research project were the RCP test in accordance with ASTM C1202 and AASHTO T277 and the concrete surface resistivity

(SR) test in accordance with AASHTO TP 95-11. A brief overview of these two test methods is covered in Sections 2.3.1 and 2.3.2. A more detailed description and literature review of these two test methods can be found in Eric Ryan's and Brian Buchanan's M.S. theses (22,59).

2.5.1 Rapid Chloride Ion Penetration Test

As stated in the official title of this test in ASTM C1202, this test method provides an "electrical indication of concrete's ability to resist chloride ion penetration" (16). The concrete's resistance to chloride attack is indicated by the determination of the electrical conductance of the concrete which will, in effect, provide a rapid indication of its resistance to the penetration of chloride ions. Despite various criticisms regarding this test, the RCP test has shown strong correlation with the chloride ponding test which is widely regarded as the "gold standard" for chloride ion penetration testing (59). Thus, the RCP test is a valid test method to perform in place of the chloride ponding test due to its relatively short duration as compared to the latter option.

Test Procedure

The RCP test takes a total of approximately thirty hours to perform from start to finish. The first twenty-four hour period of this test is spent on preparation of the samples. The last six hour period is spent on the actual testing portion. A summary of the test procedure was created by the first researcher on this project, Eric Ryan, and is outlined in accordance with ASTM C1202 as shown on the following page:

DAY 1

1. Fill pressure cooker with tap water and heat water until it begins to steam. Allow to boil for 15-20 minutes before cooling in front of fan for approximately four hours.

2. Take specimen straight from curing to cutting. Cut top two inches off of 4" x 8" concrete cylinder.

3. Dry Specimen for about 15-20 minutes with fan. The surface of the 4" diameter x 2" tall specimens should be dry to the touch before proceeding.

4. Tape top and bottom of specimens with 2" masking tape to prevent the Plasti-Dip, or synthetic rubber protective coating, in step 5 from getting on top or bottom. Trim excess tape on sides using safety knife.

5. Using latex gloves, Plasti-Dip the sides of the test cylinder using fingers to spread the material evenly. Apply one coat to the full circumference of the cylinder, let dry for 1-2 minutes then apply another coat in the same manner. Place the cylinders on top of metal specimen cups. Let the cylinders dry by the fan.

6. Replace the vacuum pump oil. This should be done before each test.

7. Change the desiccant (Drierite) before each test.

8. Remove masking tape from the dry, Plasti-Dip covered cylinders. Place cylinders in testing bowl and place top on. The cylinders should be positioned so that the Plasti-Dip sides are facing up and down (vertical). Turn on vacuum and press the top of testing bowl down to get the vacuum started. Make sure stop-cock is in closed position. Turn vacuum monitor on. Leave cylinders in vacuum for 3 hours. More

than 3 hours is fine, but needs to be within reason to keep uniformity between sets of samples (15-20mins).

9. After 3 hours, add the "de-aired" water from pressure cooker into testing bowl via the funnel and stop cock apparatus. Continue to add water until the water in the bowl completely covers the test cylinders. Leave the cylinders in the water; allow pump system to run for 1 more hour. Again, more than 1 hour is fine, but needs to be within reason to keep uniformity between sets of samples (~15 minutes). Ensure that the seal produced by the vacuum is maintained throughout this process.

10. After 1 hour, turn off vacuum pump. Open the stop-cock and let remaining water drain into bowl to break the vacuum. Remove the top of the testing bowl. Turn off the vacuum monitor. Allow specimens to soak in bowl of water for 18 +/- 2 hours.

DAY 2

11. Take the specimens out of the water, pat dry, and place in the test block cell. Do not let the specimens completely dry. Add caulking around the specimens and at the openings between the specimens and the voltage cell to ensure that a sufficient seal is created that will not allow the sodium chloride and sodium hydroxide fluids to pass through the sides. Any areas not covered by the caulking will result in a failed test and unusable data.

12. Let the cylinders and test block dry by the fan. Forty-five minutes to one hour is usually adequate time.

13. Using a funnel in the top hole of the test block, fill one side of the test cell marked "NaCl" with the NaCl solution until circle on side of test block is full. Fill the other side marked "NaOH" with the NaOH solution until the circle on the side of test block

is full. The NaCl is subjected to the top portion of the specimen and the NaOH is subjected to the bottom portion.

14. Plug the machine into the cells with the red and black cables. Place the red cable on the marked positive side and the black cable on the marked negative side. Turn on the machine by flipping the switch on the back right side of the machine.

15. Press stop and reset buttons at the same time to reset the machine; then press start button to start the test. Stay around for the first print-out to make sure the apparatus is working properly (should occur after one minute). Leave the machine running. The test will complete itself after 6 hours.

16. Disconnect the voltage cells and use hands to pull the block and cylinder apart. Begin clean-up of voltage cells for next test.

Figure 1: Rapid Chloride Ion Penetration (RCP) Testing Apparatus

Description of Test

The total charge passed through the specimen is a measure of the concrete's electrical conductance during the six-hour test period. A constant voltage of 60 V is maintained across the two ends of the test specimen throughout the test duration. The top end of the test specimen is immersed in sodium chloride, while the bottom end of the specimen is immersed in sodium hydroxide. The current passing through each test specimen is recorded initially and at 30 minute intervals for the entire six hours. Integration of the area underneath the current (amperes) vs. time (seconds) curve produced by the test specimens will provide a final value that is representative of the concrete's electrical conductance during the period of the test. The units for this value will be ampere-seconds, or Coulombs (16). Table 3 shows ASTM C1202's recommendation on the on the concrete's chloride ion penetrability classification based on charge passed.

Table 3: Chloride Ion Penetrability Based on Charge Passed (16)

Charge Passed (coulombs)	Chloride Ion Penetrability
>4,000	High
2,000–4,000	Moderate
1,000–2,000	Low
100–1,000	Very Low
<100	Negligible

Criticisms of RCP Test

Although the RCP test has been adopted as a standard test for determining chloride ion penetrability in concrete and is widely used in current practice, several criticisms exist about the test that cannot be ignored. Stanish, Hooton, and Thomas (60) state three main criticisms in regard to the RCP test: 1) the current passed is related to all ions in the pore solution not just chloride ions, 2) the measurements are made before steady-state migration is achieved, and 3) the high voltage applied leads to an increase in temperature, especially for low quality concretes, which further increases the charge passed (60). The literature goes on to explain that lower quality concretes heat more than high quality concretes because the temperature rise is based on the product of the current and the voltage. As the poor quality concrete specimen continues to heat up, the charge passed continues to increase excessively to values even higher than would typically be expected if temperatures were to remain constant. Consequently, the RCP test will make poor quality concrete appear even worse than it may actually be. In addition to the negative effects that these criticisms will have on these samples, the statistical confidence and precision of the RCP test also suffers due to the sometimes inconsistent results produced by this test.

2.5.2 Surface Resistivity Test

In recent years, the electrical resistivity of water-saturated concrete is being increasingly used to indirectly evaluate concrete characteristics such as chloride ion diffusivity, permeability, and properties of the pore water solution (49). If chloride ion penetration is used as an acceptance criterion, the amount of time that could be saved

by using the Surface Resistivity (SR) test method versus other test methods such as the Rapid Chloride Ion Penetration test is staggering. The Florida Department of Transportation comments on the efficiency of the SR test by stating that "the non-destructive nature, speed, and ease of use make the Wenner Probe technique a promising alternative test to characterize concrete permeability" (32). As of last year, the SR test method is AASHTO certified and follows in accordance with AASHTO TP 95-11, "Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration." Various research, including the work being performed by the UT research team, has proven a strong correlation between the RCP test and the SR test. Several states, Louisiana and Florida, in particular, are ahead of the curve in terms of their innovative concrete research regarding the SR test. In fact, Louisiana has even made plans to implement the SR test as part of a performance based specification for their bridge deck concrete (57).

Test Procedure

The SR test and RCP test are polar opposites in terms of the amount of time and effort that is required to complete the test. The SR test takes a total of about 15 minutes to complete once the samples are removed from the lime-water curing tank, inevitably making the Rapid Chloride Ion Penetration test seem not so "rapid." A brief summary of the SR testing procedure is summarized below in accordance with AASHTO TP 95-11:

1. Upon removal of the cylinder from curing conditions, make four indelible marks on the top surface of the mold marking the 0, 90, 180, and 270-degree points of the circumference. The specification suggests to randomly assign one mark as 0

degrees, then move counterclockwise around the cylinder. These marks are shown in Figure 2. It is also recommended to make longitudinal center marks on the sides of each cylinder to use as a visual reference during testing.

2. Blot off excess water on cylinder and transfer the sample to the specimen holder with 0 degrees on top. The UT research team used a foam sheet as the sample holder in place of the recommended specimen holder as shown in the specification.

3. Place the Wenner array probe on the longitudinal side of the sample, making sure the longitudinal center mark is equidistant between the two inner probe pins. Ensure that the probe pins are dampened prior to each reading. This step is demonstrated in Figure 3.

4. Record the measurement from the display unit after the reading becomes stable.
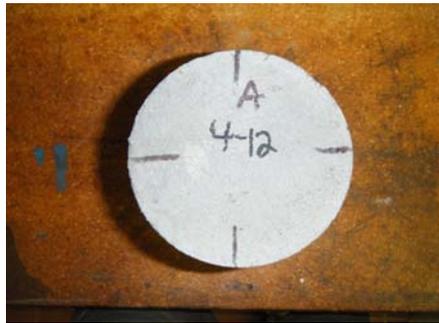
Figure 2: Top Surface Markings for SR test

Figure 3: Surface Resistivity (SR) Testing Apparatus

5. Rotate the sample from the 0 to the 90 degree mark, then 90 to the 180 degree mark, and so forth. Continue to rotate the cylinder and record the readings until one revolution around the cylinder has been completed.

6. Repeat step 5 one time. This two sets of readings will be used to obtain an average value of SR at each location.

7. Repeat steps 1-6 for the other samples in the set.

Description of Test

The Surface Resistivity test as defined by AASHTO TP 95-11 covers the determination of the electrical resistivity of water-saturated concrete to provide a rapid indication of its resistance to the penetration of chloride ions (8). This test consists of measuring the resistivity of cylinders in either 8" or 12" nominal length by 4" or 6" nominal diameter, respectively, by use of a 4-point Wenner probe array. The test specimen can either be a lab cylinder or a core sample without steel reinforcement. The test apparatus is shown in Figure 3.

The resistivity of concrete is calculated based on the current flow, sample area, and resultant potential difference between the two inner probes (8). In the SR test, current flow is generated through the concrete by an alternating current (AC) potential difference that is applied by the surface resistivity apparatus at the outer pins of the Wenner array (8). The resultant potential difference between the two inner electrodes is measured during this process. The resistance of the sample is obtained from the ratio of voltage to current as determined by the potential difference of the two inner electrodes. The resistance is then used to calculate the resistivity of the section (32).
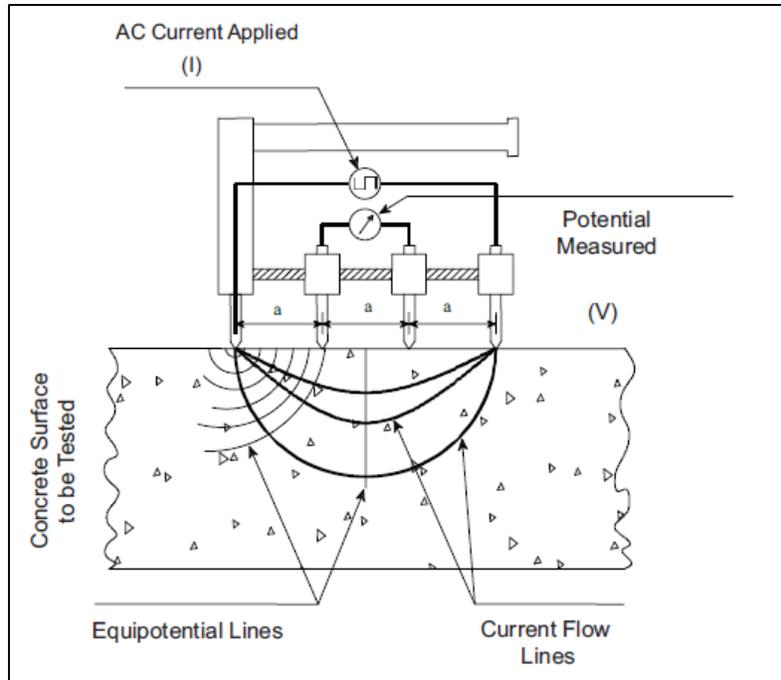
Figure 4: Four-Point Wenner Array Probe Test Setup (8)

Resistivity, ρ, of a prismatic section of length, *L*, and section area, *A*, is calculated by the following equation (32):

$$\rho = \frac{A * R}{L}$$

where R represents the resistance calculated by dividing the voltage potential, *V*, by the applied current, *I*. A visual representation of this test procedure is shown in Figure 4.

Several factors are have been found to affect concrete resistivity as well as chloride ion penetration; these include water-cementitious materials ratio, pozzolans, the presence of polymeric admixtures, air-void system, aggregate type, and degree of consolidation (8). AASHTO's recommendation on the concrete's chloride ion penetration classification based on surface resistivity is shown in Table 4.

Table 4: Chloride Ion Penetrability Based on Surface Resistivity (8)

	Surface Resistivity Test	
	100-mm × 200-mm (4-in. × 8-in.) Cylinder (kilohm-cm)	150-mm × 300-mm (6-in. × 12-in.) Cylinder (kilohm-cm)
Chloride Ion Penetration	$a = 1.5$	$a = 1.5$
High	< 12	< 9.5
Moderate	12–21	9.5–16.5
Low	21–37	16.5–29
Very Low	37–254	29–199
Negligible	> 254	> 199

a = Wenner probe tip spacing

Test Criticisms

Although the SR test offers many advantages in terms of its quickness and low difficulty level, several criticisms exist that should be discussed. Research has shown that results from Wenner Probe testing can vary significantly if the degree of saturation or conductivity of the concrete is inconsistent (32). A few techniques are suggested to achieve more uniform saturation, such as vacuum saturation or submerging the samples in water overnight. It can be assumed that this problem was mitigated on the SR tests performed by UT due to the lime-water bath curing of the test specimens. Complications with field testing of concrete with the Wenner Probe have also been reported. *In situ* field readings can give misleading results when the SR meter comes in close proximity to steel reinforcement. The reinforcing steel can cause a "short circuit" path and give a deceptively low surface resistivity reading for the concrete (32).

2.5.3 Relationship between RCP and SR Tests

It is important to understand how the RCP and SR tests relate to one another before the correlation between the two tests is discussed. As explained in Section 2.5.1, the RCP test is a measure of the electrical conductance of the concrete as an indication of its ability to resist chloride ion penetration. The SR test determines the electrical resistivity of the concrete as an indication of its resistance to chloride ion penetration. As Ryan and Buchanan explain, electrical conductivity, σ, and electrical resistivity, ρ, are inversely related to one another as shown in the following equation (22,59).

$$\rho = \frac{1}{\sigma}$$

Conductivity can be defined as an object's ability to conduct electrical current, while resistivity is the ability to resist electrical current.

In addition to the strong correlations between the SR and RCP tests that have been found by the UT research team, a review of literature shows similar results by other research groups. Kessler et al. conclude from research performed by the Florida Department of Transportation (DOT) that the SR test can not only be used as an electrical indicator of concrete permeability, but it can be used to replace the RCP test method. Kessler et al. go on to state in their research that the SR test has better precision than the RCP test (38). The Louisiana DOT also concludes that better precision exists in the SR test. Their research shows that the standard deviation of SR results is usually less than 3 kohm-cm compared to 300 to 500 Coulombs for the RCP

test (58). The combined SR/RCP results for samples tested at 28 days for the Florida DOT are shown in Figure 5. The correlation for these results is strong as demonstrated by the coefficient of determination, R^2, value of almost 0.95. Figure 6 shows the same graph for the Louisiana DOT's research results. Although not as high as Florida, Louisiana's results produce a strong R^2 value of 0.89.
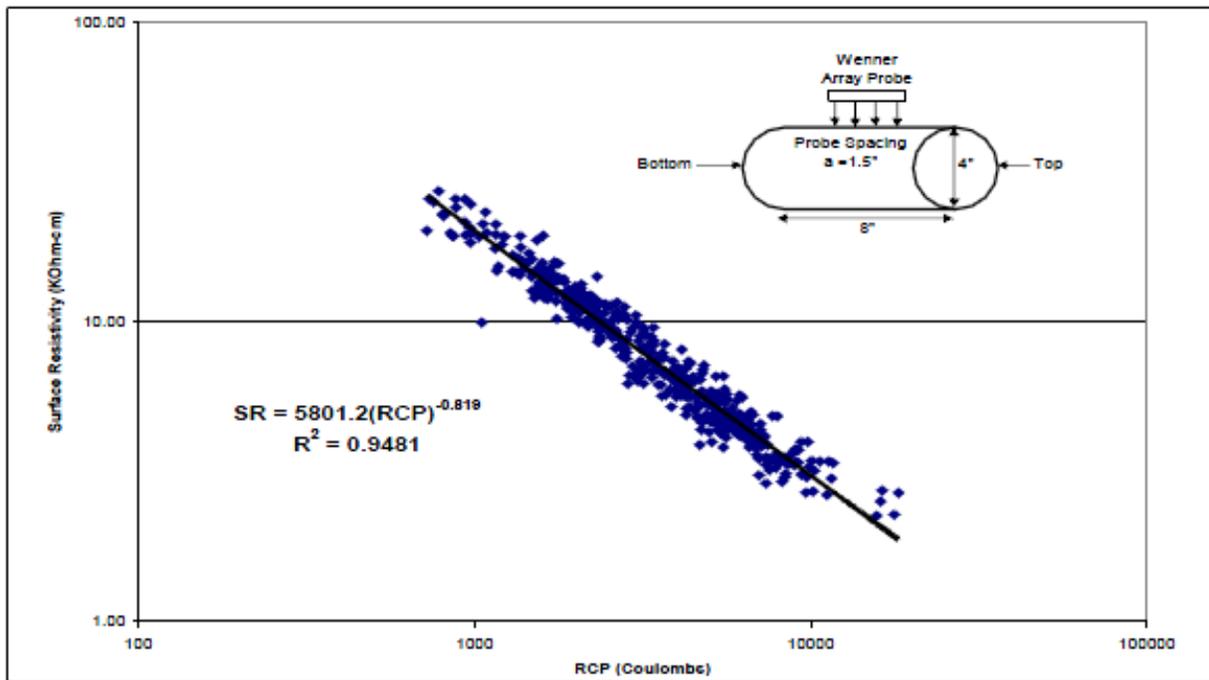
Figure 5: Florida DOT SR vs. RCP results – 28 days (39)

Figure 6: Louisiana DOT SR vs. RCP results – All Test Ages (58)

3.0   Testing Summary

This chapter provides a brief summary of the testing procedures followed by the UT research team on concrete samples prepared in both field and laboratory environments. The preparation of samples, curing methods, and description of testing procedures are addressed.

3.1   Testing of Field Samples

This section outlines the processes employed to receive the test cylinders from the field and the procedures that were followed to perform the laboratory tests once the cylinders were received.

3.1.1 Preparation and Delivery of Field Samples

Beginning in February 2010, concrete cylinders from bridge deck pours across the state of Tennessee have been received by the UT research team. The cylinders measure four inches in diameter by eight inches in length (4" x 8"), and are prepared on-site at the bridge deck by TDOT personnel before being transported to Knoxville. An efficient system of transportation of the concrete cylinders was created during Phase 1 of this project and is continuing to operate effectively. At each bridge deck pour, TDOT personnel prepare thirteen 4" x 8" concrete cylinders solely for this project. Once all thirteen cylinders are made, they are capped and placed on a flat surface for 18 to 24 hours for initial curing. The cylinders are then placed in a large marine cooler and are immersed in water for moist curing. A sealed PVC tube is included in the cooler for the necessary paperwork that TDOT will provide with information on the concrete mixture

design and acceptance results for the fresh concrete tests. The coolers are then delivered to the regional Materials and Tests center. Once the coolers reach the regional center, they are taken to the state headquarters of Materials and Testing in Nashville to await transport to the Region 1 facility in Knoxville. The coolers are placed in a controlled moist room at Region 1 until they are retrieved by the UT research team. Conveniently, TDOT's Region 1 headquarters, which encompasses all of East Tennessee, is located a short driving distance to the University of Tennessee campus, allowing the UT research team to pick up all concrete samples at this location. As would be expected, concrete samples prepared in Region 1 are able to bypass the trip to state headquarters in Nashville and are delivered directly to the Knoxville facility.

3.1.2 Curing of Field Samples

Once the coolers arrive in Knoxville and are picked up at the TDOT facility, they are brought back to UT and are subjected to a controlled curing environment. Upon removal of the cylinders from their molds, they are placed in a lime water storage tank in accordance with ASTM C31 and ASTM C511 (10,11). Curing time for the samples will vary depending on the concrete age required for a certain test and the length of time between initial mixing of the samples and their delivery date to Knoxville. For example, samples from Region 4 may take one or two weeks before they are placed in the lime water tank due to transportation delays, while samples from Region 1 may be placed in the tank within a few days after mixing. Thankfully, this variation in curing practices is assumed to be only a minor inconsistency due to the immersion of the samples in water while sitting in the coolers.

3.1.3 Description of Testing Procedures

The primary testing focus of this research is the Rapid Chloride Ion Penetration (RCP) test (16) and the Surface Resistivity (SR) test (8). The RCP test is a destructive test while the SR test is deemed to be non-destructive. Thus, both tests are able to be performed on the same cylinders enabling a valid comparison to be made between the two tests. Of the thirteen cylinders, three are used for seven-day compressive strength testing in accordance with ASTM C39 (12). These tests are generally performed at UT if the samples come from Regions 1 or 2. However, samples that come from bridge deck locations in Regions 3 and 4 will typically be tested for seven-day compressive strength prior to arrival at Region 1. Three more cylinders are tested for 28-day compressive strength at the UT facilities. Of the remaining seven cylinders, three are used for SR and RCP testing at 28 days and three are used for the same tests at 56 days. The extra cylinder is saved primarily in case another cylinder is physically defected, but it is also used for SR testing at 28, 56, and 91 days.

3.2   Testing of Lab Samples

This section outlines the preparation process, curing methods, and testing procedures for the test cylinders and shrinkage prisms prepared in the UT laboratory during this research project.

3.2.1 Preparation of Lab Samples

Several lab mixtures were prepared in the laboratory at UT to determine the effects of ternary blend concrete mixtures on concrete's ability to resist chloride ion penetration. A total of two binary concrete mixtures and three ternary concrete mixtures were prepared and tested during this research. The binary concrete mixtures resembled the TDOT mixture design currently being used on bridge decks; it contained 75% portland cement and 25% Class F fly ash. Table 5 provides details of this mixture design. The ternary concrete mixtures followed the same mixture design used by TDOT on bridge decks in Tennessee with the only exception being that slag cement was used in addition to portland cement and Class F fly ash. The cementitious materials contents of this mixture were as follows: 50% portland cement, 30% Grade 100 slag cement, and 20% Class F fly ash. Table 6 provides details of this mixture design. For each lab mixture, thirteen 4" x 8" test cylinders were prepared in accordance with ASTM C31 (11). In addition to the formation of 4" x 8" test cylinders, shrinkage prisms were

Table 5: Binary Lab Mixture Design

W/C Ratio		0.37
Total Cementitious Material Content (lb/yd^3)		620
Cement (lb/yd^3)		465
Fly Ash (lb/yd^3)		155
Water (lb/yd^3)		229
Aggregates (lb/yd^3)	#57 Limestone	1854
	Natural Sand	1204

Table 6: Ternary Lab Mixture Design

W/C Ratio		0.37
Total Cementitious Material Content (lb/yd^3)		620
Cement (lb/yd^3)		310
Fly Ash (lb/yd^3)		124
Slag (lb/yd^3)		186
Water (lb/yd^3)		229
Aggregates (lb/yd^3)	#57 Limestone	1854
	Natural Sand	1204

formed for the three ternary mixtures to determine shrinkage behavior of ternary

concrete mixtures. The shrinkage prisms were formed in accordance with ASTM C157

(9) and ASTM C490 (13). For each mixture, three different prisms were formed. Two

different prism mold sizes were used during this testing for the purpose of convenience.

The first two mixtures were formed in 3" x 3" x 11¼" molds. The third mixture was

formed in a 4" x 4" x 11¼" mold.

3.2.2 Curing of Lab Samples

One of the many benefits of laboratory samples as compared to samples

prepared in the field is the ability to properly cure the concrete samples. Once all

thirteen of the lab cylinders were made, they were capped and placed on a flat surface

for 18 to 24 hours for initial curing. Upon completion of this step, they were immediately

stripped from their molds and placed in a lime water storage tank in accordance with

ASTM C31 and ASTM C511 (10,11). Unlike samples obtained from actual bridge deck

pours, curing procedures were the same for all cylinders due to the absence of transportation delays. The cylinders were able to be fully immersed in a controlled lime water storage tank from the beginning as opposed to sitting in a marine cooler.

Curing of the shrinkage prisms was performed in a similar manner to the cylinders. Once the shrinkage prisms were formed, they were placed on a flat surface for 18 to 24 hours for initial curing. Extra care was taken to cover the prisms with moist cloths and plastic sheets to prevent loss of moisture of the prisms. Once the initial curing process was complete, the prisms were removed from their molds and placed in a lime water storage tank in accordance with ASTM standards (9-11). After seven days of moist curing, the shrinkage prisms were removed from the tank and placed in a controlled humidity room set for 50 ± 4% relative humidity and a temperature of 73 ± 3°F in accordance with ASTM C157 (9). The controlled humidity room is shown in Figure 7.

Figure 7: Storage of Shrinkage Prisms in Controlled Humidity Room

3.2.3 Description of Testing Procedures

The testing procedures for the laboratory cylinders were identical to the procedures for cylinders prepared in the field. Of the thirteen cylinders, three were used for seven-day compressive strength testing and three were used for 28-day compressive strength testing in accordance with ASTM C39 (12). Of the remaining seven cylinders, three were used for SR and RCP testing at 28 days and three were used for the same tests at 56 days. The extra cylinders were saved primarily in case another cylinder was physically damaged, but they were also used for SR testing at 28, 56, and 91 days.

The testing procedures for the shrinkage prisms were performed in accordance with ASTM C157 (9) and ASTM C490 (13). Shrinkage values for the prisms were measured using a length comparator as described in ASTM C490 (13). Figure 8 shows the testing apparatus used for this research. The initial measurements for the prisms

Figure 8: Length Comparator for Shrinkage Testing

were performed immediately after removal from the lime water bath. The prisms were

measured every day for the first week and then once a week thereafter.

4.0   Summary and Discussion of Results

This chapter summarizes and discusses the results of testing performed in the University of Tennessee (UT) lab on concrete cylinders collected from bridge deck pours across the state. A summary and discussion of results is also provided on the ternary blend lab study that was conducted by the UT research team. A standard RCP test is intended to be performed on a 95 mm, or 3.75 inches, cylinder. As noted in ASTM C1202, when using 4" x 8" cylinders for the RCP test, the total charge passed must be adjusted to account for the change in cross-sectional area. When using a 4" diameter cylinder, the total charge passed should be multiplied by the ratio of the two cross-sectional areas as described below (16):

$$Q_s = Q_x * \left(\frac{3.75}{x}\right)^2$$

$$When \; x = 4 \; inches, \qquad Q_s = Q_4 * \left(\frac{3.75}{4}\right)^2 = 0.879 * Q_4$$

With: Q_s = charge passed through a 3.75 inch diameter specimen
               Q_x = charge passed through x inch diameter specimen
               x = diameter of specimen (inches)

4.1   Field Samples

Through May 31, 2012, a total of 109 sets of cylinders have been received by the UT research team. As shown in Table 8 on page 65, 43% of the cylinders received came from Region 4 with a total of 47 sets of cylinders having been tested from this region. Region 2 is a distant second with 32 total sets of cylinders, followed by Region 1 with 21 sets and Region 3 with a meager 9 sets. A complete summary of all test

results for the field samples is located in Table A - 1 in the Appendix. Table 7

summarizes the chloride ion penetrability classification requirements for both the RCP

and SR tests as recommended by ASTM and AASHTO, respectively. As shown in

Table 8, the 28-day averages for the SR and RCP tests indicate concrete with moderate

to high chloride ion

penetrability characteristics. While the 56-day values for these tests improved as

expected relative to the 28-day values, the average chloride ion penetrability

classification for these samples was still considered to be moderate at 56 days

according to Table 7. The only exception was the 56-day SR and RCP values for

Region 1; samples from this region produced test values that were barely good enough

to be considered low penetrability according to the guidelines of Table 7. The

acceptable values for the Region 1 cylinders could be attributed to the simple fact that

these cylinders are not typically subjected to the transportation delays that may inhibit

Table 7: Chloride Penetrability Classification Based on RCP and SR tests (8,16)

Chloride Ion Penetrability Classification	ASTM C1202 Charge Passed (Coulombs)	AASHTO TP 95-11 Surface Resistivity (kohm-cm)
High	> 4,000	< 12
Moderate	2,000 – 4,000	12 – 21
Low	1,000 – 2,000	21 – 37
Very Low	100 – 1,000	37 – 254
Negligible	< 100	> 254

Table 8: Regional Breakdown of Field Results

	Number of Samples	Average 28-day Compressive Strength (psi)	Average 28-day SR value (kohm-cm)	Average 56-day SR value (kohm-cm)	Average SR Ratio (28-day / 56-day)	Average 28-day RCP value (Coulombs)	Average 56-day RCP value (Coulombs)
Region 1	21	5709	**14.5** (moderate)	**24.4** (low)	0.62	**3426** (moderate)	**1838** (low)
Region 2	32	5197	**12.4** (high)	**18.5** (moderate)	0.71	**4358** (high)	**2993** (moderate)
Region 3	9	4839	**13.9** (moderate)	**21.1** (low)	0.68	**4032** (high)	**2412** (moderate)
Region 4	47	5197	**11.2** (high)	**19.4** (moderate)	0.65	**5115** (high)	**2610** (moderate)
Averages / Totals	109	5235	**13.0** (high)	**20.9** (moderate)	0.67	**4233** (high)	**2463** (moderate)

*Words in parentheses indicate chloride ion penetrability classification.

proper curing in cylinders from other regions. Although the SR values for the Region 3 samples were barely high enough to be considered "moderate" and "low" penetrability at 28 and 56 days, respectively, not enough samples were received from this region to make a valid argument that concrete from this region was better equipped than Regions 2 and 4 to resist chloride ion penetration. Regardless of the moderate success of the Region 1 samples, the overall results for the SR and RCP testing conducted by UT demonstrate a clear need for more durable concrete on Tennessee bridge decks. The current status of bridge deck concrete is subpar in terms of the ability of the concrete to resist chloride ion penetration as shown by these results, and, if no changes are made to current concrete mixture designs, bridge decks in Tennessee can be expected to

continue to deteriorate at faster rates than would be expected if a more durable mixture were used.

The primary focus of Phase 1 of this research project was to evaluate the relationship between the SR and RCP tests and (a) to determine if there is a correlation between the results of the two tests and (b) to determine this correlation. As reported by Ryan (59) and Buchanan (22), test results thus far, in addition to numerous sources of literature, support the fact that these two tests are strongly correlated with one another. A valid way to statistically show the strength of correlation of a set of data points is to, first, draw a trend line through the data set, and, second, determine the coefficient of determination (R^2) for the trend line. The R^2 value measures how well the trend line represents the data. It has been determined by previous researchers at UT, as well as other scholars, that the strongest correlation between the SR and RCP tests is found when using an inverse power relationship. As research continues and more samples are tested, data collected by UT continue to show a strong correlation between these two tests.

As of May 31, 2012, the combined SR versus RCP plot for tests performed at ages of 28 and 56 days produces an R^2 value of 0.86. This R^2 value is consistent with research performed by the Louisiana and Florida DOT's. Figure 9 shows the SR versus RCP plot with a logarithmic scale; this can be compared to Florida's plot as seen in Figure 5 in Chapter 2. Figure 10 shows the SR versus RCP plot with a normal scale; this can be compared to Louisiana's plot as shown in Figure 6 in Chapter 2. The R^2 values were determined on both plots through the use of Excel.

66
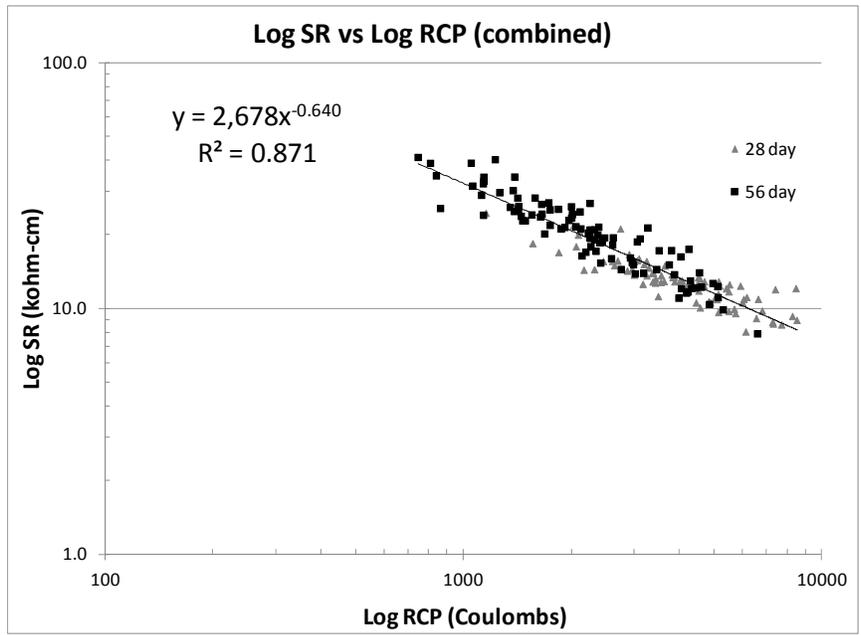
Figure 9: Combined Log SR versus Log RCP (28 and 56 Days)
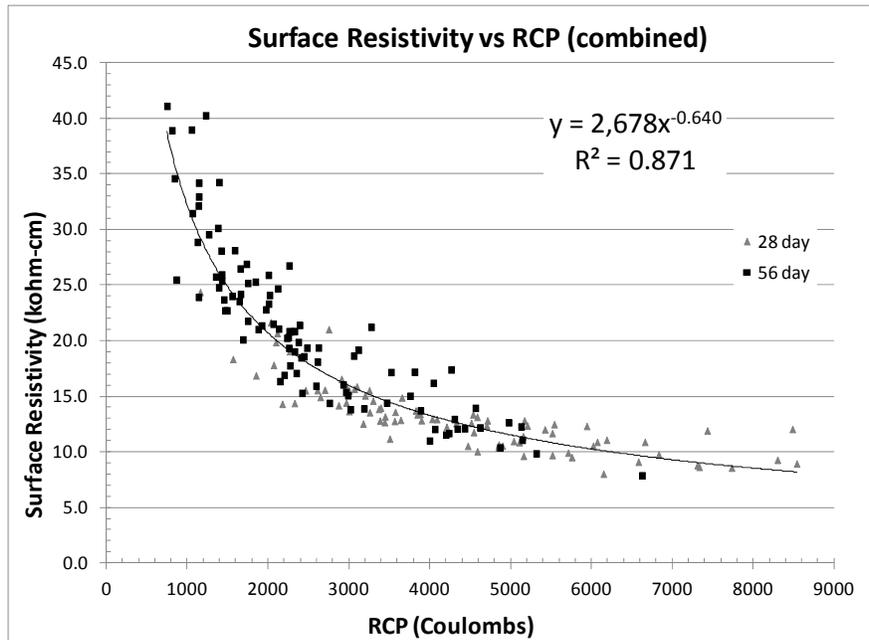
Figure 10: Combined SR versus RCP (28 and 56 Days)

Based on the values in Table 7 on page 64, a minimum SR value of 21 kohm-cm

and a maximum RCP value of 2,000 Coulombs should be attained in order for concrete

to achieve low chloride ion penetrability. Although the ASTM and AASHTO standards

do not specify an age that these values should be met in order to be considered

satisfactory, a 56-day extended moist curing period is recommended by ASTM C1202

to allow the supplementary cementitious materials present in the mixture adequate time

to reach their potential properties due to their slower rate of hydration (16). Thus, the

56-day RCP values are typically considered the acceptable values for this test. Figure

11 shows the correlation between the 56-day SR values and the 56-day RCP values.

The R^2 value of 0.84 on the plot shows that a relatively strong correlation exists

between the two tests at 56 days. In addition to the coefficient of determination, the
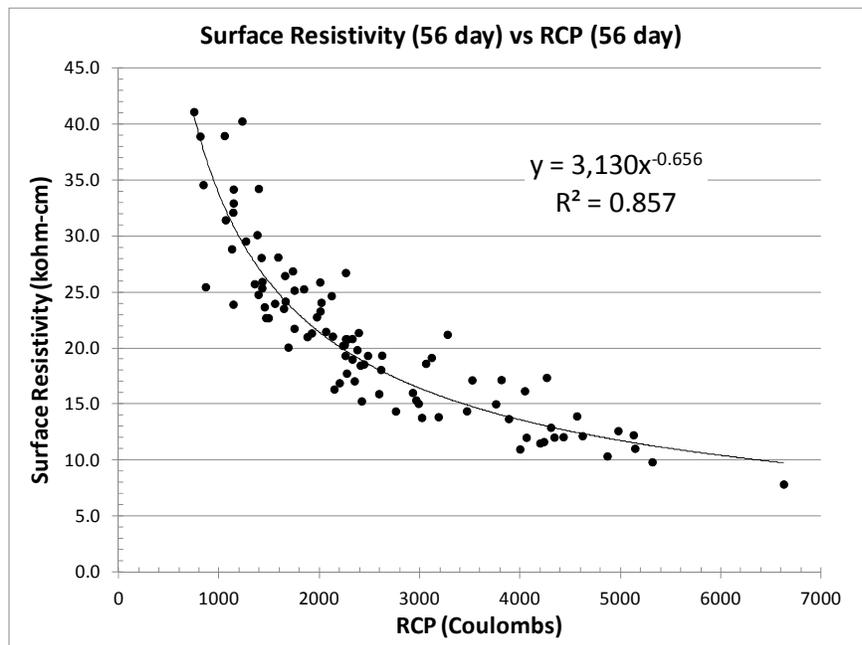
Figure 11: SR versus RCP (56 days)

validity of this correlation relative to ASTM and AASHTO limit definitions can be checked by using the equation that represents the trend line. In this equation, x is the independent variable and represents the 56-day RCP value, and y is the dependent variable and represents the 56-day SR value. When the maximum RCP value for low chloride ion penetrability of 2,000 Coulombs is substituted for x, an SR value of 21.4 kohm-cm is obtained.

$$Example: \qquad y \ = \ 3{,}130\,x^{-0.656}$$

$$y \ = \ 3{,}130 * (2{,}000)^{-0.656}$$

$$y \ = \ 21.4\ \text{kohm} - \text{cm}$$

The SR value of 21.4 kohm-cm is almost identical to AASHTO's lower limit for low chloride penetrability of 21.0 kohm-cm as classified by AASHTO TP 95-11 (8). As shown in Figures 12 and 13, the general trend lines for the SR versus RCP plots at both 56 days and all ages are closely related to the limits recommended by AASHTO TP 95-11 and ASTM C1202 (8,16). The AASHTO/ASTM limits plotted on the graphs in Figures 12 and 13 are shown in Table 7 on page 64. The cut-off limit definitions for low, medium, and high penetrability are shown in parentheses.

Several graphs detailing various correlations between the SR and RCP tests at different ages are shown in the appendix. A detailed discussion of these correlations, as well as recommendations on how to implement the SR values into a performance based specification, can be found in Master's theses written by two previous researchers on this project, Bryan Buchanan and Eric Ryan (22,59).

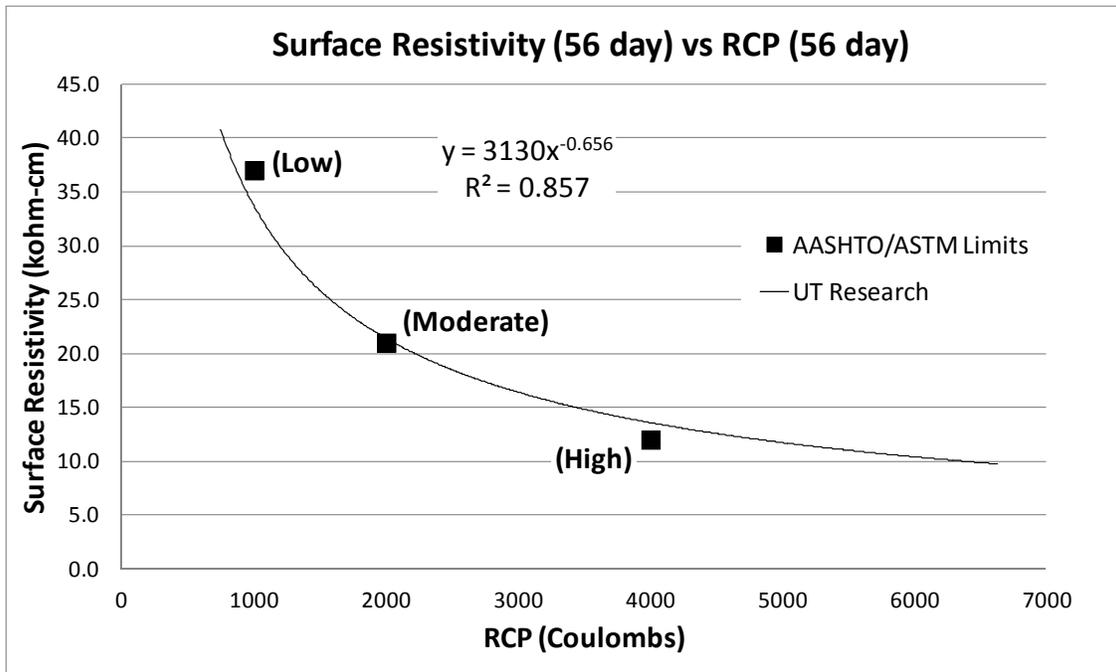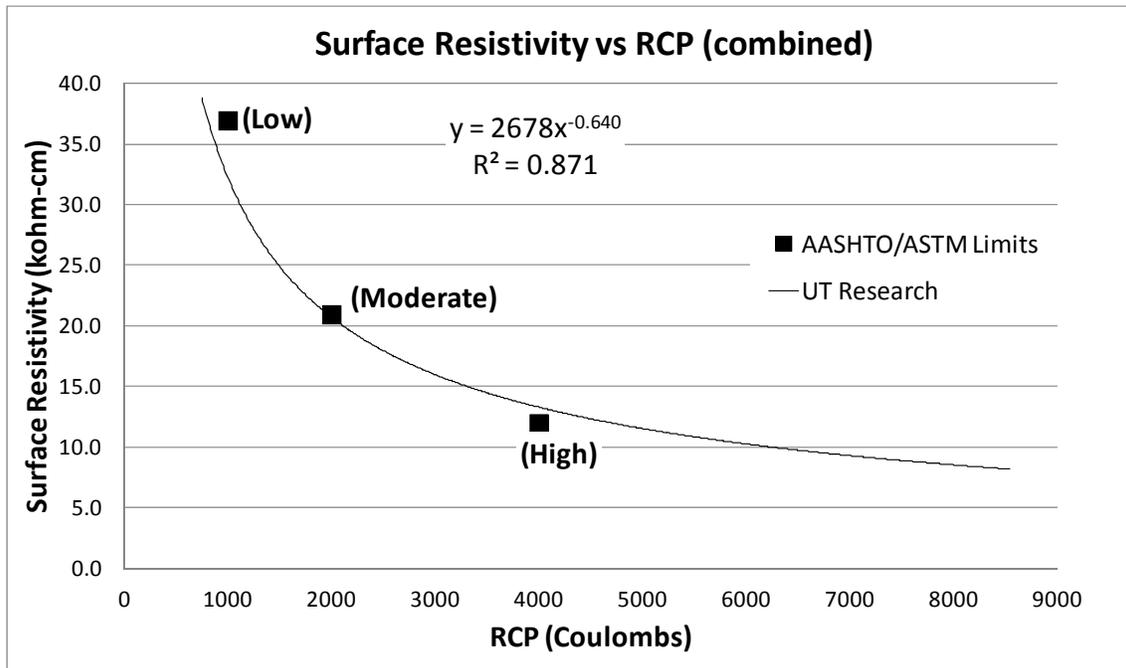**Figure 12: SR versus RCP (56 days) with AASHTO/ASTM Limits**

Figure 13: SR versus RCP (28 and 56 days) with AASHTO/ASTM Limits

4.2   Laboratory Samples

Tests performed on the laboratory samples showed great benefit in using a ternary blended concrete mixture as opposed to the typical binary, "Class D," mix currently used on bridge decks in the state of Tennessee. Results from the SR and RCP tests performed on the cylinders indicated significantly higher resistance to chloride ion penetration in the ternary mixtures as compared to the binary mixtures. Tables 9 and 10 show the compressive strength, SR, and RCP values for the three ternary mixtures and both of the binary mixtures, respectively. As shown in Table 9, the ternary lab mixtures indicated "very low" chloride ion penetrability as early as 28 days, while the binary mixtures as shown in Table 8 indicated "high" chloride ion penetrability at 28 days and only improved to "moderate" at 56 days. The SR and RCP values for the two binary mixtures closely resembled general values that are found in the "Class D"

Table 9: Ternary Lab Mixture Results

	28-day Compressive Strength (psi)	28-day SR value (kohm-cm)	56-day SR value (kohm-cm)	SR Ratio (28-day / 56-day)	28-day RCP value (Coulombs)	56-day RCP value (Coulombs)
Ternary Mix # 1	6036	**42.4** (very low)	**54.6** (very low)	0.78	**836** (very low)	**521** (very low)
Ternary Mix # 2	7541	**42.2** (very low)	**59.6** (very low)	0.71	**821** (very low)	**545** (very low)
Ternary Mix # 3	9168	**53.8** (very low)	**67.7** (very low)	0.80	**640** (very low)	**494** (very low)
Average	7582	**46.2** (very low)	**60.6** (very low)	0.76	**766** (very low)	**520** (very low)

Words in parentheses indicate chloride ion penetrability classification.

Table 10: Binary Lab Mixture Results

	28-day Compressive Strength (psi)	28-day SR value (kohm-cm)	56-day SR value (kohm-cm)	SR Ratio (28-day / 56-day)	28-day RCP value (Coulombs)	56-day RCP value (Coulombs)
Binary Mix # 1	6664	**12.9** (moderate)	**23.3** (low)	0.56	**4027** (high)	**2005** (moderate)
Binary Mix # 2	5587	**11.8** (high)	**19.0** (moderate)	0.62	**4540** (high)	**2325** (moderate)
Average	6126	**12.4** (moderate)	**21.2** (low)	0.59	**4284** (high)	**2165** (moderate)

*Words in parentheses indicate chloride ion penetrability classification.

concrete mixtures that are tested on a routine basis in the UT laboratory. Thus, it can be generally concluded that a ternary mixture will greatly enhance the concrete's ability to resist chloride ion penetration on a bridge deck as that resistance is determined by either the RCP or SR tests. The 28-day compressive strengths for the ternary blended mixtures were also significantly higher than the 28-day strengths for the binary mixtures.

While all of the correlation studies on the SR and RCP tests thus far in UT's research have only included "Class D" mixtures, it is interesting to note the impact that the inclusion of data from the three ternary blend lab mixtures has on the overall correlation between the two tests. The R^2 values on all of the correlation graphs improve when the three ternary blend lab mixtures are included in the data. While the reasoning for these improved correlations is uncertain, one possible explanation can be traced to how well the ternary lab mixtures appear to be correlated with one another. For example, ternary lab mixtures 1 and 2 have almost identical SR and RCP values at

both 28 and 56 days. In addition to the strong correlation between these ternary

mixtures, they possess significantly higher SR and lower RCP values than all of the

other "Class D" concrete mixtures, causing the data points to be located in a separate

region on the SR versus RCP plots. The inclusion of this well-correlated set of data at a

location outside of the typical area on the SR versus RCP plots helps to improve the

correlations by expanding the curve and providing the curve with more data points that

will lie in close proximity to the trend line. Figures 14 and 15 show the positive effects

that the ternary blend lab mixtures have on the correlation between the two tests at 56

days and at both test ages of 28 and 56 days, combined.
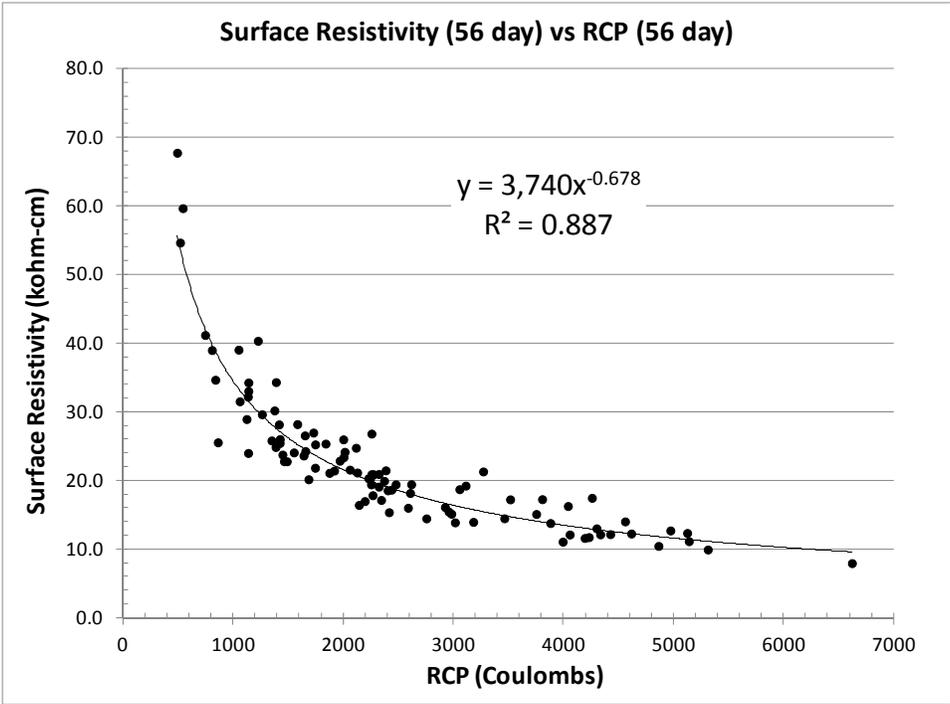
Figure 14: SR versus RCP (56 days) – With Ternary Blend Mixtures
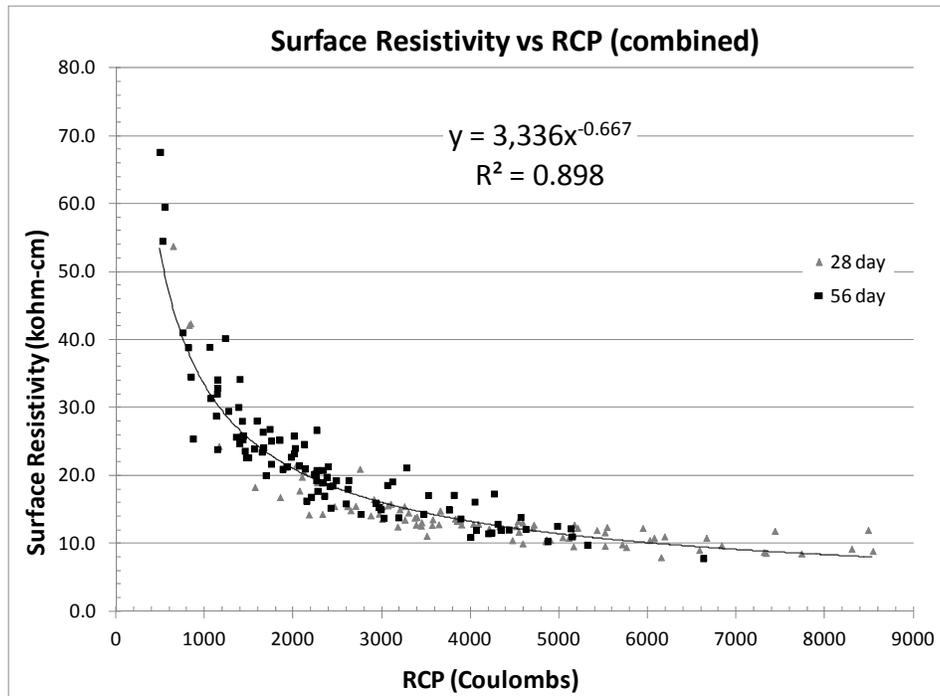
Figure 15: SR versus RCP (28 and 56 days) – With Ternary Blend Mixtures

As discussed in the review of literature in Chapter 2, the ability of concrete to resist chloride ion penetration becomes less significant if large enough cracks develop on the bridge to allow chlorides direct access to the reinforcing steel. A common cause of cracking on bridge decks is drying shrinkage of the concrete. To address concerns of drying shrinkage in ternary blend concrete mixtures, length change measurements were taken on shrinkage prisms for all three mixtures as discussed in Chapter 3. As reported by Mokarem, percentage length change of a concrete specimen containing supplementary cementitious materials should be limited to 0.0400 at 28 days and 0.0500 at 90 days (48). Shrinkage values measured by the UT research team were compared to these limits to determine the adequacy of the ternary mixtures to resist drying shrinkage.

The shrinkage results for the ternary mixtures were exceptional. At 28 days, mixtures number 1, 2, and 3 had average percentage length changes of 0.029, 0.024, and 0.030, respectively. At 90 days, mixtures number 1, 2, and 3 had average percentage length changes of 0.031, 0.025, and 0.033, respectively. Even at 90 days, these values do not exceed the recommended 28-day percentage length change of 0.0400 as specified by Mokarem (48). Figures 16-18 on the following pages show the plots of shrinkage versus time for each ternary mixture. It should be noted that several spikes in the curves exist that would appear to show positive and negative length changes for the prisms. These deviations in the curves can be attributed to either operator errors or technical errors with the length comparator. Regardless of the rare spikes in the shrinkage versus time plots, it is clear that most of the shrinkage occurs early on in the concrete's aging process, as would be expected. Also, shrinkage values for all three mixtures began to almost "level off" as the concrete aged. A viable conclusion can be drawn from the lab tests that shrinkage of ternary blended concrete should be of no concern when implementing its use on a bridge deck.
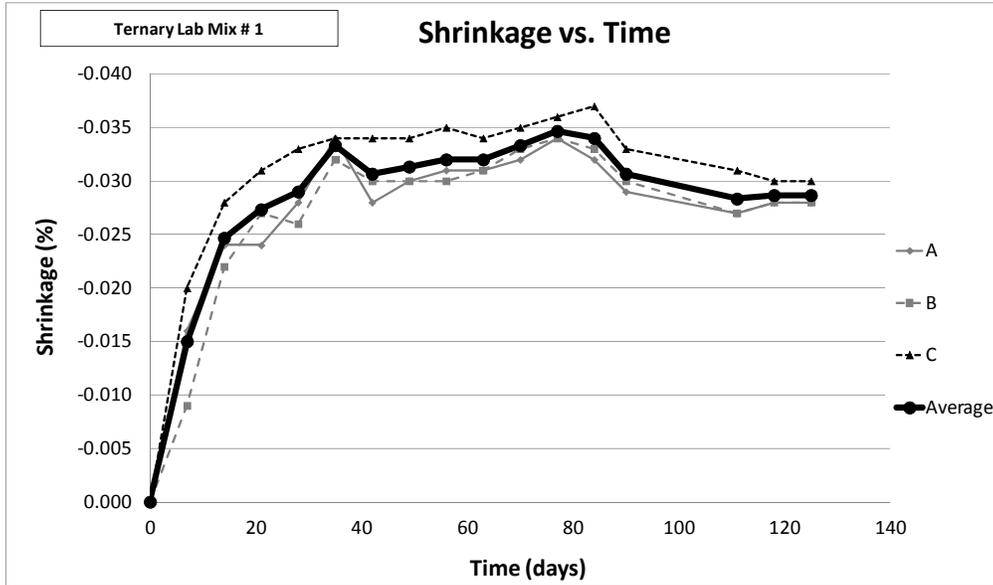
Figure 16: Shrinkage Measurements – Ternary Lab Mixture # 1
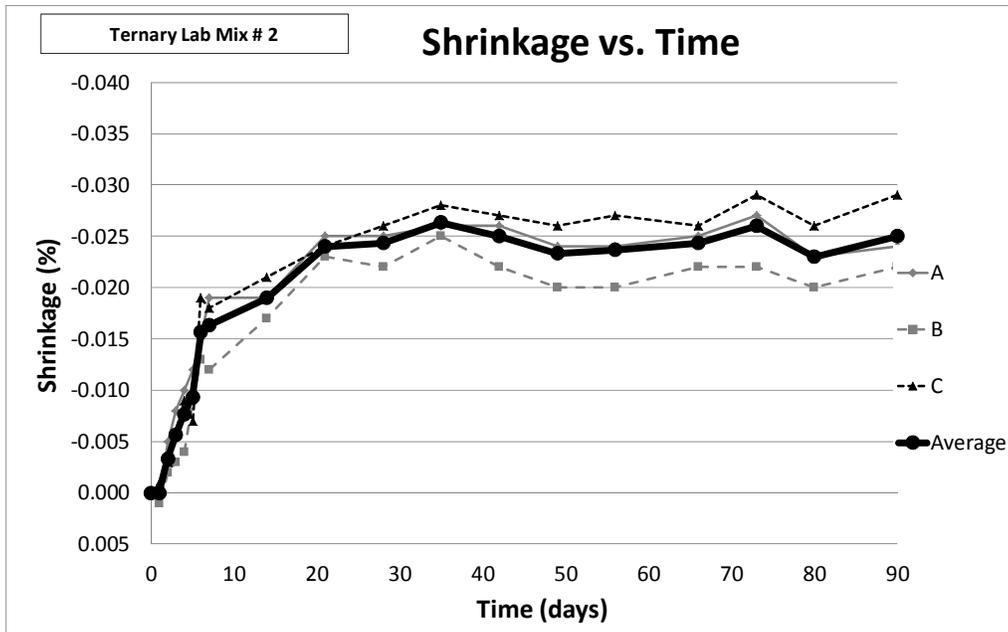
Figure 17: Shrinkage Measurements – Ternary Lab Mixture # 2
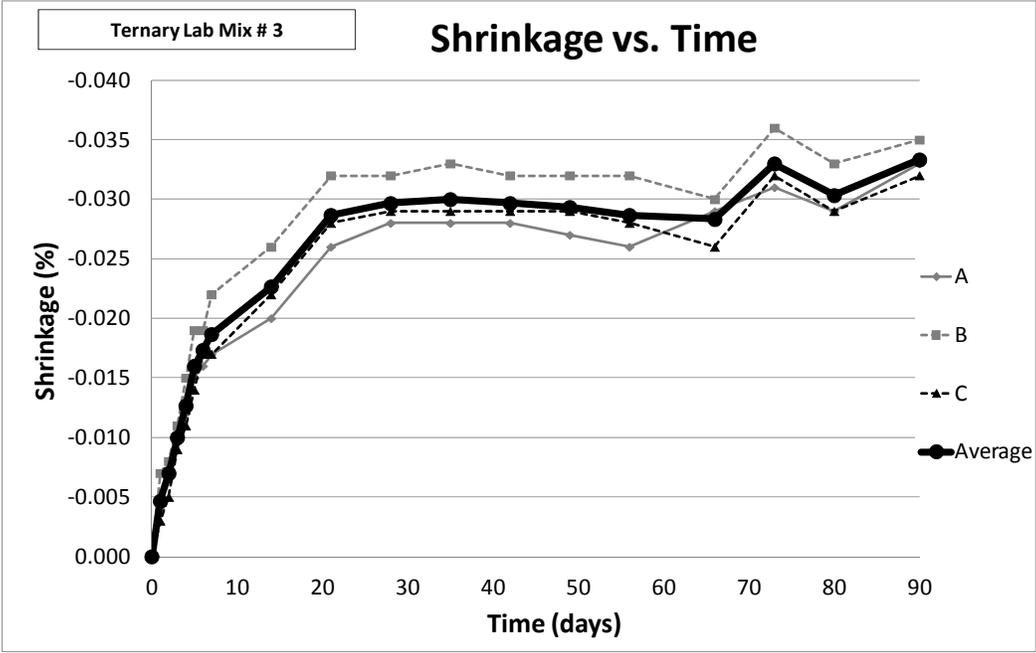
Figure 18: Shrinkage Measurements – Ternary Lab Mixture # 3

5.0 Proposal of Ternary Blend Specification

In addition to the use of quality materials and satisfactory construction practices, proper mixture proportioning is one of the key factors in producing durable concrete. As stated by Neville, concrete durability may be affected by either external causes such as the environment to which the concrete is exposed or by internal causes within the concrete itself (51). As discussed in Chapter 2, the external causes can be attributed to factors such as weathering, abrasion, freezing and thawing, or chemical attacks. The internal causes can be attributed to alkali-aggregate reactions, volumetric changes, or the permeability of the concrete. Neville suggests that permeability of concrete is the most important factor that affects concrete durability. He states that permeability "largely determines the vulnerability of concrete to external agencies, so that, in order to be durable, concrete must be relatively impervious" (51). Thus, proportioning a low permeability mixture is essential in trying to achieve durable, long-lasting concrete.

The use of supplementary cementitious materials (SCM) in concrete mixtures has been found to significantly reduce the permeability of   concrete. In addition to the benefit of lower permeability, SCMs have been found to enhance concrete's ability to resist sulfate attack and alkali-silica reaction. The lower heats of hydration of SCMs such as fly ash and slag cement are also thought to mitigate problems with thermal cracking on concrete bridge decks. Although early compressive strengths of concrete mixtures containing SCMs may be lower than typical portland cement mixtures due to slower strength gains, the ultimate strengths are generally higher than a mixture

containing only portland cement. Thus, a ternary concrete mixture offers several

advantages in terms of both strength and durability.

Although the SR and RCP tests are not direct measures of concrete permeability,

it is widely believed that the ability of concrete to resist chloride ion penetration is a

valid, albeit indirect, indicator of concrete permeability. Therefore, the SR and RCP test

results attained thus far in UT's research on bridge deck concrete durability can be used

as an acceptable indicator of the current status of bridge deck concrete in Tennessee.

As shown in Chapter 4, results from the testing show that the typical concrete mixture

currently used on Tennessee bridge decks is characterized as having "high" chloride

penetrability at 28 days and only improves to "moderate" penetrability at 56 days

according to the ASTM and AASHTO standards. Concrete of this quality is not

satisfactory in terms of protecting the reinforcing steel from corrosion.

The current specification for bridge deck concrete in Tennessee as prescribed by

TDOT is known as the "Class D" mixture. Section 604 in TDOT's "Standard

Table 11: Current "Class D" Mixture Proportions (65)

Current Class D Mixture				
Minimum 28 Day Compressive Strength (psi)	Minimum Cementitious Material (lbs/c.y.)	Maximum Water/Cem. Material Ratio (lb/lb)	Air Content % (Design ± Production Tolerance)	Slump (in.)
4,000	620	0.40	6	8 max

1. *Class D concrete shall be designed at 6% air content, acceptance range for pumping is 6.0-8.5%, acceptance range for other placement methods is 5.0-8.5%.*
2. *The slump before the addition of high range water reducing admixtures (HRWRA) shall be 3 inches maximum.*
3. *The fine aggregate shall not exceed 44% by volume calculation of the total aggregate.*

Specification for Road and Bridge Construction" provides details on the "Class D" mixture (65). The current "Class D" mix proportions are shown in Table 11. Additional guidelines as provided in the current specification are shown below the table. Currently, only one portland cement according to TDOT's specifications. Thus, ternary concrete mixtures supplementary cementitious material is permitted for use as a partial replacement to are not currently used for bridge deck applications in the state of Tennessee; binary concrete mixtures containing Class C or Class F fly ash are common but are not required. Three types of "Class D" mixtures are currently used in practice according to the TDOT mixture designs received by the UT research team for testing purposes. They are summarized as follows:

1. 100% portland cement
2. 80% portland cement, 20% Class F fly ash
3. 75% portland cement, 25% Class C fly ash

The maximum cement replacement rates for supplementary cementitious materials as required by TDOT are shown in Table 12. The term "modifier" in Table 12 refers to the

Table 12: Maximum Portland Cement Replacement Rates (TDOT) (65)

Modifier	Maximum % Cement Replacement Rate (by weight)	Minimum Modifier Cement Substitution Rates (by weight)
Ground Granulated Blast Furnace Slag (grade 100 or 120)	35.0	1:1
Class F Fly Ash	20.0	1:1
Class C Fly Ash	25.0	1:1

supplementary cementitious material. The TDOT specifications are not clear as to the permissibility of using a ternary blend mixture. The specifications specifically permit the use of a ternary blend for "Class A" concrete. While the use of a ternary blend is not specifically forbidden for "Class D," such a mixture was not used for any of the cylinders received on this project.

Upon review of the current status of bridge deck concrete in Tennessee, the UT research team is proposing that the current "Class D" specification be modified to require that a ternary blend be used on all bridge deck concrete applications. While this modification may appear to be groundbreaking in terms of TDOT's current concrete specifications, it is interestingly quite similar to the current Class A specification which is currently used on almost all structural concrete applications excluding bridge decks. Table 13 shows the mixture requirements for Class A concrete as given by TDOT. The following excerpt is taken directly from Section 604 of TDOT's specification for concrete structures:

> "Ternary cementitious mixtures (mixtures with portland cement, slag cement, and fly ash) will be allowed for Class A Concrete provided that the minimum portland cement content is 50%. The maximum amount of fly ash substitution in a ternary blend will be 20%. Substitution rates will be at a 1:1 ratio (65)."

Table 13: "Class A" Mixture Proportions (65)

Class A Mixture				
Minimum 28 Day Compressive Strength (psi)	Minimum Cementitious Material (lbs/c.y.)	Maximum Water/Cem. Material Ratio (lb/lb)	Air Content % (Design ± Production Tolerance)	Slump (in.)
3,000	564	0.45	6 ± 2	3 ± 1

The "Ternary Class D" mixture that UT is proposing can be considered a hybrid of both the Class A and Class D mixtures. The lower cementitious materials content and the ternary blend reflect the Class A mixture, while the minimum 28-day compressive strength and w/cm ratio reflects the current Class D mixture. Slump and air content requirements will follow the guidelines of the current Class A specification. It should be noted that instead of placing a minimum cementitious materials content of 564 lbs/yd^3 as specified in the Class A mixture, a maximum cementitious materials content of 575 lbs/yd^3 will be required in the "Ternary Class D" mixture. Table 14 shows the requirements for the "Ternary Class D" mixture. The percentages for portland cement replacement by the SCMs will be as shown in Table 15. An example of a "Ternary Class D" mixture with actual batching amounts is shown in Table 16.

Although no performance based requirements are included in this proposed mixture specification, it is expected that "Ternary Class D" concrete will exhibit significantly lower permeability and better resistance to chloride ion penetration. Thus, better SR and RCP values are clearly expected from implementation of this mixture. The lower amount of total cementitious material will reduce drying shrinkage as the amount of drying shrinkage is increased when the total cementitious materials and

water contents of a mixture are increased. It can also be ensured that the concrete will exhibit a high resistance to freezing and thawing damage as long as a low w/cm ratio and proper air void system are maintained in the mixture.

Table 14: "Ternary Class D" Mixture Proportions

Ternary Class D Mixture				
Minimum 28 Day Compressive Strength (psi)	Maximum Cementitious Material (lbs/c.y.)	Maximum Water/Cem. Material Ratio (lb/lb)	Air Content % (Design ± Production Tolerance)	Slump (in.)
4,000	575	0.40	6 ± 2	3 ± 1

Table 15: Cementitious Material Percentages

Cementitious Material	Percentage of Total Cementitious Material (%)
Portland Cement	50.0
Slag Cement (Grade 100 or 120)	30.0
Fly Ash (Class C or F)	20.0

As detailed in Section 604, the minimum portland cement content for a ternary mixture is 50%. The maximum amount of fly ash substitution as required by TDOT for a ternary blend is 20% (65).

Table 16: "Ternary Class D" Mixture Example

Ternary Class D Mixture	
Portland Cement	288 lbs/yd^3
Fly Ash (Class C or F)	115 lbs/yd^3
Slag Cement (Grade 100 or 120)	172 lbs/yd^3
#57 Limestone	1901 lbs/yd^3
Natural Sand	1216 lbs/yd^3
Water	230 lbs/yd^3
Design W/CM Ratio	0.40
Design Air Content	6 %
Required 28 Day Compressive Strength	4,000 psi

Course and Fine Aggregate contents came from UT-565 ternary mixture as discussed in a 2008 final report, "Development of High Performance Concrete Mixture for Tennessee Bridge Decks," by Dr. Edwin G. Burdette, Dr. J. Harold Deatherage, and Andrew Tinsley (71).

6.0   Conclusion

Long-lasting, durable, concrete bridge decks have long been the goal of state departments of transportation nationwide. The current status of bridge deck concrete in Tennessee indicates that this goal cannot be met if the current "Class D" concrete specification continues to govern bridge deck operations. The SR and RCP results from UT's testing program show that the concrete currently in place on Tennessee bridge decks is often too permeable to offer adequate resistance to chloride ion penetration. Thus, early onset corrosion of the reinforcing steel can be expected to cause critical durability issues with the bridge deck long before the expected service life has been reached.

The switch to a ternary blend specification for bridge deck concrete in Tennessee is clearly needed. The financial and technical benefits of a ternary concrete mixture on bridge decks should not be ignored as the positive effects that a ternary blend potentially has on both the strength and durability properties of concrete are significant. Three main conclusions were drawn from this research on ternary blended concrete mixtures:

1. Ternary blended concrete mixtures will exhibit lower permeability than both portland cement mixtures and binary mixtures containing two cementitious materials.

2. The difference in drying shrinkage of ternary concrete mixtures versus typical portland cement mixtures of identical cementitious materials and water contents

can be considered negligible. The factors that play larger roles in drying

shrinkage of concrete mixtures are the total cementitious materials content and

total water content.

3. While a ternary blend will exhibit lower early age strengths in typical concrete

   mixtures due to the lower heats of hydration of the SCMs, it can generally be

   expected that higher ultimate strengths will be achieved with ternary concrete

   mixtures as compared to portland cement mixtures.

Several other conclusions regarding a ternary concrete mixture were found from the

review of literature and can be summarized as follows:

1. It is generally concluded that ternary concrete mixtures offer better resistance to

   carbonation due to the lower permeability of the mixture as compared to ordinary

   portland cement mixtures.

2. A ternary blend offers better resistance to alkali-silica reaction and sulfate attack

   than a concrete mixture containing only portland cement.

3. The resistance of concrete to freezing and thawing action is relatively the same

   in ternary mixtures and typical portland cement mixtures. The main factor in

   concrete's resistance to freezing and thawing damage is proper air entrainment

   of the concrete.

4. Ternary concrete mixtures will be less susceptible to thermal cracking as compared to typical portland cement mixtures due to the lower heat of hydration of supplementary cementitious materials such as fly ash and slag cement.

5. The difference in a ternary concrete mixture's resistance to abrasion and erosion as compared to an ordinary portland cement mixture is negligible.

6. Ternary concrete mixtures containing portland cement, fly ash, and slag cement will be less expensive than a typical portland cement mixture. The cost of a ternary blend as compared to a binary mixture containing fly ash and portland cement will be more closely related due to the smaller price difference between slag cement and portland cement. Fly ash is the least expensive of the three materials. Slag cement is slightly less expensive than portland cement, on average.

7. Availability of slag cement for use in ternary concrete mixtures on bridge decks should be of no significant concern. The only issue that may arise to ready mix producers is the lack of extra silos to store more than two different types of cementitious materials. Typically, ready mix plants will have two storage silos on site, one for fly ash and one for portland cement. However, many of the larger ready mix plants are fully capable of meeting this need, and some are already stocking slag cement.

Implementation of a ternary blend concrete specification is important in reducing long-term maintenance costs and improving the overall safety and durability of bridge decks in the state of Tennessee. The positive financial implications and fewer traffic delays due to bridge deck repairs will please both government officials and the general public alike.

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Appendix

Figures and Tables

Table A - 1: Data Summary through May 31, 2012

Date	Region	County	28 day f'c (psi)	28 day SR	56 day SR	(28/56) Ratio	28 day RCP	56 day RCP	Comments
2/22/2010	4	Carroll	6239	9.8	17.1	0.57	6829	2347	Not typical Curing
3/13/2010	4	Henderson	5570	10.5	20.9	0.50	4468	2325	28 day f'c done at 24 days by TDOT
3/15/2010	2	Hamilton	5488	8.0	11.1	0.72	6146	5142	
3/16/2010	1	Cocke	5351	12.7	21.0	0.60	3438	1876	
3/17/2010	1	Knox	6737	14.4	25.8	0.56	2325	1351	
3/30/2010	2	Hamilton	5096	11.4	12.9	0.88	5152	4303	
4/6/2010	1	Carter	5358	13.4	19.2	0.70	4532	3114	Identification Scratched on tops
4/22/2010	1	Blount	5576	17.8	31.5	0.57	2066	1062	
5/3/2010	1	Knox	4230	15.5	26.8	0.58	3249	2259	
5/25/2010	4	Haywood	4249 (14day)	12.0	21.2	0.57	8483	3273	f'c taken at 14 days by TDOT
6/9/2010	2	Coffee	4653	8.9	12.1	0.74	8536	4337	
6/10/2010	2	Clay	6740	21.0	26.9	0.78	2748	1731	
6/23/2010	1	Union	4840	14.9	24.7	0.60	3653	2118	
7/2/2010	3	Williamson	3604	12.6	19.4	0.65	4511	2479	
7/2/2010 (2)	2	Polk	5610	12.3	17.2	0.72	5204	3809	
7/6/2010	3	Davidson	3743	13.6	18.6	0.73	3570	3058	Very Poor Compacted/ Not smooth
7/8/2010	4	Madison	7627 (@HQ)		7.9			6623	Rec'd After 28 days
7/15/2010	4	McNairy	4729	6.4	6.9	0.93			28 and 56 RCP Maxed out machine

Table A - 1: Data Summary through May 31, 2012 (Continued)

7/27/2010	4	Madison	4305 (14day)	12.8	20.8	0.62	5167	2278	f'c taken at 14 days by TDOT
8/10/2010	3	Davidson	4155	12.8	21.1	0.61	4710	2129	
8/14/2010	4	Henderson	4117	9.3	15.2	0.61	8298		
8/19/2010	4	McNairy	4898	7.0	8.1	0.87			Maxed out 28/56 RCP - not usable
9/1/2010	4	Lake	4393	12.8	23.6	0.54	3562	1642	
9/3/2010	1	Sevier	6483	20.7	34.6	0.60	2111	841	
9/8/2010	4	Gibson	4751	14.2	28.1	0.50	2870	1419	
9/11/2010	2	Hamilton	3835	14.4	30.1	0.48	2964	1378	
9/14/2010	1	Sevier	6076	19.9	38.9	0.51	2094	810	
9/21/2010	3	Davidson	4887	11.2	15.3	0.73	3502	2418	
9/28/2010	2	Warren	4884	12.9	21.8	0.59	3637	1747	
10/5/2010	2	Warren	5114	15.5	22.7	0.68	2460	1465	
10/12/2010	2	Warren	5219	13.7	25.4	0.54	3824	1426	
10/14/2010	2	Warren	4765	10.4	20.1	0.52	4844	1687	
10/21/2010	3	Williamson	5125	14.0	25.3	0.55	3390	1842	
10/27/2010	3	Montgomery	8948	24.4	41.1	0.59	1158	748	High Strength Concrete
11/2/2010	4	Decatur	4101	8.8	20.2	0.43		2231	Didn't do 28 RCP max out
11/4/2010	4	Shelby	9018	15.5	26.0	0.60	3824	1426	Prestressed. Missed 28 day Holidays
11/19/2010	4		5260	10.2	19.4	0.53		2620	No NaOH for 28 day test

12/22/2010	2	McMinn	5891	12.5	15.0	0.83	5537	3756	
1/4/2011	4	Haywood	4443	10.9	17.8	0.61	5105	2269	
1/19/2011	4	Gibson	5272	9.5			5754		Missed 56 day testing
1/28/2011	2	Polk	6131	15.6	15.1	1.04	3064	2984	
1/28/2011 (2)	2	Warren	4547	16.6	24.8	0.67	2906	1388	
1/29/2011	2	Warren	5728	15.6	34.2	0.46	2699	1141	
2/22/2011	2	Marion	4650	7.8	7.5	1.03			Didn't do RCP max out
3/4/2011	1	Knox	6547	13.2	14.4	0.92	3444	2758	
3/9/2011	4	Crockett	5203	15.9	33.0	0.48	3096	1141	
3/11/2011	4	Dyer	6799	8.3	16.4	0.51		2145	
3/15/2011	4	McNairy	6557	6.0	7.2	0.84			Didn't do RCP max out
3/16/2011	2		7393	13.7	17.5	0.78	2999		No NaOH 56 day test
3/22/2011	4	Shelby			28.9	0.00		1125	Rec'd After 28 days
3/29/2011	2	White	5712	10.5	12.0	0.87	4865	4060	
3/29/2011 (2)	2		5854	13.5	16.0	0.84	3254	2927	
4/12/2011	4	Hardeman	3850	9.7	19.4	0.50	5513	2256	
4/21/2011	2	Rhea	3650	10.5	17.4	0.61	6018	4262	
5/4/2011	4	Hardeman			12.1	0.00		4428	Rec'd After 28 days
5/18/2011	1	Blount	6222	16.9	23.9	0.71	1848	1139	

5/19/2011	Lab			12.6	15.9	0.79	4323	2591	6-9 Mix Design
5/20/2011	4	Carroll			9.9	0.00		5314	Rec'd After 28 days
5/23/2011	2		5094	12.2	13.7	0.89	4205	3884	
5/26/2011	Lab	UT	6027	19.0	32.1	0.59	2265	1137	7-6-10 Mix Design
5/26/2011 (2)	Lab	UT	5780	14.6	24.1	0.61	3295	2016	5-25-10 Mix Design
6/3/2011	4	Tipton	5002	9.9	15.4	0.64	5709	2960	
6/7/2011	2	Warren	4375	12.5	21.4	0.59	3173	1920	
6/9/2011	2		5291	9.6	11.7	0.82	5157	4233	
6/9/2011 (2)	2	Warren	4830	13.8	20.3	0.68	3367	2248	
6/21/2011	4	Gibson	4745	10.7	18.6	0.57	4850	2441	
6/23/2011	2		4433	13.4	21.4	0.62	3843	2388	
8/18/2011	4		4956	21.7	34.3	0.63	2031	1391	
8/23/2011	4	Haywood	3798	11.9	17.2	0.69	7430	3519	
8/25/2011	3		3771	15.1	21.5	0.70	3197	2062	
8/26/2011	4				18.5			2406	
8/29/2011	1	Blount	5019	14.9	22.7	0.66	2644	1490	
9/1/2011	3	Williamson	4751	10.9	14.0	0.78	6068	4563	
9/1/2011 (2)	3	Williamson	4566	11.1	13.5	0.82	6185		56 Day RCP Leaked
9/7/2011	4	Gibson	5704	12.0	12.6	0.95	5421	4974	

Table A - 1: Data Summary through May 31, 2012 (Continued)

9/28/2011	4	McNairy	4875	6.7	7.1	0.95			Both 28 & 56 Day RCP Maxed Out
9/30/2011	4		5334						Not Tested due to other sets being tested & holidays
9/30/2011 (2)	4	Shelby							
10/5/2011	1		6358	12.9	22.8	0.57	4085	1971	
10/11/2011	4	Hardeman	4739	10.9	12.3	0.89	6658	5127	
10/20/2011	2	McMinn	6630	11.7	16.2	0.72	5512	4044	
10/21/2011	Lab		6664	12.9	23.3	0.56	4027	2005	7-6-10 Mix Design
10/21/2011 (2)	Lab		5587	11.8	19.0	0.62	4540	2325	7-6-10 Mix Design
10/21/2011 (3)	2		4930						Not Tested – Other sets were being tested on same day
11/3/2011	1	Roane	7038	15.5	25.2	0.62	2999	1748	
11/8/2011 C	2	Dekalb	5501	7.3	10.4	0.70		4866	28 Day RCP Maxed Out
11/8/2011 S	2	Dekalb	5659	8.8	11.5	0.76	7308	4196	
11/17/2011	1	Blount	5930		29.6			1264	
12/13/2011	4	Shelby			14.4			3466	Samples not received in time for 28-day test
12/14/2011	4	Shelby			24.0			1554	
12/17/2011	4	Fayette			26.5			1654	Samples not received in time for 28-day test
12/20/2011	1	Unicoi	5368	10.0	17.8	0.56	4585		56-day RCP test did not finish due to power outage
1/20/2012	1	Johnson	4979	12.4	20.8	0.59	4701	2260	

Table A - 1: Data Summary through May 31, 2012 (Continued)

2/14/2012	Lab	UT	6036	42.4	54.6	0.78	836	521	Ternary Lab Mix
2/14/2012 (2)	4	Hardeman	5916	8.6	11.0	0.79	7330	3996	
2/21/2012	4	Shelby	5833	10.5	16.9	0.62	4896	2198	
2/23/2012	2	Dekalb			13.8			3018	Researchers were out of town for 28 day tests
2/24/2012	4	Shelby			40.3			1228	
3/6/2012	4	Weakley	4715	13.1	28.1	0.47	4583	1585	
3/6/2012 (2)	1	Carter	4805	9.1	13.9	0.65	6578	3183	
3/9/2012	2	Warren	6367	15.5	24.2	0.64	2607	1658	
3/12/2012	1	Washington	5808	12.8	25.9	0.50	3893	2003	
3/14/2012	4	Haywood			18.1			2610	Samples not received in time for 28 day tests
3/20/2012	4	Shelby	5198	18.3	25.5	0.72	1563	864	
3/30/2012	Lab	UT	7541	42.2	59.6	0.71	821	545	Ternary Lab Mix
3/30/2012 (2)	Lab	UT	9168	53.8	67.7	0.80	640	494	Ternary Lab Mix
4/9/2012	4	Shelby	5804	14.3	23.7	0.60	2173	1450	
4/10/2012	4	Hardeman	4945	11.0	12.2	0.90	5035	4619	
4/10/2012 (2)	4	Fayette	6666		39.0			1052	28 Day SR/RCP tests were not performed due to time conflicts with other tests
4/18/2012	2	Franklin	4002	13.4	19.9	0.67	3879	2374	
4/26/2012	4	Shelby	4657* (*29 day)						Samples not received in time for 28 day test

4/27/2012	2	Hamilton	4205	7.7					28 day RCP maxed out
5/11/2012	4	Obion	4036	12.3			5940		
5/15/2012	1	Cocke	5487	16.1			2954		*56 day test age not yet reached
5/15/2012 (2)	4	Hardeman	4566	8.6			7731		
5/24/2012	1	Hawkins	5964	12.8			3381		
5/24/2012 (2)	1	Sullivan							*28 Day Samples not test due to conflicts with other tests
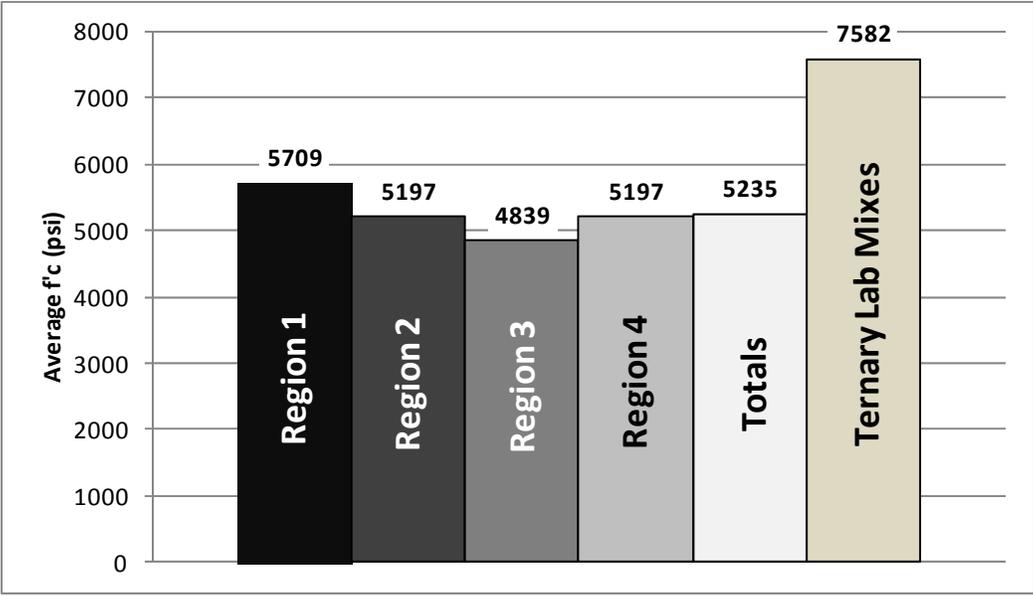
Figure A - 1: Average 28-day Compressive Strengths
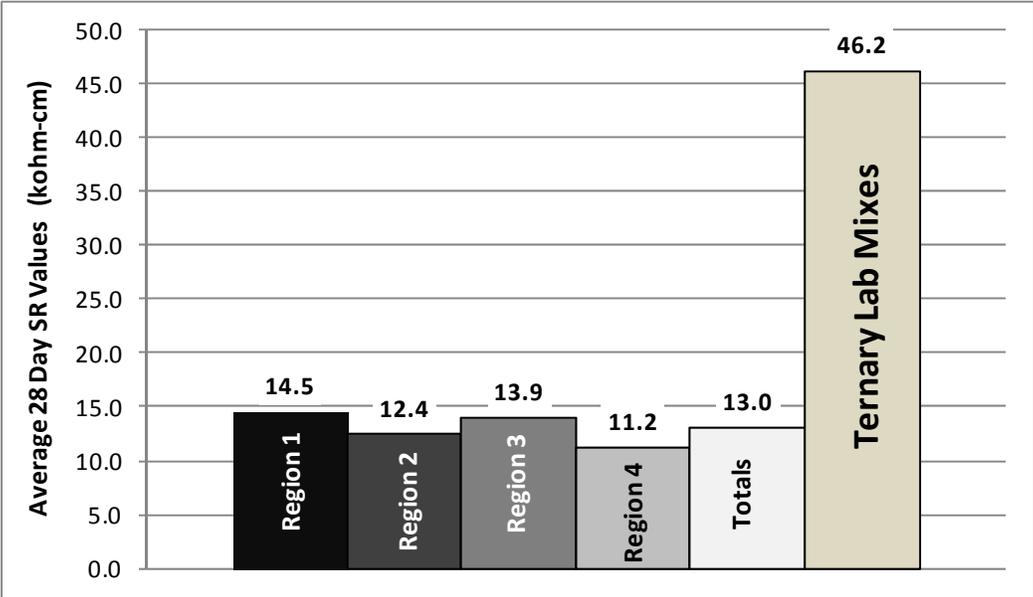
Figure A - 2: Average 28-day SR Values
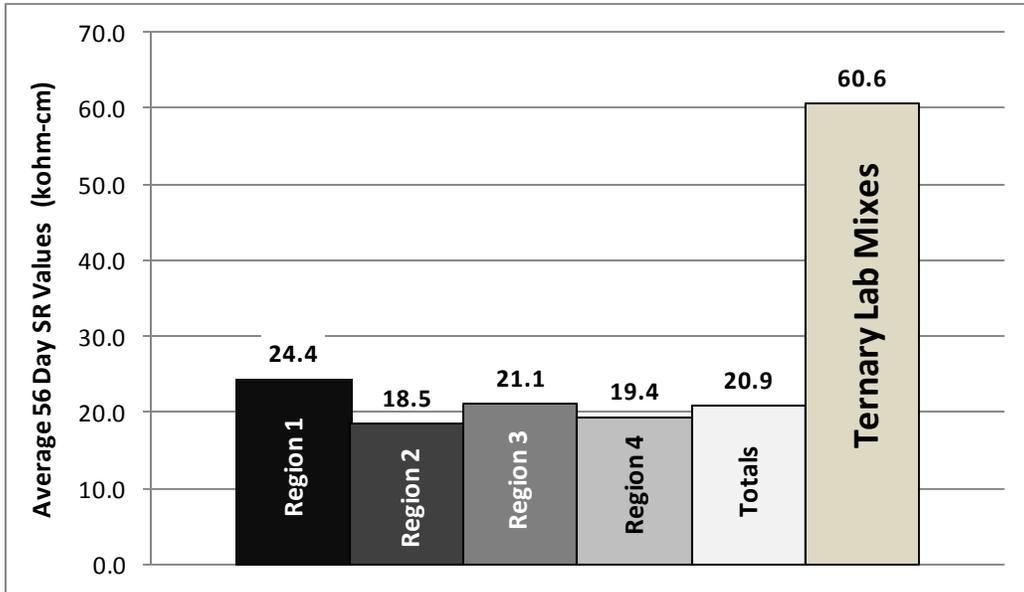
Figure A - 3: Average 56-day SR Values
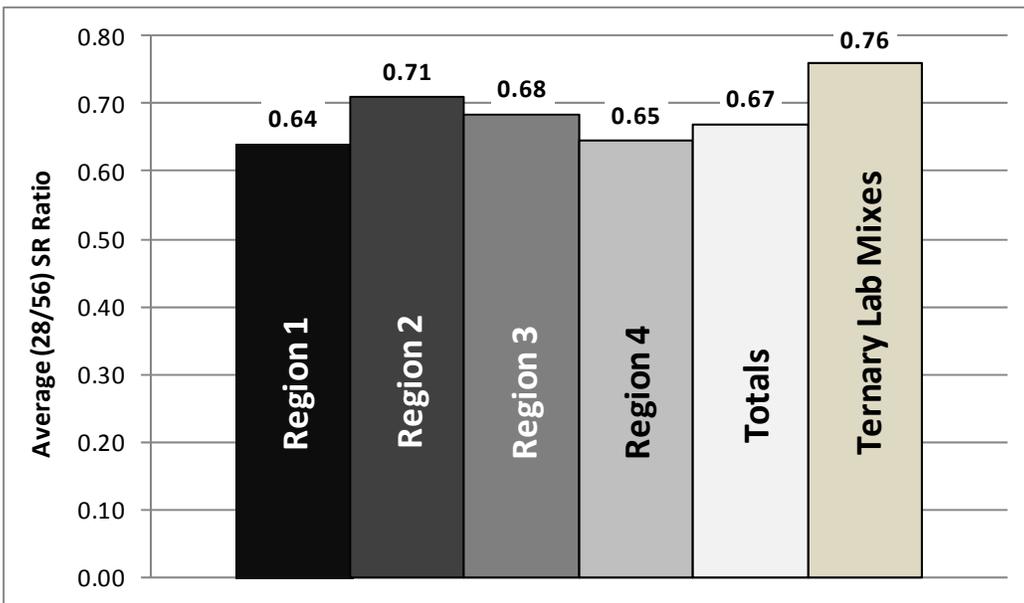
Figure A - 4: Average (28-day/56-day) SR Ratios
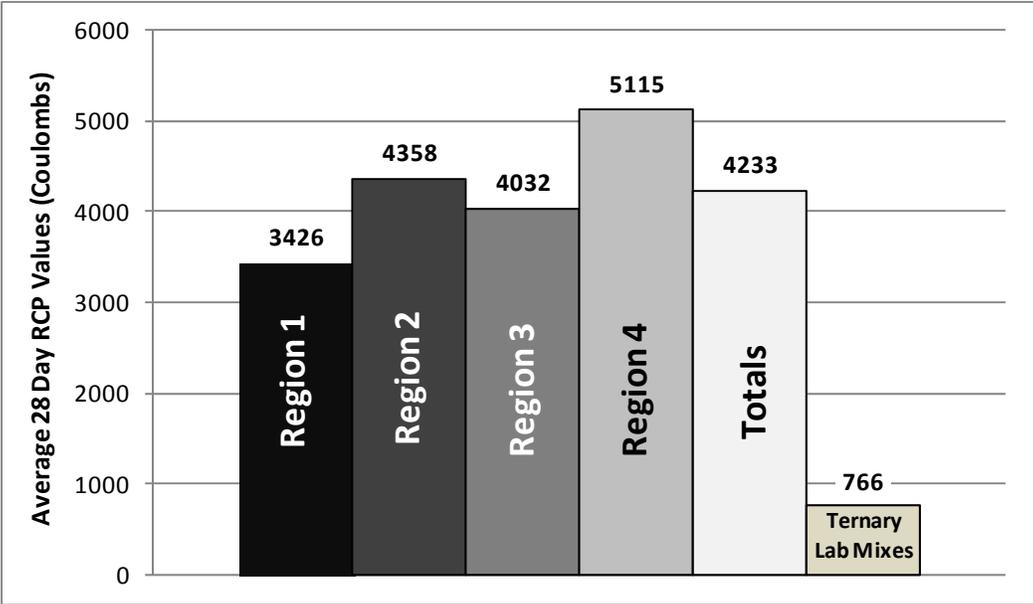
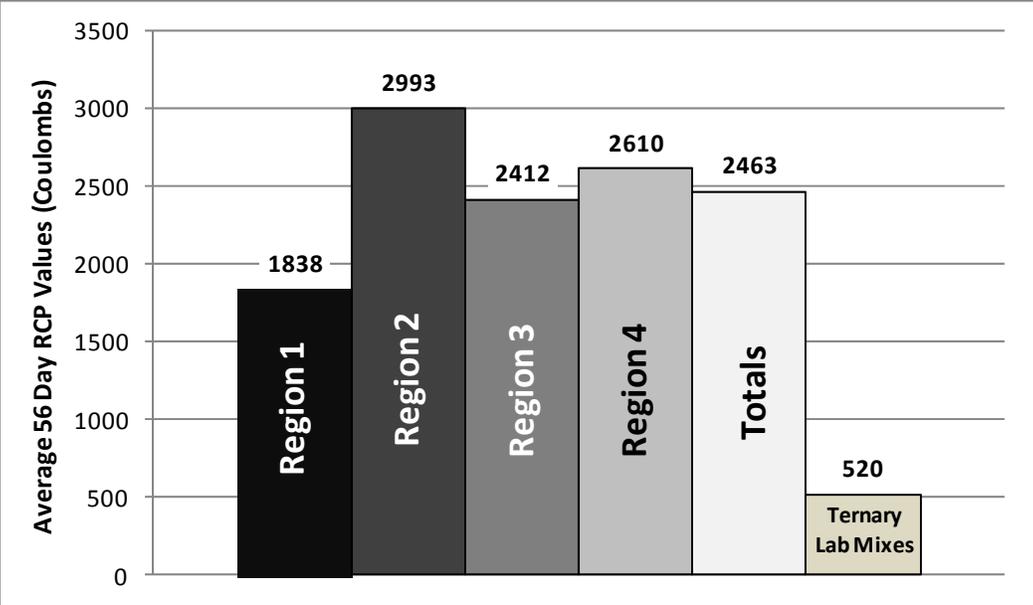**Figure A - 5: Average 28-day RCP Values**
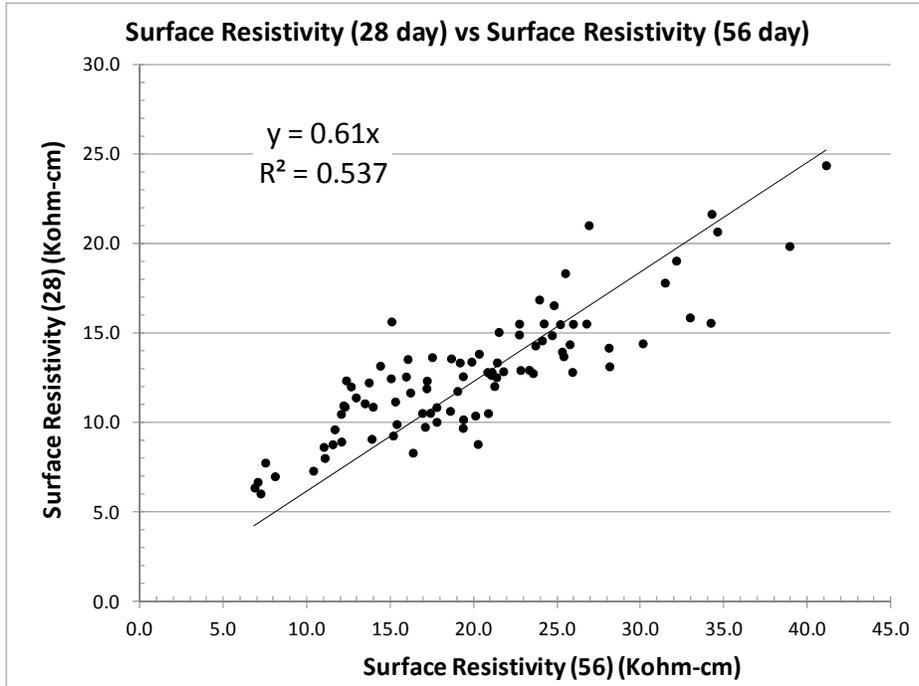

Figure A - 6: Average 56-day RCP Values

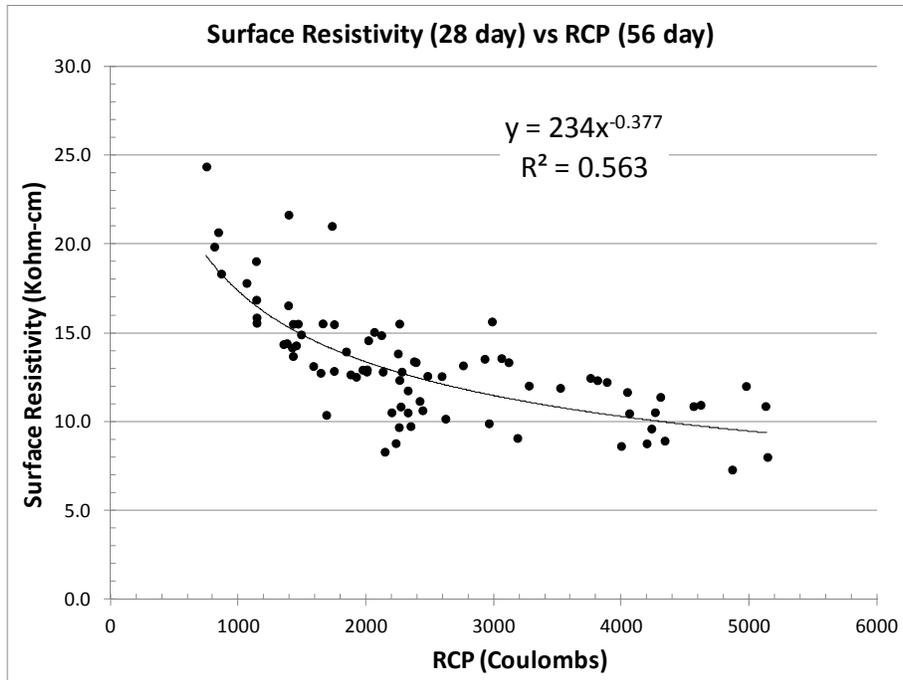**Figure A - 7: 28-day SR Values versus 56-day SR Values**

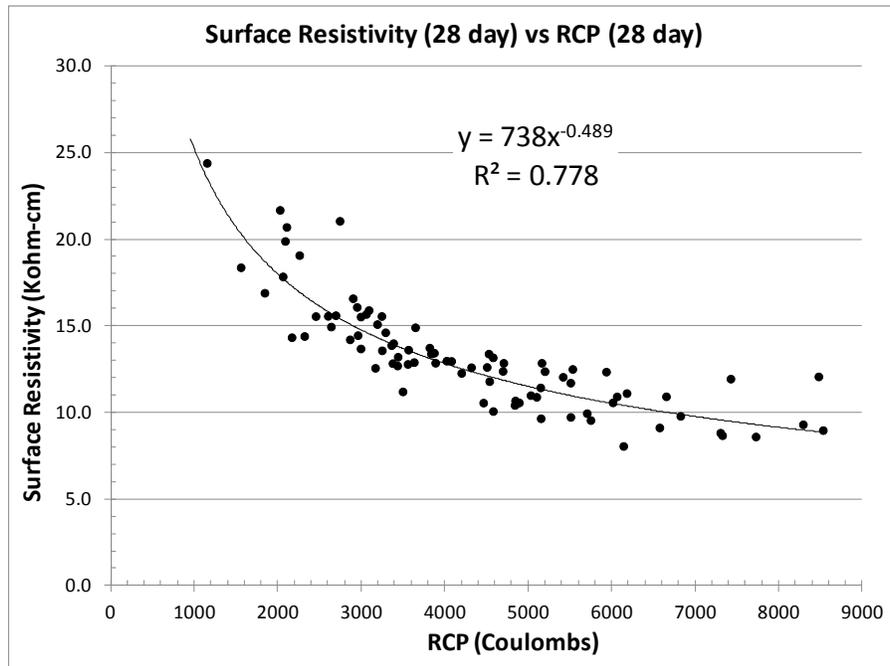**Figure A - 8: 28-day SR Values versus 56-day RCP Values**

Figure A - 9: 28-day SR Values versus 28-day RCP Values

Vita

Robert Clark Nidiffer was born in 1988 to Bob and Cindy Nidiffer in Knoxville, Tennessee. He grew up in the Karns community of Knoxville and attended school at Karns from Kindergarten through 12th grade. Upon graduation from Karns High School in the spring of 2007, the choice of where he would attend college was simple. Growing up in Knoxville, it was common for children to learn the words to "Rocky Top" before they learned to tie their own shoe. He was no different, thus, he enrolled at The University of Tennessee in the fall of 2007. After a year of debating on which type of engineering degree he would pursue, he decided that civil engineering was his true calling and went forth to receive his Bachelor of Science in Civil Engineering from the University of Tennessee in the spring of 2011. Upon completion of his undergraduate career, Robert was blessed with the opportunity to work for Dr. Burdette as a graduate assistant at the University of Tennessee. He has thoroughly enjoyed his time as a graduate student under Dr. Burdette and will graduate with his Master of Science in Civil Engineering with a concentration in Structures in August of 2012. In addition to his academic career at the University of Tennessee, Robert served as an intern with the Tennessee Valley Authority (TVA) for the last two and a half years of his undergraduate career. Upon completion of his Master's degree, he plans to begin a career with TVA as a civil engineer in August of 2012.

Robert grew up being an active member of Wallace Memorial Baptist Church in Knoxville, Tennessee. He was active in the Calvary Baptist Church college ministry

during his time at the University of Tennessee. He credits his faith in Jesus Christ for allowing him to be where he is today.