Final Report

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FULL SCALE EVALUATION OF LOW PERMEABILITY HIGH VOLUME SCM BRIDGE DECK CONCRETE

Project #: RES2013-40

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SECTION 1 - INTRODUCTION AND RESEARCH OBJECTIVE

Durability of cast-in-place concrete bridge decks represents a formidable challenge in the construction and maintenance of highway bridges and a large consideration in each year's maintenance and repair budget. Although construction and design techniques play major roles in the durability of concrete bridge decks, the hardened material properties of the concrete used for their construction play a major role as well. In particular, the permeability of the concrete, or simply its ability to resist penetration of water and other deleterious substances, plays a major role in the deck's ability to resist degradation from freezing and thawing and corrosion of reinforcing steel.

Several methods have historically been used throughout the Tennessee inventory to improve the "permeability" of the deck system, or to at least slow the ability of these deleterious substances to penetrate the deck and or reach the level of the reinforcing steel. These include the use of asphalt overlays (protection and ride quality), membranes, additional concrete cover, polymer concrete overlays, and epoxy overlays. While these methods work well, they can generally be considered expensive and serve as a visual hindrance when routine safety inspections of the deck are completed.

An additional avenue to reduce the permeability of the bridge deck system, and ultimately improve durability, is to reduce the permeability of the base concrete used to construct the deck. While the typical Class D concrete that is used for bridge deck construction (new or replacement) is high quality, significant improvement is possible when considering permeability. While several options are available to improve the permeability of a particular concrete mix, it is important to find avenues that are easy for a typical producer to implement, produce concrete that is efficient for the contractor to place, finish, and cure, and are economically efficient while maintaining high quality hardened properties. One such method is to replace a portion of the Portland cement in a typical Class D mix with high volumes of one or more supplementary cementitious materials (SCM). In previous research, this type of replacement has been found to profoundly improve the permeability of a Class D concrete mix, while not having significant negative effects on other hardened properties such as compressive strength and modulus of elasticity. These mixes have been formulated, mixed, and tested in laboratory conditions with success.

This study sought further development of low permeability mix designs each using a different SCM including silica fume, slag cement, and metakaolin. A bridge repair project consisting of two different structures was selected for full scale study of the three low permeability designs. The repair project included four phases of deck construction that allowed the use of each of the low permeability designs along with a control. These projects remained unchanged from their original design except for the concrete mix design used for the deck. Objectives of the full-scale use of the new mix designs included identification of any problems encountered by the producer or contractor (placing and finishing). Plastic and hardened properties were evaluated to validate that Class D specification requirements were achieved, and permeability testing was completed to compare performance of the low permeability mixes with the control. Field testing was completed to understand in-place permeability performance. Finally, analysis was completed to better understand the viability of surface resistivity testing for evaluation of permeability on in-place structures.

Full-scale construction and testing may lead to bridge decks with increased durability, lower repair demands, increased service lives, and overall reduced cost. Also, results of field testing of permeability may lead to in-service testing without need of a sample and providing better understanding of performance and change in performance over time.

SECTION 2 – LITERATURE REVIEW AND BACKGROUND

The 1967 collapse (46 deaths and 9 injuries) of the Silver Bridge in Ohio resulted in a congressional mandate (Federal-Aid Highway Act of 1968) to establish National Bridge Inspection Standards (NBIS) for all public highway bridges with spans of 20 feet or greater [1]. Subsequently, many other programs were authorized providing requirements related to bridge inspection. These resulted in guidelines for inspection and data collection including the "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nations Bridges" and the "Bridge Inspectors Training Manual" [2,3]. These required state DOT's to collect data related to bridge description and condition assessment on a biennial basis. Some uses of the data included determination of serviceability, safety, deterioration rate, and need for replacement [4]. Data from inspections (116 categories) is stored in the National Bridge Inventory (NBI) database [5].

The national inventory includes approximately 615,000 structures [6]. Of this, approximately 20,000 structures are in Tennessee [7]. The ASCE Report Card on Infrastructure evaluates this inventory (using NBI data) to identify its condition, indicate trends, and estimate funding needed. Since the report card started, assessment of the bridge inventory has consistently indicated that considerable funding is required for additional maintenance, repairs, rehabilitations, and replacements. The most recent rating of highway bridges resulted in a Grade of C+ for the national inventory with 9% of structures classified as structurally deficient and the Tennessee inventory received a grade of B with 5% rated as structurally deficient. Structurally deficient structures are those in need of replacement or significant maintenance or repair. Although the number of structurally deficient structures has declined (considerably in Tennessee) in recent years, it is estimated that $123 billon is needed to address these structures [8].

Of the major components (substructure, superstructure, and deck) that comprise a highway bridge, the deck usually deteriorates the most quickly and requires the most maintenance and repair because of its exposure to detrimental conditions. Maintenance, repair, and preservation of decks have proven to be costly, in some cases representing 50-80% of the owners' expenditures for bridges [9]. Deck deterioration has been identified by the National Cooperative Highway Research Program (NCHRP) as a leading cause in structurally deficient ratings [10].

The Federal Highway Administration's (FHWA) Long Term Bridge Performance (LTBP) program has identified more than 20 major issues effecting bridge performance. Six of the issues were assigned high priority status including the performance of treated and untreated bridge decks [11].

One characteristic used in NBI recording to categorize bridge decks is identification of the deck structure type. In most cases, this also identifies the material used to construct the deck. The categories include concrete cast-in-place, concrete precast panels, open grating, closed grating, steel plate, corrugated steel, aluminum, wood or timber, other, and not applicable [2].

Both nationally, and in Tennessee, the predominant material used for deck construction has been concrete (including cast-in-place and precast). NBI data indicates approximately 69% of the national inventory and 73% of the Tennessee inventory have decks constructed of concrete. Concrete as a material is subject to many forms of deterioration due to a few underlying mechanisms. Examples of typical deterioration found in the inspector's manual include cracking, spalling, delamination, and section loss [3].

Fast deterioration of concrete decks has been evident since the initiation of the NBI program. NBI data also includes identification of other aspects of the deck including the wearing surface, the type of membrane (if any) present, and deck protections that have been employed to protect and preserve the deck. Data indicates that the predominant wearing surfaces for both the national and Tennessee inventories was monolithic concrete (28% national and 38% Tennessee) and asphalt (30% national and 38% Tennessee). Also, the primary deck protection system used was epoxy coated reinforcing ranging from 15% to 27% for the national and Tennessee inventories, respectively. The NBI categories tracking these characteristics illustrate the emphasis on performance issues and the efforts to protect and preserve bridge decks. Several strategies have been employed to both improve new construction and protect existing construction in effort to prolong service life.

In the 1960's, the FHWA along with other agencies studied this issue to identify major contributing factors to deterioration. At that time, the primary issue was thought to be chloride-induced corrosion of the reinforcing [12]. Through this effort, three recommendations for construction of new concrete decks were made to reduce or slow deck deterioration and increase the time until repair or replacement. Each of the recommendations were intended to protect reinforcing by slowing the ingress of chlorides. Recommendations included an increase in concrete cover over top of reinforcing, improved deck drainage, and reduction in the water to cement (w/c) ratio of the concrete.

Overlay systems intended to protect and preserve decks were also developed beginning in the 1960's. Some of the first systems developed included the use of low slump dense concrete, latex-modified concrete, and asphalt in combination with a membrane. The aim of these systems was to protect the deck from harsh conditions that initiated or accelerated deterioration. Many additional overlay options have since been added [13].

More recent studies completed have found that poor initial quality and damage due to deicing salts, overloading, freeze-thaw cycle induced stresses, fatigue, and corrosion of reinforcing result in damage to concrete decks [14]. Freeze-thaw cycles and corrosion of reinforcing steel have been identified as the two predominant causes of deterioration [15]. These deterioration mechanisms reduce the structural integrity and ride quality of the deck with common symptoms including scaling, delamination, section loss, and cracking [23]. Freeze-thaw action may result in damage that aids in chloride ingress supporting corrosion. Concrete permeability has also been reported as a contributing factor leading to conditions conducive to corrosion [10].

Freeze-Thaw Action

Freeze-thaw action is defined as deterioration of moist materials through cycles of freezing and thawing [22]. Deterioration due to freeze-thaw action is the most detrimental environmental effect on concrete structures [16]. Both aggregates and hardened cement paste are porous materials that absorb available moisture and are therefore susceptible to freeze-thaw damage [17]. The w/c ratio, generally directly related to porosity, may play an important role in the durability of concrete when exposed to freezing and thawing [18].

Approximately 75% of a typical portland cement is comprised of dicalcium and tricalcium silicate. These compounds react with water during cement hydration to form calcium silicate hydrate and calcium hydroxide, primary components of cement paste. Calcium silicate hydrate is primarily responsible for the engineering properties of concrete, most notably strength. This compound is a poorly crystalline and highly variable material consisting of thin layers or sheets of calcium silicate with calcium ions and water in between. During hydration, capillary pores or cavities are also formed in the calcium silicate hydrate representing voids in which water can be stored and behave like bulk water [19].

During cycles of freezing and thawing, temperatures drop to a required level to begin transforming water absorbed into the void system of the concrete into ice crystals [17, 20]. Transformation of water to ice represents an expansion of approximately nine percent. As a result of the expansion during ice formation, pressure is exerted forcing the pores in the cement paste and aggregates to dilate. This dilation leads to internal stresses and micro cracking [21]. Additional cracking may occur as subsequent freeze thaw events occur. In addition to the pressures exerted due purely to the expansion of ice crystals, three other processes are thought to play a major role in freeze-thaw damage including hydraulic and osmotic pressure and desorption [17].

Generally, bridge deterioration due to frost action is easily identified during visual inspection and is typically most prevalent on reinforced concrete bridge decks. In comparison to other bridge members, the deck is relatively thin with large surface area exposed to the environment, resulting in an increased likelihood of freeze thaw cycles. Typical symptoms include scaling, spalling, and cracking. These forms of deterioration may create conditions conducive to other forms of deterioration such as chloride ingress and its effect on reinforcing steel corrosion.

Entraining air in concrete has proven to prevent damage due to freeze-thaw cycles. The entrained air provides empty space that allows expansion and water movement during a freeze event. This void space virtually eliminates the harmful pressures that deteriorate concrete due to freezing [17]. Appropriate mix design and construction practice (placing and finishing) are also important factors in freeze-thaw durability.

Corrosion of Reinforcing Steel

Corrosion of reinforcing steel is a primary deterioration mechanism of reinforced concrete, representing the most serious durability issue, and results in most of the damage to concrete structures [16, 22, 26]. Corrosion of the reinforcing steel ultimately reduces the strength, structural integrity, and functionality of the structure [21, 22]. Deck deterioration due to corrosion of reinforcing steel is considered one of the most acute durability problems throughout the inventory [17].

Common symptoms of reinforcement corrosion include delamination, section loss, cracking, and staining each readily visible through visual inspection. These symptoms are typically a result of the expansive nature of the corrosion process. The rust produced by corrosion of reinforcing steel occupies up to four times more volume than the material that was removed thus causing distress in the concrete [16, 17, 19, 21, 23, 25]. Cracks in concrete may result from a layer of corrosion one tenth of a millimeter thick [22].

Corrosion of reinforcing steel follows the same general principles as corrosion of structural steel. Through an electrochemical process, anodes and cathodes are created and the flow of ions through an electrolytic substance causes corrosion. In concrete, the anode and cathodes typically represent areas with different impurity levels in the base metal of the reinforcing, residual strains, or concentrations of oxygen or electrolytes in contact with metal [17].

Oxygen typically reaches the reinforcing steel through several means including diffusion through the concrete, cracks, or a combination of both [21]. Concretes cured and continuously submerged in water are generally not susceptible to corrosion deterioration due to the lack of oxygen present to support the process. Generally, the corrosion process is controlled by the diffusion of oxygen through the concrete [22].

Similarly, water generally reaches the reinforcing steel through diffusion or cracks present in the concrete. Water acts as the electrolyte needed for galvanic action, and therefore, permanently dry concrete will not support corrosion [14, 21]. Typically, the wider and deeper the crack, the more susceptible concrete is to deleterious elements. The ability of oxygen and moisture to reach reinforcing steel is also dependent upon the initial quality of the concrete including the density, compaction, and thickness of cover [16, 17]. Reduction in the permeability of concrete will reduce the amount of oxygen and water available to the corrosion process [21].

In comparison to structural steel, concrete construction involves several inherent properties that typically protect embedded reinforcing steel from both corrosion and the environment. As concrete hydrates and cures around reinforcing steel, the alkalinity in the cement paste causes a thin layer or oxide film to form on the surface of the steel [17, 22, 23]. The film renders the area around reinforcing steel passive, with pH ranging from 9.5 to 13 [22]. This protective film is considered very stable while it remains in a high alkaline environment with decreasing stability as the alkalinity is lowered to pH levels ranging from 9 to 11 [21,24]. Chloride contamination and carbonation are the most common reasons for the passive layer to deteriorate.

One mechanism that lowers the level of alkalinity at the reinforcing steel is carbonation. The initial stage of carbonation is the intrusion or diffusion of carbon dioxide from the atmosphere into the concrete [21, 22]. This may take place through cracks in the concrete or through naturally occurring pores in the cement paste. Once present, carbon dioxide reacts with soluble products in the pore solution such as calcium hydroxide, a typical hydration by-product, which maintains a high pH level in the cement paste [21, 22]. This reaction results in the formation of insoluble calcium carbonate that precipitates on the walls and in the cavities of the pores. The transition of calcium hydroxide to calcium carbonate results in a reduction of the alkalinity in the carbonated area to a pH level of 8 or 9, below its normal value of approximately 13 [23, 24].

Several factors affect the rate and presence of carbonation including the permeability of the concrete and presence of cracks. Also, maximum carbonation will occur between fifty and seventy percent relative humidity [21, 25]. However, for carbonation to participate in the initiation of corrosion, it must take place at or near the reinforcing steel. Two particulars instances where this is found include older bridges or those with inadequate level or quality of concrete cover [25]. Typically, well cured, quality concrete with a low w/c ratio or low permeability are subject to very shallow carbonation. The presence of cracks can increase the presence and magnitude of carbonation, especially at the level of reinforcing steel. Fully carbonated concrete near the reinforcing steel may not be subject to corrosion without oxygen and moisture [17].

The presence of chloride ions in concrete can also reduce the stability of the protective film rendering it less passive and thus more subject to corrosion [21]. Common sources of chloride include calcium chloride in accelerating admixtures, deicing salts and sea water [16, 17, 21]. These chlorides may be directly applied to the concrete bridge member, such as de-icing salts on bridge decks, or indirectly such as that as result of leaking deck joints or water proofing, splash from traffic, and from sea spray [21,25].

Chlorides typically reach the concrete surrounding the reinforcement through two main mechanisms. First, chloride present on the surface of the member or in solution in the pores may diffuse into the concrete [17, 26]. This process is typically quite slow and thought to follow Fick's second law of diffusion. The parameters that effect the diffusion are the concentration of chloride present and the permeability of the concrete. The second main transport mechanism of chloride to the reinforcing steel is through cracks present in the concrete that may be a result of temperature and shrinkage, freeze-thaw action, or overstress [17].

Regardless of the transport mechanism involved, chloride may build up over time, and eventually reach a critical level where the level of alkalinity at the reinforcing steel is diminished enough to allow the corrosion process to begin. Typical critical chloride concentrations required to reduce the high alkalinity are between one and two pounds per cubic yard of concrete but are ultimately dependent upon the actual alkalinity present at the interface of the concrete and reinforcing steel [17]. This process not only reduces the pH in the vicinity of the reinforcing steel, but also may increase the electrical conductance of the electrolyte possibly allowing the corrosion rate to increase [22]. Chloride contamination may also result in concrete that retains more moisture resulting in a more conducive environment for damage due to freeze-thaw action or corrosion [27].

Chloride may reach the level of reinforcing steel through diffusion through the pores of the concrete. Many literature sources have indicated that the diffusion process typically follows Fick's second law of diffusion as indicated in the following equation [28]. When considering chloride, C represents chloride concentration, K_d is a concrete diffusion coefficient, t is time and x is the depth from the surface of the material to a depth of interest such as reinforcing steel.

$$\frac{\partial C}{\partial t} = K_d \frac{\partial^2 C}{\partial x^2}$$

The solution of this equation for a concrete member with finite dimensions is provided below [28]. Through this equation, the chloride concentration at time t and depth x can be predicted based on the initial chloride concentration C_0.

$$C(x,t) = C_0\left[1 - erf\left(\frac{x}{\sqrt{4K_d t}}\right)\right]$$

To use the equation developed from Fick's second law, a diffusion coefficient must be known for the concrete under study. The diffusion coefficient is closely related to permeability. Numerous test methods exist for the determination of the diffusion coefficient including ASTM C 1556 – Standard Tet Method for Determining the Apparent Chloride Diffusion Coefficient of Cementitious Mixtures by Bulk Diffusion [29].

Methods Used for Deck Protection

The Tennessee DOT has implemented many of the recommendations (previously discussed) by the FHWA related to construction of durable concrete bridge decks including mix design requirements and placing of reinforcing.

Compared to typical commercial mix designs those used in Tennessee for concrete bridge decks (Class D) have considerably lower w/c ratios (0.40 maximum) and higher cementitious contents (620 lb/yd^3) for similar compressive strengths. These two parameters help to reduce the porosity and permeability of the paste. Also, compared to commercial mixes, those used for bridge decks always require air entrainment that results in more durable concrete that is less prone to degradation when compared to its typical commercial counterpart. Although air entrainment itself does not prevent corrosion, it does prevent the damage (i.e., cracking) typically associated with freeze-thaw action. The reduction of cracking may help to slow ingress of water, chlorides, and oxygen that support the corrosion environment.

One design element that intends to address corrosion, or the time until corrosion begins, is increased concrete cover. This cover provides extra thickness that detrimental substances must penetrate to reach reinforcing. The required cover for top mats of reinforcing teel in decks is considerably greater than that used for similar applications in other types of construction with less harsh conditions.

Another avenue to reduce or slow corrosion has been the use of reinforcing with improved corrosion characteristics. Tennessee, along with most other states, requires epoxy coated reinforcing for all or part of deck construction. Other types of reinforcing have also been used, or investigated, including stainless steel, zinc coted (galvanized, and fiber reinforced plastic (FRP). Each of these have resulted in varying levels of success.

Use of a protective overlay applied to the top of the deck has also been a successful and popular preservation technique used by many state DOTs, including Tennessee. According to an FHWA LTBP study, various overlays or sealers are popular methods used to protect bridge decks [11]. The treatment of decks in these manners seeks to preserve the deck (new or existing construction) and provide protection from contamination. Several situations may support the use of a deck treatment, applied either new construction or as a repair and maintenance item. One of the main reasons for application is deck protection from moisture and ingress of chlorides or other harmful materials.

The most used deck treatments across the country's bridge inventory include asphalt overlay (with or without a membrane), latex modified concrete (LMC) overlay, epoxy polymer concrete overlay, membranes, portland cement concrete overlay, silica fume (micro silica) concrete overlay, high molecular weight methacrylate (HMWM) sealer, prime coat, polyester polymer overlay, silane sealers, and low slump/dense concrete overlay [11].

Historically, bridge decks in Tennessee have been protected by four of these treatment types including asphalt overlays, reinforced portland cement concrete overlays, nonreinforced polymer modified concrete overlays, and thin bonded overlays [30]. Discrimination between the type of overlay used depends on many factors including structure specific criteria (load capacity or geometric concerns). Average service lives range from15 years to more than 30 years.

Asphalt Overlays

Two types of asphalt overlays have used as preventive maintenance: general overlay and layered overlay, the later typically referred to as sandwich seal. Both use a combination of asphalt and a bridge deck sealant (rubber-fiberglass-bitumen-polyester membrane).

Both overlays are typically placed with a total thickness of 3.25 in., but thickness can vary based on the requirements of a specific project. The sandwich seal consists of three layers of asphalt with the sealant placed between the first two layers of asphalt, hence the term sandwich seal. Similarly, the general overlay consists of three layers of asphalt with the sealant placed between the bridge deck and the first layer of asphalt concrete.

Both asphalt overlays provide protection of the bridge deck through the combination of asphalt layers and sealant. This combination protects the bridge deck form materials such as water and chlorides, which can be detrimental to bridge deck life. These methods are applicable in many situations. One of the main considerations in selection between the two methods is the construction site conditions during application.

One method may be preferred for specific conditions, such as phased construction during which laps and deterioration due to construction traffic may be problematic.

Reinforced PCC overlay

A reinforced PCC overlay consists of a monolithic layer of PCC with one layer of longitudinal and transverse reinforcing steel. This type of overlay typically is placed with a minimum thickness of 4.5 in., although a different thickness may be required for dimensional fit or structural adequacy. Typically, a Class D concrete mix is used for reinforces PCC bridge deck overlays.

This type of overlay protects the deck because of its thickness and low permeability and provides additional structural resistance. Reinforced concrete overlays are used as preventive maintenance (sometimes structural repairs) in three general cases. First, when additional construction (rehabilitation widening, or safety upgrades) is being performed on the structure, the reinforced concrete overlay provides good protection for the deck as well as structural stability and resistance for the upgrades being performed. Second, concrete overlays are used when a large portion of the deck – usually 50% or more – is deemed to need of repair. Third, an overlay of this type might be used on older bridges. Older bridge decks have four main characteristics that warrant the use of a reinforced concrete overlay: thinner cross sections than decks currently designed and constructed, less clear cover for the top mat of reinforcing steel that the current standards require, existing reinforcing steel without epoxy coating that is more susceptible to corrosion, and only one layer (bottom) of reinforcing steel may be provided. Therefore, for older but otherwise useful and adequate bridges, the concrete overlay can be used as preventive maintenance as well as to provide additional structural integrity.

Nonreinforced PMC Overlay

Nonreinforced PMC overlays are another option that can be used for preventive maintenance on highway ridge decks. PMC can be generalized as a typical concrete mix with a polymer admixture. The addition of this type of admixture can improve concrete properties of interest in overlay applications. These properties may include higher compressive strength, improving bonding to existing concrete and reinforcing steel, better resistance to water and chloride solutions, and improved freeze-thaw resistance.

TDOT specifications provide proportioning guidelines for this type of concrete mix that, in addition to a standard concrete mix, include the amount of polymer admixture that is to be added. Typically, Type 3 cement has been used in these mixes to produce a concrete mic that provides rapid set, thus reducing the amount of time required for the overlay to cure before traffic can resume.

This type of overlay typically consists of a in.-thick layer of PMC applied to the top of a prepared bridge deck. In contrast to the PCC overlay, this overlay does not contain reinforcing stee. PMC overlays provide protection for the original bridge deck through their thickness and improved properties as previously discussed. This type of overlay may be used in several situations, for example, when additional dead load needs to be avoided (versus the reinforced concrete overlay or asphalt overlays) or when the overlay needs to be completed in a short amount of time (quick turnaround when using Type 3 cement compared with other overlay options).

Thin Bonded Overlays

Unlike the three methods already discussed, thin bonded overlays typically are proprietary products. Thin bonded overlays can be of two material types: polymer-modified cementitious or polymer-modified epoxy. Both consist of polymer-modified material applied to the prepared bridge deck and, in some cases (depending on product being used), a rough fine aggregate applied on top of the material. These overlays may be placed using a broom and seed method, which consists of repeating layers of resin and aggregate placed until the specified thickness is reached. The same process may also be completed with more sophisticated mechanical equipment (may be vendor specific. Overlays of this type have short curing times. Therefore, one benefit of this type of overlay is that the roadway can be opened to traffic quickly after placement. Thin bonded overlays are approximately 3/8 in. thick.

Because of the polymer-modified materials' low permeability and resistance to freeze-thaw and chemical degradation, these overlay options provide a protective layer for the deck. They may also provide additional skid resistance. Thin bonded overlays might be chosen when additional skid resistance is desired, geometric considerations necessitate a thin overlay, or additional dead load is not acceptable.

Each of the four overlay types used in Tennessee have proven to be beneficial. While these do not improve the existing concrete deck, another layer of protection is provided. The resulting system results in reduced permeability of the deck system.

Permeability

Concrete permeability influences durability because it controls the rate that moisture enters concrete. Concrete permeability may be defined differently depending on application. In terms of bridge decks, or other flatwork, it may be defined as penetration of non-pressurized substances, such as water and chloride ions. Other uses of permeability may refer to the ability of concrete to be watertight or resist the flow of water. The same properties of concrete control both types in a similar manner. Improvements in permeability reduce the ingress of water and chlorides but also improve a concretes resistance to damage due to freeze-thaw cycles, re-saturation, sulfate attack, and other chemical attacks [17].

Diffusivity refers to the ease with which dissolved ions move though concrete. Decreases in permeability and diffusivity are correlated. Porosity, typically reported as a percentage, is a measure of the volume of voids compared to overall volume. Porosity influences permeability and diffusivity as the pore system of paste becomes discontinuous at around 30% porosity. The size and connectivity of the voids resulting from porosity have significant effect on permeability and diffusivity. The use of SCMs, because of their particle size, further divide the capillary pores so that they become more discontinuous. However, use of SCMs has little effect on overall porosity [17].

Concrete permeability is a function of several properties of hardened concrete, most notably the permeability of the paste. The parameter that has the single largest effect on durability of concrete is w/c ratio. [17]. As w/c decreases, the permeability of the paste decreases, making it more difficult for water and chlorides to penetrate the concrete. A low w/c ratio produces a paste that attains a discontinuous capillary-pore system which reduces permeability [17]. The permeability of the paste controls the overall permeability of the concrete because the other components it binds together are much less permeable in comparison.

The type and quantity of cementitious materials used also great impact on permeability of the paste. Tests show that the permeability of concrete decreases as the quantity of hydrated cementitious materials increases and is age dependent [17]. The length of moist curing also has a dramatic effect on permeability as does moisture content.

Many methods exist for measuring the permeability of concrete. Methods may be broken into two categories, including direct and indirect measurement. Indirect measurement is typically used mainly due to ease of use and reduced time to complete testing.

Direct measurement may be accomplished through use of a ponding test. One approved ponding test is AASHTO T 259 - Standard Method of Test for Resistance of Concrete to Chloride Ion Penetration [31]. This test method requires that one surface of the specimen be subjected to a specified chloride concentration for 90 days. Chloride concentrations are then measured at different distances from the surface of the specimen. The chloride diffusion coefficient may be determined based on results from these tests following ASTM C 1556.

The two commonly used indirect tests of permeability measure either conductivity or resistivity. Past research has shown that, although measuring different properties, each of the indirect measurement methods have correlated well.

Typical factors that affect the conductivity/resistivity of concrete also affect permeability, so conductivity/resistivity correlates with permeability. A linear relationship typically exists between the conductivity (amount of charge passed) and porosity of concrete indicating that as the amount of charge passed increases, the permeability increases. Conversely, as resistivity increases the charge passed increases indicating a negative correlation between resistivity and permeability.

Examples of approved test methods for measuring conductivity (in coulombs) include ASTM C 1202 - Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration and AASHTO T 277 - Standard Method of Test for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration [32, 33]. These two test methods follow very similar procedures. These methods, referred to as RCPT, are often specified for concrete used for bridge decks.

This test is typically completed using a migration cell. The sample, a 2-in. slice taken from a 4 in. diameter test specimen, is vacuum soaked and enclosed in two chambers with specific concentrations of sodium chloride and sodium hydroxide. An electric field of sixty volts is applied across the electrodes in each chamber for six hours. The current passing the specimen is recorded as a function of time. The total charge passed (area under curve) represents is total movement of ions that occurred during the test. Test results are classified using ranges that are useful for comparison. Results may be classified as high (> 4,000 coulombs), moderate (2,000 to 4,000 coulombs), low (1000 to 2000 coulombs), very low (100 to 1,000 coulombs), and negligible (<100 coulombs) permeability. Determination of the diffusion coefficient may not be possible with results from this test method.

Measurement of resistivity (kohm-cm) is inversely related to permeability. This test method, developed by the Florida DOT, utilizes a Wenner Probe to measure resistivity. One provisional test method is AASHTO TP 95 - Standard Method of Test for Surface Resistivity of Concrete's Ability to Resist Chloride Ion Penetration [34]. ASTM currently has a test method under review. This test method is more suitable for assessing in place concrete as removal of a sample is not required for testing.

Equipment used to complete this test method is commonly referred to as a "resipod". Based on the Wenner probe, the apparatus has four in-line probes. During testing, a current is applied to the outer two probes and the potential difference is measured between the two inner probes. Due to the nonhomogeneous nature of concrete, the distance between the probes influences test results with larger spacing resulting in better results. However, probe spacing is limited to avoid interference from reinforcing steel when measurements are taken on in-place concrete. Probe spacing of 2 in. is generally considered acceptable. Like RCPT methods, resistivity results may be classified into categories of permeability including high (<12 kohm-cm), moderate (12 to 21 kohm-cm), low (21-37 kohm-cm), very low (37 to 254 kohm-cm), and negligeable (>254 kohm-cm).

Methods to Reduce Permeability

As previously discussed, several changes in the design of concrete mixtures and design and construction practices have occurred with a target of corrosion prevention. Another opportunity to improve corrosion resistance of a deck system is to reduce the permeability of the concrete used in construction. This improvement should slow the ingress of the substances required to support a corrosion rich environment.

The existing concrete mix designs used in Tennessee have incorporated good practices previously discussed and result in concrete of high quality. The reduction of permeability of the mix provides another opportunity for improvement related to corrosion. Appropriate dosages of SCMs can greatly reduce the permeability and absorption of concrete. Typically, these materials are used as a replacement of a portion of cement, maintaining the total required cementitious content. Additionally, most SCMs are good at binding chloride ions preventing further migration into concrete [19].

Many studies have identified that the use of SCMs has been proven to have significant impact on the permeability of concrete or cement paste. Commonly identified SCMs having positive effect of permeability include fly ash, silica fume, slag cement, and metakaolin. SCMs, as a partial replacement of portland cement, have been found to change the pore structure of concrete. [27]. The use of these materials results in further reduction in the continuity of the capillary pores.

The average particle sizes of these SCMs are considerably smaller than that of portland cement. Portland cement particles average 15 mm. Particle sizes of these SCM's average 7-12 μm for fly ash, 1-2 μm for metakaolin, and 0.1 to 0.3 μm for silica fume. Use of these materials results in a hydration process that includes a reaction of calcium hydroxide with silica and water to form calcium silicate hydrate. This reaction is expansive with the final substance having greater volume than original. This results in a finer and less interconnected system of pores when compared to concrete containing only portland cement.

Three of the SCM's are byproducts of industrial processes. Fly ash is the most widely used SCM and is a byproduct of the combustion of pulverized coal in electric power generating plants. Silica fume is the ultrafine non-crystalline silica produced in electric-arc furnaces as a byproduct of the production of silicon metals and ferro silicon alloys. Molten slag produced in blast furnaces as a byproduct from the production of iron used in steel making is the base material used for slag cement [19].

Metakaolin is intentionally produced for use in concrete, mortars, decorative concrete, and other cement-based products. Metakaolin is produced from kaolin, a fine, white, clay mineral. Historically, kaolin has been used in the manufacture of porcelain. Calcination of this clay results in metakaolin. Of the four SCMs listed, this is the only product that is specifically manufactured for its intended purpose, allowing for tighter control and more consistent properties.

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SECTION 3: LABORATORY MIX DESIGNS

Class D Control Mix Designs

Two Class D laboratory control mixes were developed to serve as a baseline for development of three low permeability mix designs. The control mixes developed represented Class D designs like those historically used by concrete producers for bridge deck concrete.

The first control mix design was developed using only Portland cement as a cementitious material. This mix, Class D PC (PC Lab), was considered the standard mix that all remaining mix designs (1 control and 3 low permeability) were based upon.

The PC mix design was developed through use of the requirements in the standard specifications. The trial batches served to identify the correct proportions of materials, appropriate w/c ratio, and appropriate chemical admixture dosage rates to achieve the desired plastic and hardened properties. Trial batches also served to ensure that the mix design could be easily replicated with consistent properties.

The second control mix design, Class D FA (FA Lab), was based on the PC mix design and included the incorporation of Class F fly ash. Two primary goals of incorporating fly ash were to match mix designs currently used by many producers and to reduce permeability. The PC mix design was used as a starting point and modified in two ways. First, 20% of the Portland cement was replaced with the same amount of fly ash, maintaining the same total cementitious content. Second, through several trial batches, chemical admixture dosage rates were adjusted to maintain acceptable plastic properties.

The proportions of the two control mix designs are provided in Table 3-1. All materials used in fabrication of the control mix designs met requirements of the standard specifications. Comparison of the two designs reveals changes in the cementitious makeup (total cementitious remained constant) and an increase in the dosage rate of air entraining admixture (Micro Air) in the FA design. Otherwise, the remainder of both mix designs were identical. Once finalized, the designs could be easily reproduced with consistent properties across different batches. Numerous batches of both control designs were fabricated under strict control and plastic properties were measured for each batch immediately following mixing. Test specimens were also cast for hardened property testing after appropriate curing.

Plastic testing results for each of the control mix designs are provided in Table 3-2. Results from each mix design met requirements for Class D concrete listed in the standard specifications. Results for slump were 5.50 in. (PC Lab) and 6.25 in. (FA Lab), each meeting the specification of 8.00 in. or less. Air content results included 5.6% for the PC Lab design and 5.1% for the FA Lab design, each falling within the acceptable range of 4.5% to 7.5%.

Hardened properties for both control designs are provided in Table 3-3. Compressive strength results for each design are also provided in Figures 3-1 and 3-2. Compressive strength results indicate that each control design achieved the required 28-day compressive strength of 4,000 psi, with the PC Lab design reaching 4,100 psi at 3-days and the FA Lab design reaching 4,510 psi at 7-days. Strengths at 28-days were 5,980 psi (PC Lab) and 5,730 psi (FA Lab). FA Lab strengths lagged at early ages (1- and 3-days) with similar strengths at later ages (28- and 56-days).

Permeability results from the RCPT test method were 2,230 coulombs and 1,840 coulombs for the PC Lab and FA Lab designs, respectively. Based on these results the PC Lab design was classified as moderate permeability and the FA Lab design was classified as low permeability. Similarly, surface resistivity results for each mix design were classified as moderate permeability and low permeability. The PC design achieved a resistivity of 16.1 kohm-cm and the FA Lab design a resistivity of 22.6 kohm-cm. These results are illustrated in Figure 3-3 for comparison purposes.

Test results for the control mixes indicated each design met the plastic and hardened property requirements for Class D concrete. The compressive strength of the FA Lab design lagged the PC Lab design at early ages but reached and exceeded that of the PC design at later ages. Ultimately, each design attained similar strength. Results from both permeability test methods indicate that the FA Lab design resulted in a modest reduction in permeability when compared to PC Lab.

Table 3-1: Proportions – Class D Control Lab

	PC Lab	FA Lab
W/C Ratio	0.38	0.38
Coarse Aggregate (lb)	1,904	1,904
Fine Aggregate (lb)	1,140	1,140
Portland Cement (lb)	620	496
Class F Fly Ash (lb)	-----	124
Water (lb)	232.5	232.5
Design Air Voids (%)	6.0	6.0
Micro Air (oz/cwt)	0.74	1.11
Polyheed N (oz/cwt)	2.94	2.94
Glenium (oz/cwt)	2.20	2.2

Table 3-2: Plastic Properties – Class D Control Lab

	PC Lab	FA Lab
Slump w/o HRWR (in)	1.25	1.50
Slump with HRWR (in.)	5.50	6.25
Unit Weight (lb/ft^3)	144.6	146.2
Temperature (deg F)	73	71
Air Content (%)	5.6	5.1

Table 3-3: Hardened Properties – Class D Control Lab

	PC Lab	FA Lab
	Static Modulus of Elasticity (psi)	
7 Day	4,150,000	3,900,000
28 Day	4,700,000	4,650,000
56 Day	4,950,000	5,100,000
	Compressive Strength (psi)	
1 Day	2,670	2,180
3 Day	4,100	3,550
7 Day	4,740	4,510
28 Day	5,980	5,730
56 Day	6,500	6,660
	RCPT Permeability (coulombs)	
56 Day	2,230 (moderate)	1,840 (low)
	Surface Resistivity (kohm-cm)	
56 Day	16.1 (moderate)	22.6 (low)

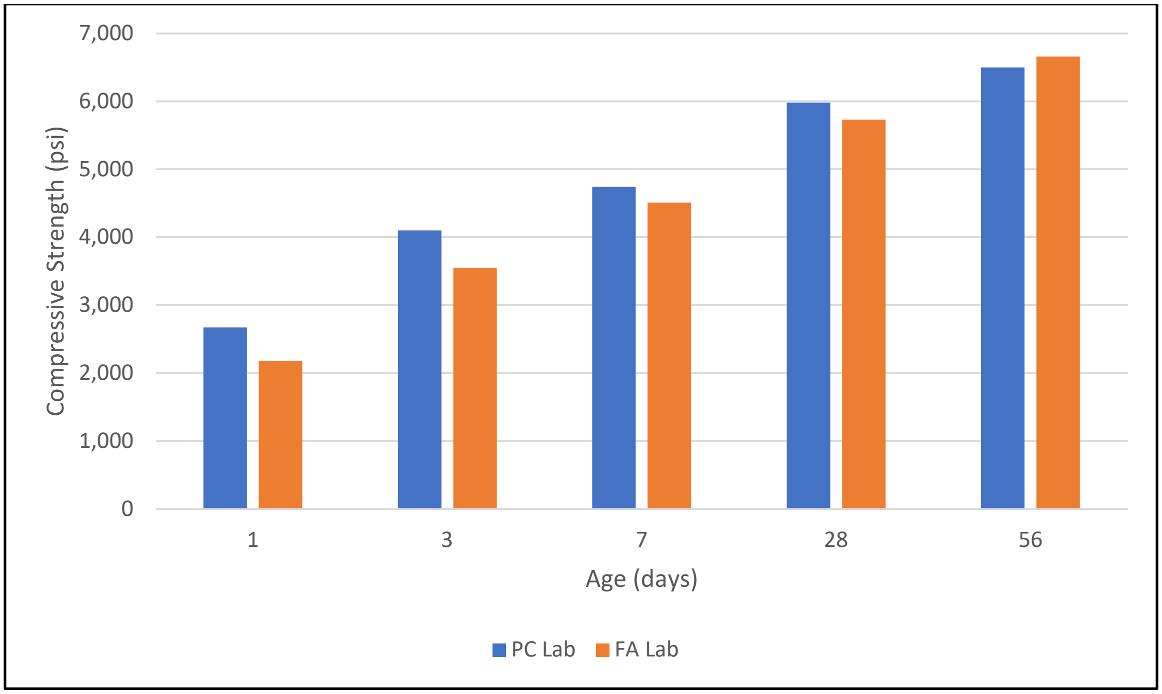

Figure 3-1: Compressive Strength – Class D Control Lab

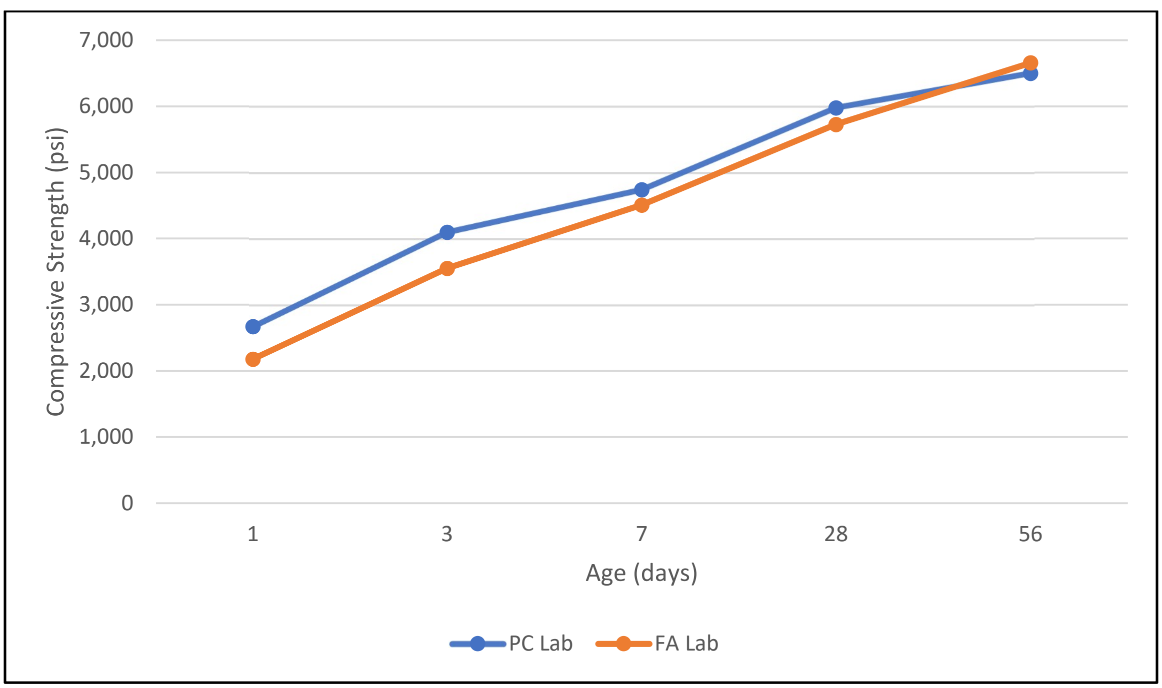

Figure 3-2: Compressive Strength – Class D Control Lab

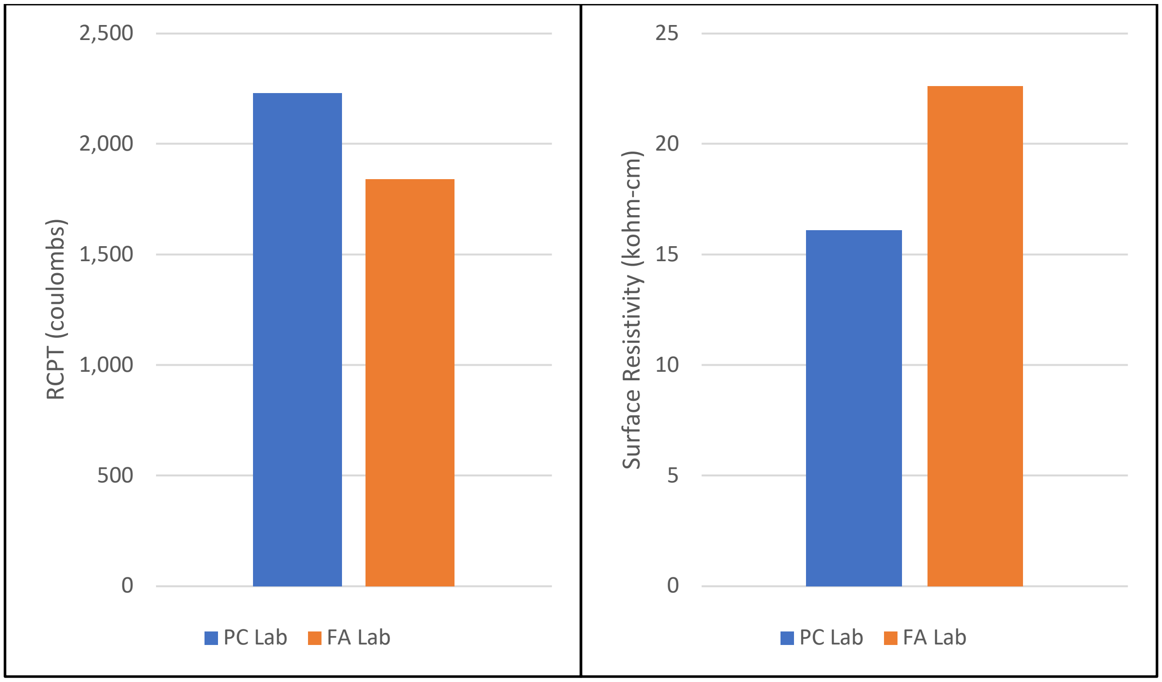

Figure 3-3: Permeability – Class D Control Lab

Class D Low Permeability Mix Designs

Three Class D low permeability mixes (Class DLP) were developed using high volume replacement with supplemental cementitious materials (SCM). The two control mixes previously discussed were used as the starting point in development. The primary purpose of the three mixes was a reduction in permeability.

Each of the mixes was developed using a different supplemental material including silica fume, metakaolin, and slag cement. Each of the mixes was based on a control mix, changing only the cementitious material makeup and admixture dosage rates.

Numerous batches of the low permeability designs were fabricated in the laboratory for the purpose of adjusting admixture rates, ensuring consistent workability and consistency, verification of acceptable plastic and hardened properties, and to ensure that each design could be consistently reproduced.

Class DLP SF

The first of the low permeability mixes, Class DLP SF (SF Lab), was developed using silica fume as a supplemental material. The design was based on the FA control mix. Ultimately, the final design replaced 3.5% of the cementitious weight with silica fume while reducing the Portland cement content by the same amount.

Class DLP MK

Metakaolin was used as the additional supplemental material for the second of the low permeability mixes. The design, Class DLP MK (MK Lab), was based on the FA control mix. Metakaolin was used as 3.5% of the cementitious content replacing the same amount of Portland cement.

Class DLP SL

The PC design was used as the basis for development of the third low permeability design. This design, Class DLP SL (SL Lab), replaced 45% of the Portland cement with slag cement.

The proportions of the low permeability mix designs are provided in Table 3-4. All materials used in each of the low permeability mix designs met requirements of the standard specifications. Cementitious makeup and admixture dosage rates changed in each design while all other components, including total cementitious content, remained unchanged.

Each of the low permeability designs were fabricated under strict control. Several batches of each design were mixed, and the standard plastic properties were measured for each immediately following mixing. Test specimens were also cast for hardened property testing after appropriate curing.

Plastic testing results for each of the low permeability designs are provided in Table 3-5. Results indicate that all plastic properties met requirements for Class D concrete listed in the standard specifications. Slump results ranged from 5.00 inches (MK Lab) to 6. 25 inches (SF Lab and SL Lab), meeting the requirement of 8.00 inches or less. Results for air content included 5.8% (SF Lab), 5.1% (MK Lab), and 5.4% (SL Lab) all falling within the 4.5% to 7.5% acceptable range.

Table 3-4: Proportions – Class DLP Lab

	SF Lab	MK Lab	SL Lab
W/C Ratio	0.38	0.38	0.38
Coarse Aggregate (lb)	1,904	1,904	1,904
Fine Aggregate (lb)	1,140	1,140	1,140
Portland Cement (lb)	474.3	474.3	341.0
Class F Fly Ash (lb)	124.0	124.0	----
Silica Fume (lb)	21.7	----	----
Metakaolin (lb)	----	21.7	----
Slag Cement (lb)	----	----	279.0
Water (lb)	232.5	232.5	232.5
Design Air Voids (%)	6.0	6.0	6.0
Micro Air (oz/cwt)	3.70	4.07	3.33
Polyheed N (oz/cwt)	2.94	2.94	2.94
Glenium (oz/cwt)	2.20	2.20	2.20

Table 3-5: Plastic Properties – Class DLP Lab

	SF Lab	MK Lab	SL Lab
Slump w/o HRWR (in.)	1.50	1.00	1.50
Slump with HRWR (in.)	6.25	5.00	6.25
Unit Weight (lb/ft^3)	145.7	144.2	145.9
Temperature (deg F)	73	71	74
Air Content (%)	5.8	5.1	5.4

Hardened testing results for each of the low permeability designs are provided in Table 3-6. Each of the low permeability designs achieved the required 4,000 psi compressive strength at 28-days of age. Results at 28-days included 4,560 psi (SF Lab), 4,700 psi (MK Lab) and 6,540 psi (SL Lab). Compressive strength results are also provided in Figures 3-4 and 3-5 for comparison and illustration of relative strength gain.

Permeability results (56-day) from the RCPT test method were 910 coulombs, 961 coulombs and 870 coulombs for the SF, MK, and SL designs, respectively. Based on these results, each of the mix designs were classified as very low permeability. Similarly, based on surface resistivity results, each mix design was classified as very low permeability. Surface resistivity results (56-day) included 47.8 kohm-cm for the SF design, 46.9 kohm-cm for the MK design, and 47.4 kohm-cm for the SL design. These results are provided in Figure 3-6 for comparison purposes.

Comparison of the low permeability designs revealed that similar plastic properties could be easily achieved and were within acceptable limits for Class D concrete. Compressive strength results met required minimum strength and were consistent across all designs at ages of 1, 3, and 7 days. The SL design achieved noticeably higher compressive strength at ages of 28 and 56 days. Permeability results were consistently categorized as very low permeability across all designs and test methods.

Table 3-6: Hardened Properties - Class DLP Lab

	SF Lab	MK Lab	SL Lab
		Static Modulus of Elasticity (psi)	
7 Day	3,450,000	3,410,000	3,550,000
28 Day	4,150,000	4,250,000	4,950,000
56 Day	4,700,000	4,600,000	5,250,000
		Compressive Strength (psi)	
1 Day	1,900	2,040	1,470
3 Day	3,150	2,490	2,630
7 Day	3,380	3,230	3,520
28 Day	4,560	4,700	6,540
56 Day	5,510	5,720	6,930
		RCPT Permeability (coulombs)	
56 Day	910 (very low)	961 (very low)	870 (very low)
		Surface Resistivity (kohm-cm)	
56 Day	47.8 (very low)	46.9 (very low)	47.4 (very low)

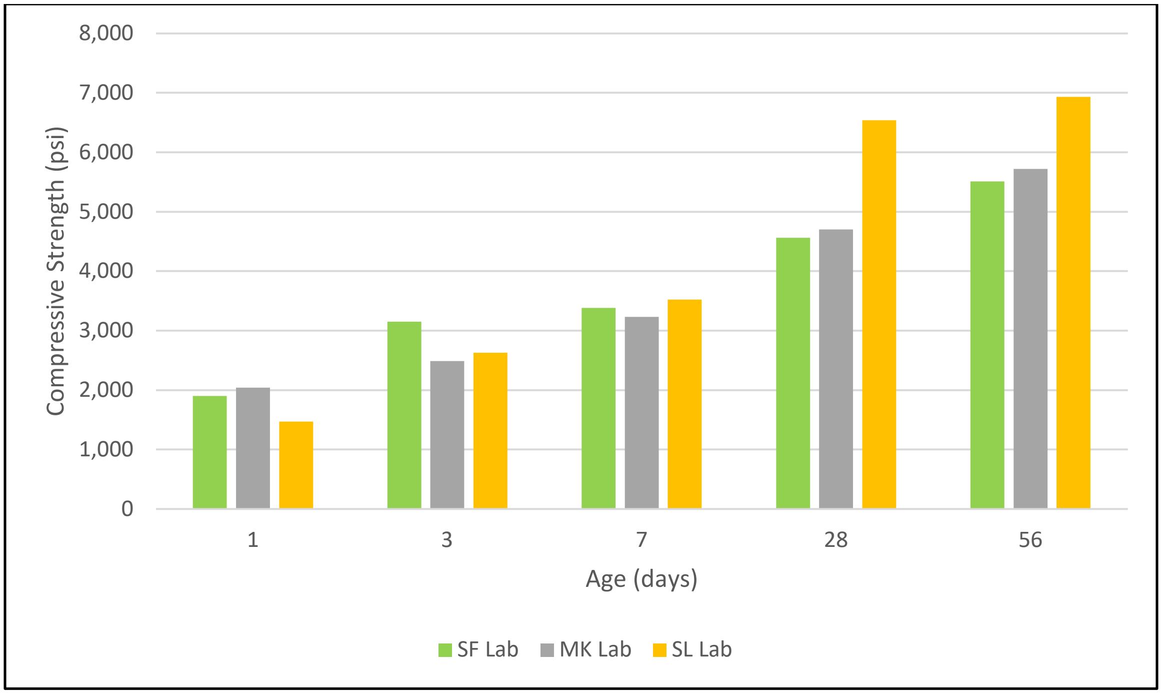

Figure 3-4: Compressive Strength – Class DLP Lab

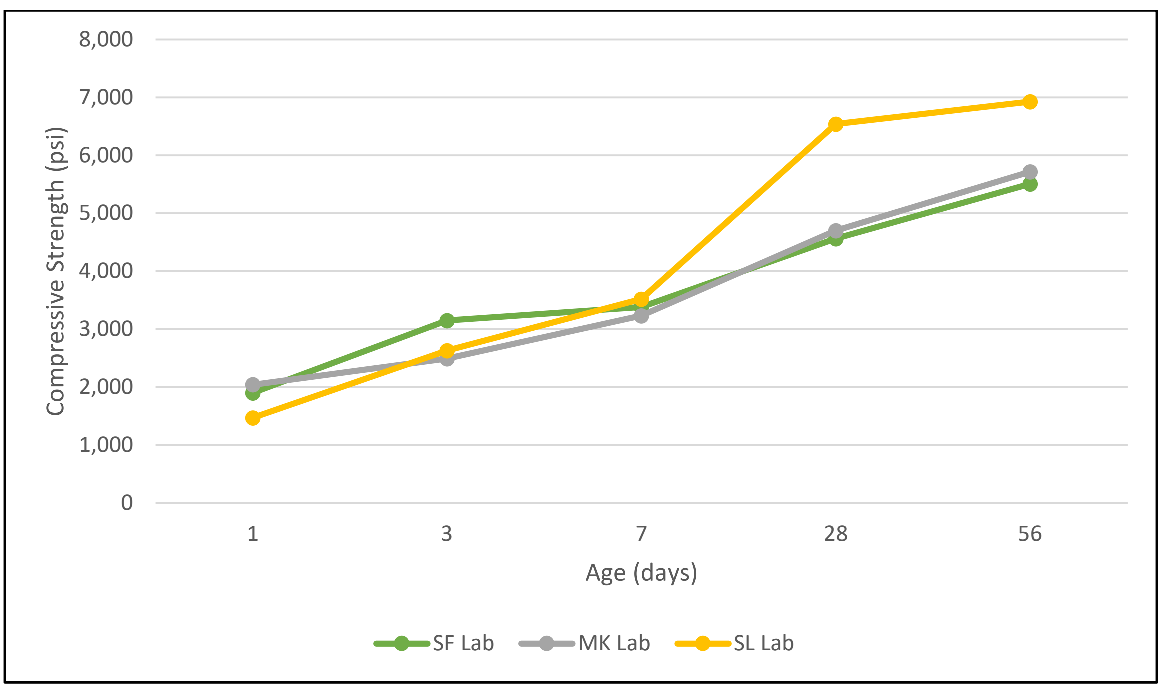

Figure 3-5: Compressive Strength – Class DLP Lab

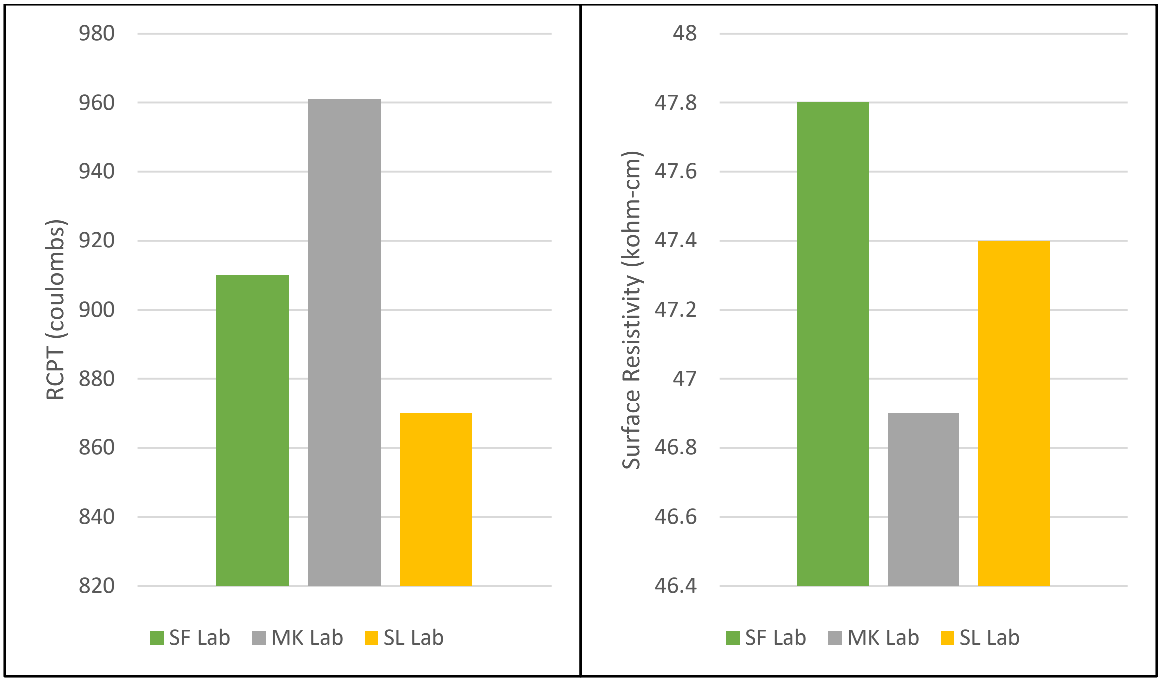

Figure 3-6: Permeability – Class DLP Lab

Control and Low Permeability Comparison

The control (2) and low permeability (3) designs were compared to illustrate the differences in makeup, compare plastic and hardened properties, and to highlight the effect of the supplemental materials on permeability. The proportions for each design are consolidated in Table 3-7.

Comparison of the proportions of all designs indicated only changes in cementitious makeup and the dosage rate of the admixture responsible for air entrainment. One design (PC) used a single cementitious material, 2 designs (FA and SL) included 2 cementitious materials, and 2 designs (SF and MK) incorporated 3 cementitious materials. Air entraining dosage changed across mix designs based on the demand of the cementitious materials used. This dosage rated was considerably elevated for each of the low permeability designs when compared to the control designs.

Plastic properties for each design, provided in Table 3-8, remained consistent across all mix designs. Slump measured after the addition of the HRWR ranged from a minimum of 5.00 in. to a maximum of 6.25 in., all less than the requirement of 8.00 in. or less. Likewise, all air content results were within the allowed range of 4.5% to 7.5%. Air content results ranged from 5.1% to 5.8%.

Table 3-7: Proportions – All Mix Designs Lab

	PC Lab	FA Lab	SF Lab	MK Lab	SL Lab
W/C Ratio	0.38	0.38	0.38	0.38	0.38
Coarse Aggregate (lb)	1,904	1,904	1,904	1,904	1,904
Fine Aggregate (lb)	1,140	1,140	1,140	1,140	1,140
Portland Cement (lb)	620	496	474.3	474.3	341.0
Class F Fly Ash (lb)	-----	124	124.0	124.0	----
Silica Fume	-----	-----	21.7	----	----
Metakaolin	-----	-----	----	21.7	----
Slag Cement	-----	-----	----	----	279.0
Water (lb)	232.5	232.5	232.5	232.5	232.5
Design Air Voids (%)	6.0	6.0	6.0	6.0	6.0
Micro Air (oz/cwt)	0.74	1.11	3.70	4.07	3.33
Polyheed N (oz/cwt)	2.94	2.94	2.94	2.94	2.94
Glenium (oz/cwt)	2.20	2.2	2.20	2.20	2.20

Table 3-8: Plastic Properties – All Mix Designs Lab

	PC Lab	FA Lab	SF Lab	MK Lab	SL Lab
Slump w/o HRWR (in)	1.25	1.50	1.50	1.00	1.50
Slump with HRWR (in.)	5.50	6.25	6.25	5.00	6.25
Unit Weight (lb/ft^3)	144.6	146.2	145.7	144.2	145.9
Temperature (deg F)	73	71	73	71	74
Air Content (%)	5.6	5.1	5.8	5.1	5.4

Hardened properties for each design are provided in Table 3-9. Compressive strength results, illustrated in Figures 3-7 and 3-8, from each design met the required minimum of 4,000 psi at 28-days. Early age strength of the SCM designs were lower than the controls (significantly in one case). Each of the SCM designs reached design strength at 28-days, lagging the control mixes at 3- (PC) and 7- (FA) days. In comparison with the PC Lab design, the SF and MK designs resulted in modestly lower (12-15%) strength and the SL design resulted in minor (7%) improvement at later ages. Results indicate that use of the supplemental materials had modest impact on early age strength, rate of strength gain, and ultimate observed strength.

Permeability results are illustrated in Figures 3-9 and 3-10. RCPT results indicated the use of fly ash results in modest improvement when compared to the PC design, improving the rating from modest permeability to low permeability across both test methods. Analysis also indicated that each of the low permeability designs resulted in considerable reductions in 56-day permeability when compared to either of the control designs with each categorized as very low permeability regardless of test method.

Each of the laboratory mix designs met all proportioning requirements, plastic properties, and hardened properties. Results indicate that the SCM designs yielded modest impacts (positive and negative) on compressive strength, though not significant enough to cause concern. Significant reductions in permeability were achieved through use of each of the SCMs. Each of the SCM designs were found to be easily replicated with consistent workability and hardened and plastic properties. Evaluation of the mix designs using SCM replacement indicated that each was suitable for full-scale construction application.

Table 3-9: Hardened Properties – All Mix Designs Lab

	PC Lab	FA Lab	SF Lab	MK Lab	SL Lab
			Static Modulus of Elasticity (psi)		
7 Day	4,150,000	3,900,000	3,450,000	3,410,000	3,550,000
28 Day	4,700,000	4,650,000	4,150,000	4,250,000	4,950,000
56 Day	4,950,000	5,100,000	4,700,000	4,600,000	5,250,000
			Compressive Strength (psi)		
1 Day	2,670	2,180	1,900	2,040	1,470
3 Day	4,100	3,550	3,150	2,490	2,630
7 Day	4,740	4,510	3,380	3,230	3,520
28 Day	5,980	5,730	4,560	4,700	6,540
56 Day	6,500	6,660	5,510	5,720	6,930
			RCPT Permeability (coulombs)		
56 Day	2,230 (moderate)	1,840 (low)	910 (very low)	961 (very low)	870 (very low)
			Surface Resistivity (kohm-cm)		
56 Day	16.1 (moderate)	22.6 (low)	47.8 (very low)	46.9 (very low)	47.4 (very low)

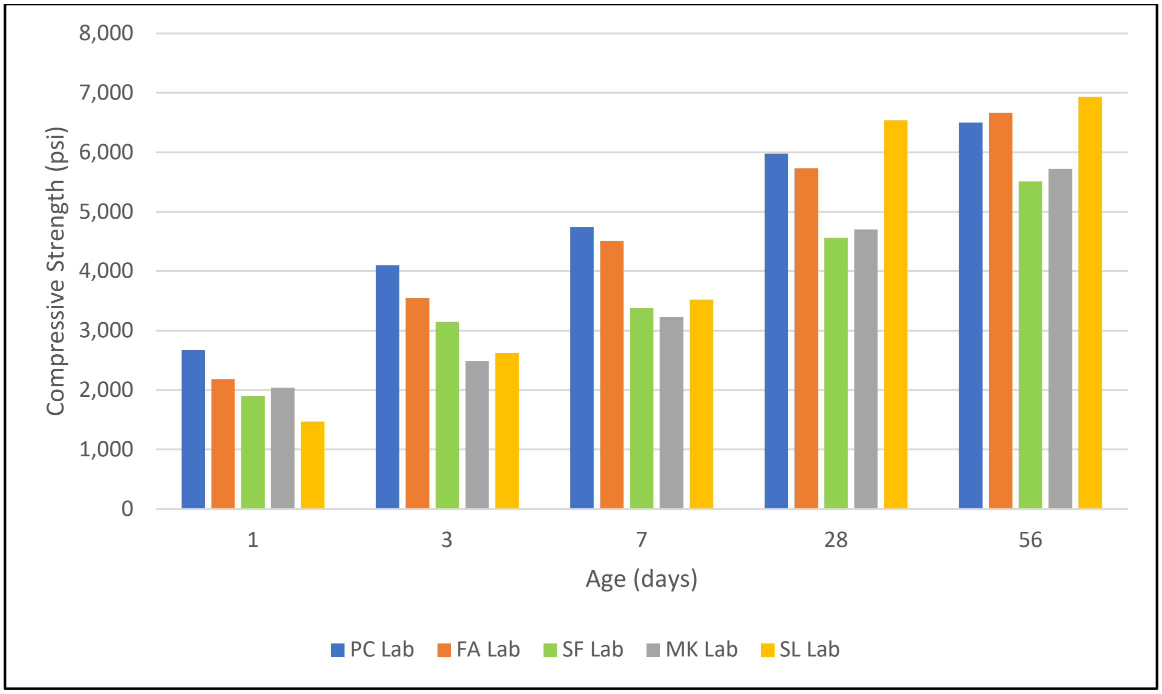

Figure 3-7: Compressive Strength – All Mix Designs Lab

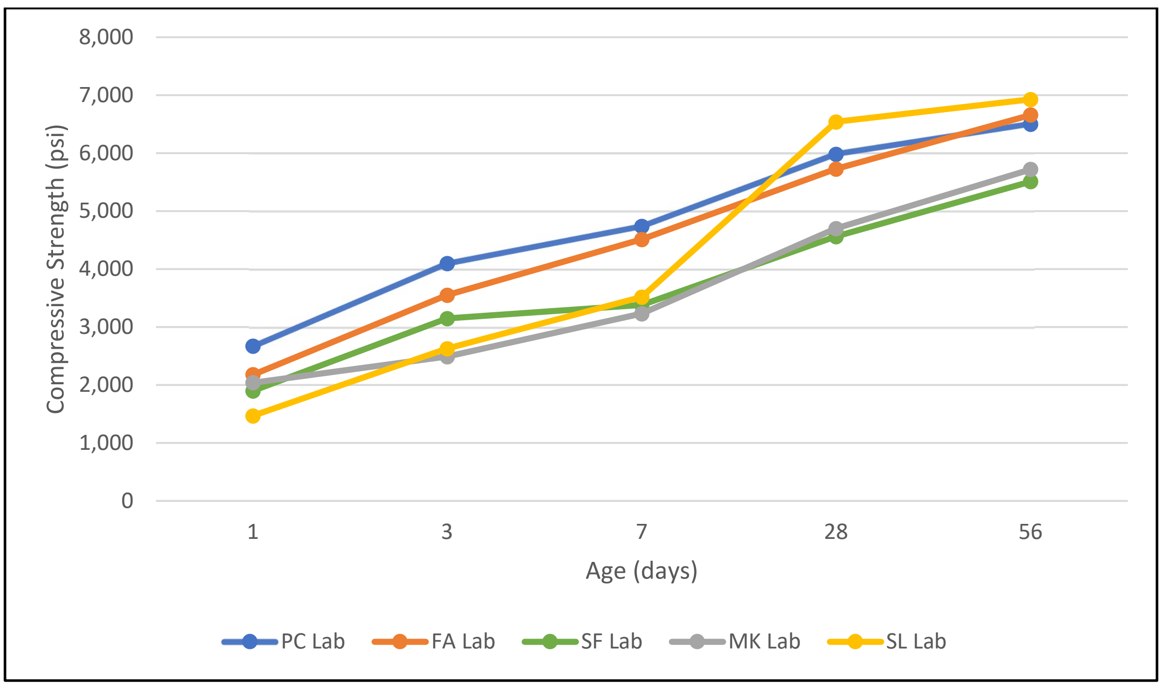

Figure 3-8: Compressive Strength – All Mix Designs Lab

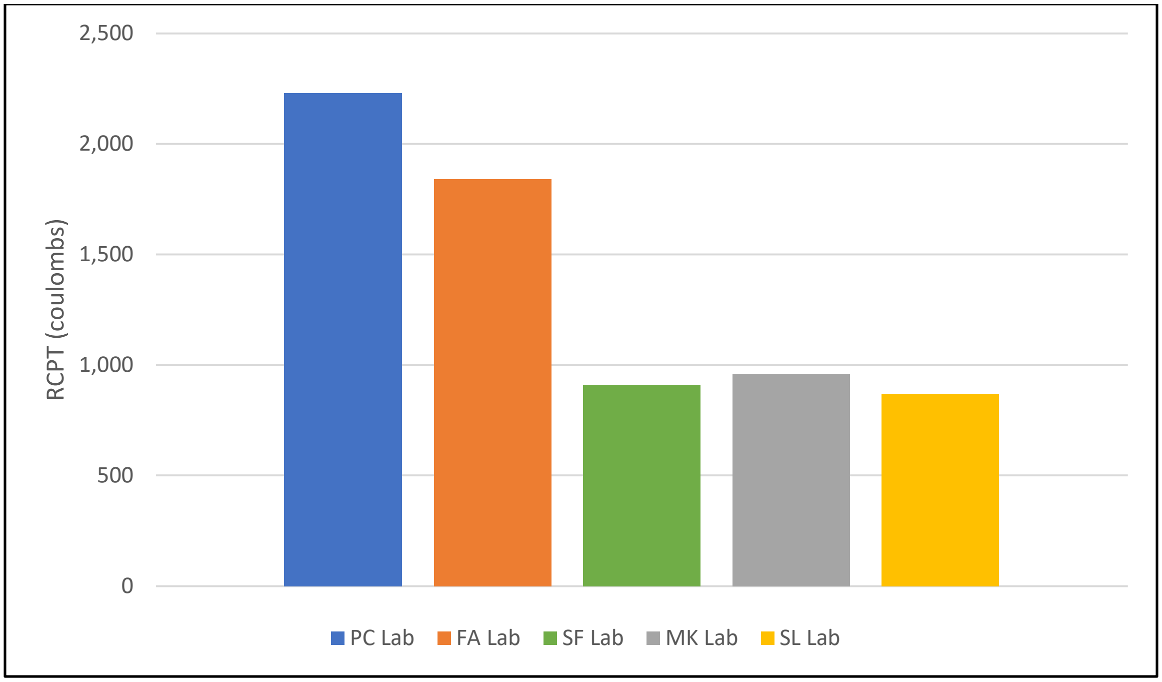

Figure 3-9: Permeability (RCPT) – All Mix Designs Lab

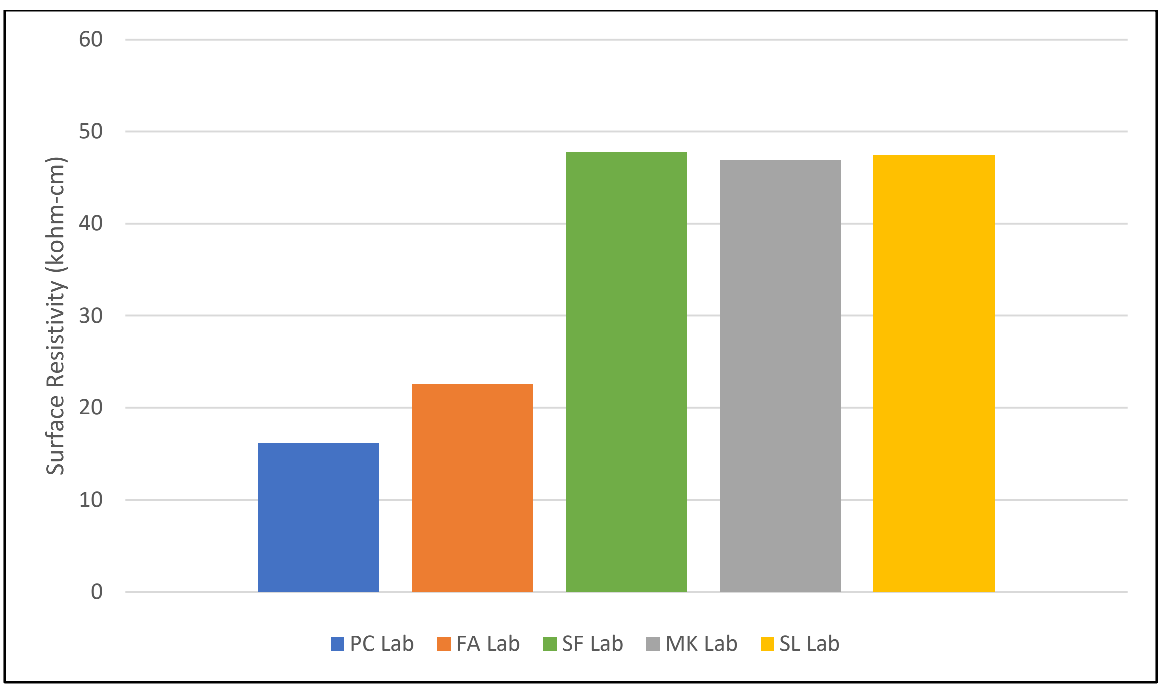

Figure 3-10: Permeability (Surface Resistivity) – All Mix Designs Lab

SECTION 4: PROJECT BRIDGES

Two bridges were selected for full scale use of mix designs (3 low permeability and 1 control) discussed in Section 3. TDOT personnel (materials and structures) invested considerable study in selection of project bridges that would meet applicable criteria and allowed both study and comparison of the low permeability mix designs with the control.

Ultimately, TDOT personnel selected the bridges included in CNP 168 – Project No. 94007-4231-04 for the study. The project chosen was a bridge repair project addressing two separate bridges on State Route 11 in Williamson County, Tennessee. Each bridge was located over and unnamed branch.

Selection of CNP 168 resulted in two structures that were located only miles apart on the same State Route resulting in very similar daily traffic and loading, salting/de-icing chemical exposure, and ambient environmental conditions. Also, each structure was of similar size (width and length) and was repaired with similar superstructure and deck designs. These similarities helped reduce the number of variables effecting long-term concrete deck performance resulting in more direct comparison and evaluation of each mix design.

The two structures included Bridge No. 94-SR11-0.59 and Bridge No. 94-SR11-0.87. Each bridge consisted of 2 spans, reinforced concrete abutments and pier, reinforced cast-in-place concrete beams, reinforced concrete deck, and asphalt overlay. The existing and proposed roadway width was 28 ft. - 0 in. for each structure. The existing width of each structure was 34 ft. - 6 in. with proposed widths of 35 ft. - 0 in.

Bridge No. 94-SR11-0.59 had an existing overall length of 57 ft. - 0 in. and proposed length of 60 ft. - 3 1/2 in. The two spans had original lengths of 34 ft. - 6 in. and proposed lengths of 35 ft. - 0 in.

Bridge No. 94-SR11-0.87 had an existing overall length of 41 ft. - 0 in. and proposed length of 42 ft. - 6 in. The two spans had original lengths of 20 ft. - 6 in. and proposed lengths of 21 ft. - 3 in.

In addition to aspects such as paving, grading, and safety upgrades, the project included repair and modification of the substructure and full replacement of the superstructure and deck of each bridge. Prestressed concrete box beams were used as superstructure replacement. The new deck consisted of reinforced concrete with two layers of reinforcing steel with typical thickness of 8 ¼ in.

Each structure included two lanes of traffic resulting in a phased approach during construction. Phasing included closing one lane on each structure while using the remaining lane for traffic in both directions using a timed traffic signal. The closed lane portion of each structure was demolished as needed, repaired/modified, and new superstructure and deck placed. After complete, the process was completed in a similar manner on the remaining lane(s). Figures 4-1 and 4-2 illustrate each structure at the beginning of one phase of construction (after asphalt removal).

In addition to functional and safety improvements/upgrades, repair and replacement was needed due to several forms of concrete deterioration. While corrosion of reinforcing is a primary deterioration mechanism of reinforced concrete, much of the steel removed from the structures during demolition exhibited minor corrosion as shown in Figure 4-3. Several likely factors resulting in deterioration included original quality of concrete, amount of reinforcing, exposure to water and de-icing salts, and leakage from the wearing surface or joints onto the structure below.

The substructure of each bridge, including 2 abutments and 1 pier, required considerable repair due to spalling, delamination, and general concrete degradation. Likely underlying causes of deterioration included exposure to deicing chemicals and water, less than adequate reinforcing steel, freeze-thaw action, and initial concrete quality. Damage was also evident at areas with substantial leaking from joints or damaged areas in the superstructure or deck. Most damage requiring repair was present along the outside edges of each structure where protection from the superstructure was not present or where additions/repairs to the original substructure had be completed. Much of the substructure was found to be sound, requiring little repair prior to receiving modification for new superstructure installation. Examples of sound and deteriorated substructure, damage removal, preparation for superstructure installation, and completed substructure modification are provide in Figures 4-4 thru 4-16.

Superstructure condition before and after demolition is illustrated in Figure 4-17 thru 4-24. Damage evident on the existing concrete beams included corrosion, cracking, and efflorescence. Much of the deterioration appeared to be due to migration of moisture and chlorides from the wearing surface and through joints. Portions of the beams were undamaged with no apparent concrete failure or reinforcing corrosion. Illustration of the replacement prestressed box beams are provided in Figures 4-25 thru 4-27.

The existing cast-in-place reinforced concrete deck suffered considerably less deterioration compared to the substructure and superstructure, possibly due to shorter service life. Much of the existing deck showed little deterioration. The predominant form of deterioration was corrosion of reinforcing steel as illustrated in Figures 4-28 thru 4-32. During demolition, the existing deck was found to be mostly sound as shown in Figure 4-33. Illustration of the deck cross section is provided in Figures 4-34 and 4-35. As shown, both the concrete and reinforcing appear in good condition with the evident corrosion resulting after exposure during demolition. The existing deck contained two layers of reinforcing with approximately double the number of bars in the top mat. Figures 4-36 to 4-43 illustrate construction of the replacement decks and completed decks are illustrated in Figures 4-44 to 4-63.

Figure 4-1: Bridge No. 94-SR11-0.87

Figure 4-2: Bridge No. 94-SR11-0.59

Figure 4-3: Reinforcing Removed During Demolition

Figure 4-4: Substructure - Deterioration

Figure 4-5: Substructure – Deterioration

Figure 4-6: Substructure – Deterioration

Figure 4-7: Substructure - Deterioration

Figure 4-8: Substructure - Repair

Figure 4-9: Substructure – Repair

Figure 4-10: Substructure – Repair

Figure 4-11: Substructure – Prepared for Modification

Figure 4-12: Substructure – Prepared for Modification

Figure 4-13: Substructure – Prepared for Modification

Figure 4-14: Substructure – Partially Repaired

Figure 4-15: Substructure – Modification

Figure 4-16: Substructure – Completed

Figure 4-17: Superstructure - Deterioration

Figure 4-18: Superstructure - Deterioration

Figure 4-19: Superstructure - Deterioration

Figure 4-20: Superstructure - Deterioration

Figure 4-21: Superstructure - Deterioration

Figure 4-22: Superstructure - Deterioration

Figure 4-23: Superstructure – Good Condition Without Corrosion

Figure 4-24: Superstructure – Good Condition Without Corrosion

Figure 4-25: Superstructure – Completed Phase

Figure 4-26: Superstructure – Completed Phase

Figure 4-27: Superstructure – Completed Phase with Modified Substructure

Figure 4-28: Deck - Deterioration

Figure 4-29: Deck - Deterioration

Figure 4-30: Deck - Deterioration

Figure 4-31: Deck - Deterioration

Figure 4-32: Deck - Deterioration

Figure 4-33: Deck - Demolition

Figure 4-34: Deck – Cross Section After Removal

Figure 4-35: Deck – Cross Section After Removal

Figure 4-36: Deck – Form Installation

Figure 4-37: Deck – Form Installation

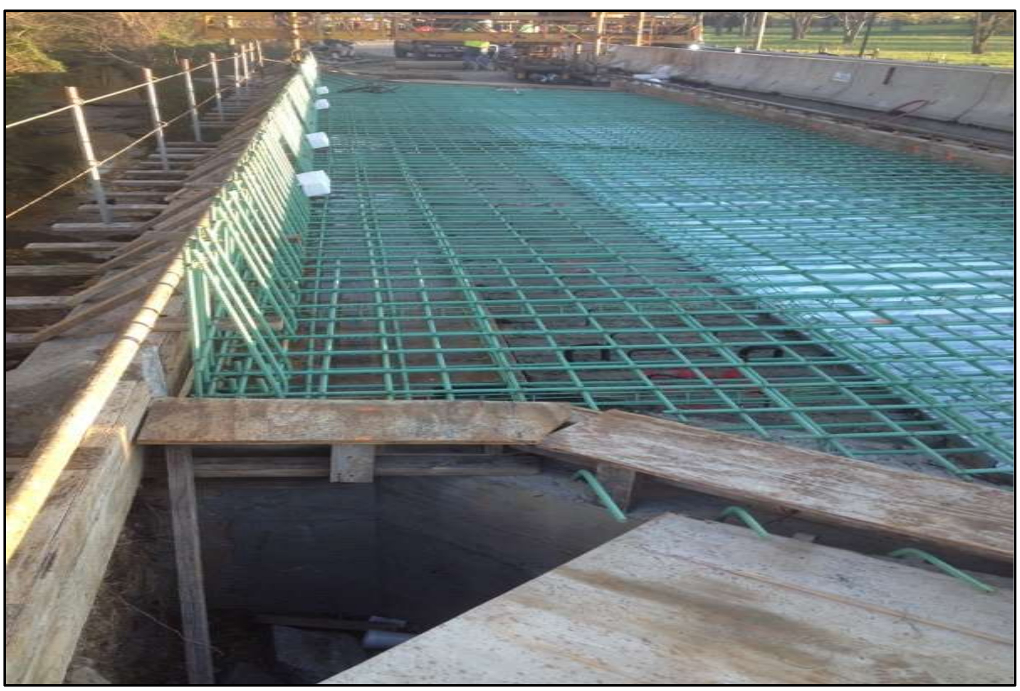

Figure 4-38: Deck – Reinforcing Installation

Figure 4-39: Deck – Concrete Placing

Figure 4-40: Deck – Concrete Placing

Figure 4-41: Deck – Concrete Placing

Figure 4-42: Deck – Concrete Placing

Figure 4-43: Deck – Concrete Placing

Figure 4-44: Completed Deck – Bridge No. 94-SR11-0.59

Figure 4-45: Completed Deck – Bridge No. 94-SR11-0.59

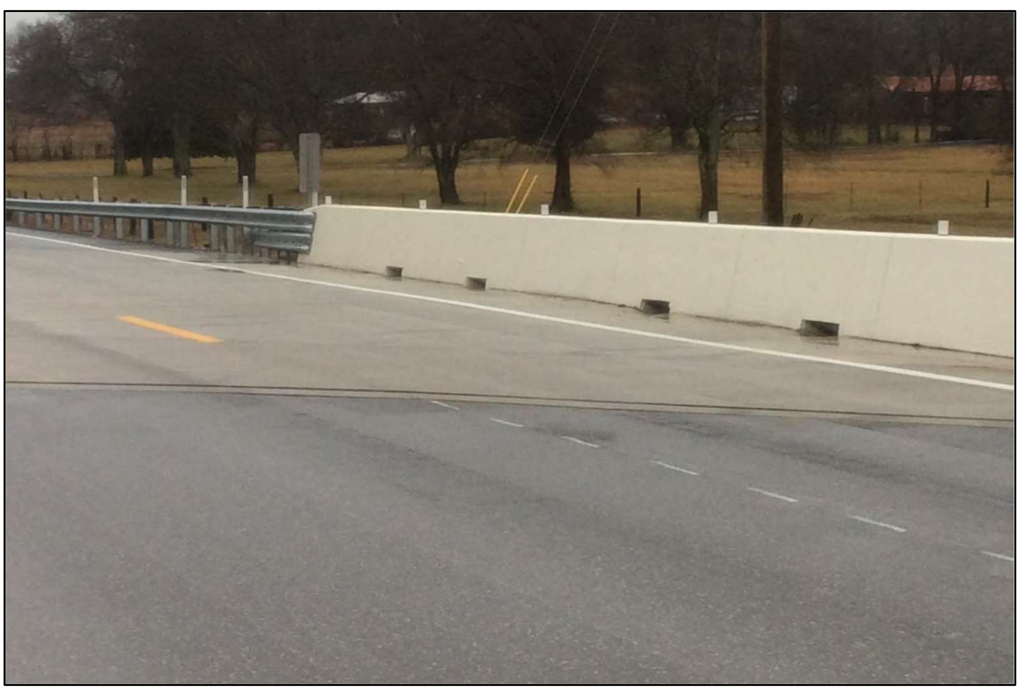

Figure 4-46: Completed Deck – Bridge No. 94-SR11-0.59

Figure 4-47: Completed Deck – Bridge No. 94-SR11-0.59

Figure 4-48: Completed Deck – Bridge No. 94-SR11-0.59

Figure 4-49: Completed Deck – Bridge No. 94-SR11-0.59

Figure 4-50: Completed Deck – Bridge No. 94-SR11-0.59

Figure 4-51: Completed Deck – Bridge No. 94-SR11-0.59

Figure 4-52: Completed Deck – Bridge No. 94-SR11-0.59

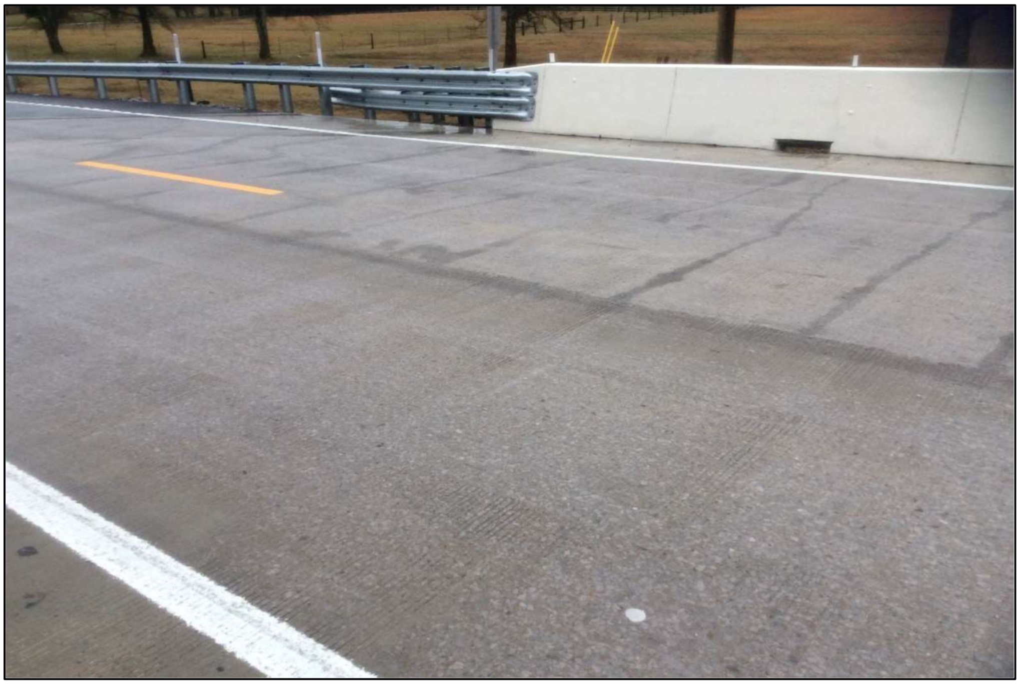

Figure 4-53: Completed Deck – Bridge No. 94-SR11-0.59

Figure 4-54: Completed Deck – Bridge No. 94-SR11-0.59

Figure 4-55: Completed Deck – Bridge No. 94-SR11-0.59

Figure 4-56: Completed Deck – Bridge No. 94-SR11-0.87

Figure 4-57: Completed Deck – Bridge No. 94-SR11-0.87

Figure 4-58: Completed Deck – Bridge No. 94-SR11-0.87

Figure 4-59: Completed Deck – Bridge No. 94-SR11-0.87

Figure 4-60: Completed Deck – Bridge No. 94-SR11-0.87

Figure 4-61: Completed Deck – Bridge No. 94-SR11-0.87

Figure 4-62: Completed Deck – Bridge No. 94-SR11-0.87

Figure 4-63: Completed Deck – Bridge No. 94-SR11-0.87

SECTION 5: TESTING RESULTS – FULL SCALE IMPLEMENTATION

The four low permeability mix designs used for full scale field implementation included Class DLP FA (FA Field), Class DLP SF (SF Field), Class DLP SL (SL Field), and Class DLP MK (MK Field). Using materials from the specific plant supplying concrete for the project, the producer performed laboratory verification of each mix design to ensure that plastic and hardened properties met Class D requirements. Mix designs discussed in Section 3 were maintained except for admixture dosage rates. Due to its similarity with mix designs typically already used for bridge decks in Tennessee, FA Field was considered the control mix and was used for comparison and evaluation of the performance of the other three low permeability designs.

The following sections include data and discussion regarding the plastic and hardened properties for each of the four mix designs. Results for each mix design are provided along with comparison to results from its respective laboratory trial mix. Results from each low permeability mix were also compared to the results of the control mix (FA Field).

Plastic properties of each mix design used were measured immediately prior to placing on each of the project's four bridge decks. The same test methods as laboratory testing were used. Plastic properties measured for each design included slump, air content, and temperature. Ambient air temperature was also measured.

Test specimens for hardened testing, including compressive strength and permeability were cast immediately following plastic testing. Specimens cured in the field immediately adjacent to the deck placed. Specimens were transported to the laboratory (at 1 and 3 days of age) and stored in curing tanks until immediately prior to testing. Compressive strength was measured at ages of 1, 3,7, 28, 56, and 128 days. Permeability testing (both methods) was completed at ages of 56 and 128 days.

The same contractor (Jamison) was responsible for placing, finishing, and curing each phase of deck construction. Likewise, the same concrete producer (IMI, Inc.) supplied concrete for each deck. Information for the contractor and concrete supplier are provided below.

Jamison Construction	**Irving Materials, Inc. (IMI)**
4532 Indian Creek Rd	4616 Hardison Mills Rd
McEwen, TN 37101	Columbia, TN 38401

All phases of deck construction were completed in a similar manner. Concrete was placed into deck forms through use of a crane and bucket system. The same vibratory roller screed was used to place, consolidate, and finish (with minimal hand work) each deck. Similar curing methods were used on each deck after placement. Any issues with concrete production, placing, and finishing are discussed along with the respective mix design.

FA Field

The Class DLP FA mix design (FA Field) was used for Phase 1 (southbound lane) deck construction on Bridge No. 94-SR11-0.87.

During construction operations, issues (including weather) that limited the use and performance of the plastic concrete were not identified. No loads of concrete were rejected. Issues that impacted concrete production and delivery were not noted by the contractor or producer. Ambient temperature was low but rising at the time of initial concrete placing. Delivery occurred on time with adequate truck spacing. Minimal onsite adjustments to the mixture were required. Concrete was placed and finished in an efficient manner. Contractor personnel responsible for placing and finishing operations did not identify any concerns with the concrete mixture. Curing procedures were implemented in a timely manner.

Plastic Properties

Immediately prior to discharge of concrete, plastic properties were measured following any final additions of water or admixtures. Results from plastic testing are provided in Table 5-1. As shown, each of the plastic properties measured met specifications for Class D concrete. Slump was reported at 6.00 in. (8.00 inches or less) and air content was reported at 6.5% (4.5% to 7.5%).

Table 5-1: Plastic Properties – FA Field

Slump (inches)	6.00
Concrete Temperature (Deg F)	62
Ambient Temperature (Deg F)	39
Air Content (%)	6.5

Hardened Properties

Results from hardened property testing are provided in Table 5-2. The FA Field mix design met (exceeded) the minimum compressive strength requirement (4,000 psi) beginning at 7-days of age (4,820 psi) with a strength of 6,680 psi at 28-days. Compressive strength results are also illustrated in Figures 5-1 and 5-2.

Permeability results at 56-days included 2,780 coulombs (RCPT) and 24.9 kohm-cm (resistivity), considered moderate permeability and low permeability, respectively. Significant reduction in permeability occurred between the 56- and 128-day testing. The 128-day results included a permeability of 747 coulombs and a surface resistivity of 51.3 kohm-cm, each considered very low permeability. Results are also illustrated in Figure 5-3.

Table 5-2: Hardened Properties – FA Field

	Compressive Strength (psi)	Surface Resistivity (kohm-cm)	RCPT Permeability (coulombs)
1 day	2,570	-----	-----
3 day	3,570	-----	-----
7 day	4,820	-----	-----
28 day	6,680	-----	-----
56 day	7,130	24.9 (low)	2,780 (moderate)
128 day	8,630	51.3 (very low)	747 (very low)

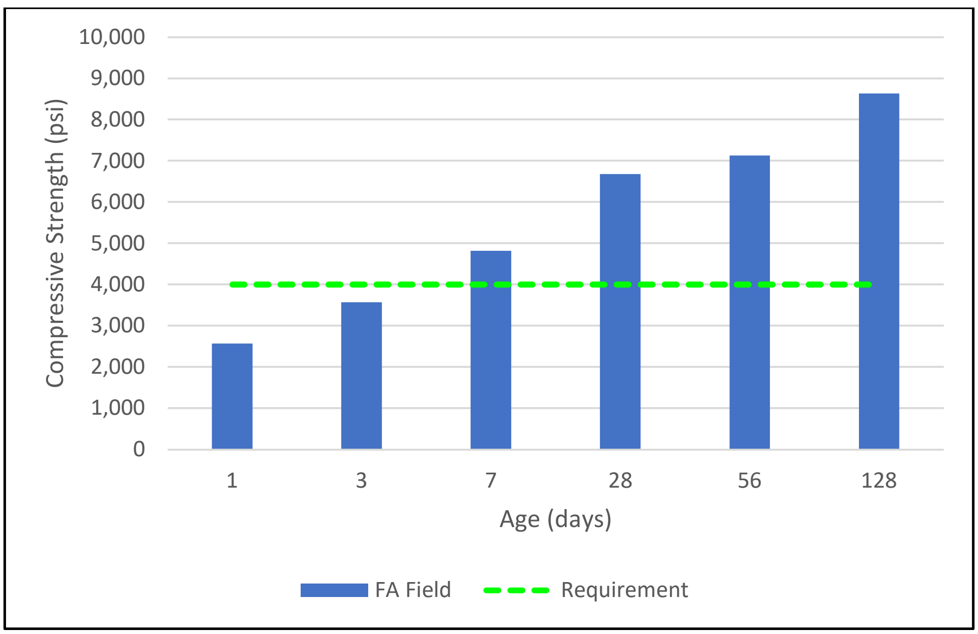

Figure 5-1: Compressive Strength – FA Field

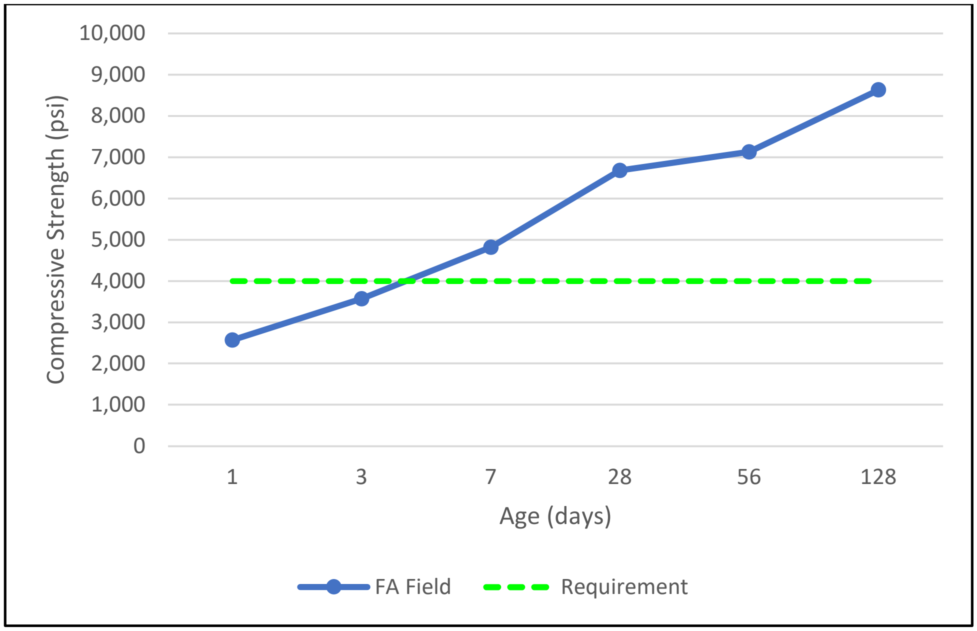

Figure 5-2: Compressive Strength – FA Field

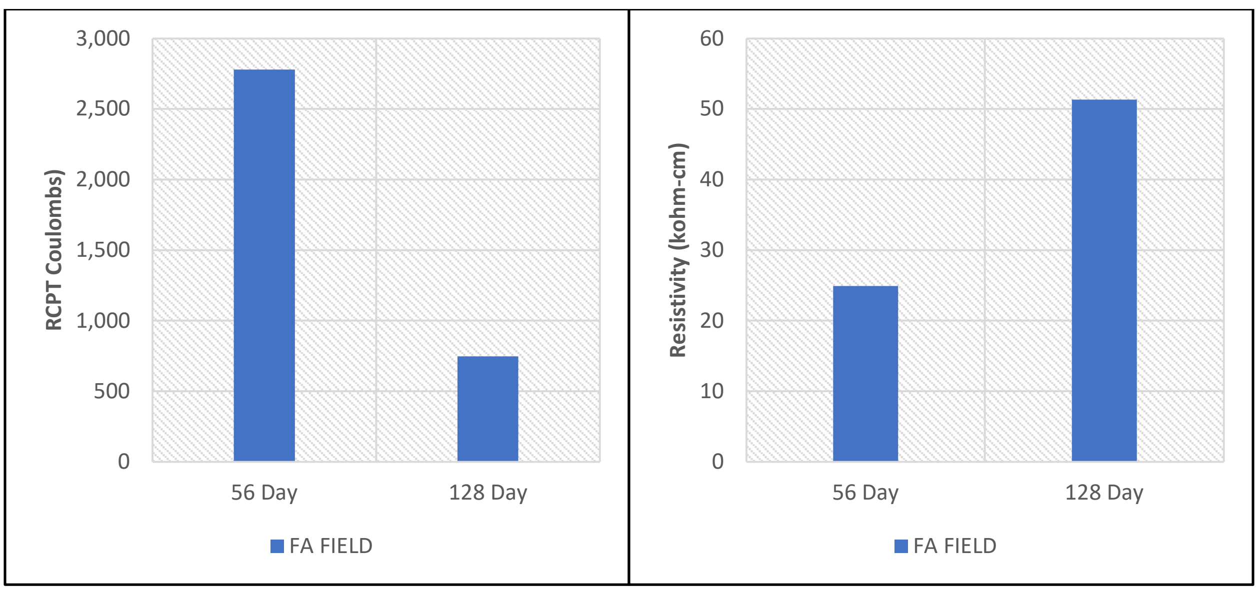

Figure 5-3: Permeability – FA Field

Comparison with FA Lab

Compressive strength performance of the FA Field and FA Lab mixes is provided in Figures 5-4 and 5-5. As shown, each mix met the minimum strength specifications. Both mixes achieved the minimum strength at the same age (7 days). Results for 28-day strength were 6,680 psi (~17% higher) for the field mix versus 5,730 psi for the lab mix. Comparison of the results indicated the field mix modestly outperformed the lab mix at all ages.

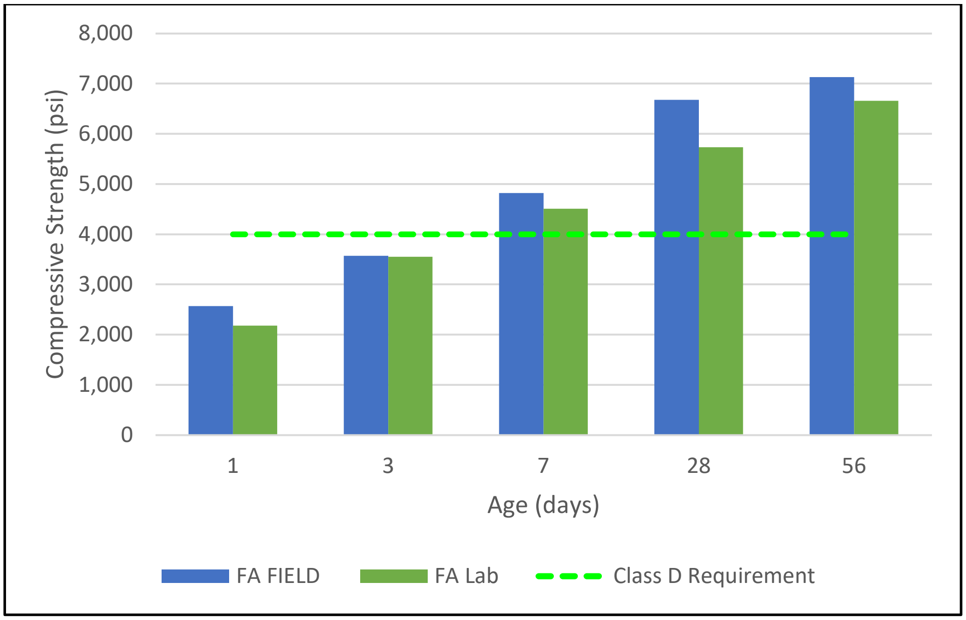

Figure 5-4: Compressive Strength – FA Field and FA Lab

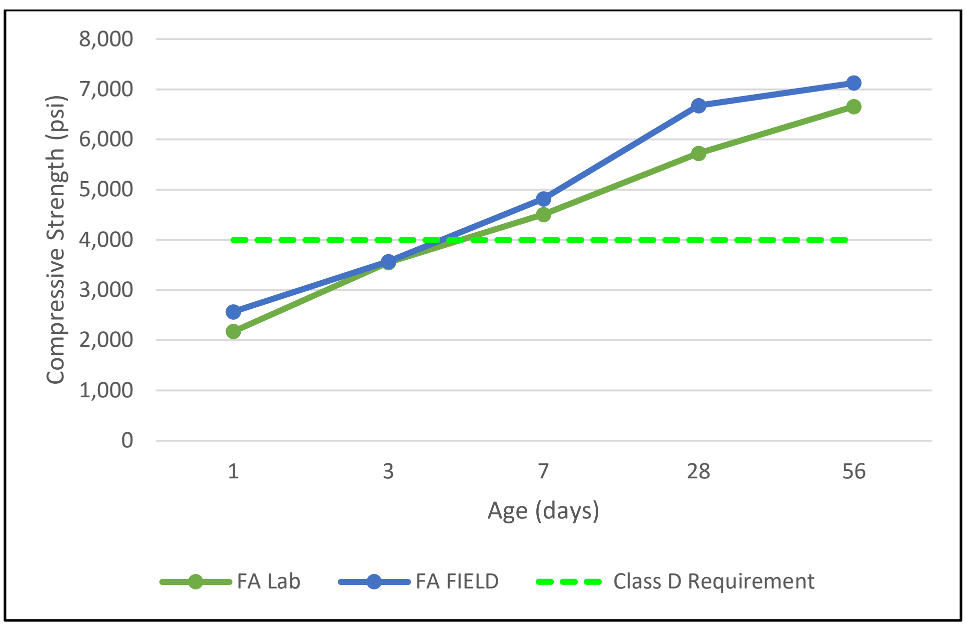

Figure 5-5: Compressive Strength – FA Field and FA Lab

Results from both permeability testing methods are provided in Figure 5-6 for the field and lab mixes at 56-days of age. Comparison indicates that the field mix modestly outperformed the lab mix regarding surface resistivity with results of 24.9 kohm-cm (low) versus 22.6 kohm-cm (low). Conversely, RCPT results indicate that the lab mix outperformed the field mix with ratings of low (1,840 coulombs) and moderate (2,780 coulombs), respectively.

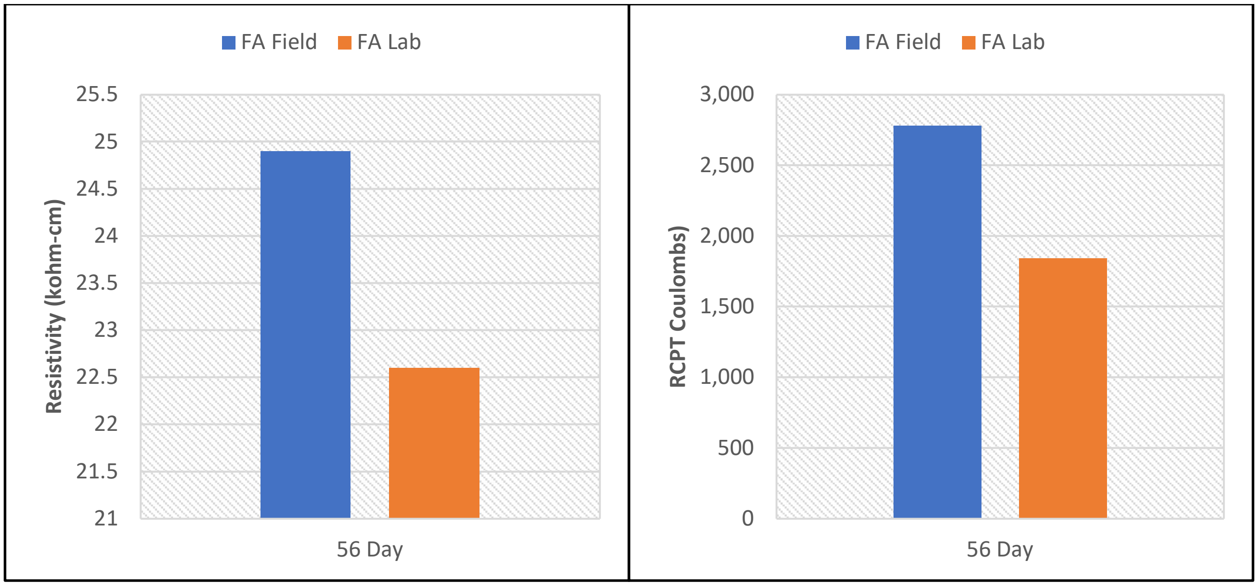

Figure 5-6: Permeability – FA Field and FA Lab

SF Field

The Class DLP SF (SF Field) mix design was used for deck construction on Phase 1 of Bridge No. 94-SR11-0.59 (southbound lane). Placing of the deck occurred on March 30, 2016.

During construction operations, issues (including weather) that limited the use and performance of the concrete were not identified. No loads of concrete were rejected. Issues that impacted concrete production and delivery were not noted by the contractor or producer. Delivery occurred on time with adequate truck spacing with minimal onsite adjustments to mixture. Concrete was placed and finished in an efficient manner. Contractor personnel did not identify any difficulties placing or finishing the mix design. Curing procedures were implemented in a timely manner.

Plastic Properties

Immediately prior to discharge of concrete into forms, the standard plastic properties were measured. Results from plastic testing are provided in Table 5-3. As shown, each of the plastic properties measured met specifications for Class D concrete. Slump was reported at 6.50 inches (8.00 inches or less) and air content was reported at 6.5% (4.5% to 7.5%).

Table 5-3: Plastic Properties – SF Field

Concrete Temperature (Deg F)	65
Ambient Temperature (Deg F)	45
Slump (inches)	6.50
Air Content (%)	6.5

Hardened Properties

Results from hardened testing are provided in Table 5-4. As shown, the SF Field mix design met (exceeded) the minimum compressive strength requirement (4,000 psi) beginning at 3-days of age (5,640 psi) with a compressive strength of 9,330 psi at 28-days of age. Compressive strength results are illustrated in Figures 5-7 and 5-8.

The 56-day permeability results included a RCPT measurement of 710 coulombs and a surface resistivity of 75.6 kohm-cm, each considered very low permeability. The 128-day test results indicated considerably reduced permeability with age. The 128-day results included a permeability of 253 coulombs and a surface resistivity of 108.3 kohm-cm, each considered very low permeability. Results from permeability testing are illustrated in Figure 5-9.

Table 5-4: Hardened Properties – SF Field

	Compressive Strength (psi)	Surface Resistivity (kohm-cm)	RCPT Permeability (coulombs)
1 Day	3,650	-----	-----
3 Day	5,640	-----	-----
7 Day	6,780	-----	-----
28 Day	9,330	-----	-----
56 Day	10,240	75.6 (very low)	710 (very low)
128 Day	10,900	108.3 (very low)	253 (very low)

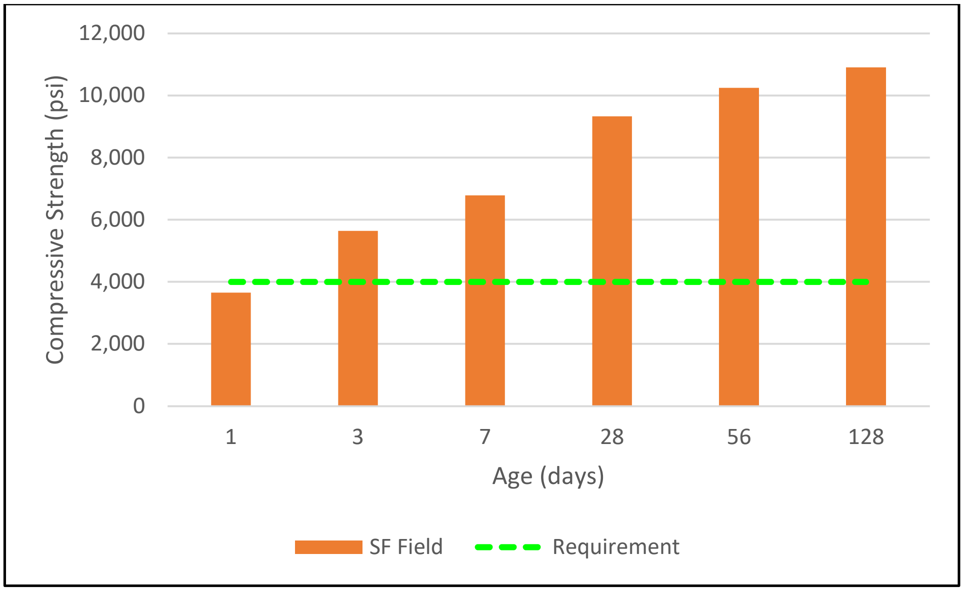

Figure 5-7: Compressive Strength – SF Field

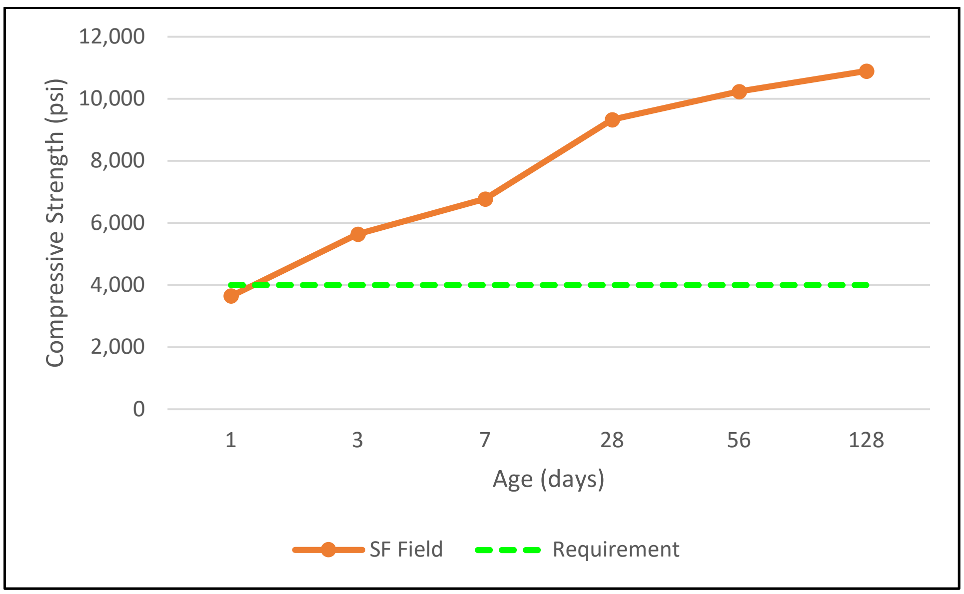

Figure 5-8: Compressive Strength – SF Field

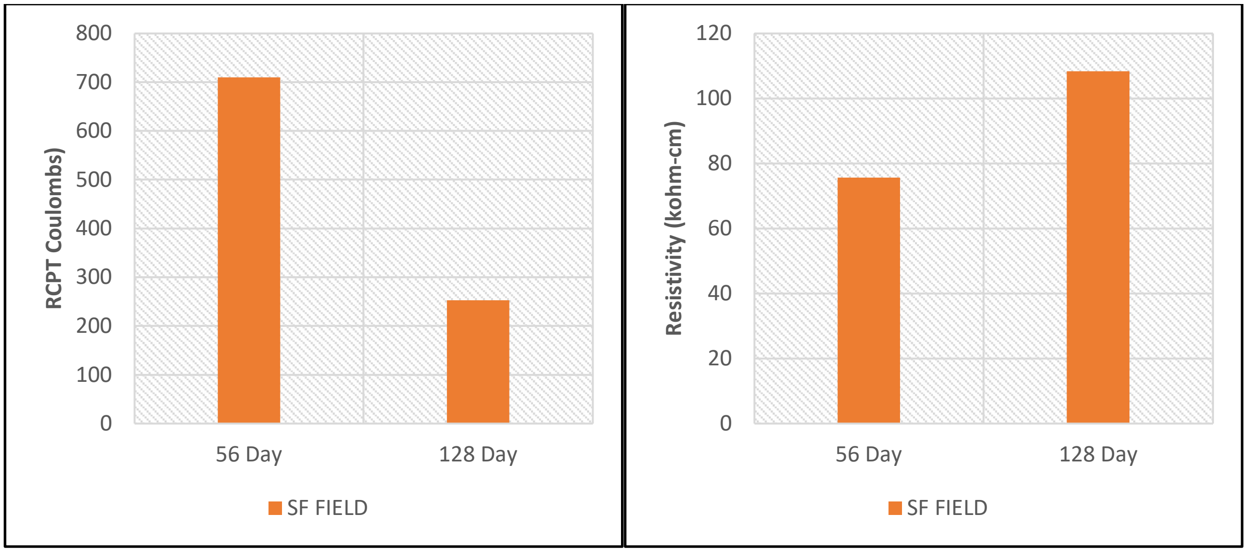

Figure 5-9: Permeability – SF Field

<u>Comparison with SF Lab</u>

Compressive strength performance of the field and lab mixes are provided in Figures 5-10 and 5-11. As shown, each mix met the minimum strength specifications. The field mix reached minimum strength at earlier age (3- versus 28-days). Results for 28-day strength were also considerably different with the field mix reaching 9,930 psi and the lab achieving 4,560 psi. Comparison of the results indicates the field mix considerably outperformed the lab mix with a faster gain of strength and compressive strengths approximately 40% higher. While results from small lab prepared and full-scale mixes are not expected to yield identical results, the magnitude of strength difference was unexpected.

Results from both permeability testing methods are provided in Figure 5-12 for the field and lab mixes. RCPT results at 56-days were 710 and 910 coulombs for the field and lab mixes, respectively. Similarly, the field outperformed the lab mix with surface resistivity results of 75.6 kohm-cm and 47.8 kohm-cm. Results from each test method and age were categorized as very low permeability. Comparison indicates that, much like compressive strength, the field mix outperformed the lab mix.

One factor resulting in the unexpected performance by the field mix was identified by the producer. Silica fume was discharged from bag form (typical delivery method) into the mixer truck. The producer indicated that from a position of safety, only whole bags were used thus increasing the amount of silica fume and cementitious materials in the mix. This extra cementitious material may have been a contributing factor in increased strength and reduced permeability.

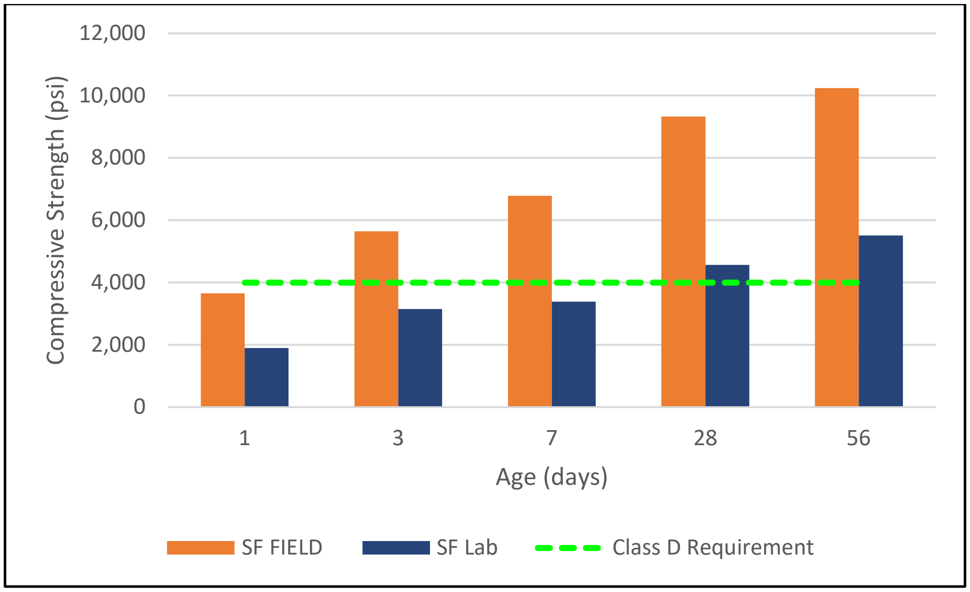

Figure 5-10: Compressive Strength – SF Field and SF Lab

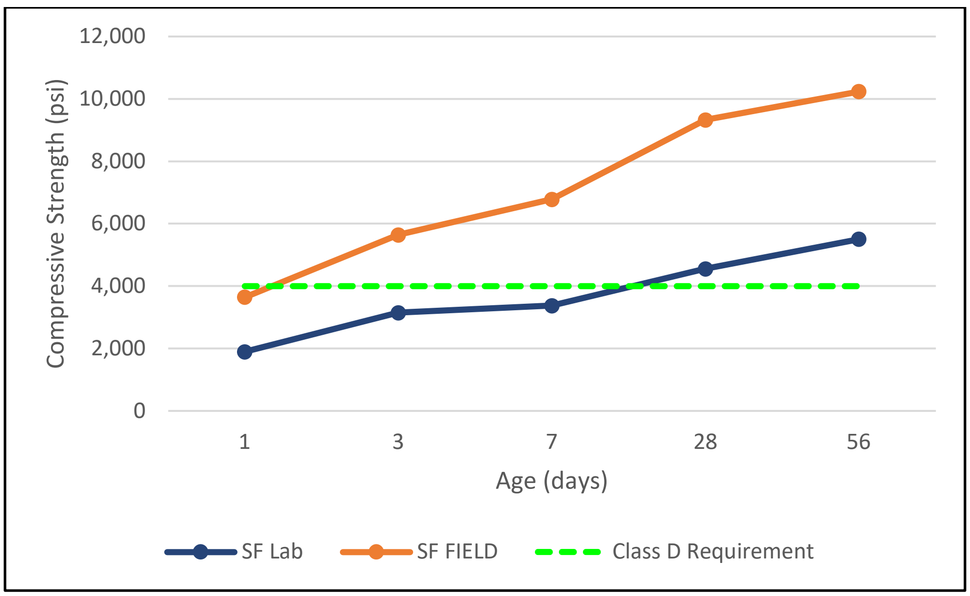

Figure 5-11: Compressive Strength – SF Field and SF Lab

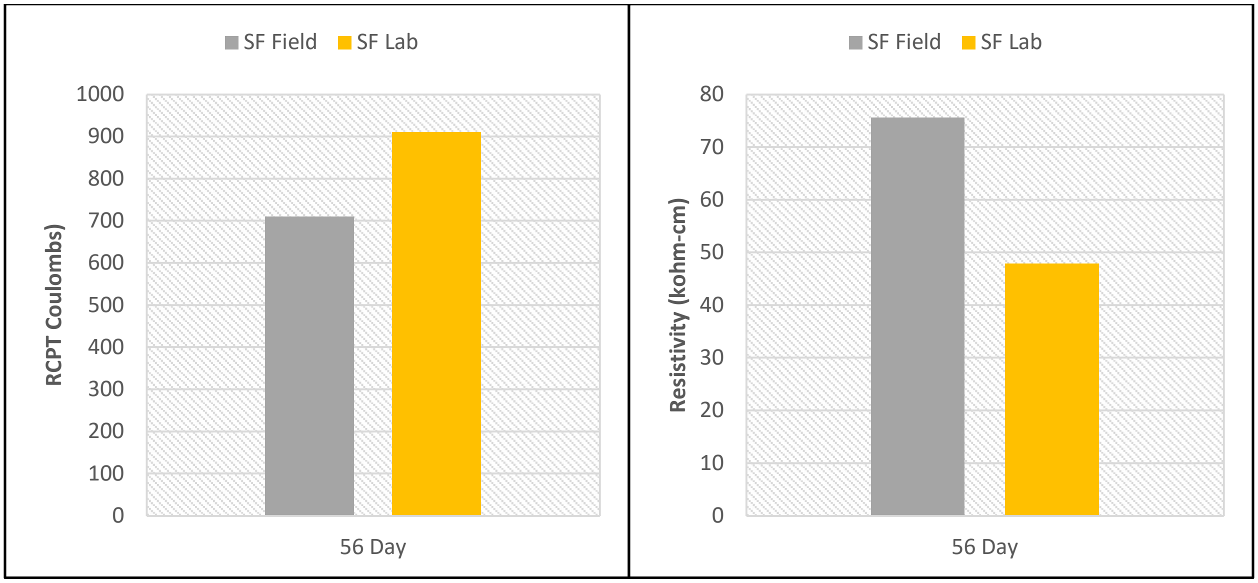

Figure 5-12: Permeability – SF Field and SF Lab

<u>Comparison with FA Field (control)</u>

Comparison of SF Field and FA Field compressive strength and permeability results indicated superior performance by the SF Field design. Compressive strength of the mix design was approximately 40 % greater than the control at each age tested and achieved specified strength 4 days faster. The 28-day strength for SF Field was 9,330 psi, approximately 40% greater than the control strength of 6,680 psi. Improved performance in compressive strength was expected as use of silica fume as a partial cement replacement typically results in improved compressive strength. Also, as previously discussed, the addition of additional silica fume contributed to the gap in compressive strength. Compressive strength results for both mixes are illustrated in Figures 5-13 and 5-14.

Permeability results for each mix are provided in Figure 5-15. Results at both 56- and 128-days indicated significantly improved performance by the SF Field as compared to the control. Results from each test method and age were categorized as very low for SF Field and moderate to very low for FA Field. Although categorization is similar, the numerical results indicated significant improvement by the SF Field design. Significant improvement was expected based on lab mix results and the addition of another SCM with smaller particle size.

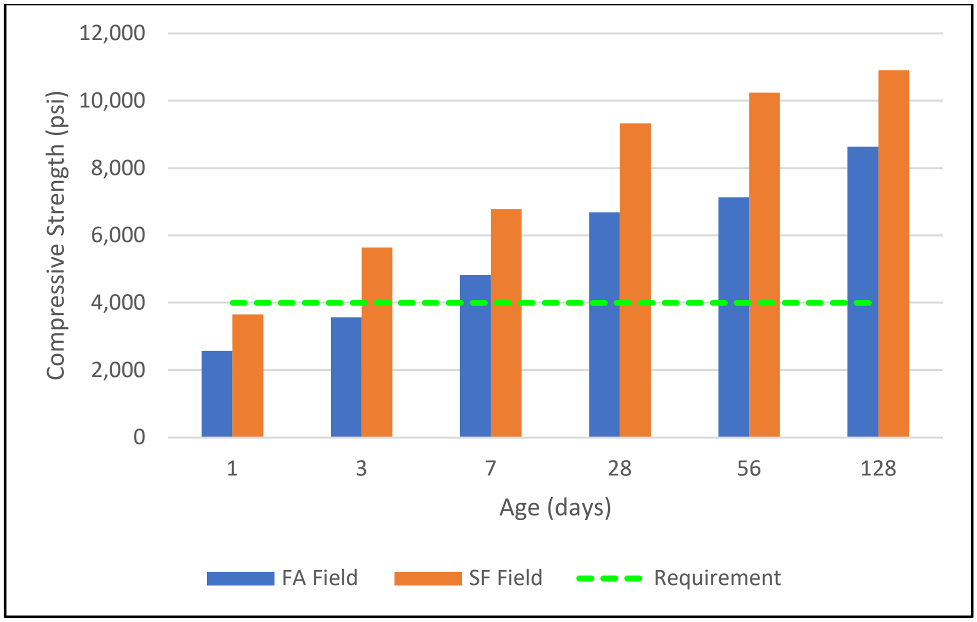

Figure 5-13: Compressive Strength – FA Field and SF Field

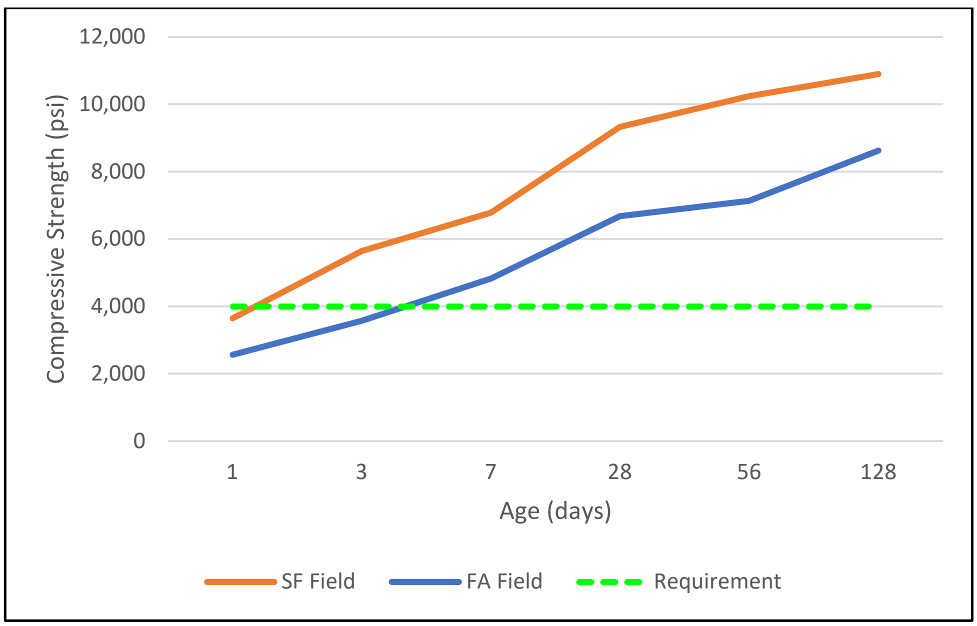

Figure 5-14: Compressive Strength – FA Field and SF Field

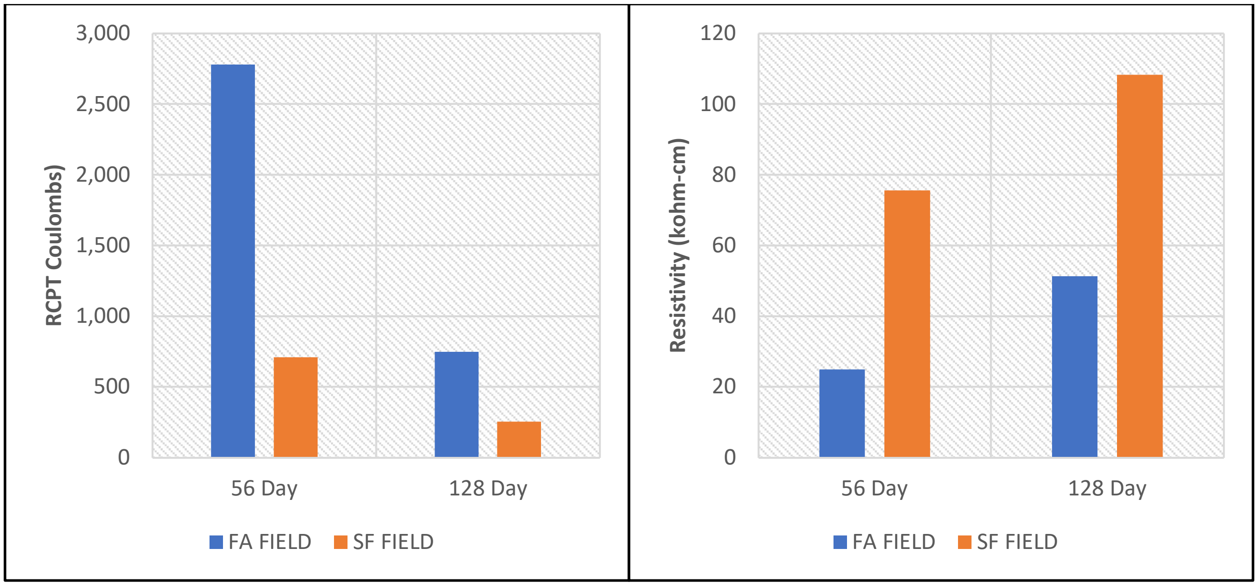

Figure 5-15: Permeability – FA Field and SF Field

<u>SL Field</u>

The deck on the second phase (northbound) of Bridge No. 94-SR11-0.59 was placed and finished on August 24, 2016. The Class D SL (SL Field) mix design was used for this phase of deck construction.

During construction operations, several issues were identified that impacted the use of the SL Field design. First, 5 loads of concrete were rejected due to plastic properties that did not meet specification upon arrival at the construction site. One load was rejected due to excessively high slump and 4 loads were rejected due to air content above the allowable range. The producer identified difficulties with the admixture dosage rates as the cause for plastic properties that were out of specification.

Second, a traffic accident occurred (unrelated to concrete mixer truck or construction equipment) during placement of concrete. The accident involved a single 18-wheel tractor trailer. The vehicle veered into the barrier wall at the edge of the bridge and became disabled on the bridge. Due to phased construction, this blocked the only lane of traffic across the structure resulting in a detour of all traffic, including the concrete delivery trucks. Due to the route between the concrete plant and the project, the delivery trucks were detoured thru a longer route to arrive at the end of bridge where the contractor was set up to receive concrete for placement using a crane and bucket. This caused a delay between successive trucks at the time of the incident and a longer time between batching and on-site arrival. This resulted in slower concrete placement along the bridge deck and greater time between adjacent placements resulting in concerns of cold joints on the deck.

All loads of concrete delivered after adjustments were made provided adequate plastic properties and were placed and finished in an efficient manner. Contractor personnel responsible for placing and finishing operations did not identify any concerns with the concrete mixture. The curing procedure used was delayed on this deck compared to the other phases of deck construction. Curing was not initiated in a timely manner, especially on the portions of deck that were placed with the first loads of concrete that arrived prior to delay. These delays in concrete arrival and beginning of curing may have negatively impacted deck properties including cracking and permeability.

Plastic Properties

Immediately prior to discharge of concrete, plastic the standard plastic properties were measured following any final additions of water or admixtures. Results from plastic testing are provided in Table 5-5. As shown, each of the plastic properties measured met specifications for Class D concrete. Slump was reported at 5.00 inches (8.00 inches or less) and air content was reported at 5.2% (4.5% to 7.5%).

Table 5-5: Plastic Properties – SL Field

Slump (inches)	5.00
Concrete Temperature (Deg F)	89
Ambient Temperature (Deg F)	87
Air Content (%)	5.2

Hardened Properties

Results from hardened property testing are provided in Table 5-6. The SL Field mix design met (exceeded) the minimum compressive strength requirement (4,000 psi) beginning at-3 days of age (4,630 psi) with a strength of 7,020 psi at 28-days. Compressive strength results are illustrated in Figures 5-16 and 5-17.

The 56-day results included a permeability of 975 coulombs and a surface resistivity of 45.5 kohm-cm, each considered very low. The 128-day test results indicated considerably reduced permeability with age. Including a permeability of 570 coulombs and a surface resistivity of 69.9 kohm-cm, each considered very low. Results are illustrated in Figure 5-18.

Table 5-6: Hardened Properties – SL Field

	Compressive Strength (psi)	Surface Resistivity (kohm-cm)	RCPT Permeability (coulombs)
1 Day	2,820	-----	-----
3 Day	4,630	-----	-----
7 Day	5,490	-----	-----
28 Day	7,020	-----	-----
56 Day	7,410	45.5 (very low)	975 (very low)
128 Day	7,740	69.9 (very low)	570 (very low)

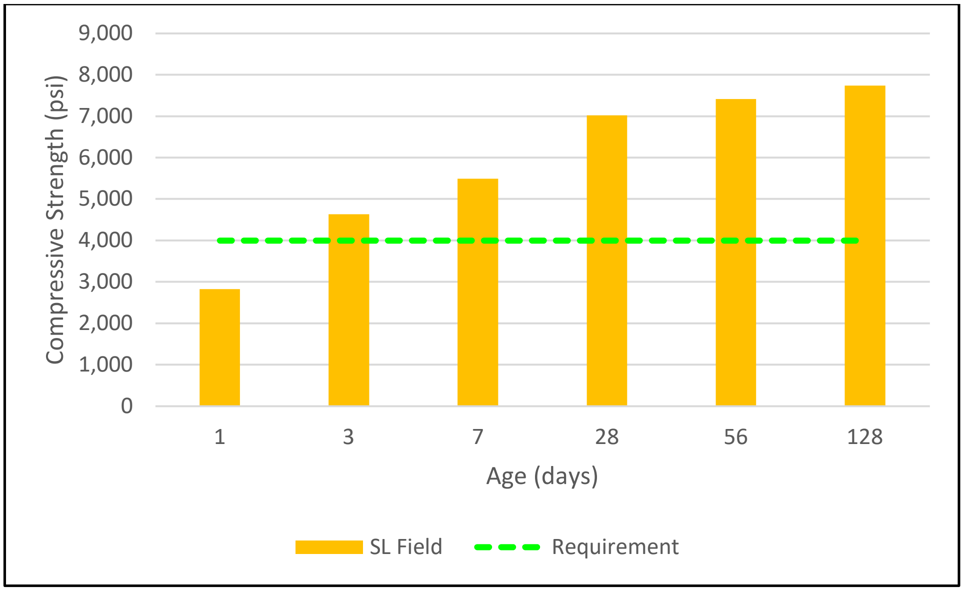

Figure 5-16: Compressive Strength – SL Field

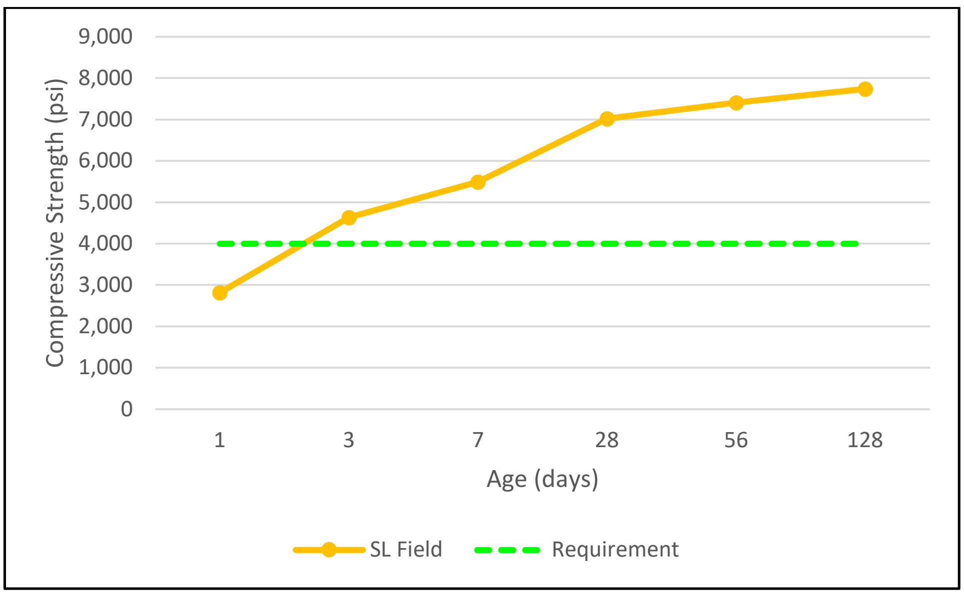

Figure 5-17: Compressive Strength – SL Field

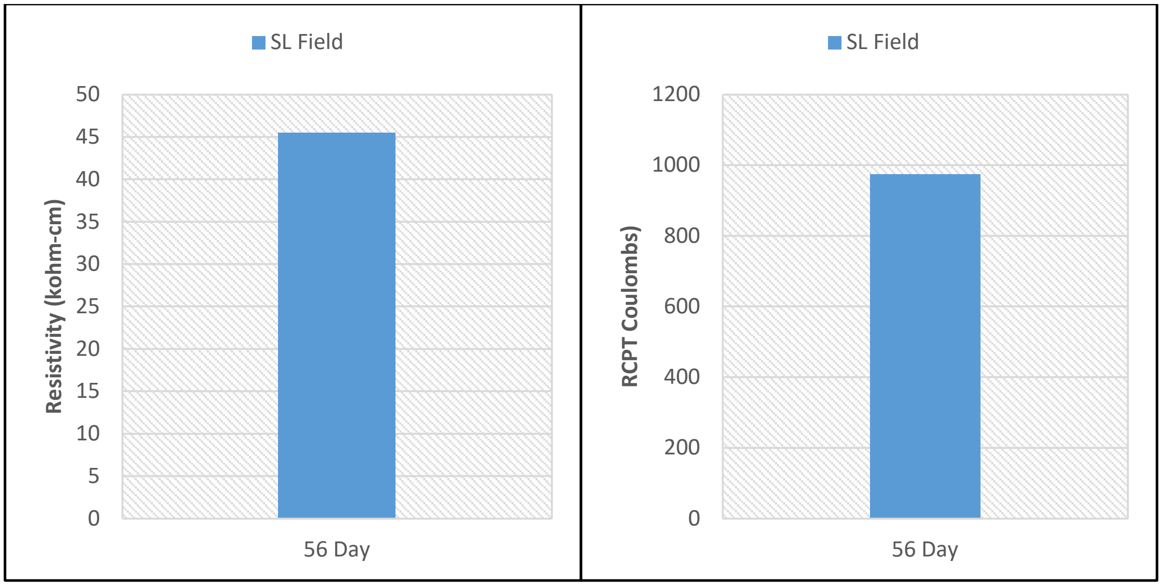

Figure 5-18: Permeability – SL Field

Comparison with SL Lab

Compressive strength performance of the SL Field and SL Lab mixes are provided in Figures 5-19 and 5-20. As shown, each mix met the minimum strength specifications. The field mix reached minimum strength at earlier age (3 versus 28 days). Results for 28-day strength were also modestly different with the field mix reaching 7,020 psi and the lab achieving 6,540 psi. Comparison of the results indicates the field mix considerably outperformed the lab mix at early ages and each mix achieved similar ultimate strengths.

Results from both permeability testing methods are provided in Figure 5-21 for the field and lab mixes. RCPT results at 56-days were 975 and 870 coulombs for the field and lab mixes, respectively, each classified as very low permeability. Similarly, surface resistivity results included 47.4 kohm-cm and 45.5 kohm-cm, each classified as very low permeability. Comparison indicates that, unlike compressive strength, the lab mix modestly outperformed the lab mix.

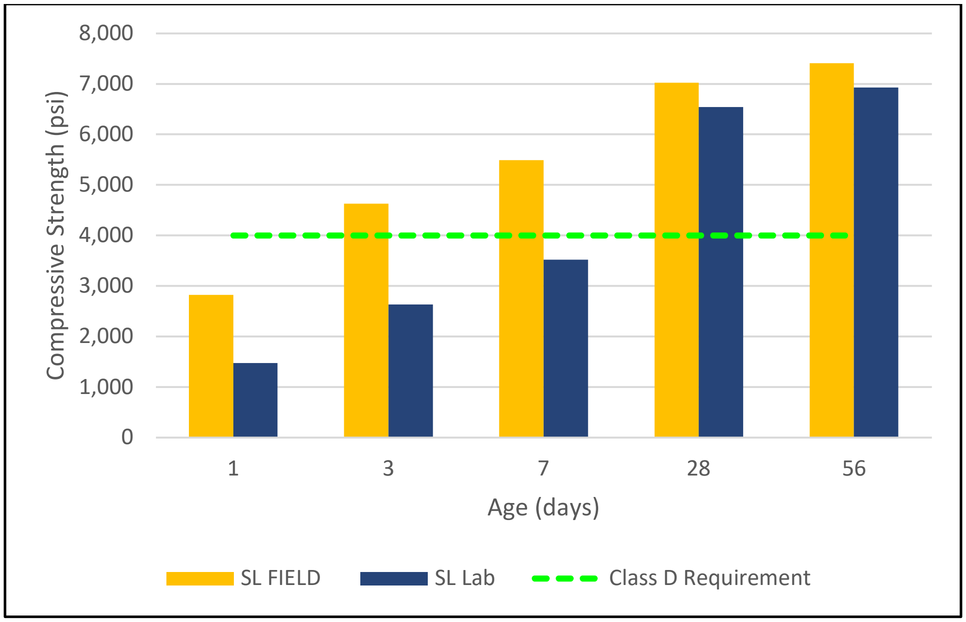

Figure 5-19: Compressive Strength – SL Field and SL Lab

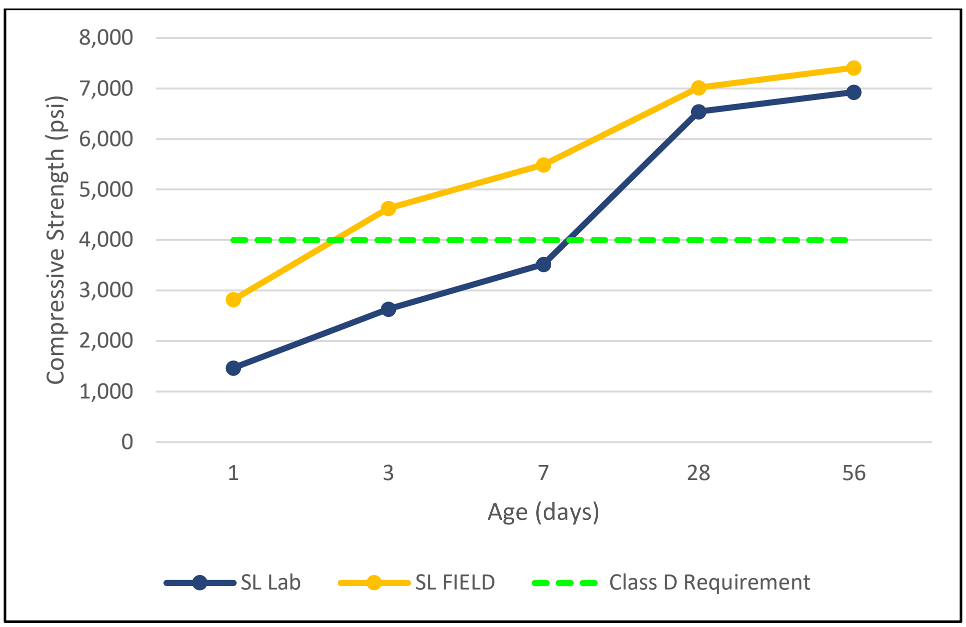

Figure 5-20: Compressive Strength – SL Field and SL Lab

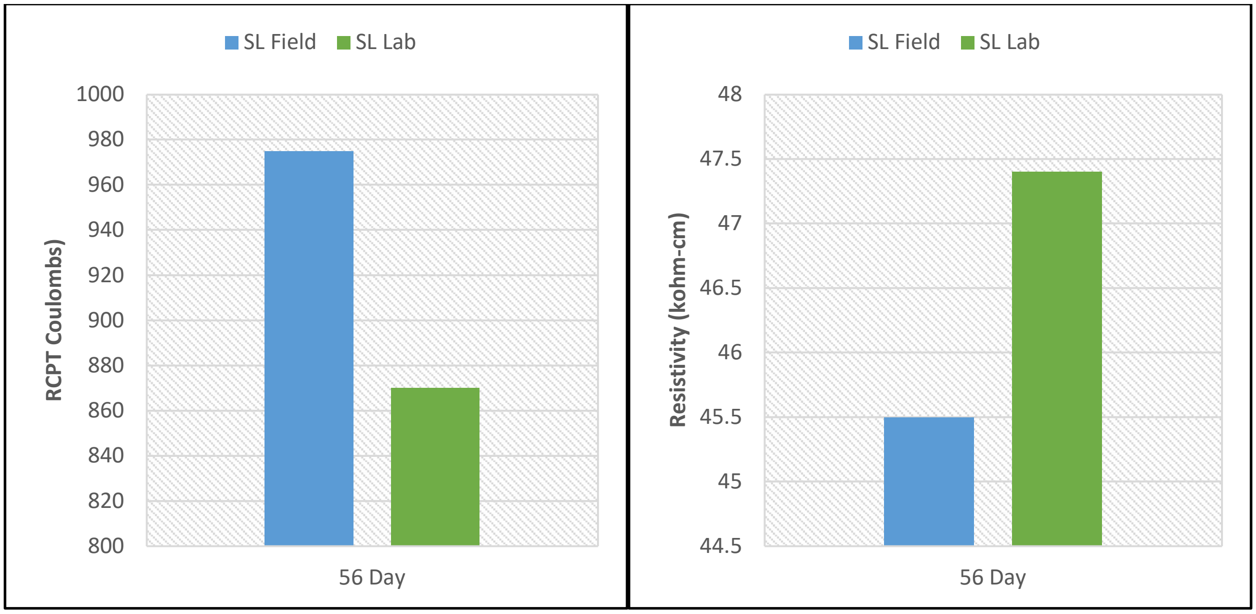

Figure 5-21: Permeability – SL Field and SL Lab

<u>Comparison with FA Field (control)</u>

Comparison of SL Field and FA Field compressive strength results indicated similar performance by both mix designs. Compressive strength of the SL Field mix achieved specified strength 4 days faster (3 versus 7 days). The 28-day strength for SL Field was 7,020 psi, approximately 5% greater than the control strength of 6,680 psi. Compressive strength results for both mixes are illustrated in Figures 5-22 and 5-23.

Permeability results for each mix are provided in Figure 5-24. Results at both 56- and 128-days indicated considerable improvement in performance by the SL Field design as compared to the control. Results from each test method and age were categorized as very low for SL Field and moderate to very low for Field FA.

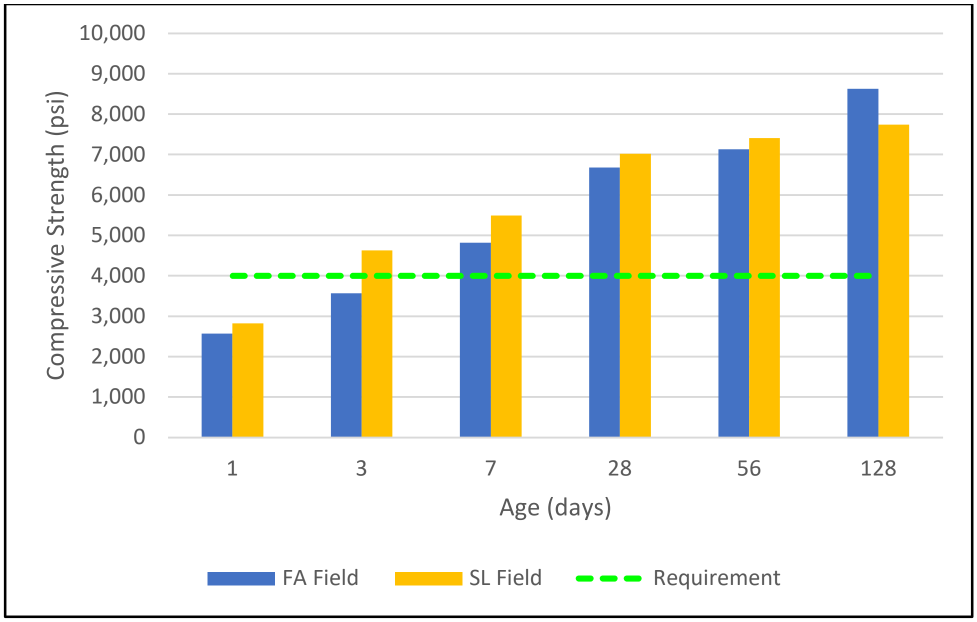

Figure 5-22: Compressive Strength – FA Field and SL Field

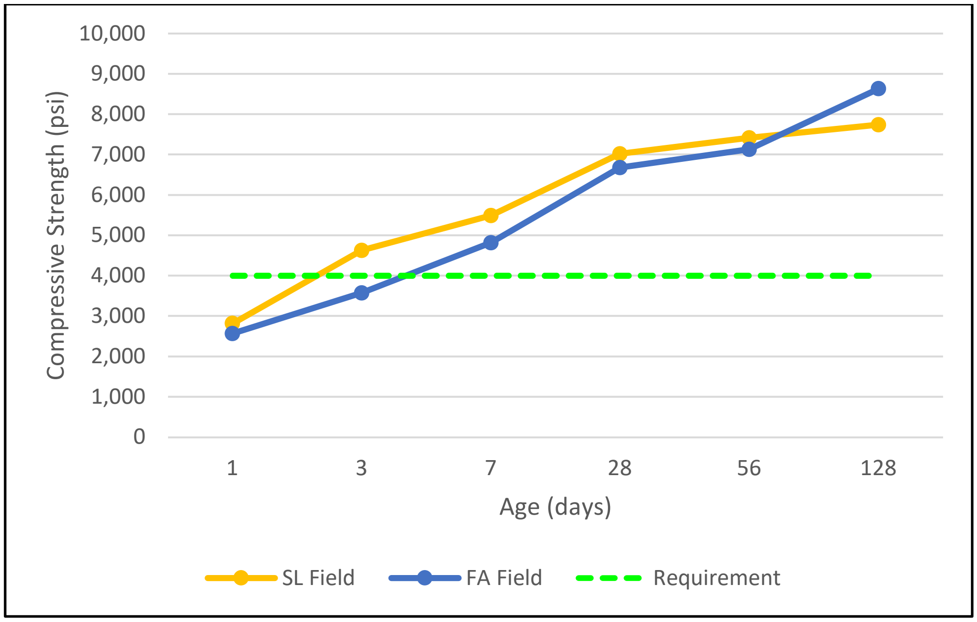

Figure 5-23: Compressive Strength – FA Field and SL Field

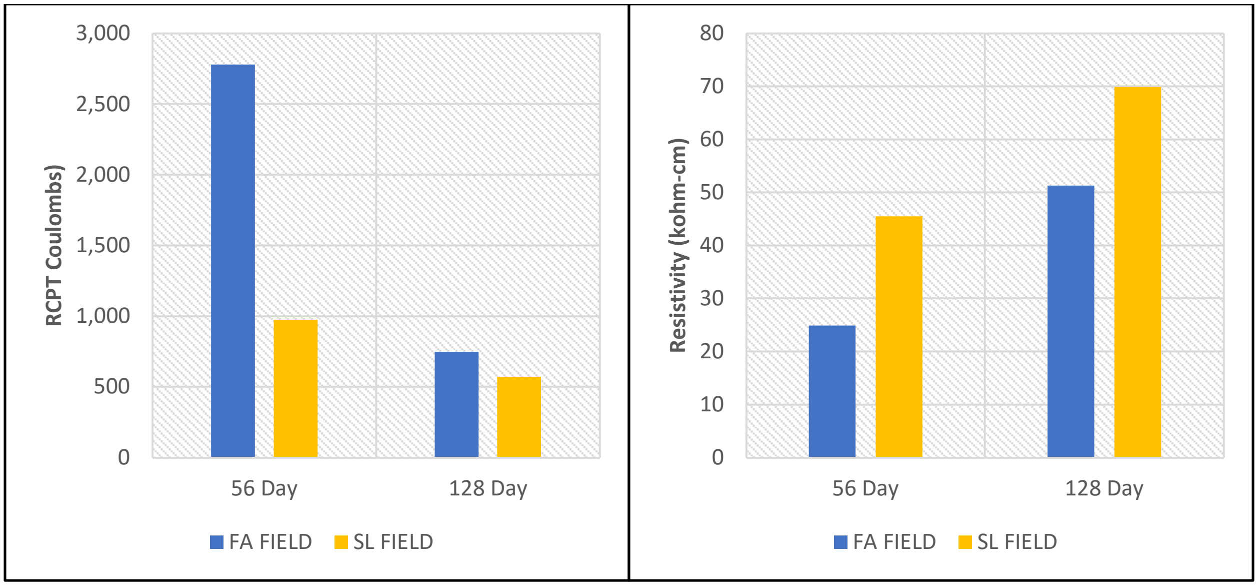

Figure 5-24: Permeability – FA Field and SL Field

MK Field

The Class DLP MK mix design (MK Field) was used on the second phase on Bridge No. 94-SR11-0.87 (northbound) This deck was placed and finished on August 25, 2016.

During construction operations, issues (including weather) that limited the use and performance of the concrete were not identified. No loads of concrete were rejected. Issues that impacted concrete production and delivery were not noted by the contractor or producer. Delivery occurred on time with adequate truck spacing with minimal onsite adjustments to mixture. Concrete was placed and finished in an efficient manner. Contractor personnel responsible for placing and finishing operations did not identify any concerns with the concrete mixture. Curing procedures were implemented in a timely manner.

Plastic Properties

Immediately prior to discharge of concrete, plastic properties were measured following any final additions of water or admixtures. Results from plastic testing are provided in Table 5-7. As shown, each of the plastic properties measured met specifications for Class D concrete. Slump was reported at 4.50 in. (8.00 in. or less) and air content was reported at 5.7% (4.5% to 7.5%).

Table 5-7: Plastic Properties – MK Field

Slump (inches)	4.50
Concrete Temperature (Deg F)	84
Ambient Temperature (Deg F)	78
Air Content (%)	5.7

Hardened Properties

Results from hardened property testing are provided in Table 5-8. As shown, the MK Field mix design met (exceeded) the minimum compressive strength requirement (4,000 psi) beginning at 3 days of age (4,390 psi) with a compressive strength of 5,590 psi at 28 days of age. Compressive strength results are illustrated in Figures 5-25 and 5-26.

Permeability results are illustrated in Figure 5-27. Permeability results at 56-days included 1,880 coulombs (RCPT) and 20.3 kohm-cm (surface resistivity), classified as low and moderate permeability. Like other mix designs, 128-day testing resulted in considerably lower assessments of permeability including 1,167 coulombs (RCPT) and 37.2 kohm-cm (surface resistivity), resulting in classification as low and very low permeability, respectively.

.

Table 5-8: Hardened Properties – MK Field

	Compressive Strength (psi)	Surface Resistivity (kohm-cm)	RCPT Permeability (coulombs)
1 Day	3,540	-----	-----
3 Day	4,390	-----	-----
7 Day	4,940	-----	-----
28 Day	5,590	-----	-----
56 Day	6,190	20.3 (moderate)	1,880 (low)
128 Day	6,960	37.2 (very low)	1,167 (low)

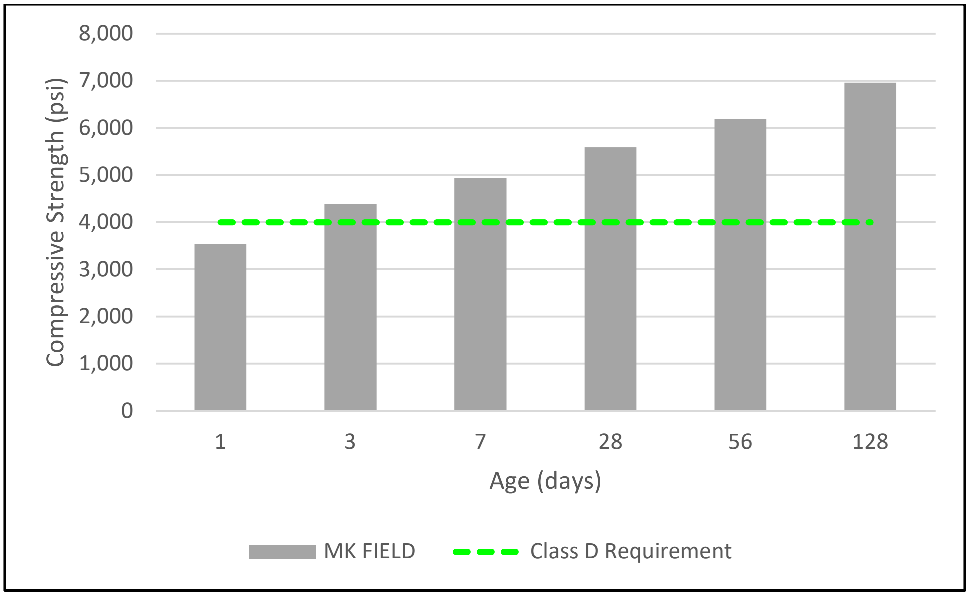

Figure 5-25: Compressive Strength – MK Field

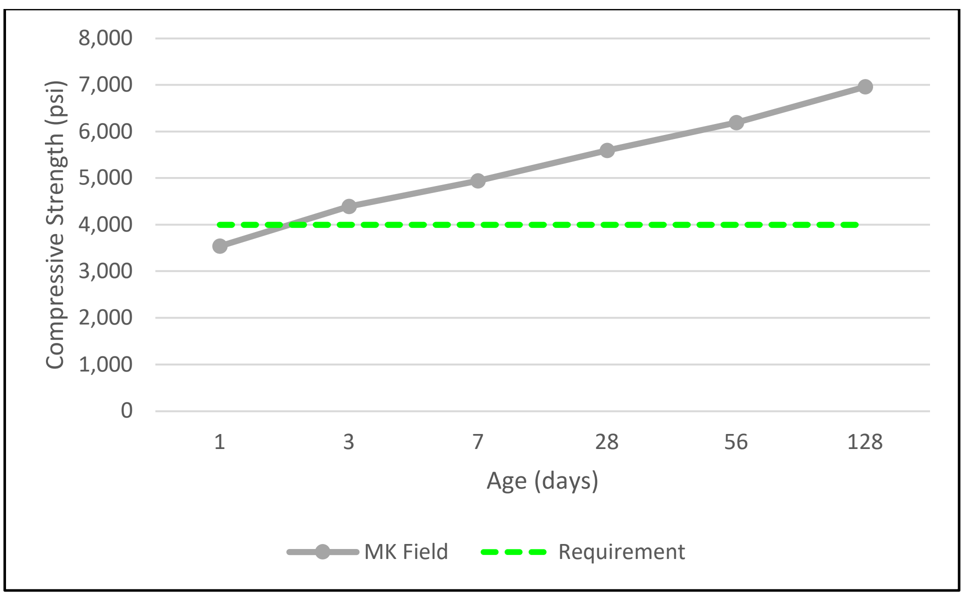

Figure 5-26: Compressive Strength – MK Field

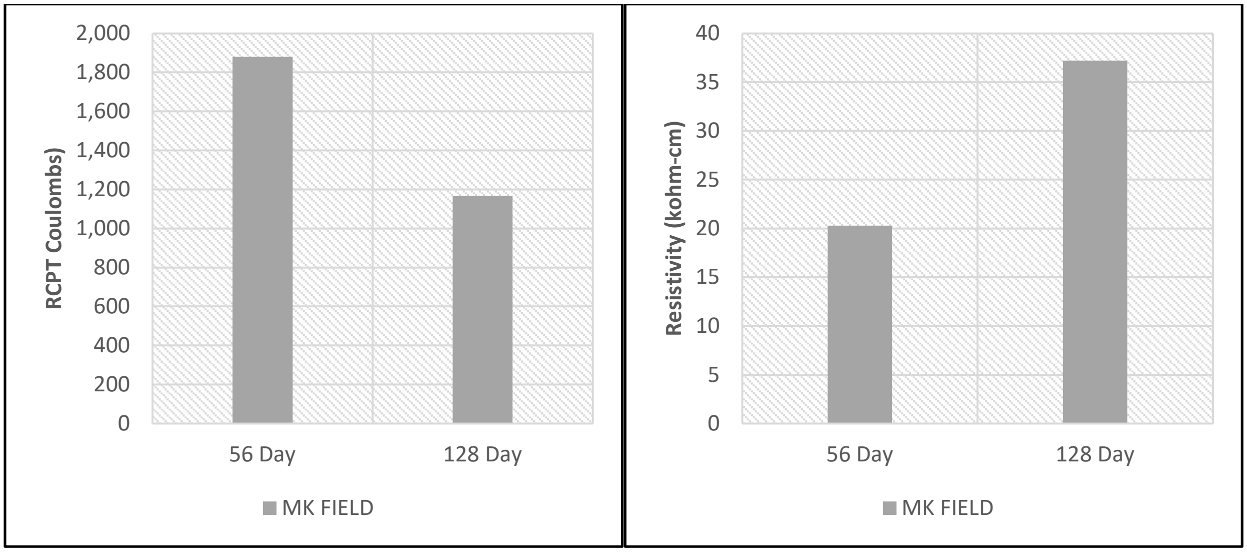

Figure 5-27: Permeability – MK Field

<u>Comparison with MK Lab</u>

Compressive strength performance of the MK Field and MK Lab mixes are provided in Figures 5-28 and 5-29. As shown, each mix met the minimum strength specifications. The field mix reached minimum strength at earlier age (3 versus 28 days). Results for 28-day strength were noticeably different with the field mix reaching 5,590 psi and the lab achieving 4,700 psi. Comparison of the results indicates the field mix outperformed the lab mix with a faster, more uniform, gain of strength and compressive strengths averaging 35% higher.

Results from both permeability testing methods are provided in Figure 5-30 for the field and lab mixes. RCPT results at 56-days were 1,880 (low permeability) and 961 (very low permeability) coulombs for the field and lab mixes, respectively. Similarly, the lab outperformed the field mix with surface resistivity results of 46.9 kohm-cm and 20.3 kohm-cm, rated as very low and moderate permeability. Comparison indicates that, unlike compressive strength, the lab mix considerably outperformed the field mix.

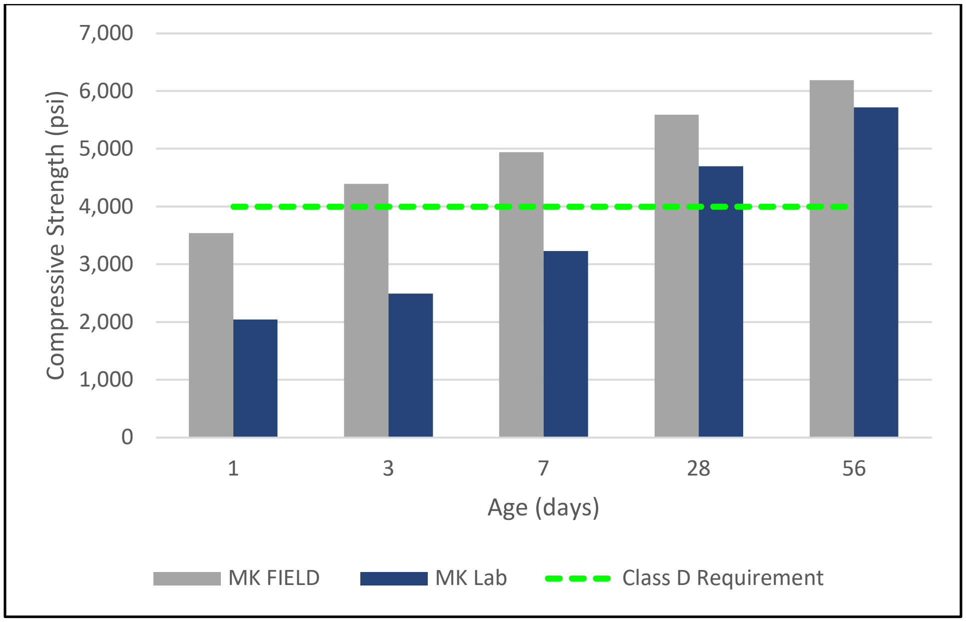

Figure 5-28: Compressive Strength – MK Field and MK Lab

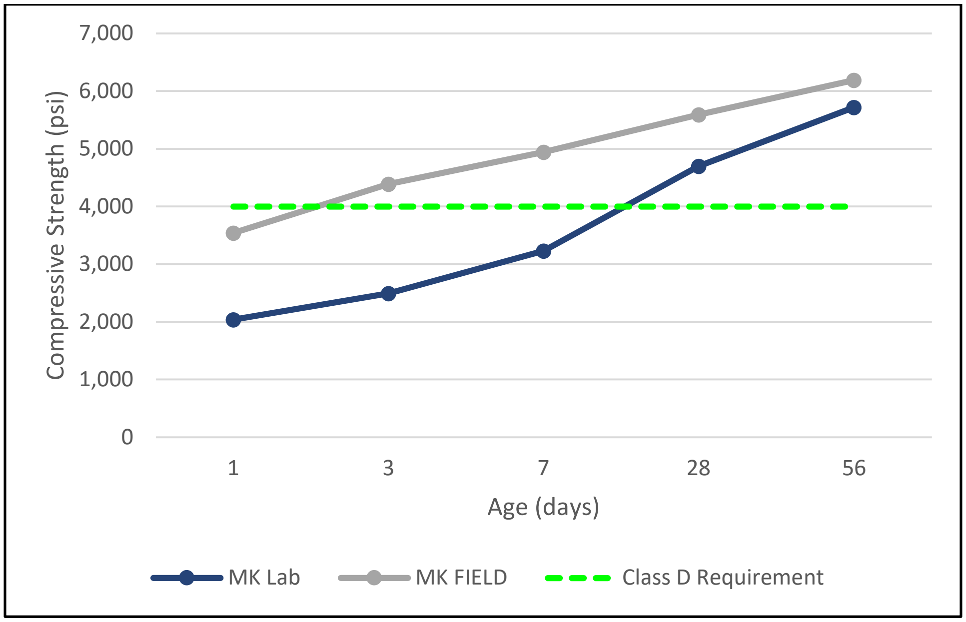

Figure 5-29: Compressive Strength – MK Field and MK Lab

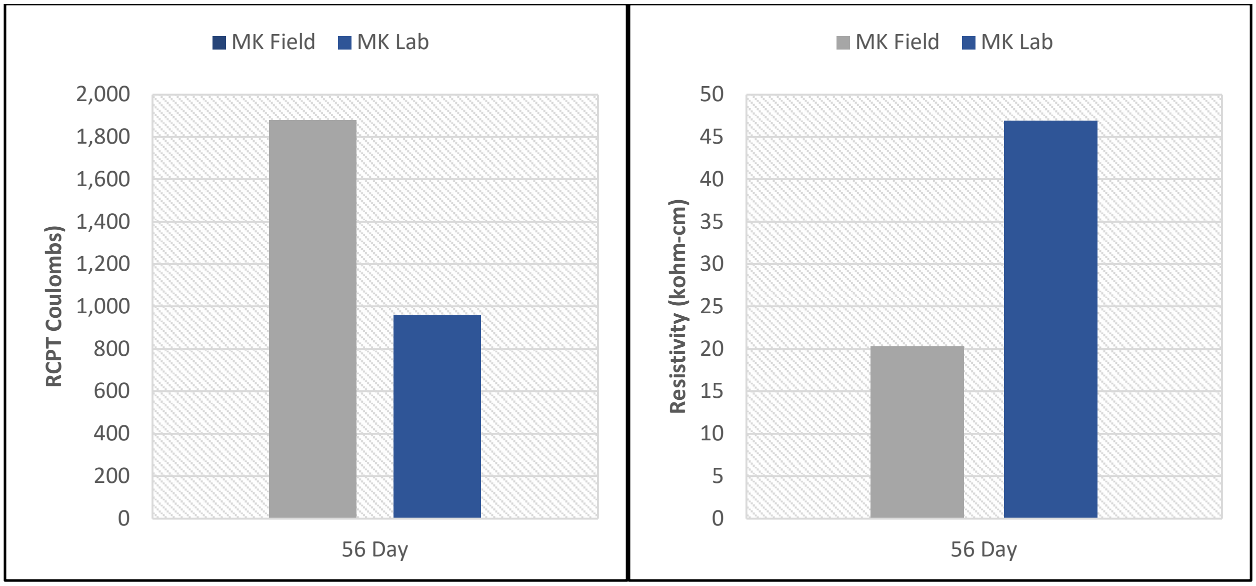

Figure 5-30: Permeability – MK Field and MK Lab

<u>Comparison with FA Field (control)</u>

Results from testing of the MK Field design were compared to those of the control design to determine if improved performance was achieved. Figures 5-31 and 5-32 illustrate compressive strength results for each mix design. As shown, MK Field slightly outperformed the control mix design at ages 1 thru 7 days and underperformed the control at ages 28 thru 128 days. However, the MK Field design considerably exceeded the compressive strength requirement for Class D concrete.

Figure 5-33 illustrates the relative performance of the MK Field and control when considering permeability and surface resistivity. As shown in Figure 5-33, results for permeability indicate the MK Field design lagged in performance at 56- and 128-days when considering surface resistivity and at 128-days when considering RCPT results. Overall, results indicate the FA Field design outperformed the MK Field design.

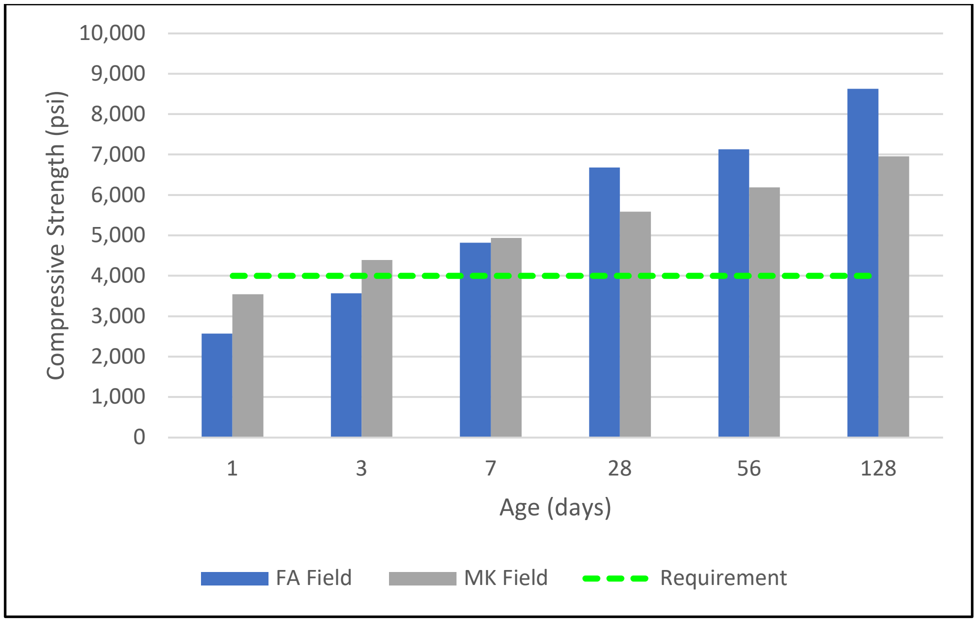

Figure 5-31: Compressive Strength – FA Field and MK Field

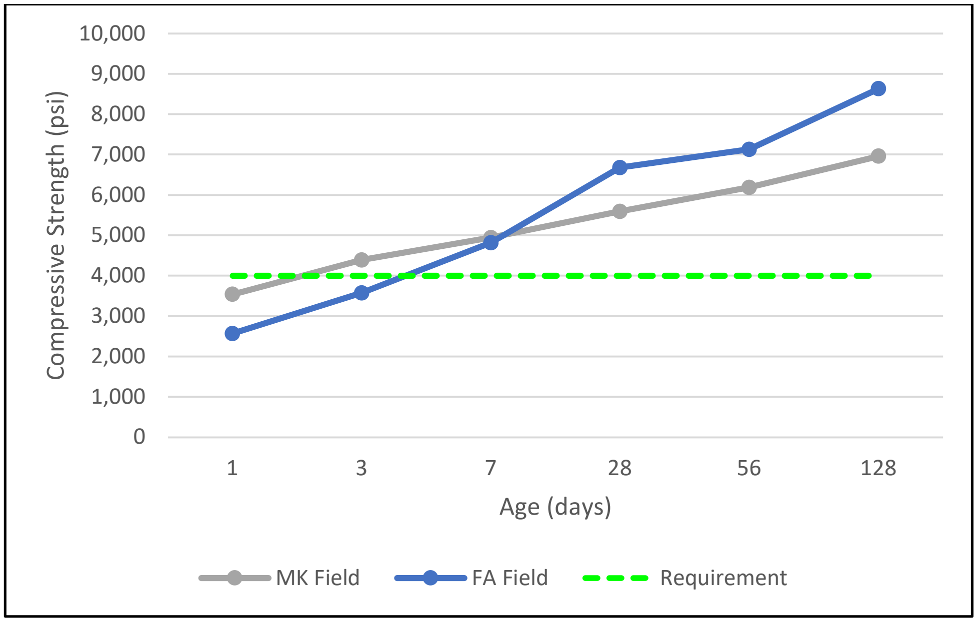

Figure 5-32: Compressive Strength – FA Field and MK Field

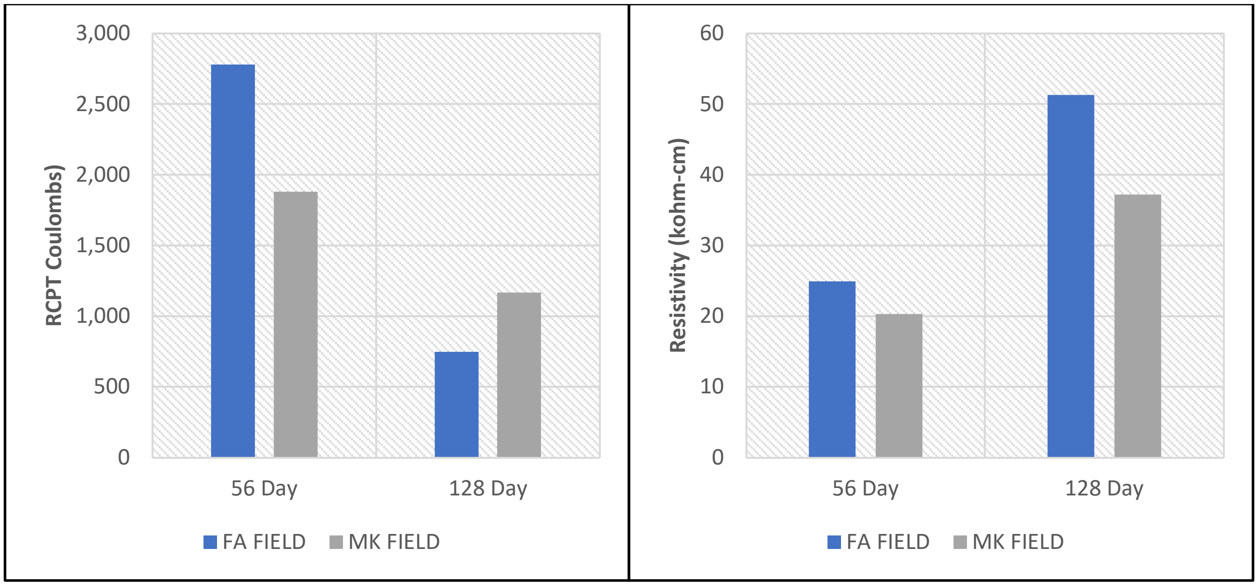

Figure 5-33: Permeability – FA Field and MK Field

<u>Field Mix Comparison</u>

Plastic properties for all field mixes are provided in Table 5-9 and illustrated in Figures 5-34 and 5-35. As shown, all plastic properties (after field adjustments) met required specification limits. As previously discussed, only the SL Field design presented issues in achieving plastic properties. All other designs met requirements upon arrival at the construction site or needed only minor adjustments. The producer indicated that after admixture dosage rates were finalized, appropriate plastic properties could be achieved by all low permeability mixes.

Table 5-9: Plastic Properties – All Mix Designs Field

	FA Field	SF Field	SL Field	MK Field
Concrete Temperature (Deg F)	62	65	89	84
Ambient Temperature (Deg F)	39	45	87	78
Slump (inches)	6.00	6.50	5.00	4.50
Air Content (%)	6.5	6.5	5.2	5.7

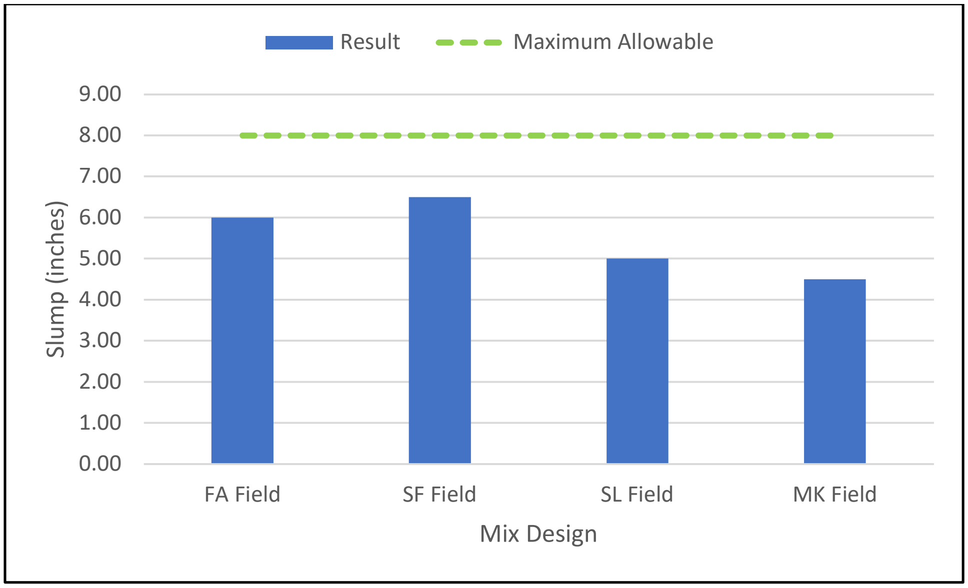

Figure 5-34: Slump – All Mix Designs Field

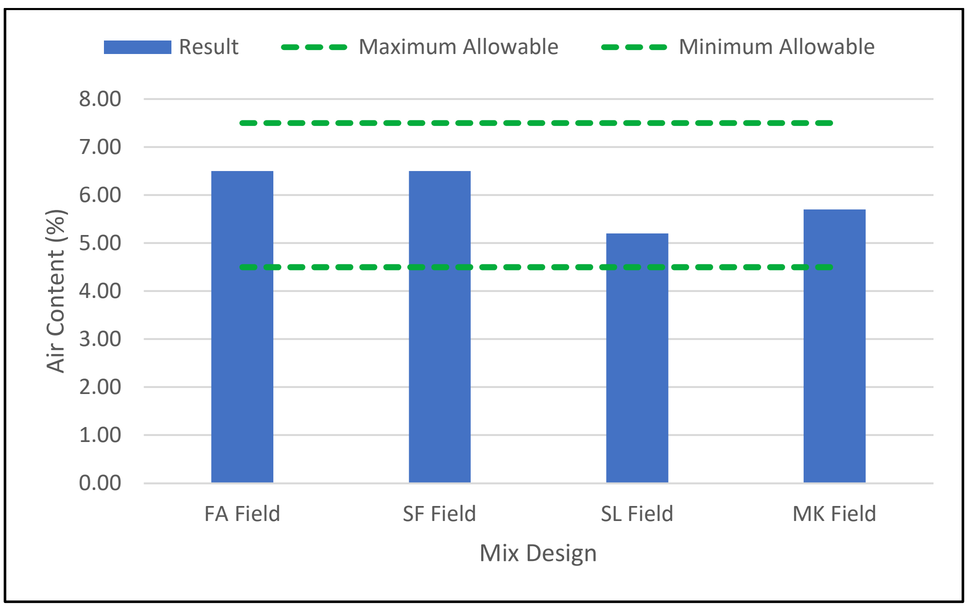

Figure 5-35: Air Content – All Mix Designs Field

Hardened properties for each of the field mixes are provided in Table 5-10 and illustrated in Figures 5-36 and 5-37. As shown, all mix designs significantly exceeded minimum strength requirements. Also, all low permeability mix designs reached the design strength by 3-days and the control design reached design strength at 7-days. While the low permeability designs achieved higher early strengths (1- to 7- days), the compressive strength of the control was higher than the MK design and similar to the SL design at later ages (28- to 128-days). The SF design significantly outperformed all other field mixes at every age. Ultimately, each of the designs used resulted in compressive strength considerably greater than required.

Figure 5-38 illustrates the RCPT results for all phases at ages of 56- and 128-days. RCPT results varied greatly at each age across all designs. Results of 56-day testing indicated that each of the low permeability designs achieved improvement when compared to the control. Specifically, the SF Field and SL Field designs provided significant improvement when compared to the control. Results from 128-day testing indicated significant improvement across all designs. The SF and SL designs continued to outperform the control whereas the MK design lagged the control. Again, the SF design outperformed all other designs.

Surface resistivity results for all mix designs are provided in Figure 5-39 for 56- and 128-day testing. Considerable variability in results was evident across all mix designs. Noticeable improvement was again evident in the 128-day results. Two of the low permeability designs (SF and SL) outperformed the control at each age while the MK design underperformed the control at each age. Like RCPT results, the SF design significantly outperformed all other designs, resulting in noticeably reduced permeability.

Table 5-10: Hardened Properties – All Mix Designs Field

	FA Field	SF Field	SL Field	MK Field
Compressive Strength (psi)				
1 Day	2,570	3,650	2,820	3,540
3 Day	3,570	5,640	4,630	4,390
7 Day	4,820	6,780	5,490	4,940
28 Day	6,680	9,330	7,020	5,590
56 Day	7,130	10,240	7,410	6,190
128 Day	8,630	10,900	7,740	6,960
RCPT Permeability (coulombs)				
56 Day	2,780 (moderate)	710 (very low)	975 (very low)	1,880 (low)
128 Day	747 (very low)	253 (very low)	570 (very low)	1,167 (low)
Surface Resistivity (kohm-cm)				
56 Day	24.9 (low)	75.6 (very low)	45.5 (very low)	20.3 (moderate)
128 Day	51.3 (very low)	108.3 (very low)	69.9 (very low)	37.2 (very low)

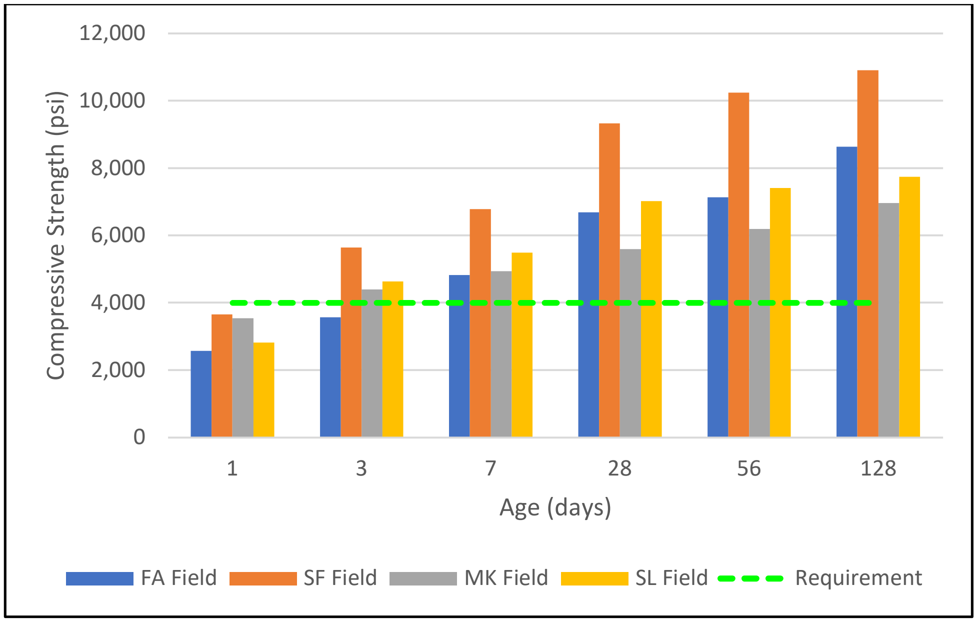

Figure 5-36: Compressive Strength – All Mix Designs Field

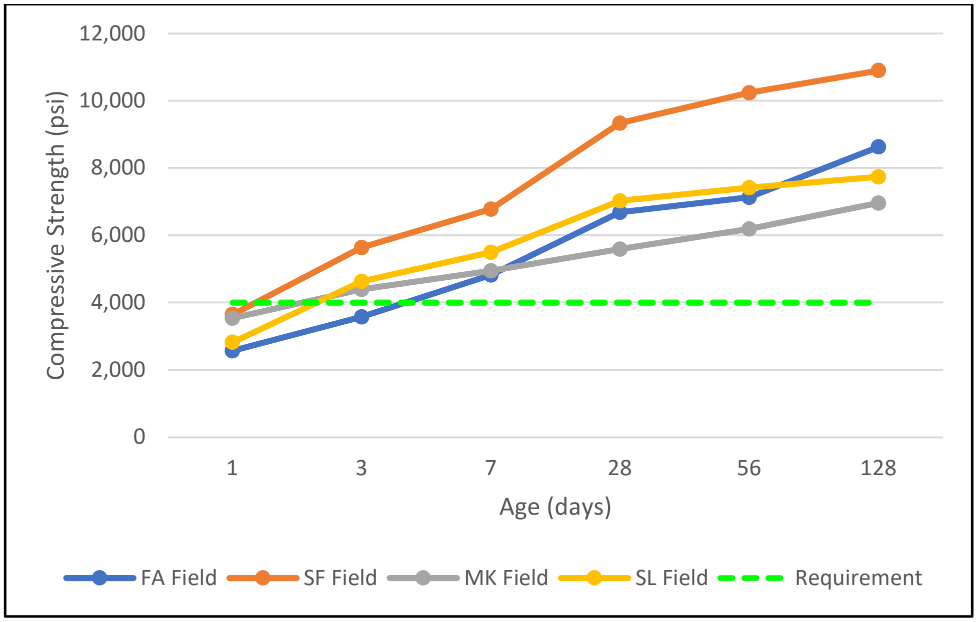

Figure 5-37: Compressive Strength – All Mix Designs Field

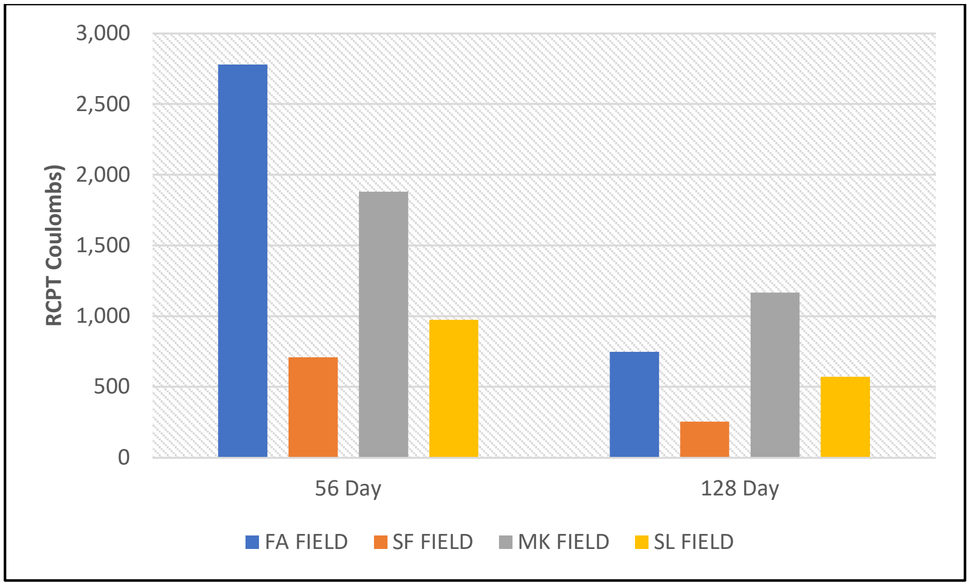

Figure 5-38: Permeability (RCPT) – All Mix Designs Field

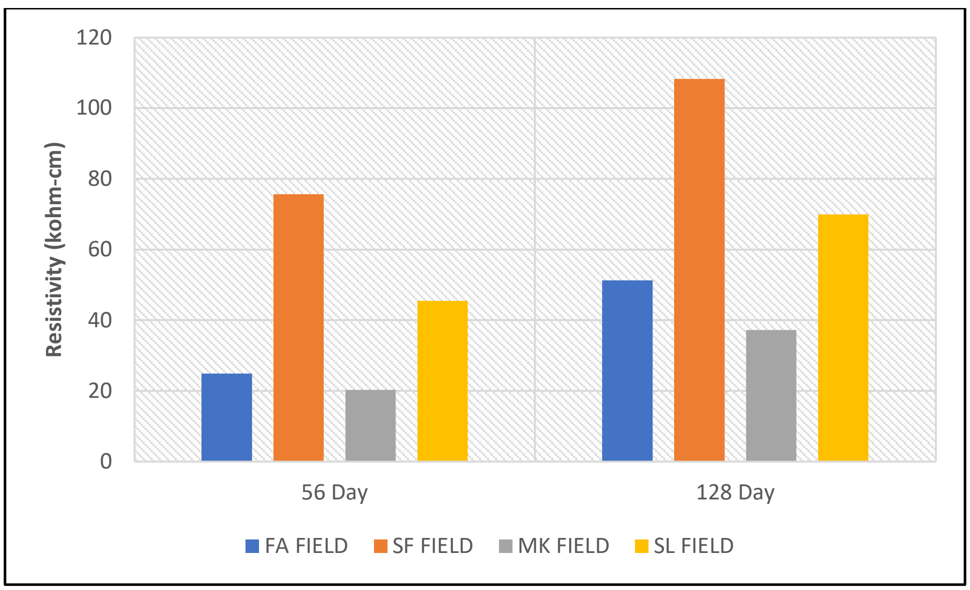

Figure 5-39: Permeability (Surface Resistivity) – All Mix Designs Field

Other Hardened Testing

After construction was complete, and each structure was in service, cores were taken from each phase of bridge deck construction to evaluate permeability through RCPT testing. Surface resistivity testing was also completed for comparison and evaluation of the two methods. Testing was completed to evaluate the in-place permeability of the concrete as placed and cured in the field and after in-service exposure to traffic and environmental conditions.

Locations for cores were chosen away from superstructure elements to avoid damage and at locations without any cracks or deformities that might interfere with testing (RCPT or resistivity). Prior to coring, reinforcing steel was approximately located to avoid unnecessary damage to the deck and prevent cores with reinforcing that would significantly alter RCPT testing results. Coring operations were completed by TDOT personnel. Immediately after coring, surface resistivity was measured adjacent to each core location. After removal of all cores, the remaining voids were repaired by TDOT personnel. Illustration of the coring and testing operations are provided in Figures 5-40 thru 5-42.

Results from each test method are provided in Table 5-11. Analysis of results indicate that the SF Field design significantly outperformed all other mix designs with results of 465 Coulombs (very low) and 91.9 kohm-cm (very low). The SL Field results were similar to the control with RCPT results of 1,040 coulombs (low) and 1,100 Coulombs (low) and surface resistivity results of 53.4 kohm-cm and 41.3 kohm-cm, each classified as very low permeability. The MK mix considerably underperformed all other mixes with a RCPT result of 2,475 coulombs (moderate) and surface resistivity result of 24.1 kohm-cm (moderate). Like results from cast test specimens, the SF Field design significantly outperformed all other designs and resulted in significant reductions in permeability.

Figure 5-40: Locating Reinforcing Steel

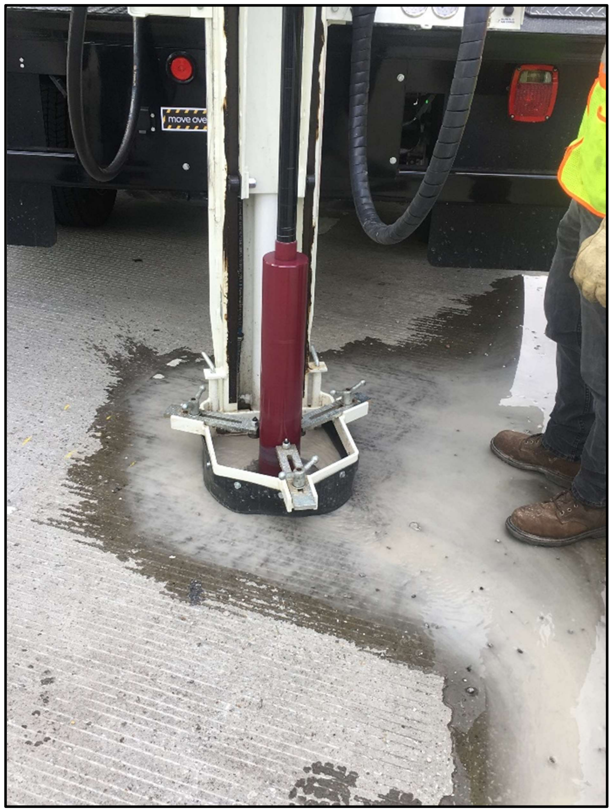

Figure 5-41: Coring Operations

Figure 5-42: Surface Resistivity Testing

Table 5-11: Permeability Results – Core and Deck Testing

	RCPT (coulombs)	Surface Resistivity (kohm-cm)
FA	1,100 (low)	41.3 (very low)
SF	465 (very low)	91.9 (very low)
MK	2,475 (moderate)	24.1 (low)
SL	1,040 (low)	53.4 (very low)

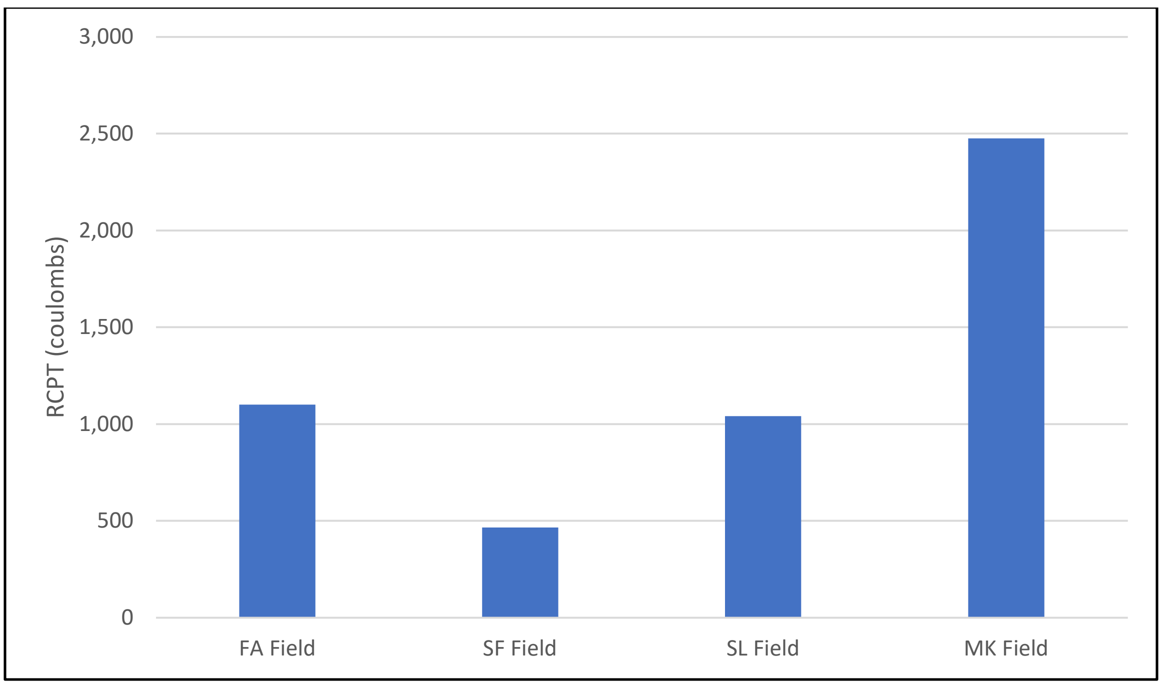

Figure 5-43: Permeability (RCPT) – Field Cores

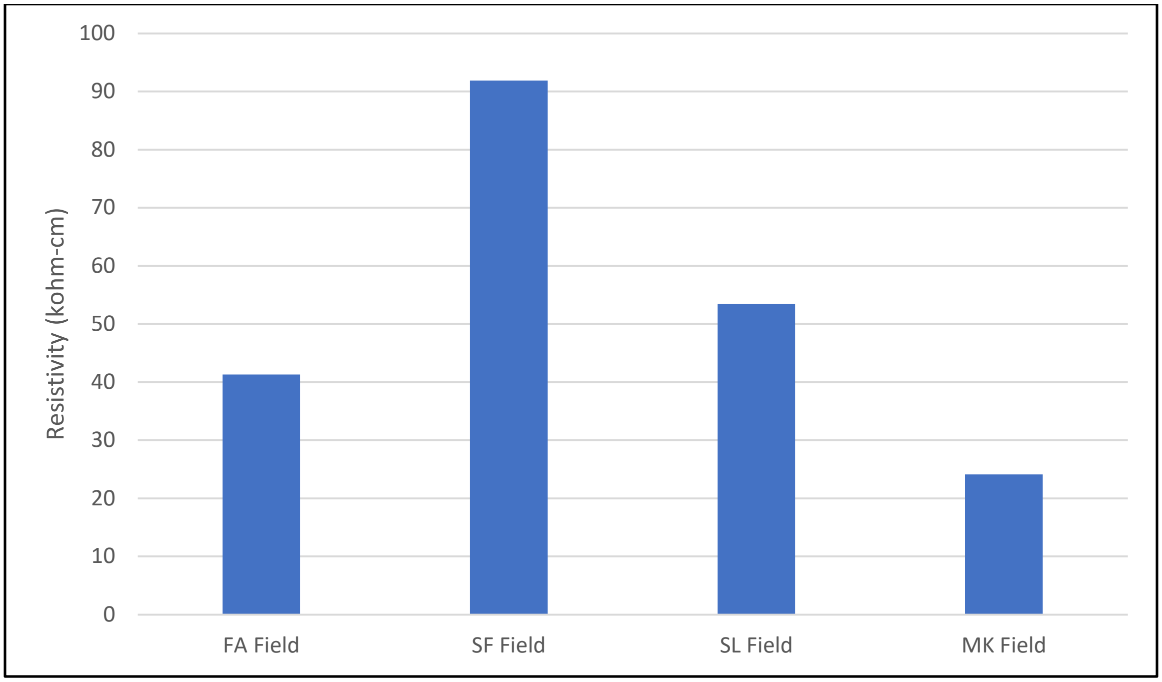

Figure 5-44: Permeability (Surface Resistivity) – Field Testing

SECTION 6: PERMEABILITY TEST METHOD COMPARISON

Results from all permeability testing were analyzed (using correlation and regression analysis) to determine if the test methods correlated or resulted in similar assessment of permeability. The analysis was also completed to identify any differences in correlation related to concrete age at testing, curing method, small versus large batch production, sample type (cylinder, core, and in-place testing), and in-service versus lab conditions. Comparison of the results from each test method was also used to better understand the viability of the surface resistivity method for in-place testing in future applications.

Permeability results for all mix designs, sample types, and testing age are provided in Table 6-1. For the purposes of analysis, 6 different groups of results were identified and are provided in Table 6-2.

Table 6-1: Consolidated Permeability Results

	Lab Mixes - Lab Cured Specimens				
	PC Lab	FA Lab	SF Lab	SL Lab	MK Lab
56 day - RCPT	2,230	1,840	910	961	870
56 day - Resistivity	16.1	22.6	47.8	46.9	47.4
	Field Mixes - Lab Cured Specimens				
	FA Field	SF Field	SL Field	MK Field	
56 day - RCPT	2,780	710	975	1,880	
56 day - Resistivity	24.9	75.6	45.5	20.3	
128 day - RCPT	747	253	570	1,167	
128 day - Resistivity	51.3	108.3	69.9	37.2	
	Field Mixes - Core and Deck Testing				
	FA Field	SF Field	SL Field	MK Field	
2 year - RCPT	----	----	1,040	2,475	
2 year - Resistivity	----	----	53.4	24.1	
2.5 year - RCPT	1,100	465	----	----	
2.5 year - Resistivity	41.3	91.9	----	----	

Table 6-2: Permeability Analysis Grouping

Group	Lab Mixes (cylinders)	Field Mixes (cylinders)	Field Mixes (cores)
1	X	---	---
2	---	X	---
3	---	---	X
4	X	X	---
5	---	X	X
6	X	X	X

Correlation Analysis

Correlation analyses yield a correlation coefficient (R) that indicates the strength of relationship between the results of each test method. The coefficients from each group, ranging from -0.815 to -0.997, are provided in Table 6-3. Results from each group indicate that a strong negative correlation exists between results of the two test methods. A negative correlation indicates that as the result from one test method increases, the result from the other decreases. Analysis indicates that parameters such as age, curing method, lab and field mixes do not significantly affect the relationship between the observed permeability when using each test method.

Table 6-3: Correlation Coefficients – Permeability Analysis

Group No.	Correlation Coefficient (R)
1	-0.997
2	-0.815
3	-0.872
4	-0.831
5	-0.831
6	-0.835

Regression Analysis

Regression analyses were completed to understand the relationship between the results from both test methods and one test methods ability to predict the measurement obtained when using the other method. Figures 6-1 thru 6-6 illustrate the results of these analyses. Results from the regression analyses included a model that best represented the relationship between results from each test method. In each group, this resulted in a model following the form of a power function. The resulting model is of similar form as previously identified by other studies. The coefficient of determination (R^2) is a parameter of the analysis that indicates the goodness-of-fit of the model to the data, with values ranging between 0 (model explains 0% of variability) and 1 (model explains 100% of variability). Analyses of the 6 groups resulted in R^2 values ranging from 0.89 to 0.99, indicating that each model accounted for much of the variability between the test methods. R^2 values for each analysis are provided in Table 6-4. Results from regression analyses indicate that parameters such as age, curing method, batch size, and curing method do not significantly affect the relationship between observed permeability when using each test method.

Each of the statistical analysis methods indicated that the two methods are strongly related and provide very similar results while measuring different indirect properties to evaluate permeability. Each analysis suggests that parameters such as age, permeability, curing method, etc. did not have great effect on the relationship between results from each method. Results indicate model fit and correlation were superior for Group 1 (only lab prepared mixes) when compared to all other groups. This is likely due to the strict control of mixing, sample preparation, and curing that may not typically be available in field conditions. Based on analysis of the statistical results, each test method yields very similar ratings of permeability. While laboratory testing provides adequate results, testing requires a sample and greater time to complete testing. Lab testing of concrete as placed and cured requires a core sample. However, resistivity testing may be completed at any age on the in-place structure without need of a sample with immediate results. Results of both analysis methods indicate that the resistivity test method may provide trustworthy results and be well-suited for permeability testing of in-place concrete.

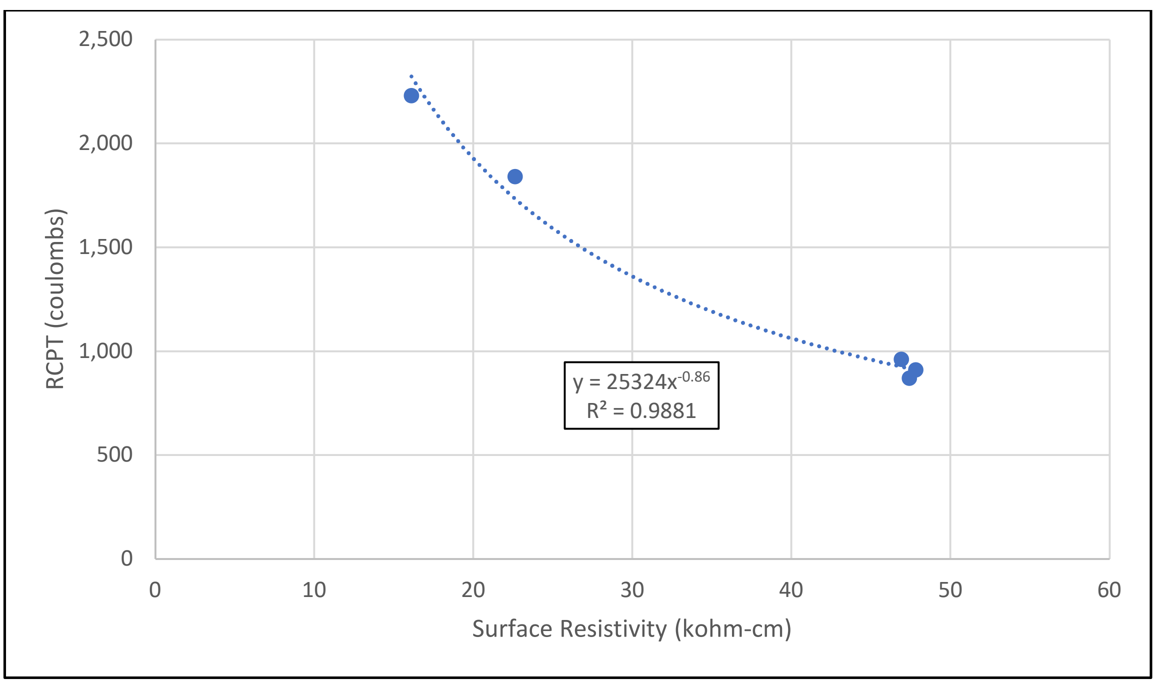

Figure 6-1: Regression Results – Permeability Group 1

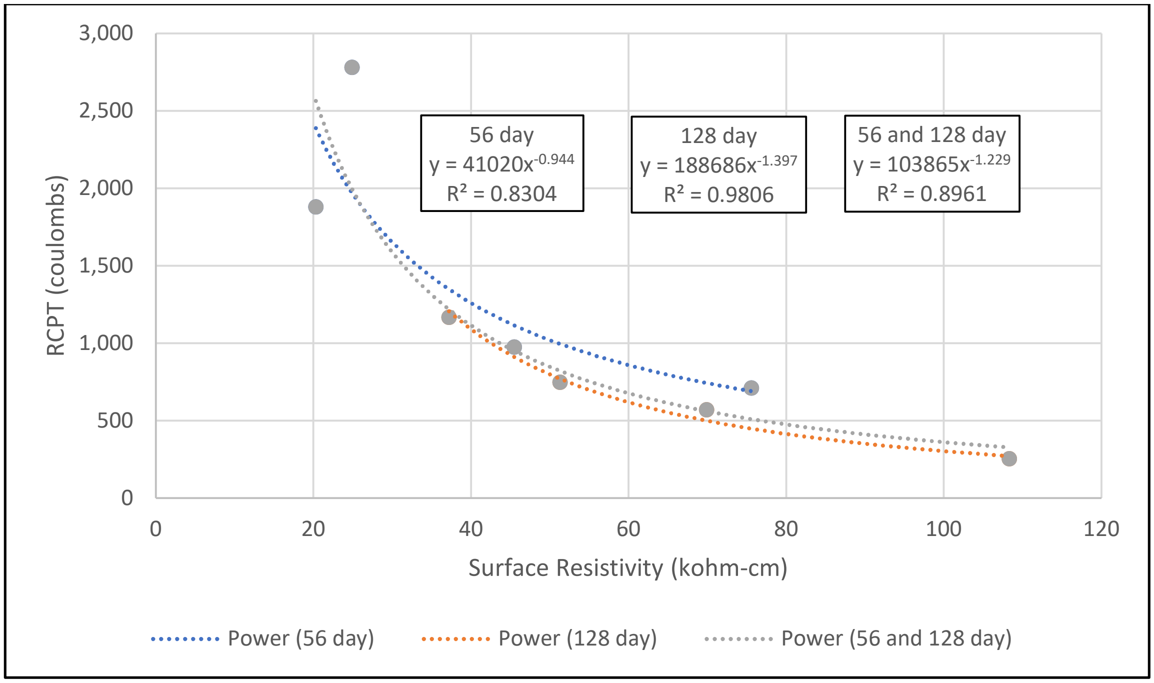

Figure 6-2: Regression Results – Permeability Group 2

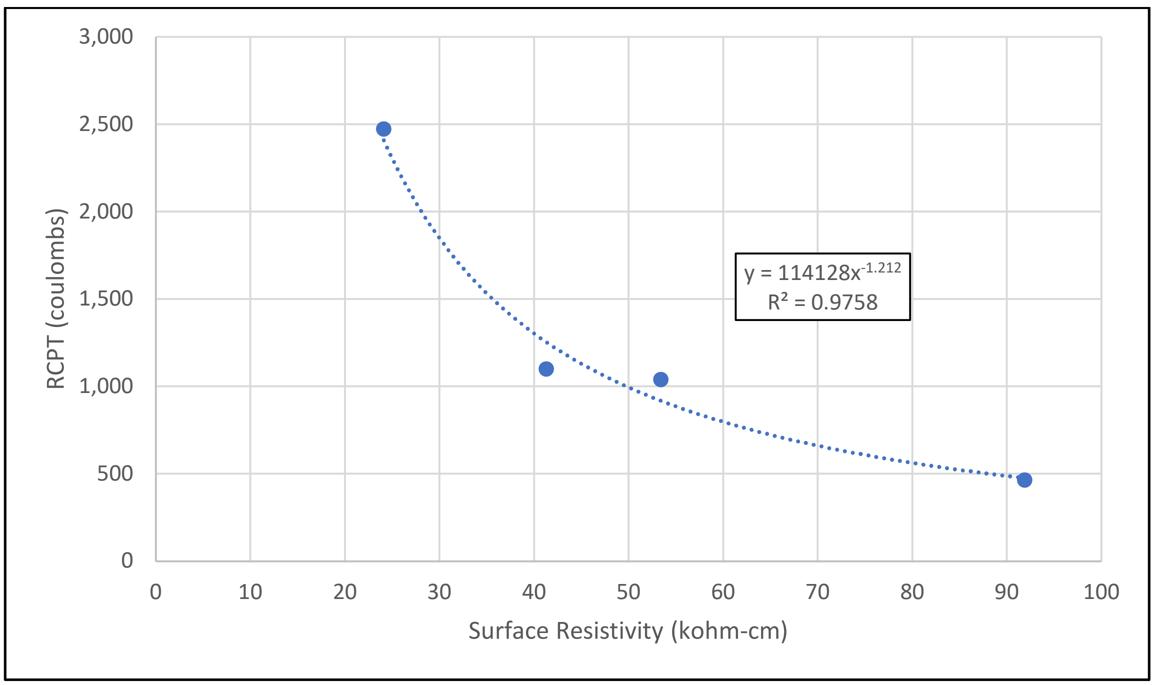

Figure 6-3: Regression Results – Permeability Group 3

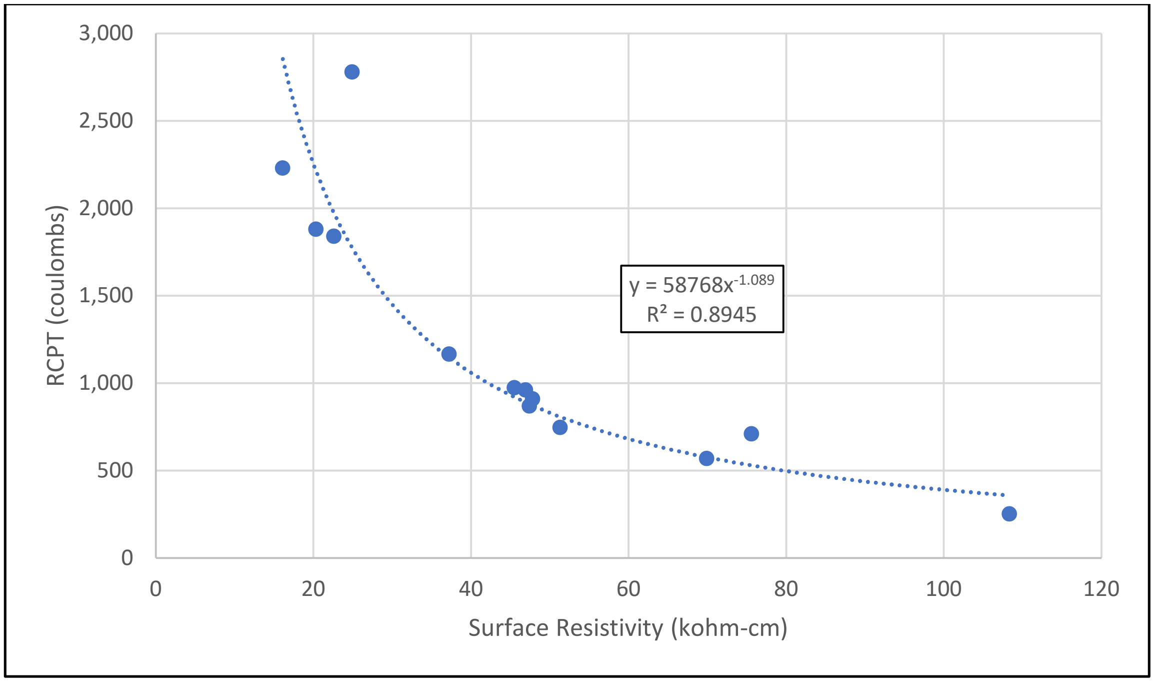

Figure 6-4: Regression Results – Permeability Group 4

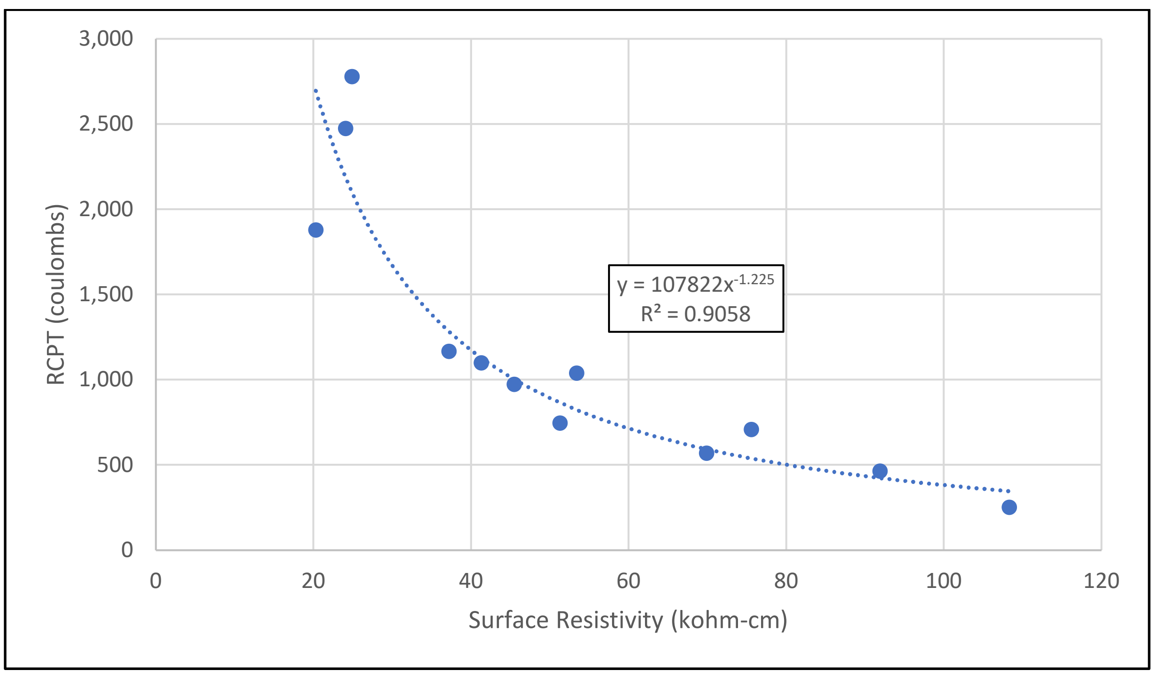

Figure 6-5: Regression Results – Permeability Group 5

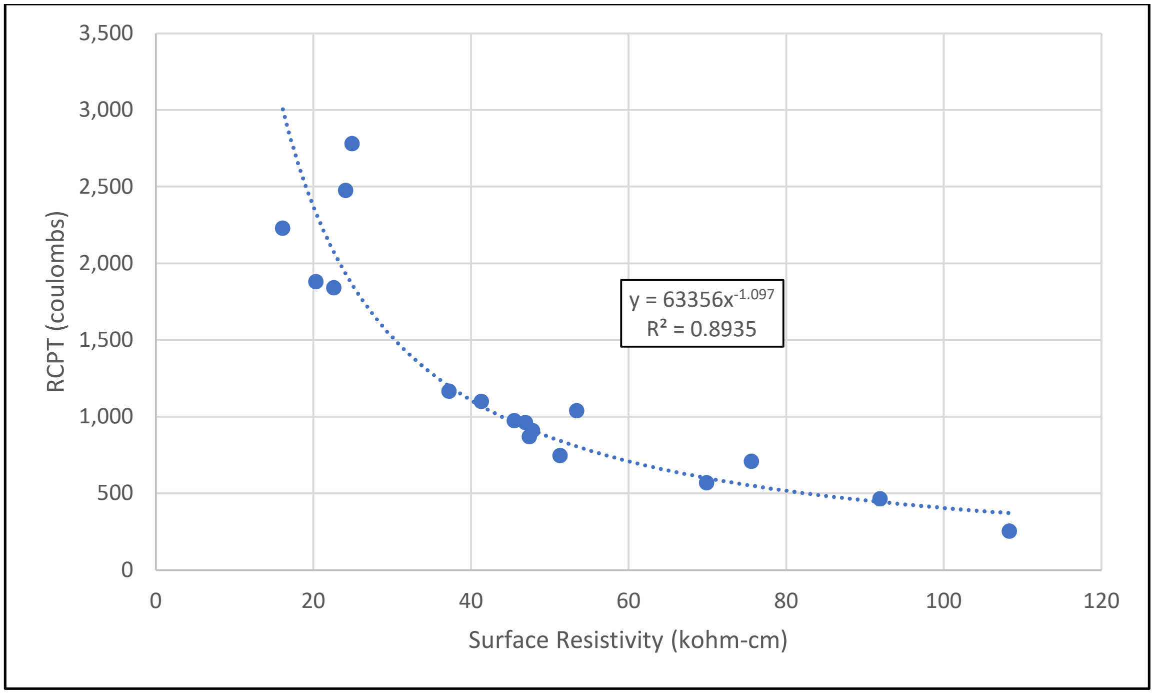

Figure 6-6: Regression Results – Permeability Group 6

Table 6-4: Coefficient of Determination – Permeability Analysis

Group No.	Coefficient of Determination (R^2)
1	0.9887
2	0.8961
3	0.9758
4	0.8945
5	0.9058
6	0.8935

SECTION 7: SUMMARY

Five mix designs were successfully developed in the laboratory. Included were control (2) and low permeability (3) mix designs. The first control design utilized Portland cement (PC Lab) as cementitious material and the second built on the first by replacing a portion of the Portland cement with fly ash (FA Lab). Each of the control designs was successful in meeting Class D requirements with the second modestly reducing permeability without appreciably effecting other hardened properties. Each of the designs were easily reproducible with consistent properties including appropriate workability and consistency. The designs were like those historically provided by producers for bridge deck construction.

Low permeability mix designs were developed based on the control designs with the aim of reducing permeability while maintaining plastic and hardened properties meeting Class D requirements. Each design, based on one of the control designs, included the addition of an SCM. The SCMs included silica fume (SF Lab), slag cement (SL Lab), and Metakaolin (MK Lab). In each case, the designs developed maintained required plastic properties and were easily reproduced with consistent and appropriate workability and consistency. Compressive strength in two of the low permeability designs was somewhat improved and in one case was negatively impacted in a minor amount. In all cases, compressive strength was well above that required for Class D concrete. Each design resulted in significantly reduced permeability with ratings of very low permeability based on two different test methods.

Four of the mix designs developed (FA, SF, SL, and MK) were used for full scale implementation on a bridge repair project consisting of two bridge structures each with two lanes of traffic. A different mix design was used for each lane of bridge deck for evaluation of the low permeability designs and comparison with the control design (FA). The laboratory designs were provided to the concrete producer prior to construction of the deck. Trial baches were completed with the producer's materials and admixtures to ensure all properties, workability and consistency were acceptable prior to full scale use. No changes to the original laboratory designs were made except for admixture dosage rates. Results from trial batches were acceptable and like those obtained in initial laboratory testing.

Deck construction using the FA Field, SF Field, and MK Field designs was successful. The producer identified no issues in production and delivery of each mix design. All batches of concrete delivered to the jobsite were acceptable and were not rejected. Also, the contractor responsible for placing and finishing each of the decks did not identify any issues. Delivery, truck spacing, placing, and finishing presented no issues. Curing of each deck was accomplished in a timely manner.

Construction of the deck using the SL Field design was ultimately successful, but several issues on the day of placing caused difficulties. Numerous batches (5) were rejected upon arrival at the constructions site. A single batch was rejected due to high slump and 4 batches were rejected due to higher than acceptable air content. The producer indicated that the rejected batches were result of admixture dosage rates. After adjustment of dosage rates, plastic properties were acceptable for the remainder of batches delivered. Additionally, a traffic accident occurred during placement of the deck that blocked the single lane of traffic that was open to traffic (phased construction). The closure of the traffic lane resulted in a detour of traffic around the construction site. Due to the detour, concrete delivery trucks were required to take a longer route to arrive at the construction site resulting in longer truck spacing. This resulted in longer time between successive placements and thus approaching cold joints in the deck. Also, initial curing operations were delayed and not begun in a timely manner consistent with the other decks placed, that possibly resulted in plastic cracking and impacts on permeability.

Testing results indicated that each of the designs used for deck construction met the plastic and hardened properties required for Class D concrete. Compressive strength results indicated that the SL Field design performed similar to the control, the SF Field design significantly outperformed the control, and the MK Field slightly underperformed the control.

Permeability testing (testing field cast specimens) indicated that, when compared to the control, the SL Field results in modestly reduced permeability and the SF Field design resulted in significantly reduced permeability. Conversely, the control design resulted in lower permeability than that found with the MK design. Field testing of each in-place deck (cores and testing complete directly on deck surface) indicated similar results in that the SL design modestly outperformed the control, SF field significantly outperformed the control, and the MK Field design significantly underperformed the control. Comparison of these results (cast specimens versus actual deck testing) indicated the same comparative results considering permeability.

Statistical analysis of the results from each permeability test method (RCPT and surface resistivity) indicated that results from each test method are highly correlated and result in very similar rating of permeability. Parameters such as age of test specimen, sample type (cast, core, or in-place deck) did not have significant impact on the relationship between the two methods. Results of the analysis support the use of the surface resistivity method for in-place evaluation of permeability without the need for a cast specimen or core.

Based on results and analysis of the full-scale project, several recommendations are made. First, while the low permeability designs achieved varying level of success, the SF Field design achieved far superior results when compared to the control and remaining low permeability designs. Additional use of this design is recommended for future small bridge deck projects to provide further study and evaluation. Additionally, a survey of producers concerning their ability to produce this design would provide insight into availability and ensure that the design can be used without competitive disadvantage. Second, additional study of the surface resistivity test method is recommended on future projects to further validate its use for evaluation permeability of in-place concrete. The limited results presented indicate that the test method appears to be a viable test method for assessing permeability of in-place concrete, but additional data is needed to gain certainty. This may include testing stored cast specimens, cores, and performing surface resistivity at varying ages to ensure that the surface resistivity test method provides consistent results and can be relied upon solely for permeability testing.

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