Evaluation of Current WI Mixes Using Performance Engineered Mixture Testing Protocols

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16. Abstract

This study investigated performance related tests with respect to current WisDOT concrete mixtures. Performance tests included the super air meter, hardened air voids, resistivity, formation factor, porosity, optimized gradations, vibrating Kelly Ball, box test, coefficient of thermal expansion, and compressive and flexural tests. Several project locations were selected that represented various regions and aggregate sources throughout the state of Wisconsin. Various testing regiments were performed using various consolidation methods for the SAM test, and various conditioning methods for resistivity testing. Results prove the value of using the optimized gradation curve (Tarantula Curve) by increasing workability, reducing cementitious without sacrificing performance. Recommendations include the use of the MinT for SAM consolidation and the accelerated curing method for resistivity.

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Executive Summary

This study investigated performance related tests with respect to current WisDOT concrete mixtures. Performance tests included the super air meter (SAM), hardened air voids, resistivity[1], formation factor, porosity, optimized gradations, vibrating Kelly Ball, box test, coefficient of thermal expansion, and compressive and flexural tests. There were two phases to this project. Phase I included visiting eight project locations which represented various regions and aggregate sources throughout the state of Wisconsin. During Phase I, material was tested at the plant, before and after the paver. The various tests included: SAM, hardened air, resistivity, formation factor, porosity, vibrating Kelly ball and the box test. During Phase I it was concluded that additional research was needed regarding optimized gradation, consolidation methods for the SAM and conditioning methods for resistivity. In Phase II of the project, a large lab study was conducted to investigate performance using various aggregate sources and blends. During the lab study, the research team investigated the common use of 1.5-in. (38.1 mm) stone in Wisconsin as it pertains to the Tarantula Curve and performance properties. The various optimized blends were tested using slump, box test, shrinkage, and resistivity. Additionally, during Phase II, five more field projects were visited where testing was conducted using various consolidation methods for the SAM test, and various conditioning methods for resistivity testing. Results demonstrate the value of using the optimized gradation curve (Tarantula Curve) by increasing workability, reducing cementitious products without sacrificing performance. Recommendations include the continued option to use 1.5-in. (38.1 mm) stone, use of the Tarantula Curve (with a warning band) for all WisDOT mixtures, design and production targets for the SAM meter, using the MinT for SAM consolidation and the accelerated curing method for resistivity.

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[1] Resistivity - In this document, the term resistivity is used to describe the electrical resistivity of a material. Electrical resistivity is a fundamental property of a material that measures how strongly it resists electric current. The term bulk or uniaxial resistivity refers to the electrical resistance of a material that is measured from end to end of a sample corrected for geometry. Surface resistance is measured using a 4-probe configuration. The surface resistance needs to be corrected to account for the shape of the electrical field and the confined geometry of a cylinder. If the confined cylinder geometry is not used the term 'apparent surface resistivity' is used by convention in the concrete community. When the test is properly performed, the sample is properly conditioned, and the geometry is properly accounted for; the resistivity measured using the bulk/uniaxial geometry (TP119) and the surface resistivity geometry (TP 358) are generally interchangeable.

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1.0 Introduction

Construction specifications, as provided by a state highway agency (SHA), describe the characteristics of the materials to be used and/or a process for performing work on a project. While specifications cover many aspects of construction, the ideal specification should provide a link between measurable properties of concrete and anticipated performance of the concrete in a given environment. Most concrete specifications have historically been prescriptive. In other words, these specifications have typically been a means and methods approach for the construction of concrete pavement. One of the main concerns with this approach is that most of the risk/liability for the performance of the structure is on the SHA and therefore limits the incentive for contractors to innovate their product. A potential alternative that is being evaluated is performance-related specifications (PRS) [1]. A PRS gives the expected performance of a concrete mixture and then relies on the contractor/producer to provide this material while permitting for less restrictions on the constituents of a mixture. The implementation of these new innovations has shown the ability to improve performance while decreasing the unit cost of the material and still meeting performance requirements.

PRS's can take many forms [1, 2, 3, 4, 5, 6, 7, 8, 9, 10]. Goodspeed et al. [2] developed a PRS for high performance concrete by establishing performance grades based on eight standard tests. Ozyilidirim [3] discussed the development of quantitative relationships linking tests to performance. National Ready-Mix Concrete Association (NRMCA) developed performance-based standards that relate hardened concrete requirements to established testing criteria [1]. PRS have also been developed in the late 1990s for pavements [4, 5, 6] that link tests, performance, and repair costs. While not a specification per se, The International Federation for Structural Concrete (fib) has developed a model code that outlines calculations of several durability related distresses [7], as have others [9, 10, 11]. Furthermore, the Illinois Tollway has been implementing performance-based specifications, which incorporate dowel bar alignment, thickness, strength, and smoothness into a payment calculation.

In response to a directive from the Federal Highway Administration (FHWA), an expert task group was established to examine Performance-Engineered Mixtures (PEM) [6]. This effort has produced the AASHTO PP-84 guide specification that uses a series of standardized tests to evaluate the concrete mixtures. These tests can be used either to design, qualify the mixture, or accept the mixture.

Recent research has shown that blending of different aggregate sizes allows for a reduction in the paste content of a concrete mixture while still showing satisfactory constructability performance by using the Tarantula Curve [12, 13, 14, 15, 16]. The Tarantula Curve is a tool to evaluate the performance of the mixtures. Also, several emerging test methods, Box Test, the V-Kelly, resistivity (bulk or surface), and the Super Air Meter, have become more practical and useful that give important insights into the constructability and the long-term durability of the concrete. Since these tests can be completed quickly and are not overly costly, the tests have great potential.

2.0 Research Objectives

This research was divided into two phases. The recommendations from Phase I were incorporated into Phase II, along with an expansion of some PEM testing. Phase I was conducted from 2017 to 2019, where Phase II was conducted from 2019 to 2021.

The objectives of Phase I were to:

1) Use performance-based testing methods on current WisDOT mixtures, and
2) Collect a comprehensive database of results on several WisDOT mix designs and assess how they compare to proposed Performance-Engineered Mixtures (PEM) specifications. This objective was to be completed using the following three tasks:
    i. Perform field-testing of plastic concrete from the plant and in front and behind the paver using PEM test methods.
    ii. Perform lab testing on hardened concrete specimens using PEM test methods.
    iii. Evaluate how current Wisconsin mixture designs fit into proposed PEM specifications.

Based on the WisDOT request for proposal the research focused on gaining new information on three primary properties:

Strength Properties

i. Flexural vs. Compressive Strength.

Durability Properties

i. Resistivity,
ii. Porosity,
iii. Formation Factor,
iv. Coefficient of Thermal Expansion,
v. Super Air Meter, and
vi. Hardened Air Voids

Workability Properties

i. Vibrating Kelly Ball
ii. Box Test

Based on the recommendations of Phase I, the Phase II objectives were to:

1) Evaluate the 1.5-in. (38.1 mm) aggregate in the Tarantula Curve.
2) Evaluate the MinT consolidation compared to rodding during the SAM testing.
3) Test Resistivity per AASHTO TP119-21 (bulk resistivity) and AASHTO T358-21 (surface resistivity) using the following curing conditions:
    i. Bucket Curing in pore solution (AASHTO TP 119-21 – Option A)
    ii. Accelerated Curing (AASHTO T358-21)
    iii. Lime Curing (AASHTO R39-21)
    iv. Sealed Sample (AASHTO TP119-21 - Option C)
4) Perform field testing on Wisconsin mixtures to verify Phase I recommendations and perform additional resistivity conditioning.

3.0 Test Methods and Project Selection

In order to produce more complete and representative concrete performance datasets from Wisconsin, Behnke Materials Engineering (BME) worked with the Project Oversight Committee (POC) and the Wisconsin Concrete Pavement Association (WCPA) to select project locations in different regions throughout the state. This widespread assortment of projects allows for the analysis of concrete products consisting of different aggregate types such as igneous and glacial gravels, limestones, and dolomites. Project locations with basic aggregate source information is presented in the following section followed by testing procedures and sample preparation.

3.1 Project and Testing Breakdown

This section discusses general project information such as project location, aggregate sources, and how testing sessions were performed for both Phase I and Phase II.

3.1.1 Project Selection Information

For Phase I, eight total project site visits were conducted throughout the 2017 and 2018 construction seasons. For Phase II, five total project visits were conducted throughout the 2021 construction season. An effort was made to select projects in all WisDOT regions to incorporate the variety of different materials (i.e.: aggregate, cementitious products) and blending of materials (i.e.: optimized/non-optimized gradations) used throughout the state. Final project selection was approved by the Project Oversight Committee (POC) prior to the field visit. Table 1 shows the outline of the projects visited for this research effort, and Figure 1 shows a map of where these projects are located.

Table 1: Project and Aggregate Source Information

					Aggregate Information			
Project ID	**Roadway**	**Location**	**Region**	**Year Visited**	**Coarse Agg. Source**	**Specific Gravity***	**Absorption %***	**Classification**
PHASE I								
1517-07-83	USH 10 to STH 441	Appleton	NE	2017	Ben Carrie	2.768	0.846	Limestone
2025-13-71	STH 190	Capitol Drive	SE	2017	Genessee	2.708	1.346	Gravel
1401-02-71	STH 16	City of Columbus	SE	2017	Michels Columbus	2.584	2.440	Dolomite
8680-00-71	USH 2 / Belknap Street	City of Superior	NW	2017	Robertson	2.753	1.035	Quartzite
2788-00-72	Summit Avenue to Northview Road	West Waukesha Bypass	SE	2017	Lafarge Colgate	2.736	0.827	Gravel
1003-10-84	IH 39; Illinois State Line - Madison, STH 11 to CTH O	I-39 Rock County	SW	2018	Townline	2.646	1.575	Gravel
1007-11-71	IH 39; Illinois State Line - Madison, E Church Road to Church St. NB	I-39 Dane County	SW	2018	Prairie Ave Concrete	2.672	1.399	Gravel
1022-08-72	IH 94; 250th Street - Wilson Creek	I-94 Menomonie	NW	2018	Hughes	2.594	2.585	Dolomite

PHASE II								
1058-02-73	STH 29; Wittenberg – Shawano, CTH U Intersection	STH 29 Shawano County	NC	2021	Peters Chase	2.756	0.879	Carbonate
1310-10-70	STH 50; 75th Street – IH 94 to 74th Avenue	STH 50 Kenosha County	SE	2021	LaFarge Dyer Lake	2.673	1.50	Gravel
1440-15-71	STH 23; USH 151 - Seven Hills Road	STH 23 Fond Du Lac County	NE	2021	Thackray	2.748	0.967	Carbonate
1007-12-79	IH 39; USH 12/18 Interchange/SB Core	IH 39 Dane County	SW	2021	Townline	2.646	1.614	Carbonite and Igneous Gravel
1060-33-84	USH 45; Zoo IC – North Leg	USH 45 Milwaukee County	SE	2021	Lannon	2.746	0.936	Limestone

*Specific Gravity and Absorption data acquired from WisDOT Approved Aggregate Source List.

Figure 1: Project Highway Location Map. Phase 1 projects are shown as black stars. Phase 2 projects are shown as red stars.

Phase I of this research project originally intended to investigate pavement mixtures that did not contain the Wisconsin DOT optimized graded specification (Tarantula Curve). It was discovered in the first year of Phase I where mixtures that did not meet the optimized graded specification showed variable performance in the Super Air Meter and Box Test. In the second year of Phase I, projects that met the Wisconsin DOT optimized gradation specification were investigated and subsequently the observed variability decreased. All Phase II projects used the optimized graded specification. Details for the mixtures are discussed below.

3.1.2 Test Sessions

3.1.2.1 Phase I Test Sessions

For the Phase I field visits the concrete was sampled at the plant, before the paver, and after the paver. These sampling techniques were respectively characterized as "Plant", "Before-Paver", and "After Paver". This was completed four times for each project except for Columbus and are referred to as "sessions" herein. These sampling sessions were performed over two days in both the morning and afternoon (numbered chronologically – sessions 1 through 4) to determine the variability of the material on the project. For example, session 1 indicates the samples (including at the plant, before-paver and after-paver) were taken on the first day in the morning, whereas session 4 indicates the samples (including at the plant, before-paver and after-paver) were taken on the second day in the afternoon. This approach is an attempt to capture any changes of properties as plant and field conditions vary throughout the day, and day-to-day.

3.1.2.2 Phase II Test Sessions

For the 2021 Phase II field visits the concrete was sampled only before the paver, as this is consistent with current WisDOT plastic concrete testing. This was completed three times for each project. These sampling sessions were performed over one day (numbered chronologically – sessions 1 through 3). If a project was scheduled to finish early on the day of our visit, the sessions were expedited, sometimes back-to-back, to ensure at least three sessions were completed in the time allowed. On all projects, except the Zoo Interchange, 3 sessions were tested.

3.2 Sample Preparation

This section describes sampling techniques and methods, justification for consolidation methodology choices, as well as organizational procedures.

3.2.1 Sampling

An objective of Phase I was to better understand how the concrete mixtures changed from production at the plant, to placement before the paver, and then after the paver consolidated the concrete. Plant sampling was typically conducted from the chute of a mixing truck, except Superior where material was sampled from a partially filled loader bucket. Before-paver samples were taken on-grade after the material had been discharged but prior to the material entering the paver. After-paver samples were taken immediately after the paver passed over the material but before the finishing crew handled the material. The before and after-paver samples were taken at the same location for each session so the material sampled would be the same, however, the after-paver sample had been run through the paver. Due to timing issues, the plant samples and the field samples were not taken from the same truck. All Phase II samples were taken before the paver. The appendix (Appendix A1) lists all the sampling sessions with corresponding testing.

3.2.2 Consolidation

Various consolidation methods were used throughout the project including rodding, a battery-powered vibrator, a 120V_{AC} powered vibrator, and the Miniature Vibration Table (MinT). The testing plan for the strength specimens (compressive strength and beam modulus of rupture) required consolidation using a battery-powered vibrator because that is standard practice for WisDOT. For the hardened specimens, AASHTO T 23 was followed for casting and curing while consolidation was accomplished using either rodding or a battery-powered vibrator. For the plastic testing, both the vibrating Kelly Ball (V-Kelly) and the box test required the use of the $120V_{AC}$ powered vibrator; however, these two tests require different vibration speeds and vibration heads. The V-Kelly requires a 13/16 in. (20.6 mm) square head vibrating at 8,000 vibrations per minute (VPM) while the box test requires a 1in. (25.4 mm) square head at 12,500 VPM. The variable speed *Wyco WVG1 SureSpeed Electric Concrete Vibrator Motor* was used for all testing.

The SAM Testing consolidation methods were modified from Phase I to Phase II. At the beginning of Phase I, consolidation was not considered a primary testing variable. However, high variability of test results was observed with the SAM meter during field visits, which drove the research team to evaluate consolidation procedures as a method to reduce variability. In the Phase II field visits, the research team explored consolidation of the SAM using the MinT and rodding.

The MinT is a portable vibration table that utilizes external vibration to consolidate the SAM. The source of the vibration is generated by inserting the vibration head of a battery-operated concrete vibrator in a metal slot underneath the device. To ensure efficient transfer of vibration energy, the vibration head is married to the device by tightening a series of bolts to hold it in place during operation. Figure 2 depicts what the device looks like. The MinT requires the minimum vibration speed of the concrete vibrator to be greater than 12,000 vibrations per minute (VPM) but less than or equal to 14,000 VPM. For this project, the concrete vibrator has a vibration speed between 12,500-13,000 VPM. The consolidation method calls for two lifts of concrete to be placed in the SAM bowl with each layer being vibrated for 50 seconds.

Figure 2: MinT.

3.2.3 Casting and Curing

Specimen preparation was based primarily on AASHTO T 23 with a few exceptions that will be discussed in later sections. Due to the 1.5 in. (38.1 mm) maximum aggregate size of most mixtures evaluated, all concrete cylinders were 6 in. x 12 in. (152.4 mm x 304.8 mm) size for consistency. Disposable plastic molds were used for all cylinders with a tightly fitting cap. Reusable steel or composite molds were used for casting beams with moist burlap set on the surface with a tarp over the top to reduce moisture loss. Per AASHTO T 23 all hardened specimens were cured onsite for 24-48 hours. The samples were then transported back to the testing facility. AASHTO T 23 allows a maximum transportation time of 4 hours, however, the Phase I Superior project was outside of the 4-hour range.

3.2.4 Conditioning

The Phase I 6 in. x 12 in. (152.4 mm x 304.8 mm) resistivity cylinders were conditioned in a cure room at the S.T.A.T.E. Testing facility in Illinois. For Phase II, the resistivity cylinders were conditioned at the Behnke Materials Engineering (BME) facility in Beloit Wisconsin using four different methods:

1) Bucket Curing in pore solution (AASHTO TP 119-21 – Option A)
2) Accelerated Curing (AASHTO T358-21)
3) Lime Curing (AASHTO R39-21)
4) Sealed Sample (AASHTO TP119-21 - Option C)

Additionally, six solutions were collected from the curing tanks at BME in glass vials for testing at Oregon State. Three of these were labeled "CHEM" and the other three were labeled "LIME". The LIME solutions were taken from large water tanks (that are intended to represent calcium hydroxide saturated solutions) and the CHEM refers to the simulated pore solution after approximately 6 weeks containing a concrete cylinder. A picture of these samples are shown in Figure 3.

Figure 3: Solutions as Received.

Resistivity of the solutions ($\rho_{Solution}$) was measured using a VWR Symphony B40PCID benchtop meter. The measurements were all taken in terms of solution conductivity ($\sigma_{Solution}$) in mS/cm. Conductivity (σ) is inversely related to resistivity as described in equation 1

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

The probe for the VWR Symphony B40PCID benchtop meter is stored in a KCL solution when not in use. Before testing the probe is calibrated using solutions of known resistivity. The sensor was calibrated before testing the "LIME" solutions using a 25 mS/cm calibration standard and the sensor was calibrated before testing the "CHEM" solutions using a 200 mS/cm calibration standard.

3.3 Strength Properties

This section describes the strength performance testing methods and procedures used on this project.

3.3.1 Compressive Strength

Compressive strength of the concrete was measured via AASHTO T 22 method using 6 in. x 12 in. (152.4 mm x 304.8 mm) cylindrical specimens. During Phase I, eleven specimens per field project were cast for strength testing and all eleven specimens were cast in testing session 2. Two specimens were tested at 3, 7, 14, 28, and 90 days after casting.

For Phase II, three specimens were cast for strength testing during each session (total of twelve specimens per project). Each specimen was tested at 28 days after casting.

3.3.2 Modulus of Rupture

The modulus of rupture (MOR) for the concrete beams were measured according to AASHTO T 97 which uses three-point bending. For Phase I, one beam was tested at 7, 14, and 90 days after casting while three beams were broken at 28 days after casting. The reduced number of replicates for 7, 14, and 90 days was due to the increased manpower and hauling costs for the additional samples. It was decided that the budgeted funds were better spent visiting more projects for greater data diversity rather than additional MOR replicates. For Phase I, all samples were cast from material that was sampled in testing session 1 for each project and from material that was sampled from the before-paver location. For Phase II, three specimens were cast for flexural testing during each session (total of twelve specimens per project). Each specimen was tested at 28 days after casting.

3.4 Durability Properties

Durability is the ability for a material to last for extended periods of time without significant deterioration to its physical structure or properties. Materials that are durable are desirable for many reasons such as cost savings, conserving resources, reducing waste, and the environmental impacts of repair or replacement. These properties are particularly important to concrete products that are exposed to the environment because concrete will be subjected to weathering, chemical attack, and abrasion; all of which will deteriorate the concrete's engineering properties over time. The following test methods are designed to assess the durability of concrete products.

3.4.1 Surface Resistivity

For the Phase I and II resistivity testing performed by BME, each mixture's surface resistivity was measured according to AASHTO T 358 using 6 in. x 12 in. (152.4 mm x 304.8 mm) cylindrical specimens. During Phase I, nine cylinders were cast for each project location, where three samples were produced during each testing session for the first three sessions. Cylinders were stored in a temperature-controlled (69.8° - 77°F / 21°C - 25°C) cure room at 100% relative humidity when not being actively tested. During Phase II twelve cylinders were cast for each project location, except STH 29 (Shawano) where only three cylinders were cast and conditioned in lime water tanks. It was after STH 29 that the additional conditioning methods were added to the work plan.

All cylinders were tested at intervals of 7, 14, 28, 56, and 90 days after casting, except for STH 29 which were tested at 28, 56 and 90 days. For Phase I, samples were removed from the cure room for testing, but ambient temperature was not recorded during testing, although, the samples were tested in a temperature-controlled laboratory environment. Samples were returned to the cure room immediately after testing. The samples were reused, so all nine cylinders from each project location were tested at each of the required curing intervals. For Phase II, samples were removed from the tanks and tested immediately.

Table 2 shows the AASHTO defined levels of chloride ion penetration for a given geometrically corrected surface resistivity measurement. Resistivity is inversely related to chloride ion penetration, where higher resistivity corresponds to lower chloride ion penetration, which is a desirable property. The typical trend is for the apparent surface resistivity reading to increase as the concrete has longer curing times (i.e., increased hydration). This means that the potential for chloride ion penetration decreases with curing time. Figure 4 shows the device used to measure the apparent surface resistivity.

Table 2: Chloride ion penetration levels as specified by AASHTO T358 corrected for 6 in. x12 in. (152.4 mm x 304.8 mm) cylinders.

Corrected Chloride Ion Penetration - T 358	
Level	**Resistivity (kΩ-cm)**
High	<6.9
Moderate	6.9-12
Low	12-21.1
Very Low	21.1-144.6
Negligible	>144.6

Figure 4: Surface Resistivity Apparatus measuring a 6 in. x12 in. (152.4 mm x 304.8 mm) cylinder.

For the Phase II lab testing, conducted at Oklahoma State University (OSU), three 4 in. x 8 in. (101.6 mm x 203.2 mm) concrete cylinders were made from each mixture and cured according to ASTM C192 for surface resistivity testing AASHTO T 358, and an additional 3 were kept as reserve. These samples were cured in the environmentally controlled room at a temperature of

73°F (22.8°C) and 100% relative humidity according to ASTM C192 and kept in their molds until testing. Four lines were marked on the circular face of each concrete cylinder at 0, 90, 180, and 270 degrees and measured in terms of kΩ-cm over those lines. These four measurements were averaged for each cylinder.

3.4.2 Porosity, Bulk Resistivity, and Formation Factor

Phase I concrete samples from Rock County, Dane County, and Menomonie projects were examined for porosity and bulk electrical resistivity measurements.

3.4.2.1 Porosity

Porosity measurement was conducted on the cylindrical specimens with a diameter of 6 in. ± 0.08 in. (152 mm ± 2 mm) and a thickness of 2 in. ± 0.04 in. (51 mm ± 1 mm). The volume of permeable pores was determined according to ASTM C642-13 with the exception that the concrete specimens were saturated by vacuum, instead of being placed into boiling water (Note: a new AASHTO standard (AASHTO TP 135) has been created for the determination of porosity based on the approach used here and this will be available later this year). Vacuum saturation has been shown to be a comparable method of sample conditioning which enables the saturation of all air voids in the specimen. After the specimens were oven dried at 221°F ± 3.6 °F (105°C ± 2 °C), the mass was measured and then they were placed into the vacuum chamber with a vacuum level of 0.135 psi ± 0.039 psi (933 Pa ± 266 Pa) for 3 hours. Saturated limewater was drawn into the vacuum chamber and specimens were maintained in vacuum for another hour. The specimens were kept submerged for another 48 hours after the vacuum session. The mass of the saturated surface dry (SSD) samples and their apparent mass under water were measured to calculate the porosity. Eight cylinders of each mixture were tested for porosity.

The calculation for porosity (per AASHTO TP 135 – Section 9.1) is as follows:

$$\text{Volume of permeable pore volume, percent} = \frac{B-A}{B-C} \times 100 \qquad (2)$$

Where:

A = mass of oven-dried sample in air, g;

B = mass of saturated, surface-dry sample in air after vacuum, g; and

C = apparent mass of sample in water after vacuum, g.

3.4.2.2 Bulk Electrical Resistivity

The bulk electrical resistivity was measured on the concrete specimens with a diameter of 6 in. ± 0.08 in. (152 mm ± 2 mm) and a thickness of 2 in. ± 0.04 in. (51 mm ± 1 mm). Specimens were conditioned using Option A (immersion in a calcium hydroxide saturated simulated pore solution) of AASHTO TP 119-17. The calcium hydroxide simulated pore solution consists of 7.60 g/L NaOH (0.19M), 10.64 g/L KOH (0.19M), and 2.0 g/L $Ca(OH)_2$. For Phase I, eight cylinders of each mixture were tested for bulk electrical resistivity. Specimens were immersed in a 5-galon bucket with enough calcium hydroxide saturated simulated solution to cover the specimens by 1.5 in. (38 mm). The resistance of the specimens was measured using a resistivity meter with a frequency of 1 kHz at 73.4°F ± 3.6°F (23°C ± 2°C). The bulk electrical resistivity was calculated according to AASHTO TP 119-17. Figure 5 shows the device used to measure bulk electrical resistivity, although, this can also be done using any surface meter [17].

Figure 5: Bulk Resistivity Apparatus.

3.4.2.3 Formation Factor

The intent of the formation factor is to describe the pore network and its connectivity. Ideally less pore connectivity leads to higher durability as less material is able to penetrate the concrete and cause corrosion. The formation factor (F) is calculated as the ratio of the electrical resistivity (obtained through Bulk or Surface Resistivity) of the saturated concrete (ρ_C) that has been saturated with simulated pore solution to that of the resistivity of only the simulated pore solution:

$$F = \frac{\rho_C}{\rho_{ps}} \tag{3}$$

where ρ_{ps} is the electrical resistivity of the simulated pore solution. Given this relationship, high Formation Factor values are desired to increase the predicted durability. Table 3 summarizes the relationship between chloride ion penetration susceptibility and measured resistivity, and formation factor.

Table 3: Relationship between Chloride Ion Penetration Susceptibility, Resistivity, and Formation Factor

Chloride Ion Penetration – TP119-19		
Level	Resistivity (kΩ-cm)	Formation Factor
High	< 5.2	520
Moderate	5.2 - 10.4	520 - 1,040
Low	10.4 - 20.8	1,040 - 2,080
Very Low	20.8 - 207	2,080 - 20,700
Negligible	> 207	20,700

3.4.3 Coefficient of Thermal Expansion

The coefficient of thermal expansion/contraction describes how much concrete will expand or contract under changing thermal conditions. This is particularly important in paving applications. Because concrete is usually sawcut to control shrinkage cracking during curing. These slabs will warp (convex or concave) depending on the thermal gradient throughout the depth of the concrete slab. In cases where the concrete is highly susceptible to thermal expansion or contraction, premature damage to the concrete slabs can occur. It also can become a road hazard to the motoring public in the most severe cases. This is particularly important in paving applications where the concrete is subject to severe thermal cycling. It is generally assumed that higher coefficients of thermal expansion are not desired because this will cause the least volumetric stability under changing thermal conditions. Both the cement paste and coarse aggregates play a significant role in the thermal expansion and contraction properties of the mixture. Limestone aggregates typically have the lowest expansion, and quartz-based aggregates typically have the highest coefficient of thermal expansion.

Concrete cylindrical sample specimens were tested to determine the coefficient of thermal expansion (CTE) according to AASHTO T 336. Specimens were conditioned for no less than 48 hours by submersion in limewater, and until two successive weighs of the surface-dried sample at intervals of 24 hours showed an increase in weight of less than 0.5%. Specimen lengths were measured at room temperature after removal from the conditioning tank and then placed into the measuring apparatus. CTE values are determined by taking measurements with the LVDTs at 50°F (10°C) and 122°F (50°C) and then 50°F (10°C) again to simulate heating and cooling segments, where a segment is the measured length change for a given heating (50°F to 122°F / 10°C to 50°C) or cooling change (122°F to 50°F / 50°C to 10°C). A cycle is two consecutive segments. The following equation is used to calculate the CTE:

$$CTE = \frac{CTE_1 + CTE_2}{2}, \left[reported\ units = \frac{\mu\varepsilon}{°C}\right] \tag{4}$$

Where:

$$CTE_n = \frac{\left(\frac{\Delta L_a}{L_0}\right)}{\Delta T} \tag{5}$$

$$\Delta L_a = \Delta L_m + \Delta L_f \tag{6}$$

$$\Delta L_f = C_f \times L_0 \times \Delta T \tag{7}$$

ΔL_a = actual length change of specimen during temperature change, mm

L_0 = measured length of specimen at room temperature, mm

ΔT = measured temperature change, °C (increase = positive, decrease = negative)

ΔL_m = measured length change of specimen during temperature change, mm (increase = positive, decrease = negative)

ΔL_f = length change of the measuring apparatus during temperature change, mm

C_f = correction factor accounting for the change in length of the measurement apparatus with temperature, $\frac{\frac{mm^{-6}}{mm}}{°C}$

3.4.4 Air Void System

It is very common in the production of concrete to include chemical admixtures called air entrainers. Air entrainers are primarily liquid surfactants that are added to the mix water, and occasionally introduced as a powder to the cement prior to adding the mix water. Air entrainment's purpose is to encourage the formation of small spherical air bubbles during mixing of plastic concrete that will be evenly distributed throughout the hardened cement paste. It can also help improve the workability during the mixing and forming stages. Ultimately, the purpose of these air voids is to decrease the cracking susceptibility of the concrete from freezing water within the pores. When pore fluid freezes, it expands, and the entrained air void system provides extra space for the ice to expand into. If the pressure induced by the freezing water exceeds the tensile strength of the concrete, the cavities will dilate and rupture causing cracks, scaling, and crumbling. Ideally, the entrained air voids should be closely spaced, yet still isolated from each other and from large capillary pores. This reduces the possibility for the voids to become saturated with water, rendering them ineffective. Additionally, if concrete is not properly consolidated, larger "entrapped" air voids may be present which contribute to the overall air content, however, they do not provide the same durability benefits that entrained air voids provide. Entrapped air should be minimized in all concrete mixtures.

3.4.4.1 Super Air Meter (SAM)

The SAM is similar to the ASTM C231 Type B air meter with some modifications, the main difference being the SAM meter will quantify the distribution of the entrained air. The SAM uses six retaining clamps to account for the increased operational pressures and a digital pressure gauge for testing. The device is shown in Figure 6.

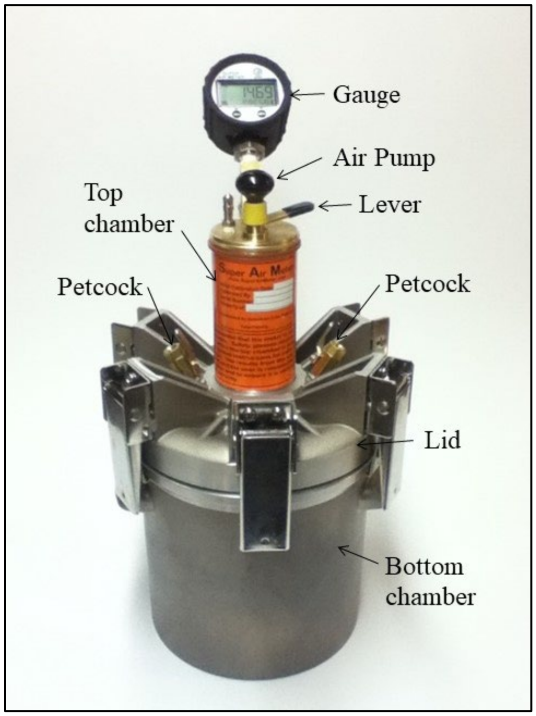


Figure 6: SAM testing device.

Similar to the Type B meter, the first step is to fill, consolidate, and strike off the plastic concrete in the bottom chamber in accordance with ASTM C231. The top rim of the bottom chamber and the bottom ring of the lid are cleaned thoroughly to ensure a proper seal. The rim of the bottom chamber should be free of any concrete, aggregates, or paste. This cleaning is important for a proper seal between the lid and bottom chamber since the pressure increments are higher for the SAM than the Type B meter.

The clamps then fasten the lid to the bottom chamber and water is added through the petcock valves to fill the void between the bottom of the lid and the top of the concrete. Once all the air bubbles are out of the bottom chamber, the petcocks are closed. Next, the top chamber is pressurized to 14.50 psi ± 0.05 psi (99.97 kPa ± 0.345 kPa) and allowed to equalize. The bottom chamber is struck with a rubber mallet while the lever is pressed to allow the two chambers to equalize. The lever is held down for 10 seconds to reach equilibrium. The pressure value on the gauge is recorded and used to calculate the volume of the air in the concrete [18, 19, 20, 21, 22]. The top chamber is then pressurized up to 30.00 psi ± 0.05 psi (206.84 kPa ± 0.345 kPa) without opening the petcocks. The lever is held down for 10 seconds to reach equilibrium while the bottom chamber is hit on all sides. The top chamber is then pressurized to 45.00 psi ± 0.05 psi (310.26 kPa ± 0.345 kPa) without opening the petcocks. The lever is then held down for 10 seconds and the sides of the bottom chamber are hit with a rubber mallet. This pressure value should be recorded and will be called, Pc1. The pressure is then released from the bottom chamber by opening the petcock valves. The lid is left attached while water is added to the bottom chamber through the petcocks to fill the space between the lid and the concrete and the procedure is repeated. After completing the 45 psi (310.26 kPa) pressure step, the equalized pressure is recorded as Pc2. The test takes an experienced user between 8-10 minutes to complete. Figure 7 shows a typical data set and a video of the test is available [23].

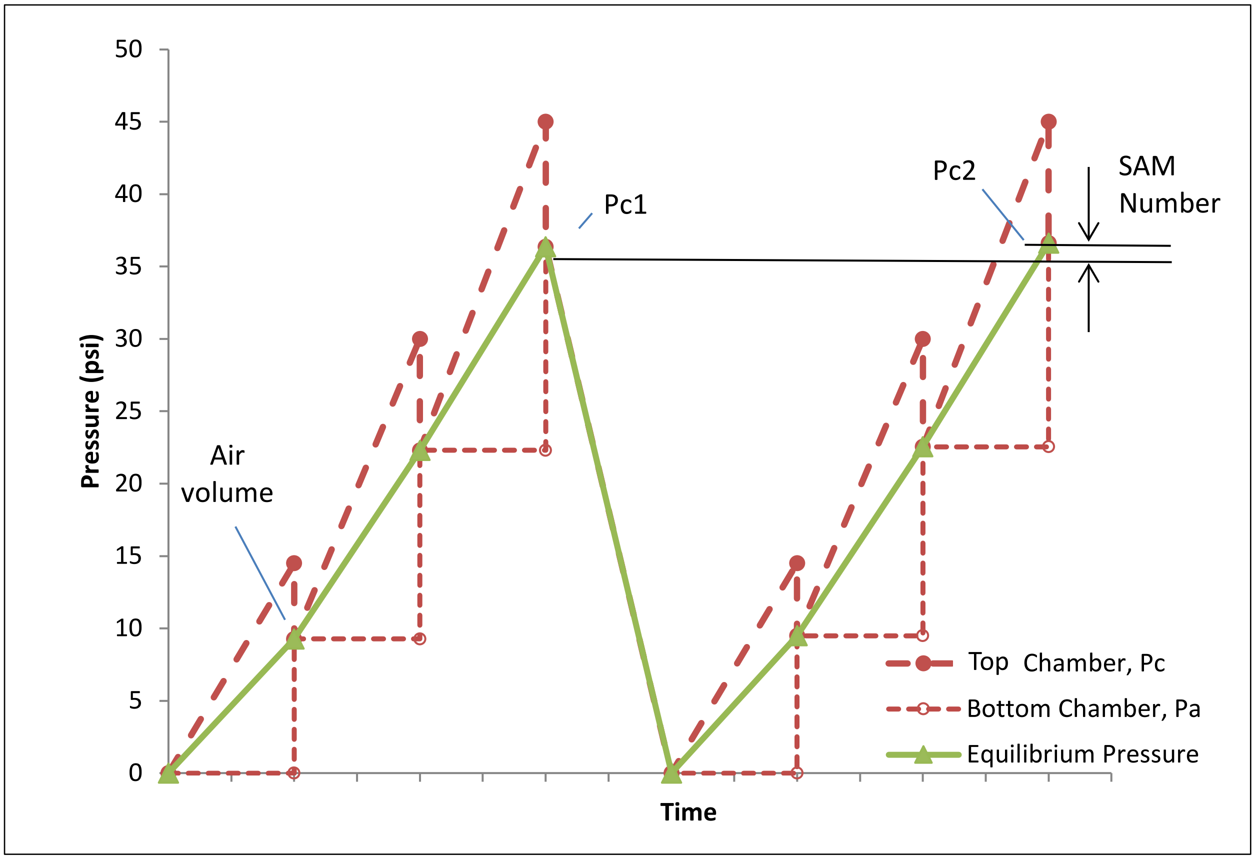


Figure 7: SAM pressure steps graphically shown for the top and bottom chambers.

The SAM Number is the term used to quantify the differences in the two pressure curves shown in Figure 7. Mathematically, this term is shown as: SAM Number = Pc2 – Pc1.

Pc1 is the first equalized pressure at 45 psi (310.26 kPa) and Pc2 is the second equalized pressure at 45 psi (310.26 kPa). The SAM Numbers ranged from 0.03 to 0.78 for the mixtures represented. The SAM Number is an empirical number that will be correlated to other parameters such as Spacing Factor. The Spacing Factor is a numerical value that is obtained from the hardened air void analysis that provides an indication of the air void spacing within the concrete. This is a widely used parameter in the concrete industry and so it is useful to compare to the SAM Number. The SAM Number is used as a correlative number because it is unitless.

3.4.4.2 Hardened Air Voids

The ASTM C457 hardened air void analysis was completed by Oklahoma State University. Concrete samples were cut into ¾ in. (19.05 mm) thick slabs and polished with sequentially finer grits. The surface of the sample was preserved with an acetone and lacquer mixture to strengthen the surface before it was inspected under a stereo microscope. After an acceptable surface was obtained, the sample is cleaned with acetone. The surface was then colored with a black permanent marker, the air voids were filled with less than 0.0000394 in. (1 µm) white barium sulfate powder, and the air voids within the aggregates were blackened under a stereo microscope. This process makes the concrete sample black and the voids in the paste white. Sample preparation details can be found in other publications [24, 25]. The samples were analyzed with ASTM C457 method C by using the Rapid Air 457 from Concrete Experts, Inc. A single threshold value of 185 was used for all samples in this research, and the results do not include chords smaller than 30 μm (0.0011811 in.). These requirements have shown to provide similar techniques by others and satisfactory results with the materials and instrument used [25, 26, 27].

3.4.5 Shrinkage Testing

Four 4 in. x 4 in. x 11.25 in. (101.6 mm x 101.6 mm x 285.75 mm) beam samples were prepared for drying shrinkage testing. Two of the samples were used for mass change measurements, while the other two samples were used to measure length change (ASTM C157). The two length change samples contained vibrating wire strain gauges that were cast into the samples during concrete placement. The strain gauges allowed strain measurements to be taken as the concrete was hydrating, which is not possible when using the ASTM C157 standard procedure for shrinkage measurements. To keep the gauges centered in the beam forms, two holes were drilled in the forms and thin wires were loosely wrapped around each end of the gauge and each wire was strung through each hole in the wooden sides.

After casting, all four samples were demolded, and an initial mass reading was taken for the two mass change samples. Then, all four samples were wet cured for 7 days in an environmentally controlled room that was kept at a temperature of 73°F (22.8°C) and 100% relative humidity. The four samples were demolded after one day, and the initial mass measurements were recorded. The samples were stored in the drying room, which was kept at a temperature of 73°F (22.8°C) and 50% relative humidity. The mass loss of each sample was measured at the time intervals specified in ASTM C157 with an accuracy of 0.1 g (0.00022 lbs.). The gauges recorded strain measurements from the samples every hour. Since strain measurements were being recorded for two samples, the measurements were averaged and plotted with error bars showing one standard deviation.

3.5 Workability

Adequate and application-specific workability of concrete is essential to slipform paving operations. If the mixture does not respond well to vibration, then the material will not properly consolidate, and this will leave unintentional “entrapped” air voids at the surface and within the pavement. If the concrete is too workable then the material will not retain its shape after passing through the paver. The traditional method of quantifying workability is through the slump cone test under AASHTO T 119; however, this does not characterize how the material behaves under the influence of vibration. This is important to consider as slipform pavers rely on vibration for forming and consolidating the mixture which requires the workability to be in a specific range to obtain proper consolidation while allowing the material to stay formed without slumping after the paver passes. It is also important to note that not all material with the same slump performs the same under vibration which identifies a need for a new procedure to be implemented to characterize workability for slipform mixtures.

3.5.1 Vibrating Kelly Ball (V-Kelly)

The V-Kelly test method is aimed at characterizing the properties of the plastic concrete under vibration. The V-Kelly test is a relatively new concept that is based on a penetration-based test that was developed as an alternative to the slump cone test [28, 29, 30]. The original method involved placing a frame and weighted semispherical ball as shown in Figure 8. The penetration of the ball is monitored over time. The original test was formerly standardized in ASTM C360-92, The Standard Test Method for Ball Penetration in Freshly Mixed Hydraulic Concrete. Due to lack of adoption of the test method the test procedure was discontinued from ASTM in 1999.

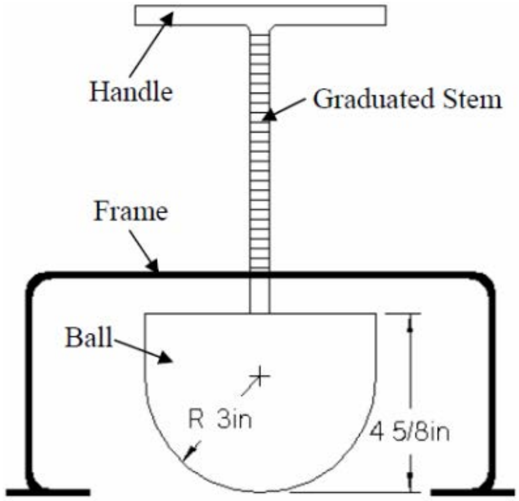


Figure 8: Schematic of the testing apparatus for ASTM C360-92. [31]

The ASTM C360 procedure was then modified by Iowa State University to add vibration to the test. The test is similar, but a vibrator is attached to the ball and the rate of penetration is measured over 30 seconds. Once the data is obtained a curve fit is used to determine the V-Kelly Index (V_{Index}) from the following relationship:

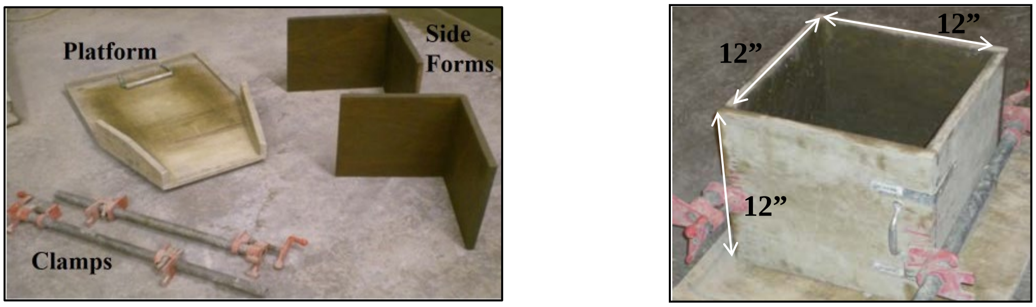
$$D_{Pene} = V_{index} \times \sqrt{t} + c \quad (8)$$

Where:

D_{Pene}:	Penetration at time *t* (in)
V_{Index}:	V-Kelly Index
t:	Time (sec)
c:	Initial Penetration (in)

3.5.2 Box Test

A mixture for a slipformed paver requires the concrete to be flowable enough for consolidation but still hold an edge. The Box Test is a useful tool for evaluating the response of a concrete mixture to vibration, while also holding an edge [32]. The Box Test, (AASHTO TP-137), was developed to visually quantify the workability based on comments from the slipformed paving industry. The components of the Box Test are shown in Figure 9(a). The two L-shaped forms have been made to form a hollow box on top of the platform as shown in Figure 9(b). The two pipe clamps with a span of 18 in. (457.2 mm) or a ratchet strap can be used to hold the L-shaped forms together. The vibrator parameters of frequency and head size have been specifically designed to be comparable to the consolidation of a slipformed paver, and therefore it is imperative to use the proper vibrator when conducting this test. A 1-in. (25.4 mm) square head electric vibrator operating at 12,500 VPM is required to provide the consolidation to the concrete.



(a) (b)

Figure 9: (a) components of the Box Test and (b) dimensions of the Box Test.

After the components of the Box Test have been assembled, concrete can be uniformly hand scooped into the box up to a height of 9.5 in. (241.3 mm). Then a 1 in. (25.4 mm) square head vibrator running at 12,500 VPM is inserted and allowed to vibrate downward into the concrete for three seconds. Then the vibrator continues vibration while raising the vibrator upward for three seconds and then finally removing the vibrator from the box of concrete. Immediately, the clamps are to be removed from the side wall forms and then both side wall forms are removed. This process can be described as a four-step process as shown in Figure 10.

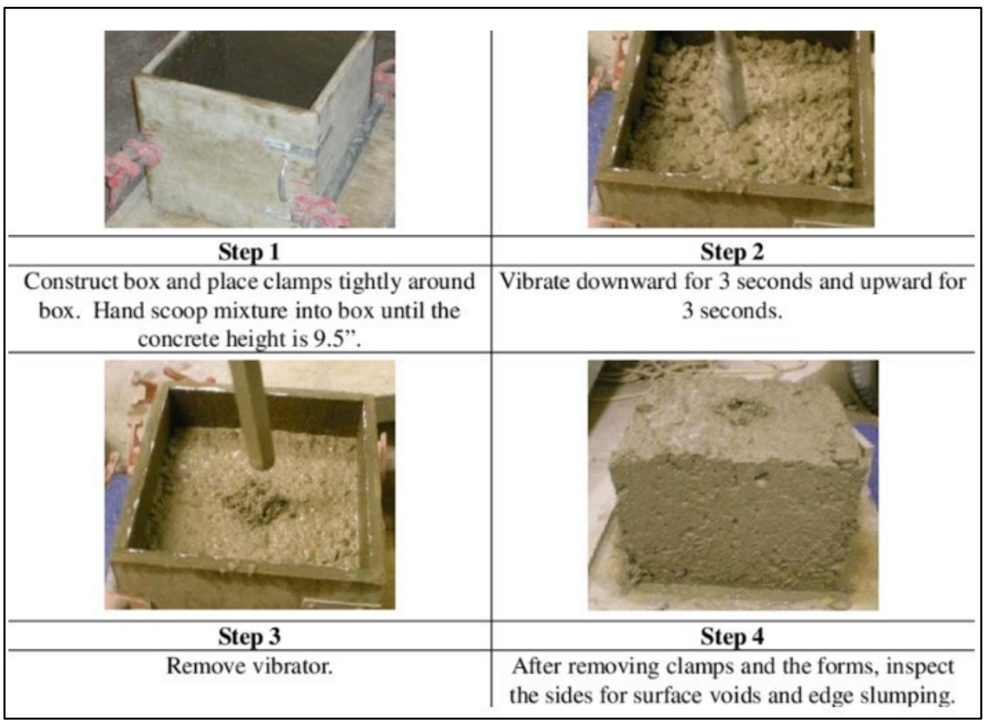


Figure 10: The four steps of the Box Test [32].

The response of a mixture to vibration can be assessed by the surface voids observed on the sides of the box using Figure 11. If a mixture responded well to vibration, the overall surface voids should be minimal because the vibration waves were able to propagate through the concrete and remove these voids. If, however, the sides of the concrete mixture had large amounts of surface voids, it did not respond well to vibration. The average surface voids for each of the four sides were estimated with a number ranking using Figure 11 and an overall average visual ranking was given to each test. For this testing, a mixture was assumed to have good workability performance if the edge slumping was less than 0.25 in. (6.35 mm) and the sides had less than 30% surface voids measured visually. This performance criterion will be referred to as “passing the Box Test.” These requirements are discussed in a past publication by Cook et al. [32].

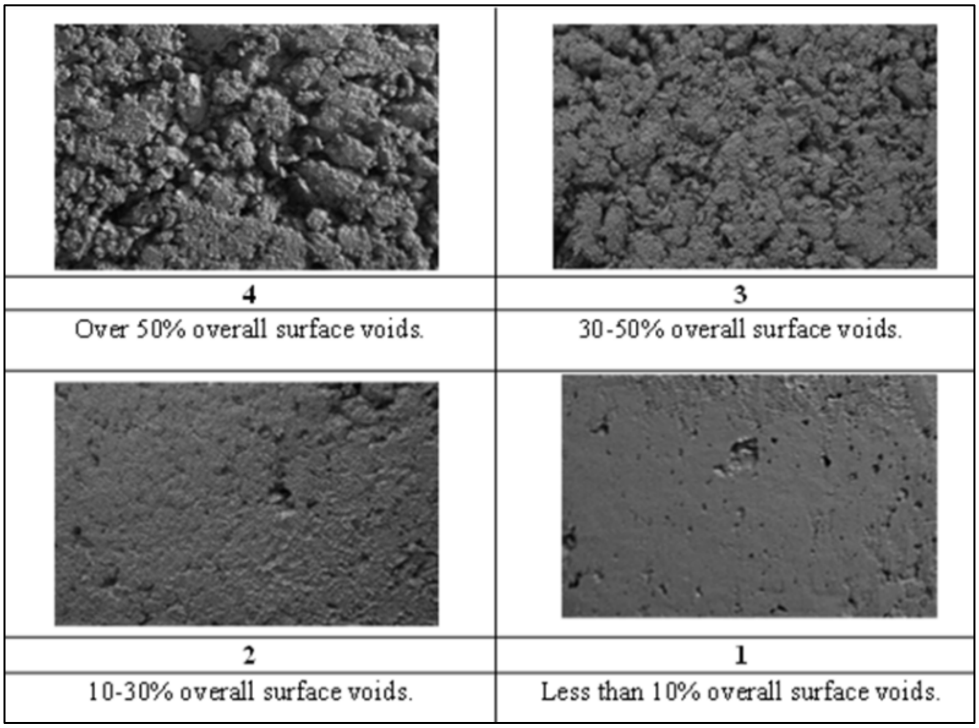


Figure 11: The Box Test Ranking Scale [32].

The Box Test can provide insight into possible edge slumping issues. The top and bottom edge slumping can be measured to the nearest 0.25 in. (6.35 mm) by placing a straightedge at each corner and using a tape measure to find the length of the highest extruding point. It is common for well-performing paving mixtures to have less than 0.25 in. (6.35 mm) of edge slumping with the Box Test.

3.5.3 Slump Test

The Slump test is used to help provide insight into the consistency of workability of concrete mixtures. The Slump Test, AASHTO T119, has been the most specified workability test; however, it simply measures the ability of a cone of concrete to deflect after removing forms [33]. This test is not sensitive enough to accurately predict the workability of mixtures with low flowability, such as pavement mixtures [28].

4.0 Phase II Lab Evaluation of Wisconsin Aggregates

During Phase II, the Oklahoma State University (OSU) lab evaluated Wisconsin aggregates. Not only does Wisconsin have varying aggregate sources throughout the state, as a practice, most contractors use 1.5-in. (38.1 mm) stone in their PCC mixtures. Wisconsin was moving towards an optimized gradation requirement; however, the original Tarantula Curve was not evaluated for larger than 1-in. (25.4 mm) aggregates. OSU was tasked with evaluating various aggregate sources and gradations for strength and performance properties to determine if Wisconsin specifications needed to be adjusted.

4.1 Selection of Aggregate

Researchers worked with the POC and the Wisconsin Concrete Pavement Association (WCPA) to identify four different sources of coarse aggregates. The intent was to collect materials from a mixed aggregate in south central Wisconsin, a crushed limestone in Milwaukee area, an igneous gravel in northwest Wisconsin, and an igneous quarry material in north central Wisconsin. The four sources collected were the following:

1) Hass Quarry in Eau Claire County – quartz diorite/gneiss
2) Lathers Pit in Rock County – crushed carbonate gravels
3) Badgerland Aggregates in Manitowoc County – crushed carbonate gravels
4) Franklin Quarry in Kenosha County – limestone/dolomite

Materials were gathered throughout WI and shipped to Oklahoma State University for evaluation of the 1.5-in. (38.1 mm) coarse aggregate requirement, adjustment of the Tarantula Curve, review of the MinT, and surface resistivity.

4.2 Literature Review of the Impact of Larger Coarse Aggregates on the Performance of Slip Formed Concrete

The nominal maximum aggregate size is used as an input in some concrete mixture design methods. Previous literature on flowable concrete states that the increase in the nominal maximum aggregate size increases the slump of the mixture for a given w/c and paste content [34, 35, 36]. These publications go on to state that because of the increase in workability then the paste content of the mixture can be reduced, and this would in turn improve the performance of the concrete at the same workability. Additionally, reduced cementitious material requirements can reduce CO_2 emissions from the production and hauling of the cementitious materials. Economic benefits may also be realized in reduced spending on cementitious materials.

It is important to realize that these previous publications only use the slump test to measure the workability of the concrete and they focused on mixtures with a slump > 4 in. (101.6 mm). Unfortunately, the slump test is not helpful to investigate slip formed concrete. Paving concrete is typically placed with a slip formed paver and the workability of this material depends on how the concrete responds to vibration [32]. Because of the inadequacy of the slump test to measure paving concrete, the Box Test was developed. By using the Box Test, previous publications show that a larger nominal aggregate size did not always improve the workability of the concrete [37].

To simplify and improve aggregate proportioning for both flowable and slip formed concrete, the Tarantula Curve was developed. This is a tool that uses three criteria to guide an aggregate gradation. First, there is a combined aggregate gradation band that limits specific sizes, next there is a minimum amount of coarse sand (the sum of the aggregate retained on the #8, #16, and #30 sieve). The final criteria is the amount of fine sand (the sum of the aggregate retained on the #30, #50, #100, and #200 sieve). All of these criteria must be met for an aggregate gradation to provide satisfactory workability. If these criteria are not met, then the mixture will not have satisfactory performance as outlined in Figure 12.

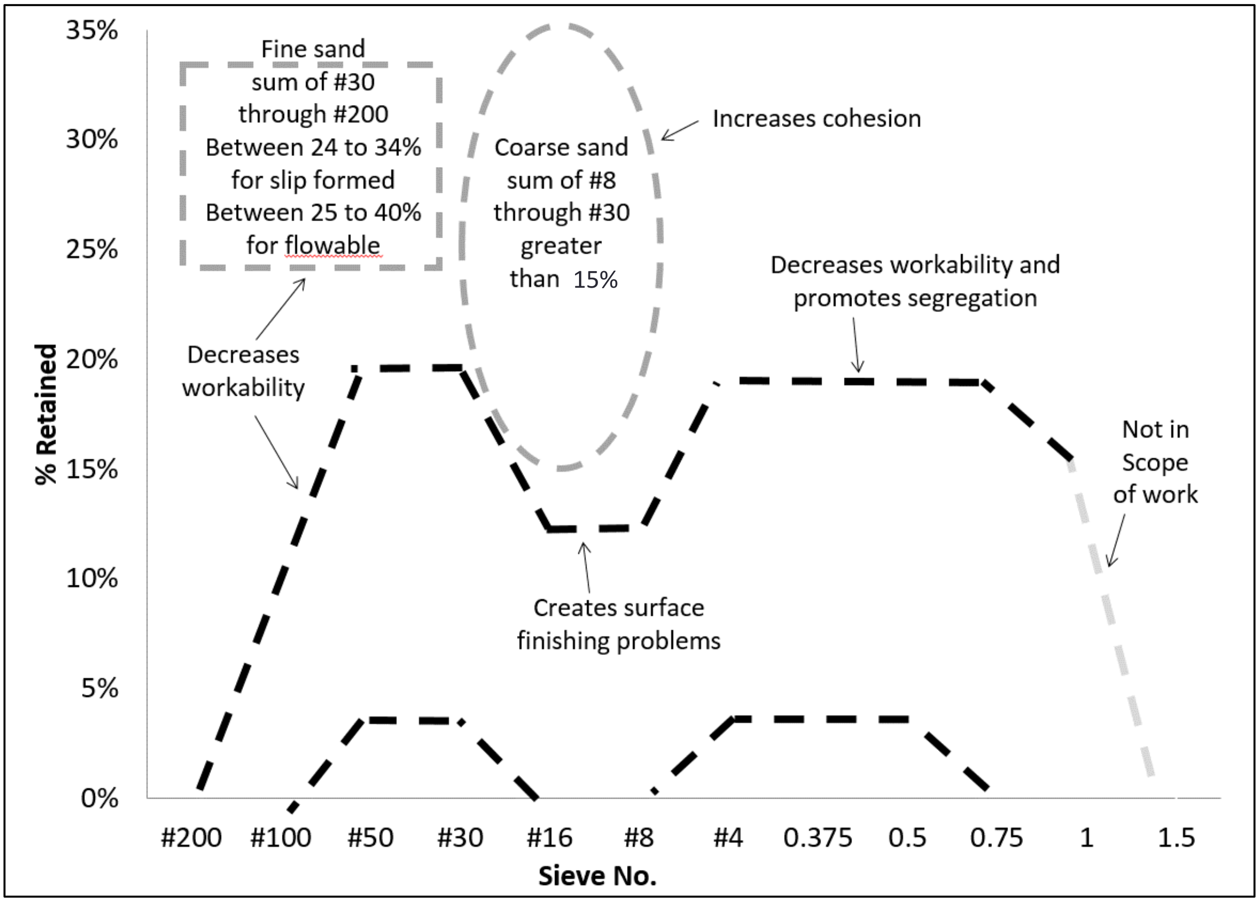


Figure 12: The Tarantula Curve with the three design criteria highlighted and the description of what will happen if these criteria are not met.

When developing the Tarantula Curve, there was only a limited amount of work that was done with larger aggregates because they are not widely used for paving projects in most states. This work aims to provide more workability, strength, surface resistivity, and shrinkage of concrete mixtures that use 1.5 in. (38.1 mm) maximum nominal aggregates. New recommendations for the Tarantula Curve are also made based on the work with larger coarse aggregates.

The research is presented in two parts. In Part I, a detailed study is done with the Haas coarse aggregate from Wisconsin and in Part II these findings are checked with three other quarries from Wisconsin. Finally, a recommendation is given about the use of larger aggregate in Wisconsin concrete mixtures used for pavements. Some testing is also done with the MinT – a miniature vibrating table used to consolidate concrete for the SAM.

4.3 Experimental Methods

This section describes the experimental methods used to evaluate the incorporation of the larger, 1.5-in (38.1 mm) aggregates into the Tarantula Curve.

4.3.1 Materials

All of the concrete mixtures in this research were prepared using a Type I ordinary Portland cement that meets the requirements of ASTM C150. A Class C fly ash that met the requirements of ASTM C618 was used at a 30% cement replacement by weight. The oxide analyses for the cement and fly ash, as well as the Bogue calculations for the cement, are shown in Table 4. All mixtures used an ASTM C494 mid-range water reducer (MRWR) Type A, F. One 1.5-in (38.1 mm). nominal size coarse aggregate, one 3/4-in. (19.1 mm) nominal size coarse aggregate and one fine aggregate were used in the mixtures. The fine aggregate was from a local source in Oklahoma, but the coarse aggregates are from Wisconsin. Sieve analysis for each of the aggregates is shown in Table 5. Pictures of the coarse aggregate are shown in Figure 13. Absorption and specific gravity of each aggregate followed ASTM C127 for a coarse aggregate or ASTM C128 for a fine aggregate. A summary of the properties of the aggregates is shown in Table 6.

Table 4: Chemical Composition of Type I Portland Cement and Class C Fly Ash.

Material	**Oxide Percentages**								**Phase Concentrations**			
	SiO_2	**Al_2O_3**	**Fe_2O_3**	**CaO**	**MgO**	**SO_3**	**Na_2O**	**K_2O**	**C_3S**	**C_2S**	**C_3A**	**C_4AF**
Cement (%)	21.1	4.7	2.6	62.1	2.4	3.2	0.2	0.3	48	24	8.1	7.9
Fly Ash (%)	25.3	19	5.2	33	7.8	2.6	3.4	0.6	-	-	-	-

Table 5: Properties and Sieve Analysis of Each Aggregate Type.

		Aggregate Source and Type								
		1.5" Nominal Max Coarse				**3/4" Nominal Max Coarse**				**Fine**
		Haas	Lathers Pit	Franklin	Badgerland Aggregates	Haas	Lathers Pit	Franklin	Badgerland Aggregates	River Sand
Percent Passing the Sieve Number	**1.5"**	87%	96%	93%	99%	100%	100%	100%	100%	100%
	1"	20%	22%	49%	45%	100%	100%	100%	100%	100%
	3/4"	3%	1%	12%	9%	99%	94%	98%	98%	100%
	1/2"	1%	0%	2%	1%	66%	18%	55%	64%	100%
	3/8"	0%	0%	1%	1%	41%	5%	30%	32%	100%
	#4	0%	0%	1%	1%	5%	1%	2%	3%	98%
	#8	0%	0%	1%	1%	2%	1%	1%	2%	92%
	#16	0%	0%	1%	1%	2%	1%	1%	2%	74%
	#30	0%	0%	1%	1%	2%	1%	1%	2%	41%
	#50	0%	0%	1%	1%	2%	1%	1%	2%	10%
	#100	0%	0%	1%	1%	2%	1%	1%	2%	1%
	#200	0%	0%	1%	1%	2%	1%	1%	2%	0%
	Pan	0%	0%	0%	0%	0%	0%	0%	0%	0%

Quarry: Haas

County: Eau Claire

Quarry: Lathers Pit

County: Rock

Quarry: Franklin

County: Kenosha

Quarry: Badgerland Aggregates

County: Manitowoc

Figure 13: Pictures of the Coarse Aggregate Used in the Testing.

Table 6: Aggregate Properties as Measured by the Research Team.

	Aggregate Source	**Bulk Specific Gravity (SSD)**	**Absorption (%)**
1.5" NM Coarse	Haas	2.72	0.88
	Lathers Pit	2.72	1.37
	Franklin	2.64	1.68
	Badgerland Aggregates	2.80	0.57
3/4" NM Coarse	Haas	2.73	1.14
	Lathers Pit	2.68	1.53
	Franklin	2.68	1.46
	Badgerland Aggregates	2.80	0.76
Fine	River Sand	2.65	0.55

4.3.2 Concrete Mixture Design

To evaluate and compare the performances of multiple concrete mixtures, the paste content and water-to-cementitious materials ratio (w/c) were held constant. The mixtures were designed to have 5.5 sacks (517 lbs. / 234.5 kg) of cementitious material per cubic yard of concrete, a paste volume of 23.2%, and a w/c of 0.42. Different mixture designs were investigated for every aggregate source. A total of 6 concrete mixtures were produced to determine the influence of gradation on specific properties of the concrete. The mixture designs for all 6 mixtures in Part I are shown in Table 7 and the mixtures investigated in Part II are shown in Table 8.

The Tarantula Curve was used to design the concrete mixtures. All the mixtures were designed to intentionally hold the paste and mid-range WR dosage constant and vary the gradations of the mixtures. Additionally, the fine aggregate proportions were held constant, while the coarse aggregate proportions were varied. This allowed the impact of the larger coarse aggregate sizes on the workability and the hardened properties to be investigated and measured. The workability was evaluated with the AASHTO TP 137 Box Test results. The Box Test measures how responsive a concrete mixture is to vibration, specifically for slip-form applications. The combined aggregate gradations of all mixtures for every aggregate type were plotted in a percent retained chart and will be presented in the results section.

Table 7: Concrete Mixture Proportions at Saturated Surface Dry (SSD) for Part I.

Mix	1.5" NM Coarse (lbs/yd^3)	3/4" NM Coarse (lbs/yd^3)	Sieved Coarse (lbs/yd^3)	Fine (lbs/yd^3)	Cement (lbs/yd^3)	Fly Ash (lbs/yd^3)	Water (lbs/yd^3)	Mid-Range WR (oz/cwt)
Haas 1	0	2050	0	1350	362	155	217	11.5
Haas 1B	0	1800	250 (3/4" size)	1350	362	155	217	11.5
Haas 2	250	1800	0	1350	362	155	217	11.5
Haas 3	500	1550	0	1350	362	155	217	11.5
Haas 4	750	1300	0	1350	362	155	217	11.5
Haas 5	1000	1050	0	1350	362	155	217	11.5
Haas 6	1250	800	0	1350	362	155	217	11.5

Table 8: Concrete Mixture Proportions at Saturated Surface Dry (SSD) for Part II.

Mix	1.5" NM Coarse (lbs/yd^3)	3/4" NM Coarse (lbs/yd^3)	Sieved Coarse (lbs/yd^3)	Fine (lbs/yd^3)	Cement (lbs/yd^3)	Fly Ash (lbs/yd^3)	Water (lbs/yd^3)	Mid-Range WR (oz/cwt)
Franklin 1	600	1350	0	1400	362	155	217	11.5
Franklin 2	1250	690	0	1400	362	155	217	11.5
Badgerland 1	690	1250	0	1400	362	155	217	11.5
Badgerland 2	1050	1000	0	1400	362	155	217	11.5
Lathers Pit 1	900	850	0	1625	362	155	217	11.5

4.3.3 Concrete Mixing Procedure

Aggregates were collected from outside storage piles and brought into a temperature-controlled room at 73°F (22.8°C) for at least 24 hours before mixing. Aggregates were placed in the mixer and spun, and a representative sample was taken for a moisture correction. At the time of mixing all aggregate was loaded into the mixer along with approximately two-thirds of the mixing water. This combination was mixed for three minutes to allow the aggregates to approach the saturated surface dry (SSD) condition and ensure that the aggregates were evenly distributed. Next, the cement, fly ash, and the remaining water was added and mixed for three minutes. The resulting mixture rested for two minutes while the sides of the mixing drum were scraped. After the rest period, the mixer was started, the water reducer was added, and the concrete was mixed for three minutes.

4.3.4 Sample Preparation and Testing

After preparing the mixture, plastic properties of the concrete were tested, which included Slump ASTM C143, Box Test (AASHTO TP-137), Unit Weight ASTM C138, and Air Content ASTM C231. Then samples were prepared for Drying Shrinkage testing ASTM C157, Compressive Strength ASTM C39, and Electrical Surface Resistivity (AASHTO T 358). The number of samples with the test method can be summarized in Table 9 and additional information about the sample preparation and testing can be found in the proceeding subsections.

Table 9: Concrete Testing Information.

Test Property	Test Method	Sample Size	Sample Count
Slump	ASTM C143	---	2
Box Test	AASHTO TP-137	---	2
Unit Weight	ASTM C138	---	1
Air Content	ASTM C231	---	1
Drying Shrinkage	ASTM C157	4 in. x 4 in. x 11.25 in. (101.6 mm x 101.6 mm x 285.75 mm)	4 (2 for mass, 2 for length)
Compressive Strength	ASTM C39	6 in. x 12 in. (152 mm x 304.8 mm)	8 (3 at 7- and 28-days, plus reserve)
Electrical Surface Resistivity	AASHTO T358	4 in. x 8 in. (101.6 mm x 203.2 mm)	6 (3 at 3-, 7-, 14-, 28-, 56-, 90-, and 120-days, plus reserve)

4.4 Results and Discussion

This section discusses the results of the Phase II lab evaluation of Wisconsin aggregates.

4.4.1 Part I – Haas Aggregates

This work aims to systematically change the coarse aggregate gradation of the Haas aggregate while holding the paste volume, WR dosage, and w/c constant and determine the impact on the plastic and hardened properties of the concrete. A systematic evaluation will be done with the Haas aggregate and this will be checked with the other aggregates in Part II. The combined individual percent retained gradations is plotted on the Tarantula Curve for Mixture 1 and 1B is shown in Figure 14 and the combined gradation for the other 6 mixtures is plotted on the Tarantula Curve in Figure 15. In all mixtures, the fine aggregate was held constant with a fine sand volume of 31% and a coarse sand volume of 24%. The total volume of coarse and intermediate aggregate was held constant in each mixture but the relative proportion of the two was changed for each mixture. This means that when the larger coarse aggregate is reduced then the intermediate aggregate must be increased. This allowed mixtures to be created with very high amounts of larger coarse aggregate and low amounts of intermediate aggregates and vice versa. By systematically changing the amount of each of these materials the changes in workability or hardened properties can be determined.

Figure 14 shows two mixtures that are very close to the Tarantula Curve. Mixture 1 is slightly outside the limits and Mixture 1B has a reduced amount of #4, 3/8 in. (9.5 mm), and ½ in. (12.7 mm) and an increased amount of ¾ in (19.1 mm). These slight changes in gradation will make a big difference in the performance of the workability of the mixture. Notice that the dashed line means the test failed the Box Test and the solid line means that it passed the Box Test.

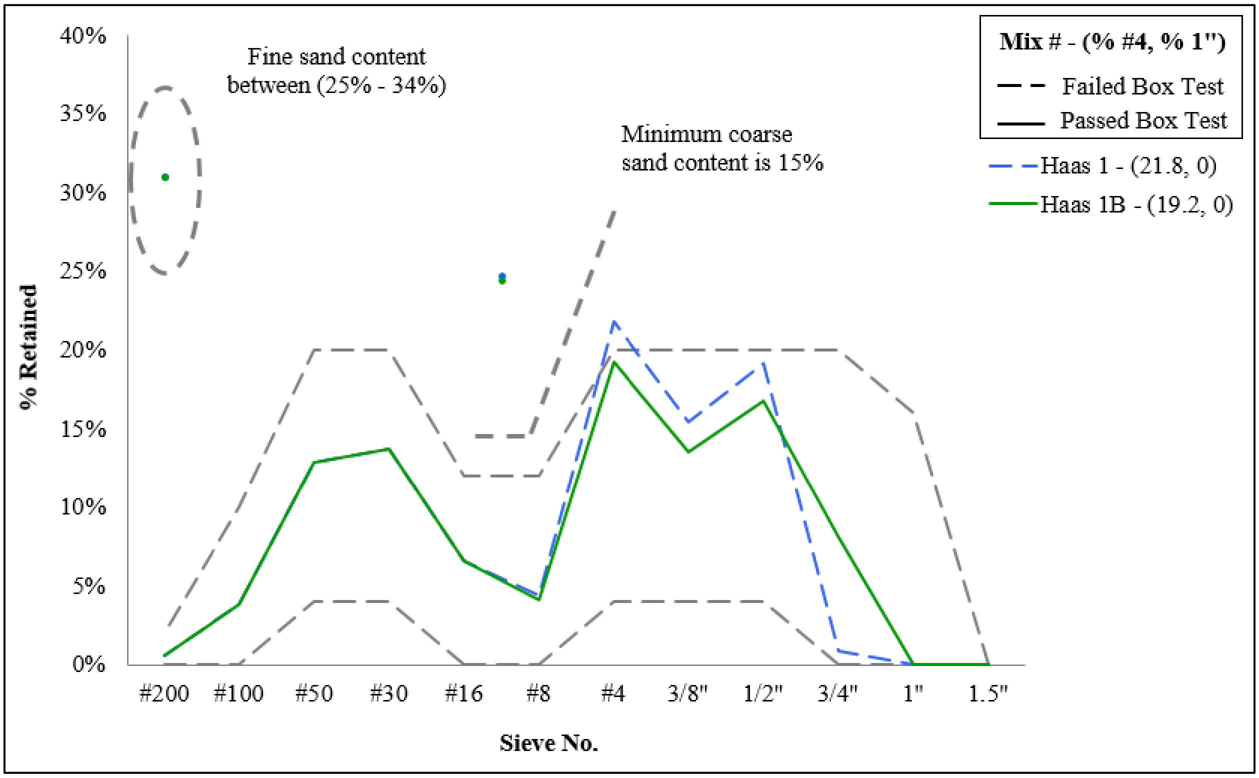


Figure 14: Mixture 1 and 1B Plotted on the Tarantula Curve. Dashed Lines Failed and Solid Lines Passed the Box Test.

As shown in Figure 15, the combined gradations of mixtures 2, 3, and 4 are within the limits set by the Tarantula Curve. Mixtures 5 and 6 both exceed the 16% limit on the 1-in. (25.4 mm) size, and mixture 1 exceeds the 20% limit on the #4 size and contains no rock larger than 3/4-in. (19.1 mm). This mixture with only 3/4-in. (19.1 mm) aggregate is expected to be less workable because of the smaller amount of larger aggregate that is used.

It should be noted that the legend in the figure denotes the mixture number followed by the percent retained on the #4 sieve size and the percent retained on the 1-in. (25.4 mm) sieve size. This code will be used in the legends of all remaining figures shown in this work. The legend also denotes the mixtures that passed the Box Test with a solid line and the mixtures that failed the Box Test with a dotted line.

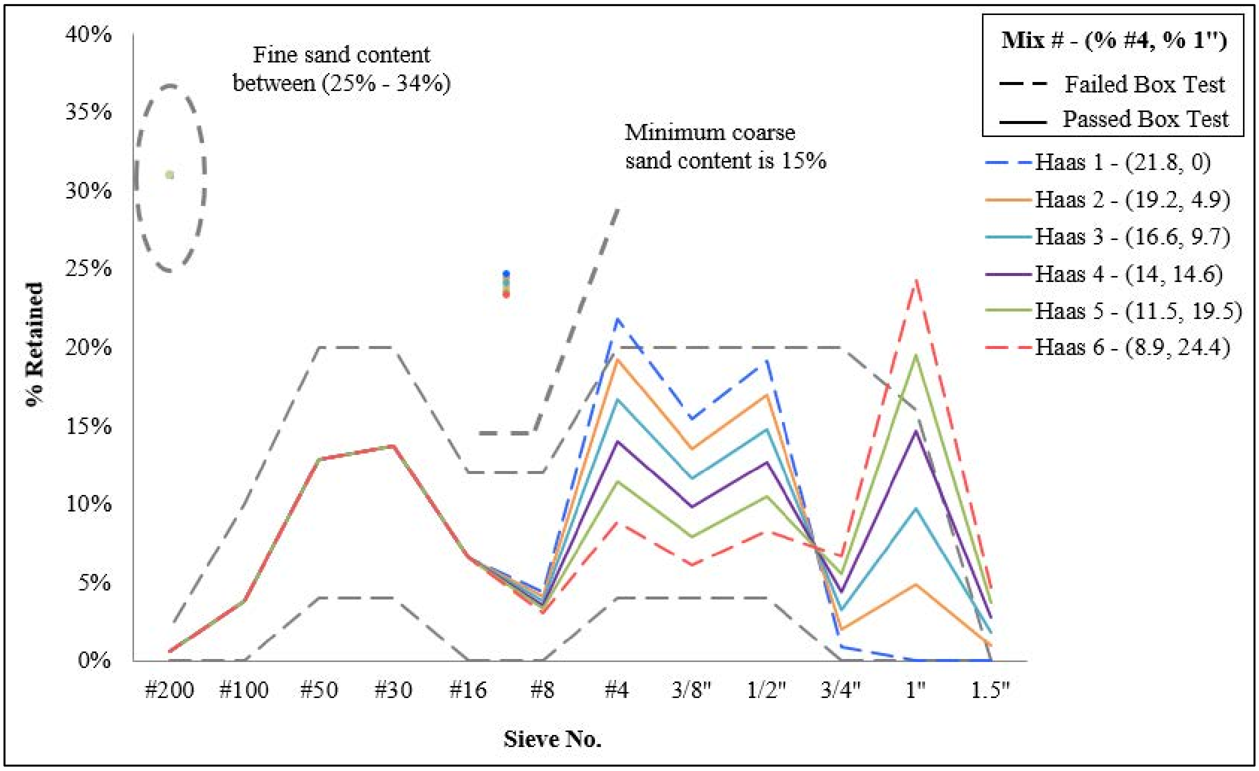


Figure 15: The Mixtures Plotted on the Tarantula Curve for Mixtures 1 through 6. Dashed Lines failed and Solid Lines Passed the Box Test.

4.4.1.1 Box Test Performance

The average Box Test results from the four sides of the box and one standard deviation are displayed in Figure 16 below. According to Figure 16, mixtures 1B and 2 through 5 passed the Box Test but mixtures 1 and 6 did not. Mixture 6 was the only mixture that exhibited edge slumping during the Box Test.

In Figure 14, the combined gradation for mixture 1 shows that the #4 size had more than 20% retained, which exceeds the limit for this size. From previous research by Cook et al., if a single sieve size of coarse aggregate (#4 and larger) retained more than 20%, the workability performance of the concrete would decrease [37]. Thus, the Box Test performance for mixture 1 reinforces this idea from the Tarantula Curve design method. In Mixture 1B some slight changes were made to the aggregate gradation and the performance in the Box Test has significantly improved. This highlights the importance of these limits and how slight changes in aggregate gradation can make a big difference in workability performance.

Mixtures 5 and 6 exceeded the limit on the 1-in. (25.4 mm) size, which is known to create workability issues in a given concrete mixture. From Cook et al.'s previous research, the Tarantula Curve limit for the 1-in. (25.4 mm) sieve size was found to be 16%. As previously shown in Figure 15, mixture 5 had 19.5% retained on the 1-in. (25.4 mm) size, while mixture 6 had 24.4% retained on the 1-in. (25.4 mm) size. Mixture 5 passed the Box Test, but mixture 6 did not. This means that for these materials and mixtures there is a limit for the 1-in. (25.4 mm) size that is between 19.5% and 24.4% retained. The mixtures in this study used a lower w/c, higher water reducer dosage, more fly ash, and a different coarse aggregate source than the previous study and so this may impact why the limit is slightly different between the mixtures.

Edge slumping was exhibited in mixture 6. As stated before, this mixture had low amounts of intermediate sizes and a high amount retained on the 1-in. (25.4 mm) size. Intermediate aggregate sizes provide cohesion to concrete mixtures. Since the mixture was lacking in these sizes, the mix was less cohesive which led to edge slumping during the Box Test.

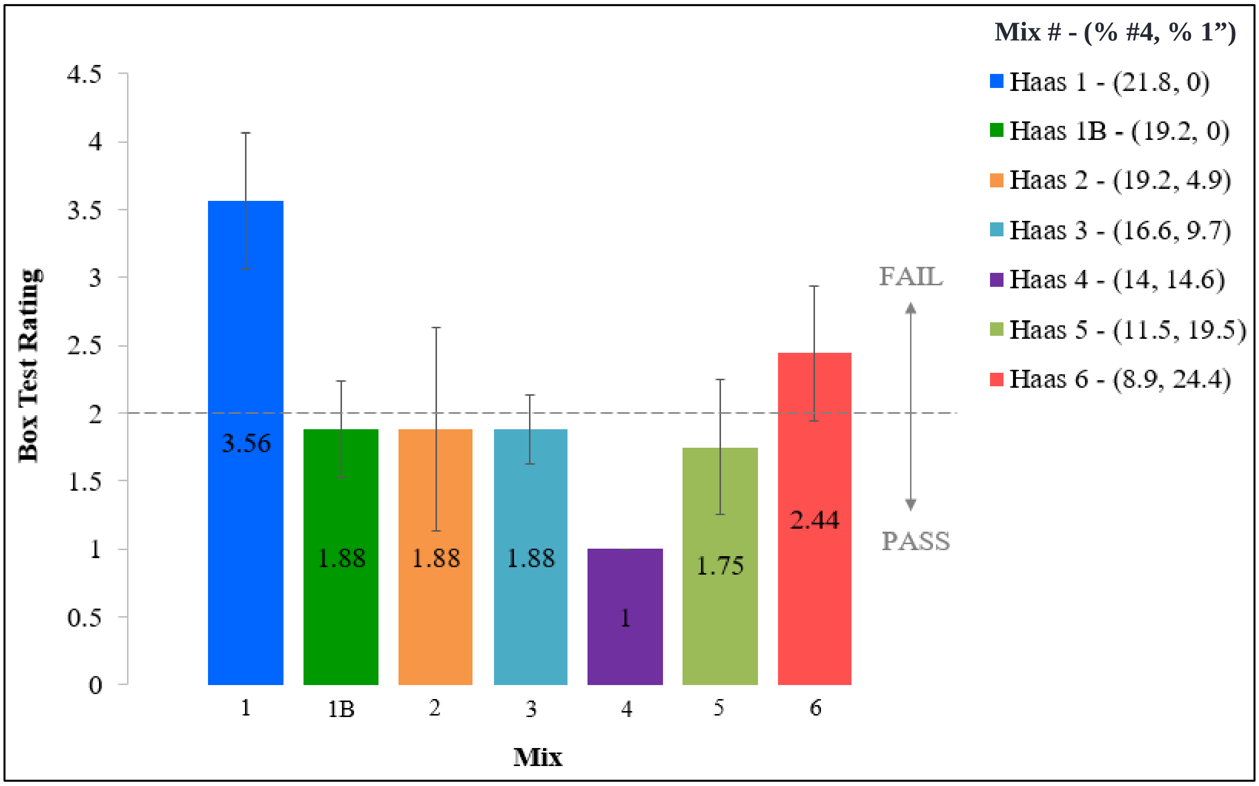


Figure 16: Average Box Test of all Mixtures.

4.4.1.2 Slump Test Performance

The Slump Test results for the mixtures are displayed in Figure 17 below. The standard deviations for the slump results are only reported for mixtures 1, 1B, and 6. This is due to only one slump measurement being collected for the other four mixtures. The standard deviation for mixture 1 is zero, which is why no error bars are shown in Figure 17.

Figure 17 shows that mixtures 1, 1B, 2, and 3 had the lowest slumps. When looking at the combined gradations in Figure 14 and Figure 15, these mixtures had the highest amount of intermediate aggregate out of all 6 mixtures. Mixtures 1B, 2, and 3 are approaching the limit for the Tarantula Curve on the #4 size, and mixture 1 exceeds the #4 size limit. According to the Tarantula Curve, excessive amounts of intermediate sizes decrease workability and promote segregation. This could be causing the lower slump results for these mixtures. Mixture 6 had the highest slump of 1.75 in. (44.5 mm), but it is important to note the high standard deviation of the slump measurement (0.71 in. / 18 mm). This large standard deviation is likely caused by the segregation caused by the high amount on the 1" (25.4 mm) sieve. Research by Ghazal et al. showed that concrete mixtures with 20% coarse material retained on a single sieve size created a mixture with a non-uniform aggregate distribution [38]. X-ray CT scans were able to observe areas with high concentrations of aggregates and other areas with almost no aggregate. This poor distribution of coarse aggregate will cause segregation within the concrete and poor response to vibration; however, there may be an increase in the slump because the material may collapse when the walls of the slump cone are removed. Past research has shown that the slump of concrete does not accurately represent how it will consolidate and finish in the field, which are important aspects of pavement concrete [32, 37].

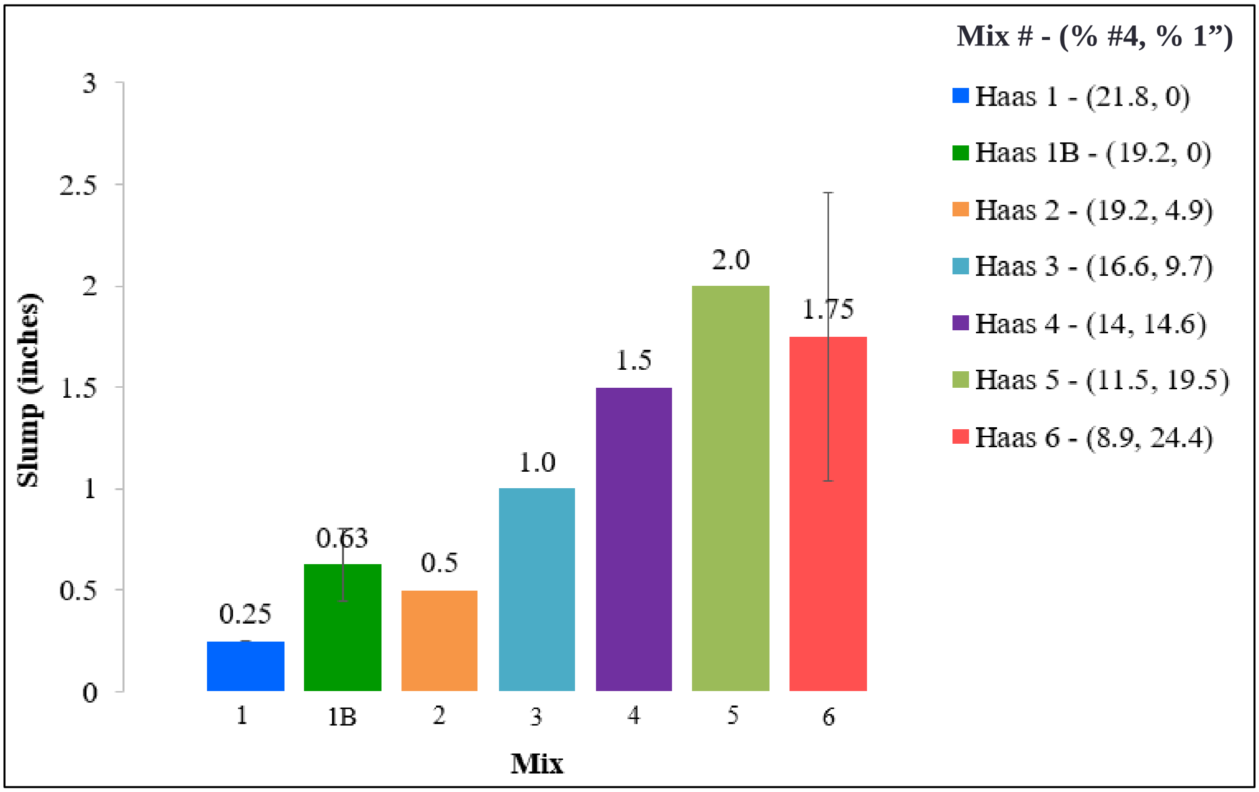


Figure 17: Average Slump Test Results for the Mixtures.

4.4.1.3 Compressive Strength

The average compressive strength and standard deviation are shown in Figure 18. The average strength of all the mixtures is depicted as a solid black line, and the average plus or minus the standard deviation is depicted as a dotted gray line. This is used as a method to make a relative comparison between the different mixtures. A Student's t-test was used to determine which mixtures had compressive strength values that were statistically different than the average strength. At 28 days, mixture 6 was the only mixture found to have an average compressive strength that is statistically lower than the average. This mixture also had the highest amount of larger stone in the mixture. This shows that mixtures with higher amounts of coarse aggregate did not increase the compressive strength.

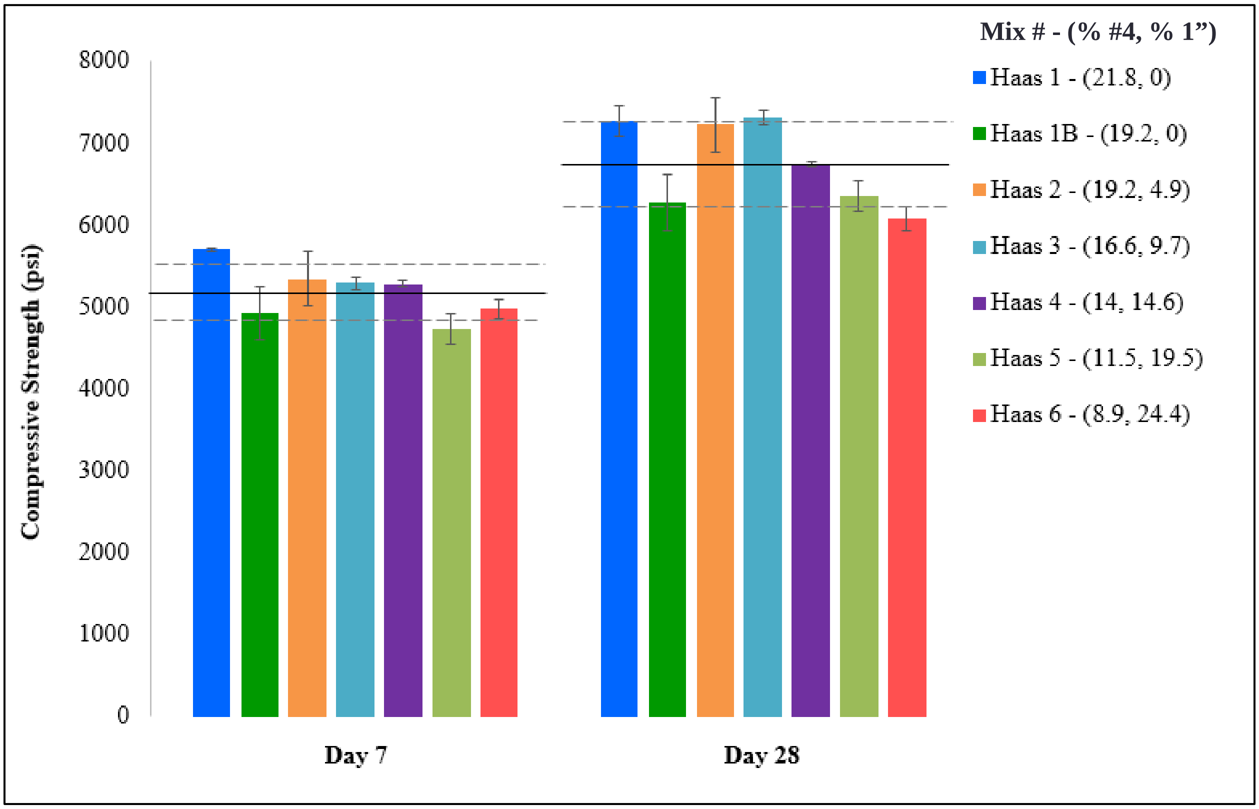


Figure 18: Average Compressive Strength of the Mixtures on Each Day of Interest.

4.4.1.4 Electrical Surface Resistivity

The electrical resistivity results for the mixtures are shown in Figure 19. The average apparent surface resistivity is depicted as a solid black line, and the average plus or minus the standard deviation is depicted as a dotted gray line. A Student t-test was used to investigate if any mixtures were statistically different than the average. The Student t-test found no difference in the measurements. This shows that there is no significant difference in the electrical apparent surface resistivity of a mixture regardless of the amount of large aggregate in the mixture.

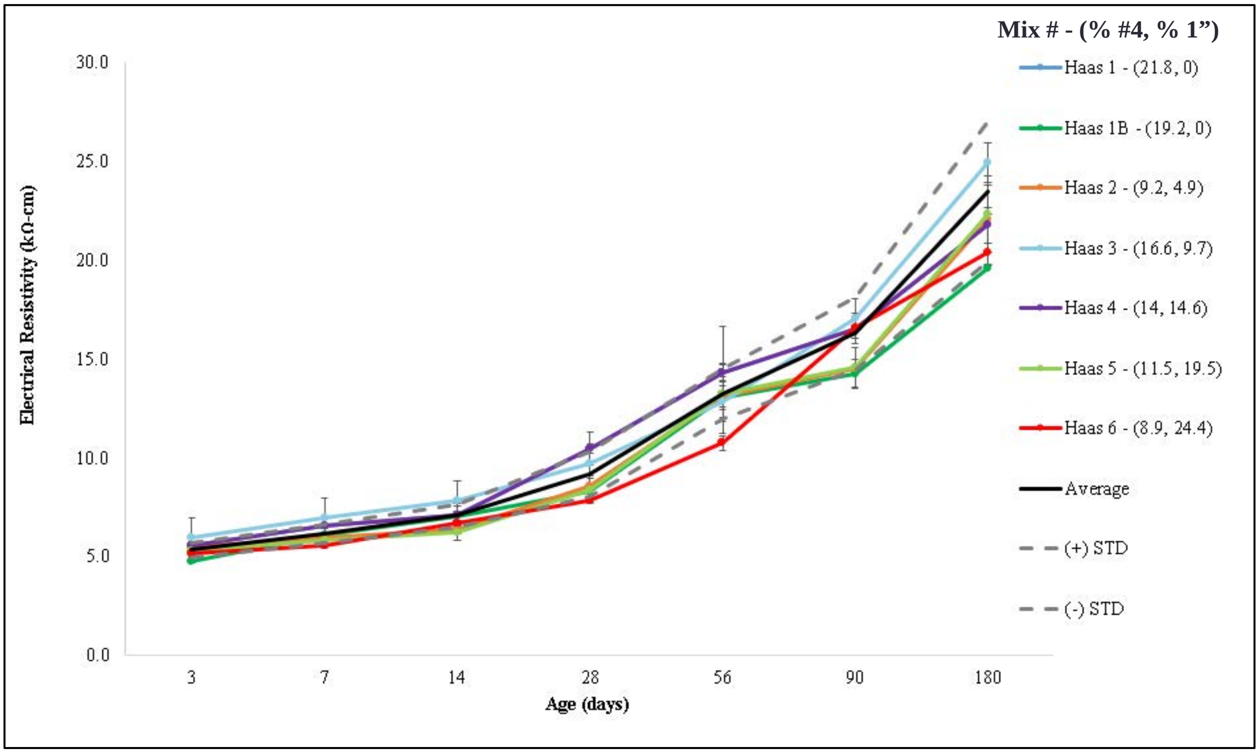


Figure 19: Average Apparent Surface Resistivity Measurements for Mixtures on Each Day of Interest.

4.4.1.5 Mass Change from Drying

The average mass change and standard deviation from drying are presented in Figure 20. The increase in mass during curing is also shown. The average mass loss of all the samples is shown by a solid black line and the standard deviation is shown by a dashed line. A Student t-test at 145 days of drying showed that there was no statistical difference in the mass loss between any of the samples. This shows that there is no significant difference in the mass loss from drying of a mixture with a constant paste volume and w/c, regardless of the amount of large aggregate in the mixture.

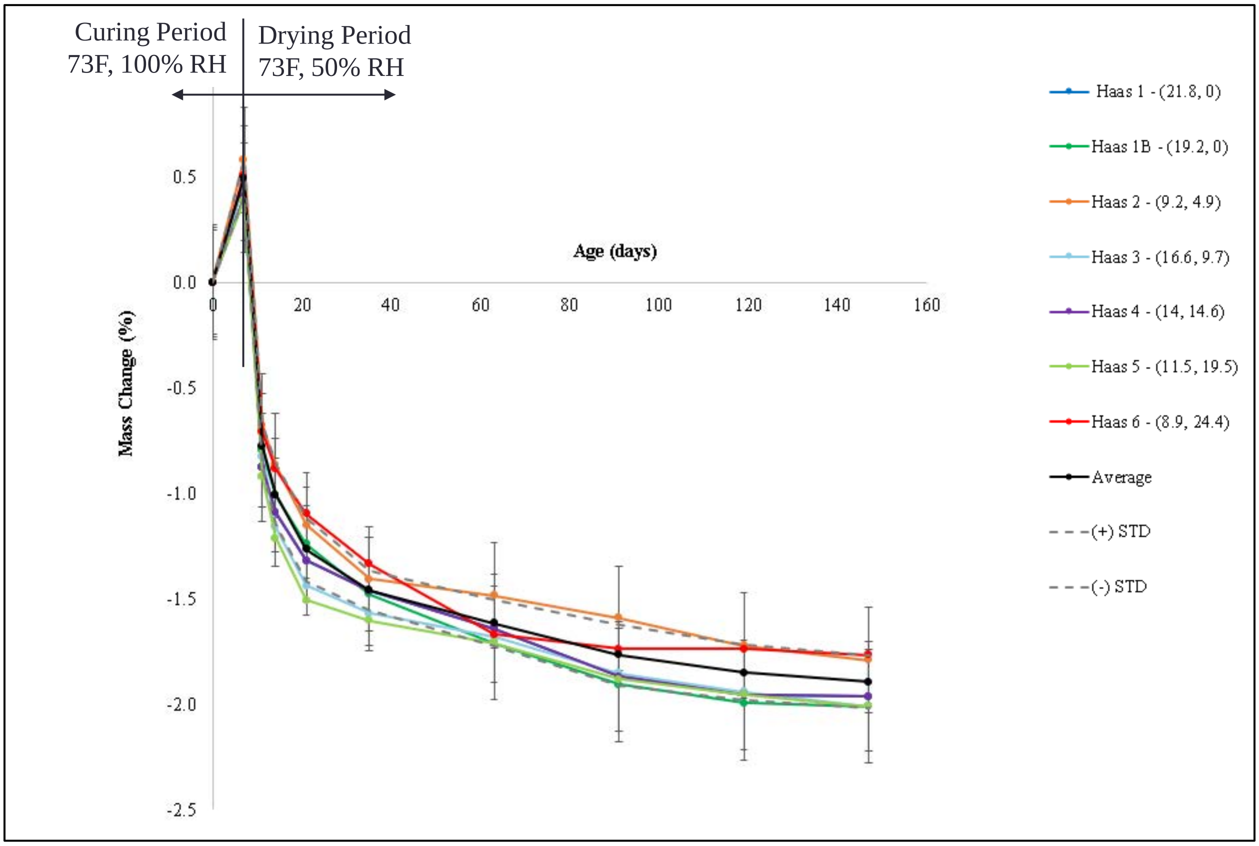


Figure 20: The Mass Change of the Mixtures on Each Day of Interest from Drying for the Testing in Part I.

4.4.1.6 Shrinkage Strain

The shrinkage strain from storage in a 50% relative humidity and 73°F (22.8°C) room as measured by the strain gauges over the first 90 days is shown in Figure 21. The gauges measure initial strains within the concrete that occur as it is being placed in the molds and during hydration. After the initial strains, the gauges continue measuring the strains during the curing process and the drying process. The strain gauge readings show that all of the samples expanded during the curing period, and then began to shrink after the samples were placed in the drying room. The drying room is maintained at 73°F (22.8°C) and 50% relative humidity.

Since the strain was found to be almost constant after 90 days, the datalogger was used for other experiments. To learn more about the long-term shrinkage, the data logger was used to measure the samples after about 200 days of drying. Because the samples were cast at different times it was necessary to interpolate some of the measurements. Because of this interpolation, it was not possible to calculate a standard deviation directly and so instead the mean absolute deviation was used. The mean absolute deviation uses the standard deviation at two different points to estimate the standard deviation between these two measurements. The results are shown in Table 10. Because a standard deviation could not be calculated then a Student t-test could not be used. However, all of the average strain measurements are within 8% of one another and so this is considered to be similar.

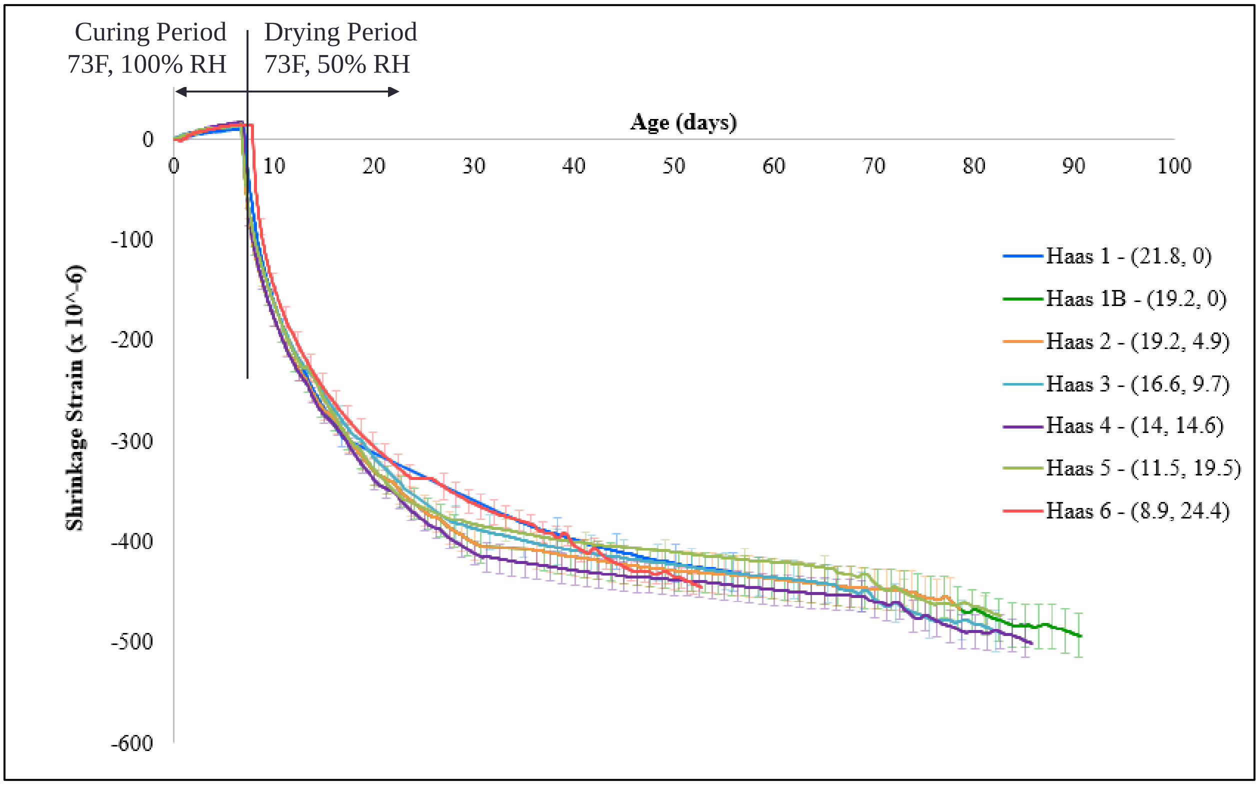


Figure 21: Average Shrinkage Strains from Mixtures During Curing and Drying Periods.

Table 10: The 200 Day Shrinkage Measurements and the Mean Absolute Deviation for the Seven Different Mixtures from Part I.

Sample	200 Day Shrinkage	Mean Absolute Deviation
Haas 1	495	24
Haas 1B	490	19
Haas 2	493	22
Haas 3	497	26
Haas 4	498	27
Haas 5	462	9
Haas 6	503	32
Average	491	23

4.4.2 Summary of Part I – Haas Aggregates

This work used Haas aggregates to provide additional insights into how using aggregate gradations of 1-in. and larger changed the performance of a concrete mixture. The following conclusions were made from this work:

- Using more than 20% coarse aggregate on the #4 and 1-in. (25.4 mm) sieve sizes created mixtures with poor Box Test performance.
- The slump of concrete mixtures increased as the percent retained on the #4 sieve size decreased but this did not correspond to performance in the Box Test.

- Significant changes in the aggregate gradation did not change the compressive strength, electrical surface resistivity, mass change during drying, or the shrinkage of the concrete mixture.

These findings show that despite there being a large difference in the amount of 1-in. (25.4 mm) and larger stone in a concrete mixture that there is no measurable difference in the compressive strength, electrical surface resistivity, mass loss from drying, and drying shrinkage. This shows that the presence of larger maximum nominal aggregate sizes does not change the physical properties of these concrete mixtures when the paste volume and w/c are held constant. Also, the work shows that the amount of 1-in. and larger stone can impact the workability of the concrete mixture, but this only occurs when these limits exceed the boundaries of the Tarantula Curve.

4.4.3 Validation with Other Aggregate Sources

In this section, the mixtures and testing will be repeated with three other coarse aggregate sources from Wisconsin. These mixtures will use the highest and lowest amount of large aggregate possible while still being within the Tarantula Curve to compare the performance. This means if the change in the amount of large stone impacts the workability or physical properties of the concrete then there should be a large difference in the behavior between the mixture with a high amount of 1-in. (25.4 mm) aggregate retained and a low amount of 1-in. (25.4 mm) aggregate retained. Because of the gradation of the Lathers Pit, it was only possible to investigate a mixture with a large amount of stone retained on the 1-in. (25.4 mm) sieve. Two of the Haas mixtures were included so that the results from Part I could be compared to Part II. Since the workability and physical testing have already been discussed, the methods used to complete the testing will be presented without significant discussion.

4.4.3.1 Box Test Performance

Figure 22 shows the combined gradations compared to the Tarantula Curve. Since none of the lines in Figure 22 are dashed, this means that there are no mixtures that failed the Box Test. Figure 23 gives the detailed measurements for the Box Test. This shows that the limits of the Tarantula Curve are useful to predict the performance in the Box Test, which is supported by previous research [12, 13, 14, 15, 16]. Also, it is important to emphasize that there was no difference in workability performance despite there being drastically different amounts of stone retained on the 1-in. (25.4 mm) sieve.

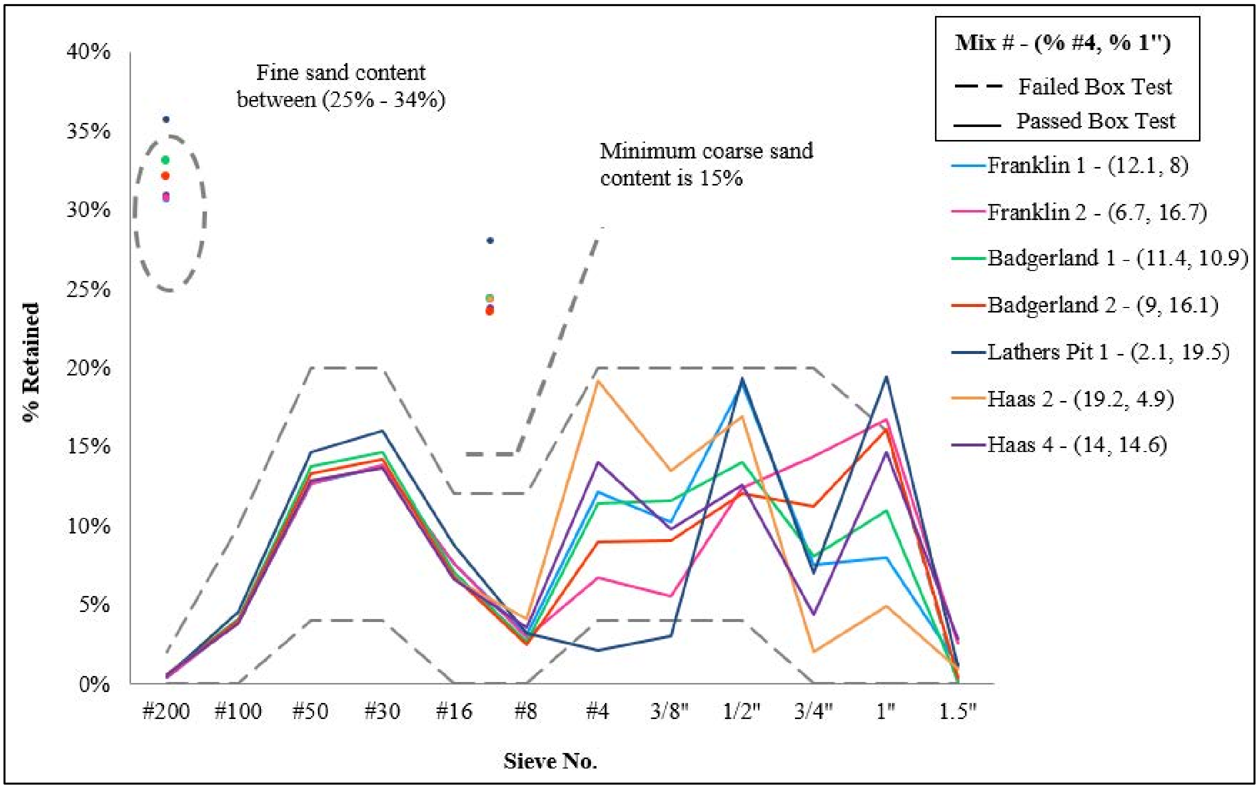


Figure 22: The Mixtures Plotted on the Tarantula Curve for the Mixtures from Part II. Dashed Lines Failed and Solid Lines Passed the Box Test.

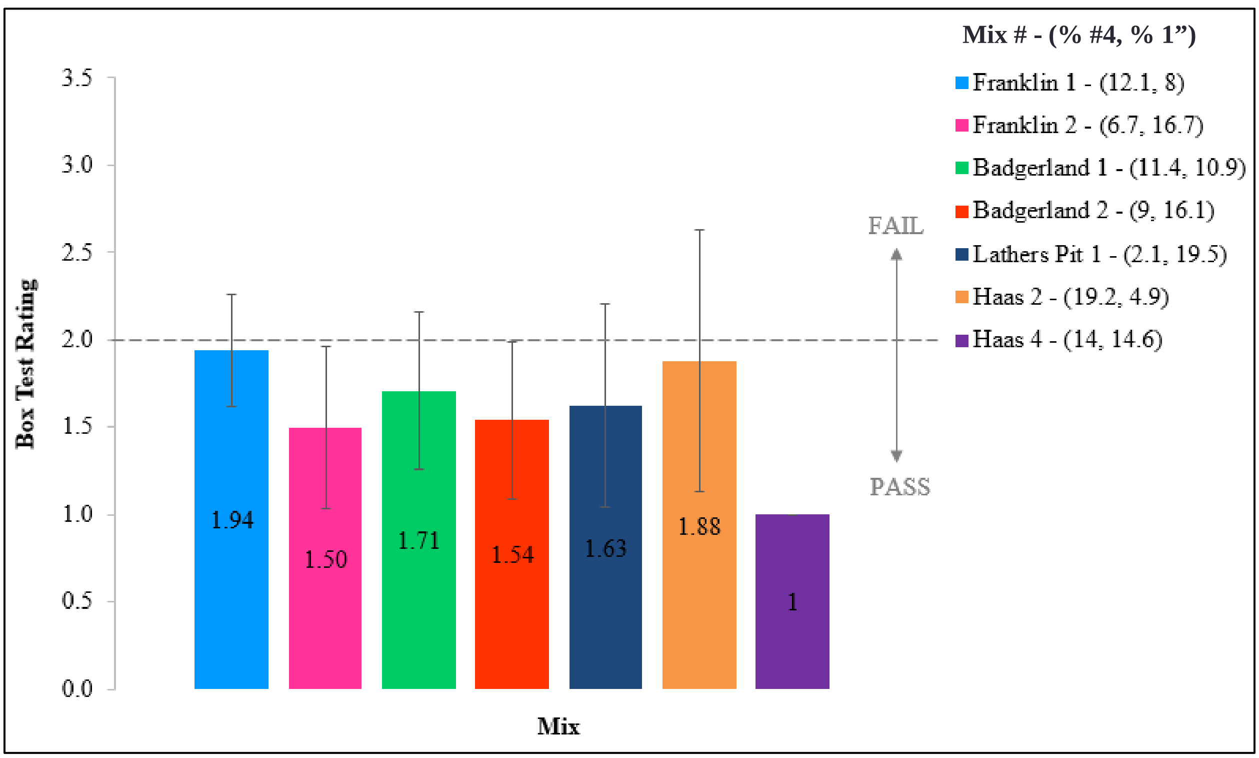


Figure 23: Average Box Test Results from Part II.

4.4.3.2 Slump Test Performance

Figure 24 shows the slump measurements. Currently, WisDOT has a maximum limit for slip formed paving of 2.5 in. (63.5 mm) (Section 415.2.1 Concrete Materials 9 paragraph 3). Note that the slump measurements are variable and that three of the mixtures have a slump higher than 2.5 in. (63.5 mm). This means that several of these mixtures would not be allowed by WisDOT, but they are shown to have satisfactory performance in the Box Test. It will also be shown that these same mixtures will have satisfactory physical properties. This emphasizes that the slump limits used by WisDOT do not represent performance in concrete.

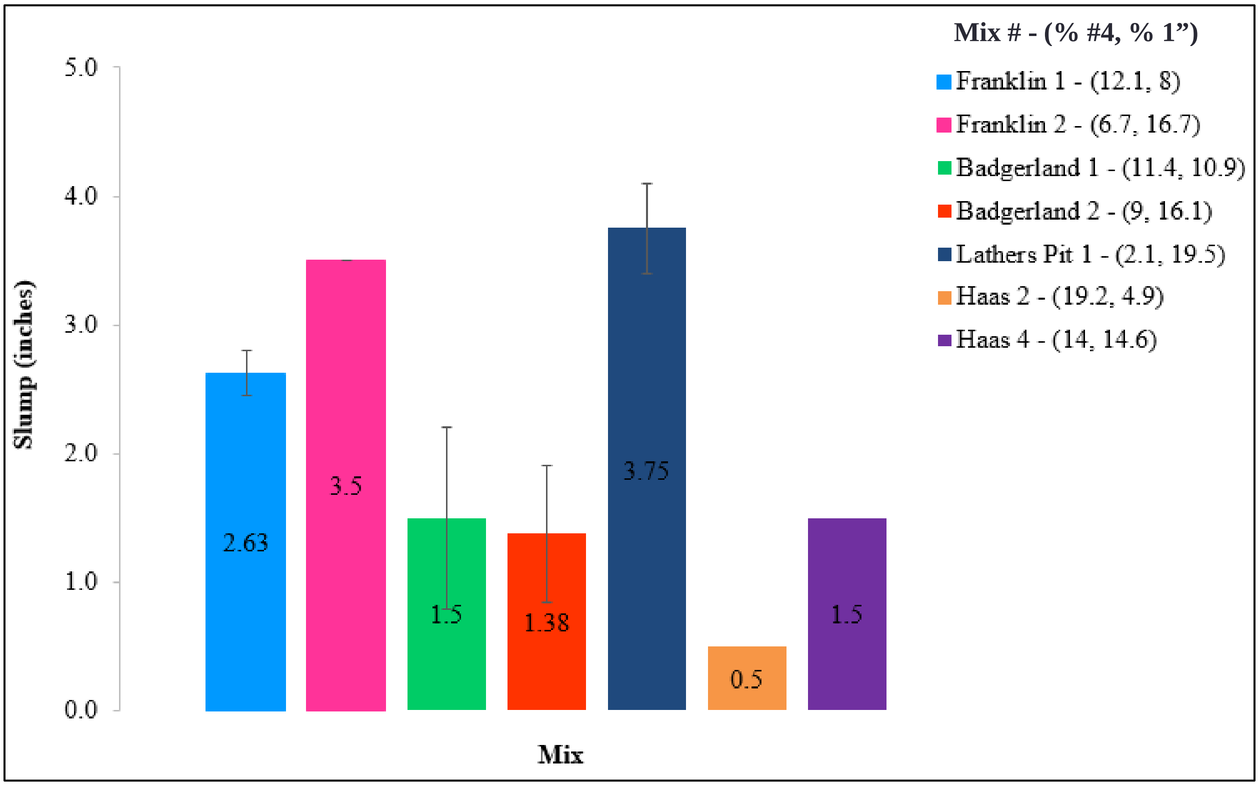


Figure 24: The Slump Test Results for the Mixtures in Part II.

4.4.3.3 Compressive Strength

The compressive strength data is shown in Figure 25. At 7 days the data varied from 4,000 psi to 5,500 psi (27.6 MPa to 37.9 MPa) and at 28 days the data varied from 6,500 psi to 8,500 psi (44.8 MPa to 58.6 MPa). It is not surprising that these strengths are variable because of the different properties of the coarse aggregates that are used. Since the coarse aggregates make up roughly 33% of the volume of a concrete mixture then changes in the strength of the coarse aggregate will change the resulting strength of the concrete. Despite this variation, by comparing mixtures with the same aggregates and high and low amounts of 1 in. (25.4 mm) retained there is minimal difference in the observed strength. This reinforces the findings from Part I, where it was clearly shown that despite changes in the amount of larger stone in a mixture that there is no difference in the strength of the concrete for a constant w/c and paste content.

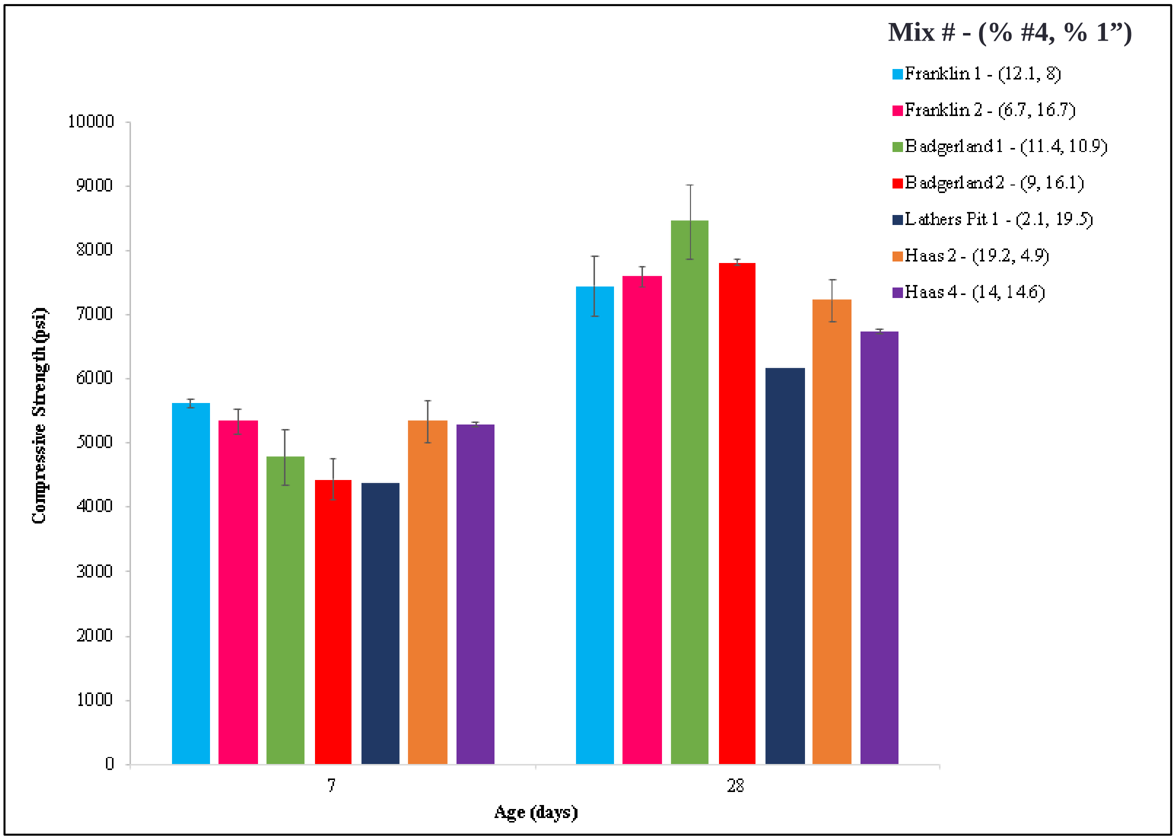


Figure 25: Average Compressive Strength of the Mixtures on Each Day of Interest.

4.4.3.4 Electrical Surface Resistivity

Figure 26 shows the apparent surface resistivity of the different aggregates and their performance. The results are very similar over the first 28 days and then there are some differences in the apparent surface resistivity at 90 days and 180 days. While it is hard to know what causes these differences, all of the apparent surface resistivity measurements are increasing over time and would be much higher than a mixture that only contained Portland cement. If you compare the results from the mixtures that use the same aggregates, but different amounts retained on the 1-in. (25.4 mm) sieve then the performance is almost identical between the mixtures. This again highlights that the use of different amounts of the larger aggregate does not impact the properties of the concrete if the w/c and paste volume is held constant.

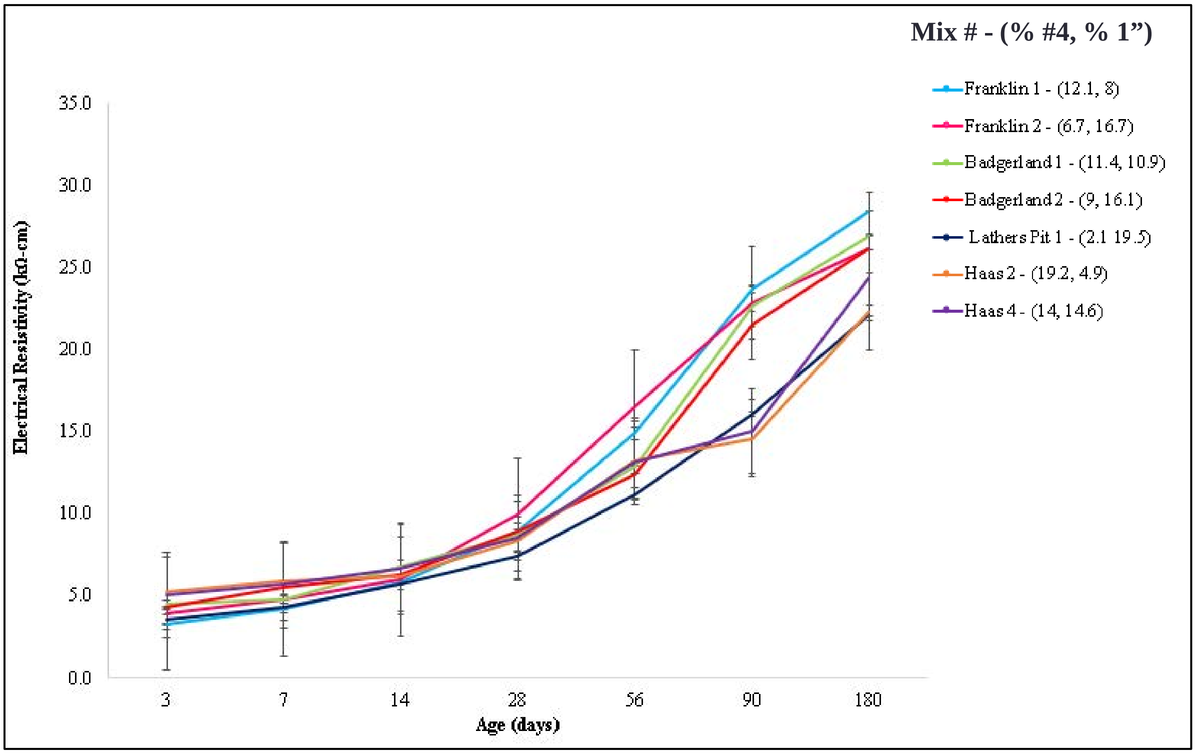


Figure 26: Average Apparent Surface Resistivity Measurements for Mixtures on Each Day of Interest.

4.4.3.5 Mass Change from Drying

The mass loss from drying is shown in Figure 27 and the drying subsequent drying shrinkage at 200 days is shown in Table 11. The mass change over time shows that there is a significant difference between several of the mixtures; however, the 200 days shrinkage is comparable between the mixtures.

The reader should remember that these mixtures use different aggregates, and these aggregates have different absorption and also different stiffness. If an aggregate has more absorption capacity, then that means that it can contain more moisture within the pores. This means that when the concrete dries that there will be more moisture loss when higher absorption aggregate is used. Based on Table 6, the Lathers and Franklin aggregates have the highest absorption at 1.37% and 1.68% and the mixtures with these aggregates also have the highest mass loss in Figure 27.

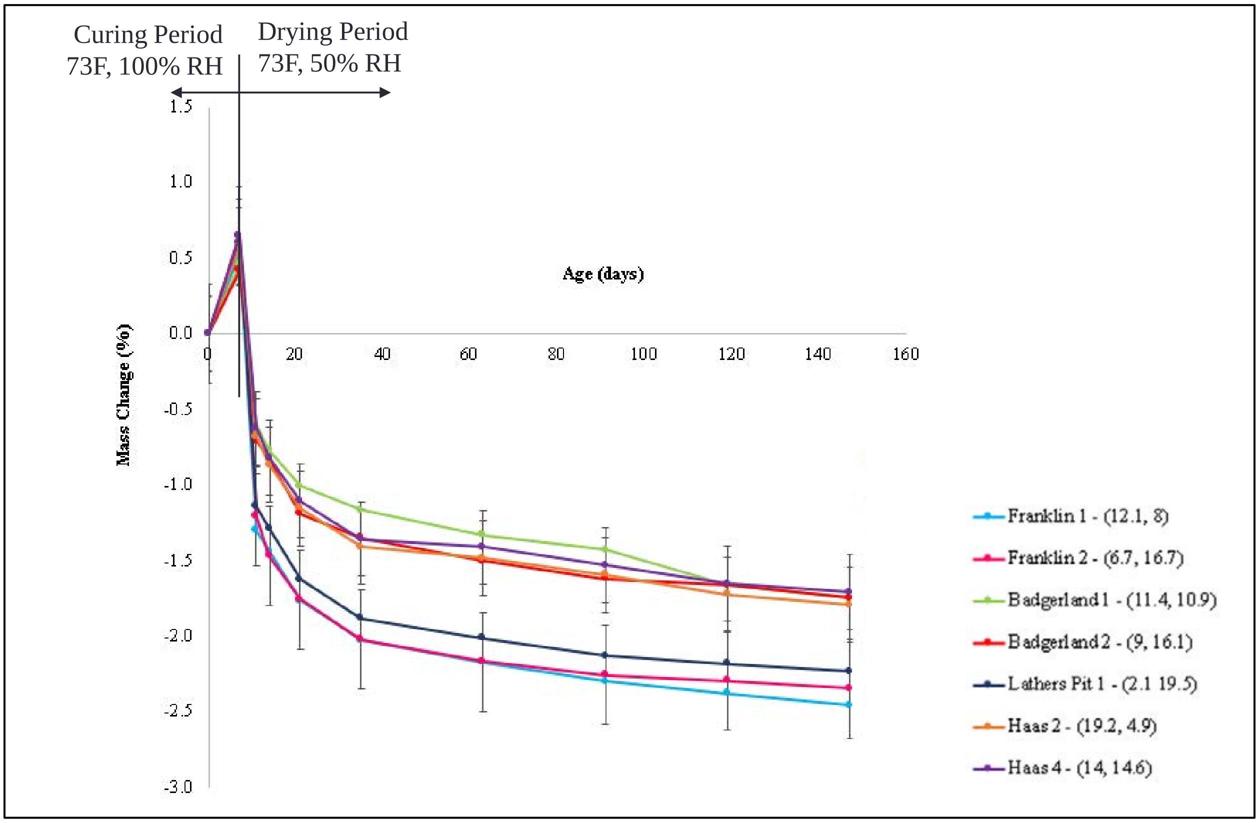


Figure 27: The Mass Change of the Mixtures on Each Day of Interest from Drying for the Testing in Part II.

Table 11: The 200 Day Shrinkage Measurements and the Mean Absolute Deviation for the Mixtures from Part II.

Sample	200 Day Shrinkage	Mean Absolute Deviation
Franklin 1	407	64
Franklin 2	414	57
Badgerland 1	439	32
Badgerland 2	479	8
Lathers Pit 1	478	7
Haas 2	493	22
Haas 4	498	27
Average	458	31

All of the aggregates investigated have a similar 200-day shrinkage except the shrinkage for Franklin is lower. The Franklin and Hass aggregates have roughly a 20% difference in shrinkage. This difference is likely because of the differences in stiffness between the aggregate sources. The aggregates in a concrete mixture restrain the shrinkage of the paste as it dries. If the aggregates have different stiffness, then this will cause differences in the measured shrinkage [39, 40, 41]. However, if one compares the shrinkage results from mixtures that use the same aggregate source but with different amounts of aggregate retained on the 1-in. (25.4 mm) sieve, the differences are all less than 9% and most have a difference of about 2%. This highlights that there is minimal difference in performance for mixtures that use different amounts of material retained on the 1-in. (25.4 mm) sieve.

4.5 Recommended Changes

Based on the testing in this work the following recommendations are made:

1. The slump limit of 2.5 in. (63.5 mm) in the current WisDOT specification for slip formed concrete should be removed. Mixtures with a slump higher than this were shown to have satisfactory performance in the Box Test. Also, this work shows that the slump did not correlate to the Box Test. It is recommended to require the Box Test results during the trial batching and that the visual ranking be less than 2.0. The Box Test could also be used in test pours or trial batching in the field to verify that the material responds to vibration. The slump test may be able to be used for consistency of slip formed concrete, but this was not a focus of this work.
2. The work showed that there are minor differences in the Box Test, strength, surface resistivity, or shrinkage of mixtures with the same paste content and w/c but different amounts of large aggregate in the mixture. Based on this work, the WisDOT requirements of requiring a minimum amount of coarse aggregate retained on the 1.5 in. (38.1 mm) sieve is not improving the performance of the concrete. Instead, it is recommended that the only requirement is that the aggregate gradation meets the Tarantula Curve limits.
3. Based on the findings in this work, new boundaries are recommended for the Tarantula Curve for using up to 1.5 in. (38.1 mm) diameter stone. This new Tarantula Curve is shown in Figure 28, and the limits of which are listed in Table 4. This new curve places a limit of 5% on the 1.5 in. (38.1 mm) sieve size. Please note, this new curve and warning band are used in all Tarantula Curve graphs and analysis throughout this report. While it may be possible to use higher amounts of material on the 1.5 in. (38.1 mm) sieve, this is the limit of the testing in this project.

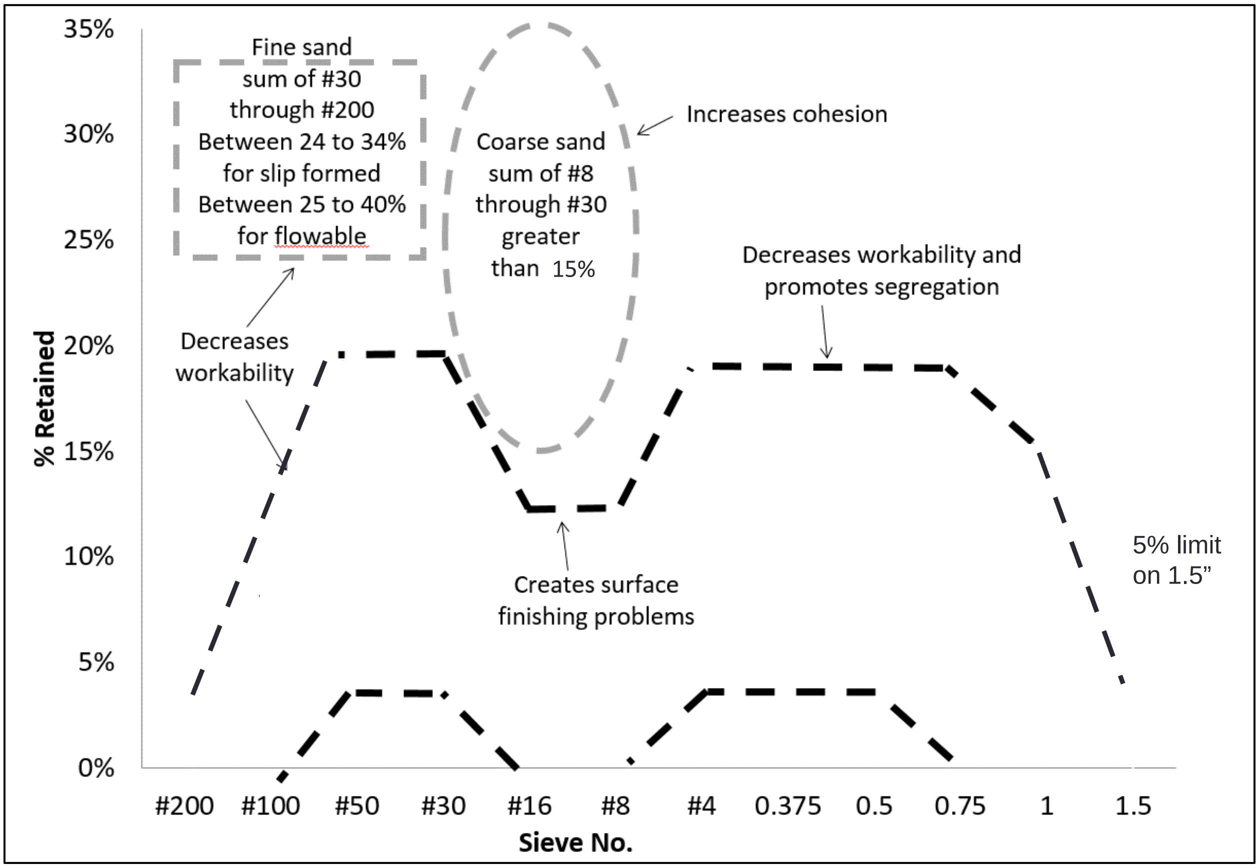


Figure 28: The Tarantula Curve Limits are Adjusted for Slip Formed Concrete that Uses Up to 1.5-in. (38.1 mm) Stone in the Concrete Mixture.

4.6 Use of the MinT with Wisconsin Materials

Previous research by Finnell showed that the variability of the air volume, unit weight, and SAM Number is increased for low workable mixtures, such as slip formed pavements [42]. It should be emphasized that the research found that the average didn't change with the different consolidation methods but the variation or spread in the measurements did. This means that if only a few measurements are taken then the results may be further apart than what is anticipated by the user. It also means that more measurements are needed to determine an accurate measurement. This can be problematic because field concrete is not currently sampled with a large number of replicates to determine an appropriate average value.

In the previous research to establish the MinT, a mixture was prepared with Oklahoma aggregates with a ¾ in. (19.1 mm) maximum nominal aggregate size. Mixtures were made that showed satisfactory and unsatisfactory performance in the Box Test. These concretes were tested with internal vibration, the MinT, and conventional rodding. The MinT is a platform that vibrates the SAM with a special platform that uses a battery powered vibrator. The vibrator allows the bottom plate to act as a vibrating table and help consolidate the concrete. Table 12 shows the results from the research with the ¾ in. (19.1 mm) maximum nominal aggregate size. As can be seen in the table the average values from the three measurements for the SAM are practically the same but the standard deviation for the results with the MinT is 50% lower than the other two consolidation methods.

Table 12: Summary of the Average SAM Number and Standard Deviations for Several Replicate SAM Tests That Use Different Methods of Consolidation for ¾ in. (19.1 mm) maximum nominal size aggregates [42].

	Number	Avg SAM	Standard Dev	COV (%)
Rodding	33	0.19	0.075	39
Internal Vibration	13	0.21	0.078	37
MinT	35	0.19	0.051	26

In this work, a concrete mixture with the Lathers aggregate was created that had a Box Test value that is above the limit and a slump of 1.5 in. (38.1 mm) A w/c of 0.42 is used with 517 lb/cy (306. 7 kg/m^3) of the total binder with a 30% fly ash replacement. The total aggregate gradation plotted on the Tarantula Curve is shown in Figure 29. This mix is harsh as it did not perform well in the Box Test. More than 15 measurements were made for the Unit Weight, air volume, and SAM Number and the standard deviation and coefficient of variation are compared between the mixtures and is shown in Table 13.

The results show that the standard deviation is practically the same for the air volume and the SAM Number for both mixtures from the previous research done with Oklahoma materials and the current research done with Wisconsin materials. It should be noted that the standard deviation for the unit weight container is higher for the Wisconsin materials. This suggests that the larger stone impacts the measurement of the unit weight but not the air volume or the SAM Number. Since the unit weight is not currently used to accept or reject concrete then this increase in variation may not be important.

Based on these findings, for mixtures that have a failing Box Test, it is recommended to use the MinT to decrease the variability of the measurement of the air volume and the SAM Number. This lower variability will help ensure that contractors and Department tests have the highest probability of correlation and best represent the concrete being produced.

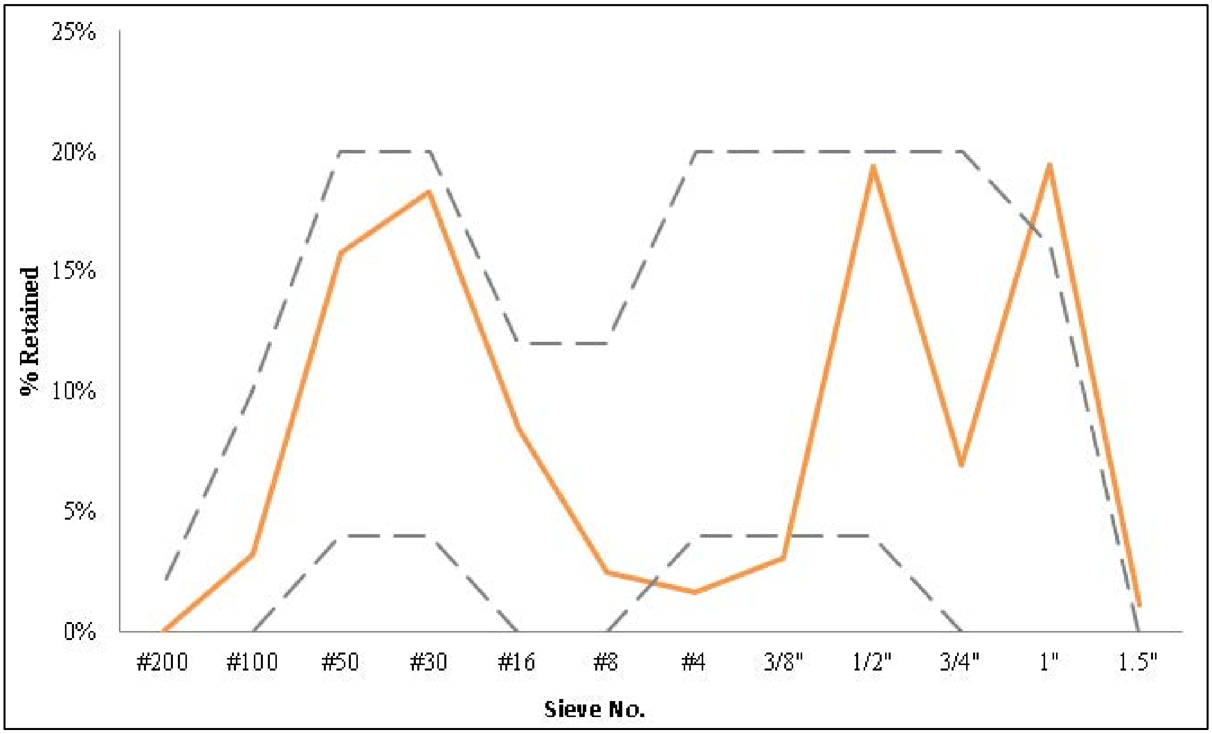


Figure 29: The Combined Aggregate Gradation Used for the Testing.

Table 13: Comparison of the SAM, Air Volume, and Standard Deviation for the Mixtures with Different Amounts of Large Aggregate.

	Number	Avg. SAM	Standard Dev	COV (%)
3/4" NMA	36	0.19	0.051	26
1" NMA	15	0.25	0.060	26
	Number	Avg. SAM	Standard Dev	COV (%)
3/4" NMA	35	5.3	0.43	8
1" NMA	19	8.2	0.39	5
	Number	Avg. SAM	Standard Dev	COV (%)
3/4" NMA	35	149	1.0	0.7
1" NMA	23	148.2	2.2	1.5

5.0 Field Evaluation of Mix Design

This section evaluates the mix design criteria based on the Optimized Aggregate Gradation/Tarantula Curve as defined in Wisconsin's Construction and Materials Manual (CMM) 8-70.2.2.3 (rev. June 2019) and its relationship with workability.

5.1 Mix Design Analysis

As discussed in Section 4, the Tarantula Curve boundary and the coarse and fine sand limits were chosen to help designers produce concrete mixtures with good workability. Any mixture within the curve would be expected to have good workability with enough cementitious and water contents. It has been found in previous research that mixtures that are not within the Tarantula Curve limits generally require a higher paste content (binder and water) or higher water reducer dosage to obtain successful performance. Depending on how far the mixture is outside the Tarantula Curve, the paste content should be increased. Figure 30 below represents the Tarantula Curve guidelines for workable mixtures, and will be used throughout this section.

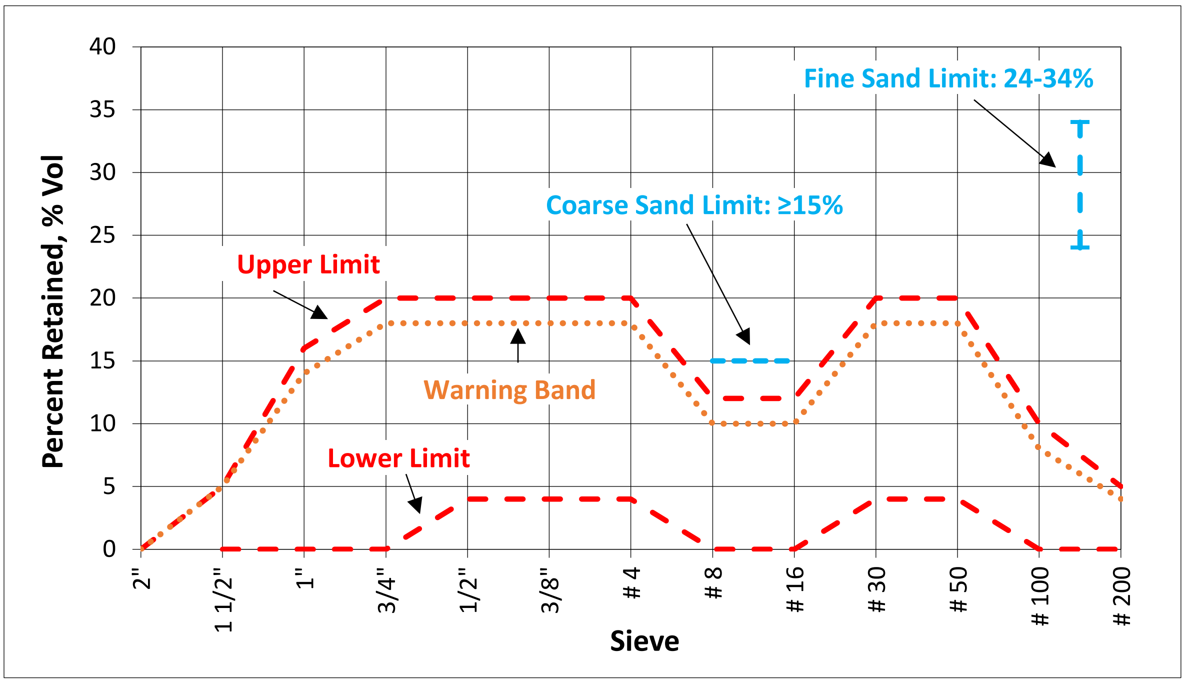


Figure 30: Tarantula Curve guidelines for workable mixtures.

It is important to realize that aggregate gradations will vary in practice. This means that mixtures that are designed to be close to the limit of the Tarantula Curve may violate the limits as the gradation naturally varies during production. This may require a higher amount of cement paste in the mixture to obtain satisfactory performance of the mixture. One way to address this is to encourage producers to make their aggregate gradations be far enough away from the limits that they will not violate the boundary under typical variations in the gradation. One way to encourage this is to have an inner boundary, known as the warning band, that helps account for the variability in the aggregate gradation. The warning band was conceptualized to be used as a tool in this study to help identify mixtures that may start exhibiting differing performance due to their close proximity to specification limits. The Tarantula Curve was used to produce concrete mixtures with adequate workability as shown in Figure 30, with the gradation master range shown in Table 14.

Table 14: Tarantula Gradation Master Range

SIEVE SIZES	PERCENT RETAINED[3]	SUGGESTED WARNING BAND
2 in. (50.8 mm)	0	0
1 1/2 in. (38.1 mm)	$\le$ 5	5
1 in. (25.4 mm)	$\le$16	14
3/4 in. (19.1 mm)	$\le$ 20	18
1/2 in. (12.7 mm)	4-20	18
3/8 in. (9.5 mm)	4-20	18
No. 4	4-20	18
No. 8[1]	$\le$12	10
No. 16[1]	$\le$12	10
No. 30[1] [2]	4-20	18
No. 50 [2]	4-20	18
No. 100 [2]	$\le$10	8
No. 200 [2]	$\le$ 5	4

[1] Minimum of 15% retained on the sum of the #8, #16, and #30 sieves.

[2] Conform to 24-34% retained of fine sand on the #30-200 sieves.

[3] 2022 WisDOT Standard Specification Table 501-4 Optimized Aggregate Gradation.

Table 14 represents the Tarantula Curve requirements and suggested warning band that is applied to all Tarantula graphs within this report.

During the project, it was realized that although producers were using the WisDOT Optimized Gradation spreadsheet, some of the mixtures were not within the limits of the Tarantula Curve. This means that the workability of these mixtures may be impacted. This section discusses the gradation for each project and compares the performance to the Box Test. Table 15 and Table 16 summarize each mixture design for the Phase I and Phase II field projects.

Table 15: Mix Design Summary Table – Phase I Field Projects.

		Phase I Field Location							
		Appleton	**Capitol Drive**	**Columbus**	**Superior**	**W. Waukesha Bypass**	**I-39 Rock County**	**I-39 Dane County**	**Menomonie**
General Design	**Coarse Sand Spec: ≥ 15%**	13.6%	18.1%	18.4%	25.7%	24.7%	24.3%	19.0%	24.2%
	Fine Sand Spec: 24-34%	28.3%	30.0%	24.7%	25.0%	23.1%	30.2%	29.9%	32.4%
	Total Cementitious	565 lbs. (256.3 kg)	565 lbs. (256.3 kg)	565 lbs. (256.3 kg)	530 lbs. (240.4 kg)	565 lbs. (256.3 kg)	565 lbs. (256.3 kg)	520 lbs. (235.9 kg)	520 lbs. (235.9 kg)
	Cement Source	La Farge	La Farge Alpena, MI	St. Mary's - Charlevoix, MI	La Farge - Alpena, MI	St Mary's	La Farge - Alpena, MI	La Farge - Alpena, MI	La Farge
	w/c	0.41	0.37	0.39	0.42	0.41	0.40	0.40	0.38
Supplementary Cementitious Materials	**Fly Ash**	19%	30%	30%	23%	0%	30%	30%	30%
	Slag	0%	0%	0%	0%	30%	0%	0%	0%
	SCM Source	La Farge Edgewater	La Farge Elm Road	La Farge - Portage, WI	NMC - Silver Bay	St. Mary's	La Farge - Portage, WI	La Farge - Portage, WI	La Farge
Admixtures	**Air Entrainer Name**	General Resource Technology	Polychem SA - General Resource Technology	Polychem SA - General Resource Technology	Polychem SA - General Resource Technology	Air 360 - Sika	Polychem SA - General Resource Technology	MasterAir-AE 90 - BASF	N/A
	Water Reducer Name	General Resource Technology	400NC - General Resource Technology	400NC - General Resource Technology	400NC - General Resource Technology	Plastocrete 161 - Sika	400NC - General Resource Technology	MasterGlenium 7511 - BASF	N/A
Tarantula Curve	**OAG Spec. Required?**	NO	NO	NO	NO	NO	YES	YES	YES
	Meets OAG Spec.?	NO	YES	NO	YES***	NO	YES	YES	YES
	Warning Band Exceeded?	YES	YES	YES	YES	YES	NO	NO	NO
Workability*	**Box Test**	1.19 (0.40)	1.19 (0.40)	1.58 (0.51)	1.56 (0.63)	2.06 (0.25)	1.38 (0.72)	1.81 (0.54)	1.00 (0.00)
	V-Kelly**	0.89 (0.10)	0.80 (0.13)	0.51 (0.15)	0.71 (0.11)	0.72 (0.12)	0.62 (0.08)	0.55 (0.21)	0.70 (0.11)

*Note: For the workability values, the value appearing in parenthesis is the standard deviation.

**Note: V-Kelly units are $\left(\frac{in}{sec}\right)^{\frac{1}{2}}$

***Note: Superior was not required to use the OAG spec. but opted to anyways to reduce total cementitious content.

Key: Red boxes indicate values that have exceeded specification, regardless of requiring the OAG spec. Yellow boxes indicate values that are near proposed or required specification limits.

Table 16: Mix Design Summary Table – Phase II Field Projects

		Phase II Location				
		STH 29 Shawano	**STH 50 Kenosha**	**STH 23 Fond Du Lac**	**I39/90 Dane**	**Zoo IC Milwaukee**
General Design	**Coarse Sand Spec: ≥ 15%**	16.6%	22.6%	26.7%	19.4%	23.7%
	Fine Sand Spec: 24-34%	31.7%	29.9%	25.6%	27.5%	26.9%
	Total Cementitious	535 lbs (242.7 kg)	530 lbs (240.4 kg)	520 lbs (235.9 kg)	520 lbs (235.9 kg)	521 lbs (236.3 kg)
	Cement Source	St. Mary's Charlevoix	Lafarge Alpena	St. Mary's Charlevoix	Buzzi Festus	Buzzi Festus
	w/c	0.41	0.38	0.39	0.37	0.42
Supplementary Cementitious Materials	**Fly Ash**	15%	0%	30%	30%	30%
	SCM Source	LaFarge Edgewater	N/A	Schahfer Wheatfield, IN	LaFarge Elm Road	LaFarge Oak Creek
Admixtures	**Air Entrainer Name**	GRT SA	Sika Air 360	Mapei Polychem SA	Mapei Polychem SA	MB AE 400
	Water Reducer Name	GRT Polychem	PlastoCrete	Mapei Polychem 400 NC & Dynamon SX	Mapei Polychem 400 NC & Dynamon SX	MB POZZ 80 & MB DELVO
Tarantula Curve	**OAG Spec. Required?**	YES	YES	YES	YES	YES
	Meets OAG Spec.?	YES	YES	YES	YES	YES
	Warning Band Exceeded?	NO	NO	NO	NO	NO
Workability*	**Box Test**	1.83 (0.39)	1.00 (0.00)	2.08 (1.31)	2.08 (0.90)	1.50 (0.58)
	V-Kelly	N/A	N/A	N/A	N/A	N/A

*Note: For the workability values, the value appearing in parenthesis is the standard deviation.

5.1.1 Appleton

The Appleton gradation, as shown in Figure 31, was within the Tarantula Curve on all limits except for the coarse sand limits. This project was, however, not required to use the Optimized Aggregate Gradation specifications found in the CMM 870.2.2.3. Further, the amount on the ½ in. (12.7 mm) sieve is very close to the recommended limit. The coarse sand is responsible for cohesion within the mixture. In some of the Box Test results, shown in Figure 32, the corners of the box did not remain in place. This shows the lack of cohesion of the mixture. The other results of the Box Test show good performance with some surface voids but minimal edge slumping. This yields a Box Test score of 1.3 – which corresponds to very good workability. It is recommended to increase the coarse sand content of this mixture to be within the Tarantula Curve limits and improve cohesiveness.

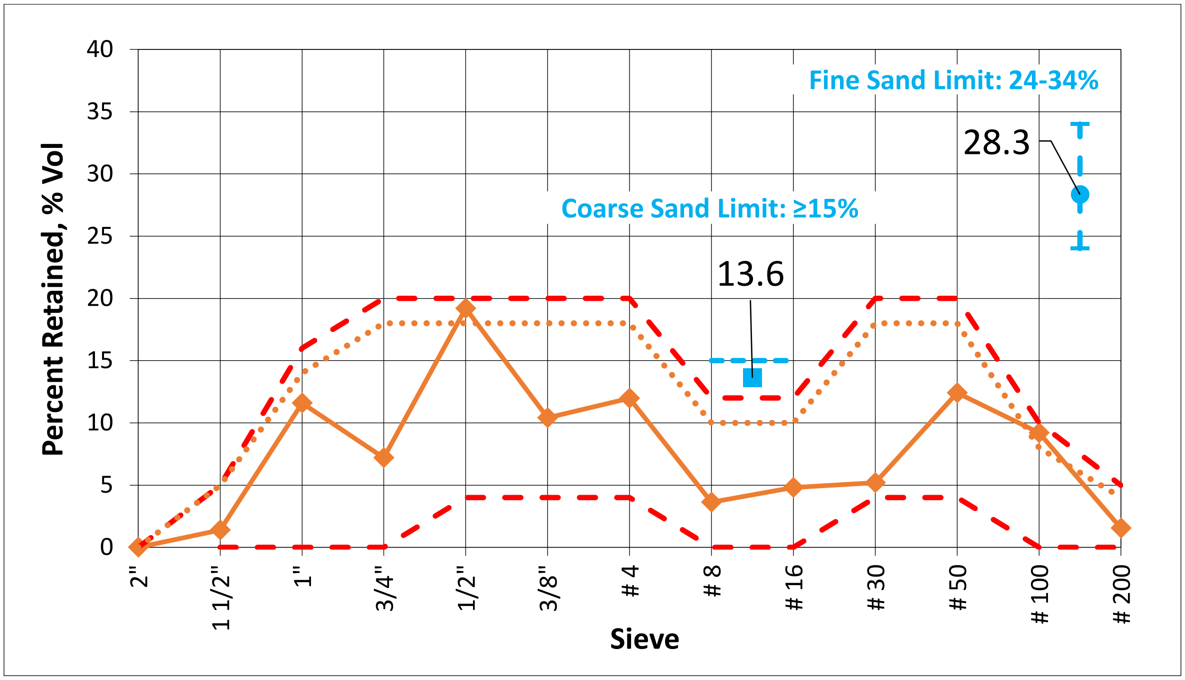


Figure 31: Appleton Tarantula Curve.

Figure 32: Appleton Box Test Photos.

5.1.2 Capitol Drive

The mixture for Capitol Drive, as shown in Figure 33, meets the Optimized Aggregate Gradation specification, but is right at the limit for the ½ in. (12.7 mm) sieve. This project was, however, not required to use the Optimized Aggregate Gradation specifications found in the CMM 870.2.2.3. The performance in the Box Test, shown in Figure 34, looks to be very good in the images, yielding a Box Test score of 1.8. Because this mixture is within the limits of the Tarantula Curve the cementitious content is recommended to be reduced while maintaining a w/c < 0.43. This reduction in cementitious content can reduce cost and improve the durability of the concrete. This mixture used a much higher cementitious content of 565 lbs. (256.3 kg) and w/c = 0.37. This low w/c is not desirable as it will cause autogenous cracking of the concrete. This low w/c was probably necessary to make the concrete mixture hold an edge because of the higher cementitious content. It is recommended that the cementitious content be reduced for future mixtures to reduce the cost and improve the durability without impacting the workability for the concrete.

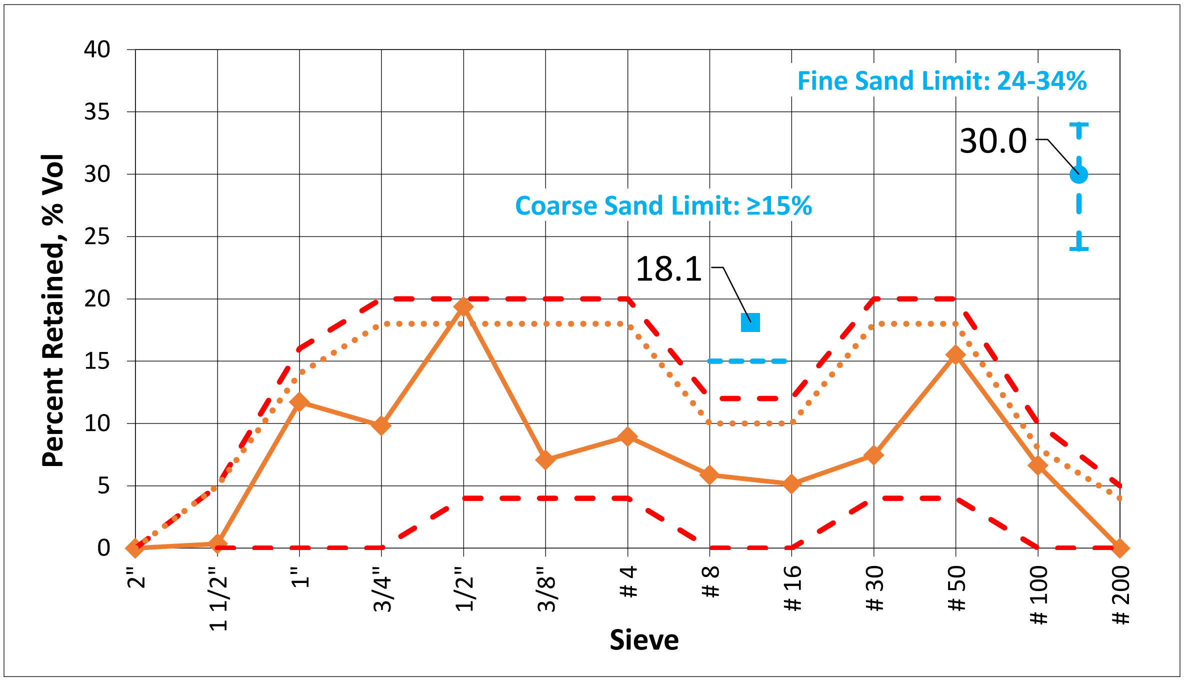


Figure 33: Capitol Drive Tarantula Curve.

Figure 34: Capitol Drive Box Test Photos.

5.1.3 Columbus

The mixture for the Columbus project, as shown in Figure 35, violated the specification limit for the ½ in. (12.7 mm) sieve. This project was not required to use the Optimized Aggregate Gradation specifications found in the CMM 870.2.2.3. Most of the images from the Box Test, shown in Figure 36, show a higher amount of surface voids than would be preferred but the results are acceptable scoring a 1.8 in the Box Test. When these voids are within the volume of the concrete, they will then be expected to lower the strength and decrease the long-term performance of the pavement. Since this mixture is outside the Tarantula Curve then a cementitious content close to the 565 lbs. (256.3 kg) currently allowed by WisDOT is recommended. This matches what is provided. The mixture performance may be able to be improved further with slight changes to the gradation.

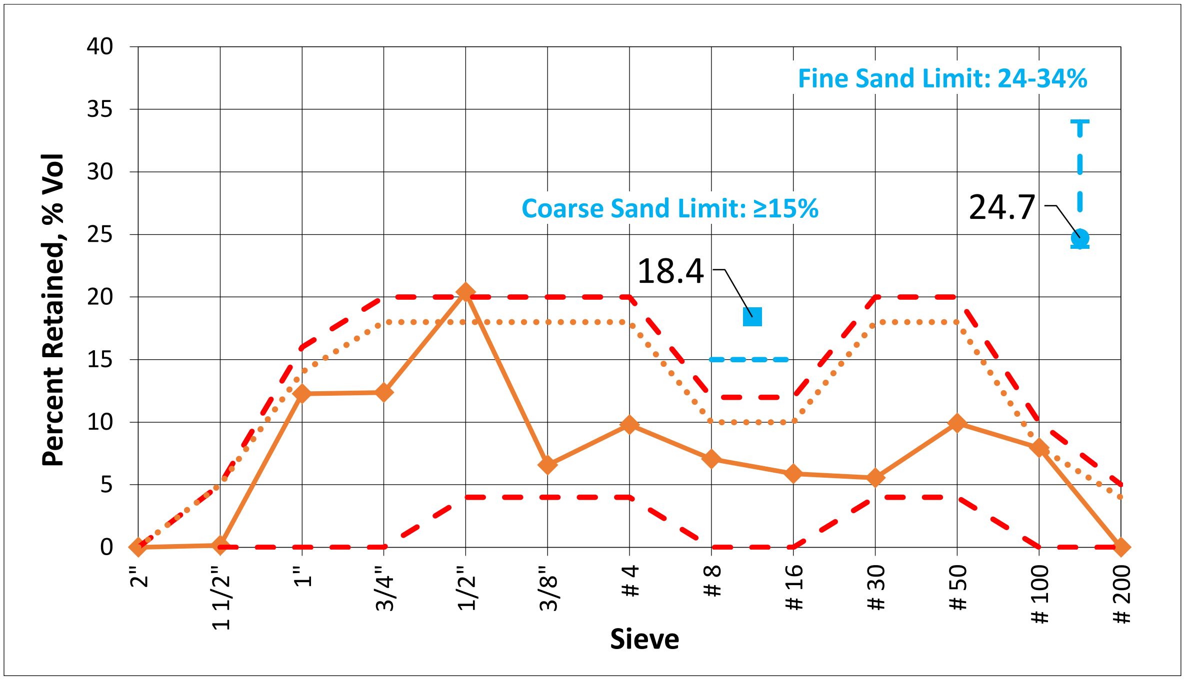


Figure 35: Columbus Tarantula Curve.

Figure 36: Columbus Box Test Photos.

5.1.4 Superior

The mixture for the Superior project, shown in Figure 37, was right at the limits for the ½ in. (12.7 mm) sieve and the fine sand content but was still within the Optimized Aggregate Gradation specifications. This project was not required to use the Optimized Aggregate Gradation specifications found in the CMM 870.2.2.3, however they opted to use the Optimized Aggregate Gradation standard special provision to lower their total cementitious content. If a mixture near the limits of the Tarantula Curve that will be used for production concrete, then one would expect to produce some mixtures that do not have satisfactory performance. A design is expected to be out of the suggested limits about 50% of the time if the gradation used in the design is the average for the aggregate. This is caused by normal variation in aggregate gradation during production proportioning, and the reason why the warning band concept was implemented. The warning band helps identify mixes that have a higher probability of going outside the Tarantula Curve during production. Most of the Box Test results, shown in Figure 38, look good with some edge slumping on the last few images resulting in a Box Test score of 2.1. If the gradation had stayed within the Tarantula Curve limits, then the minimum cementitious content would be acceptable.

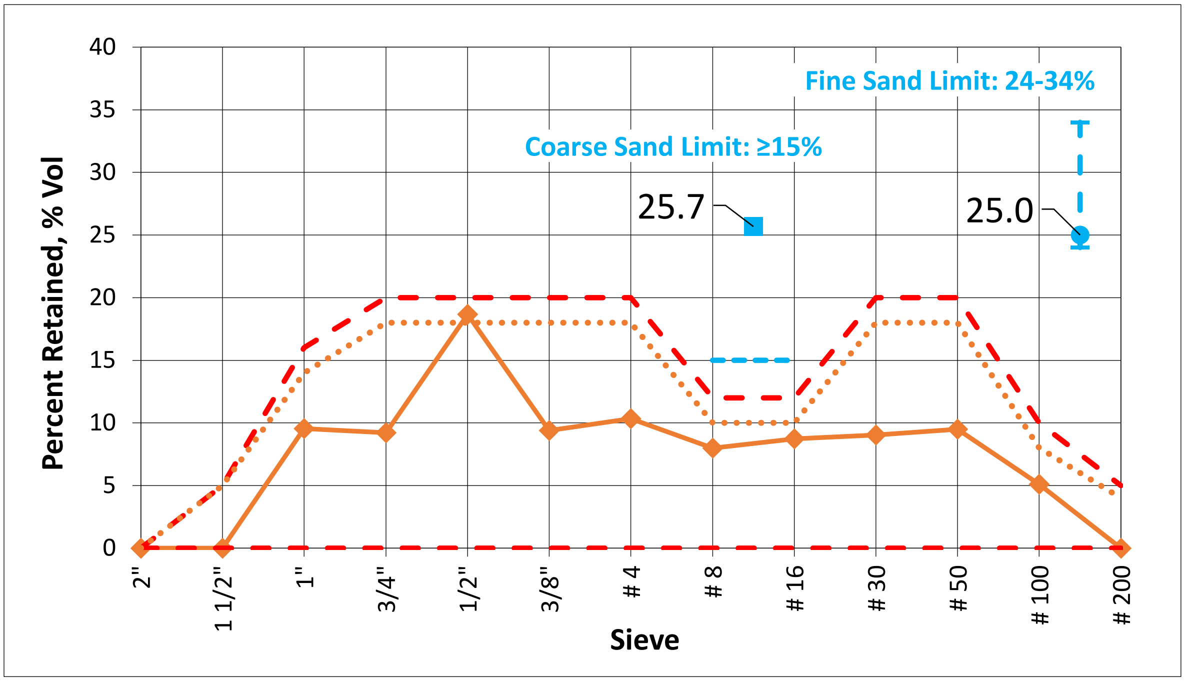


Figure 37: Superior Tarantula Curve.

Figure 38: Superior Box Test Photos.

5.1.5 West Waukesha Bypass

The mixture for the West Waukesha Bypass was out of the Tarantula Curve on the fine sand and very close to the limit in the 1 in. (25.4 mm) and #8 sieve, as shown in Figure 39. This project was not required to use the Optimized Aggregate Gradation specifications found in the CMM 870.2.2.3. This would predict that the mixture would have consolidation issues. The Box Test performance, shown in Figure 40, demonstrates this. In nearly every image one can see large voids on the surface of the Box Test resulting in a score of 1.9 in the Box Test. This means that these large voids are within the concrete. These voids will reduce the strength of the concrete and can lead to premature cracking. It is recommended that the gradation of this mixture be adjusted by increasing the sand content and decreasing the coarse aggregate. It may also be desirable to decrease the intermediate aggregate as well. The research team would need more information about the gradation of the individual aggregates and how they vary.

The mixture used the minimum cementitious content of 565 lbs. (256.3 kg). Based on the results the cementitious content should likely be increased if this gradation is used. While it may be possible to fix this issue with a larger amount of cement paste in the mixture, it is not desirable for the durability, sustainability, and cost of the concrete.

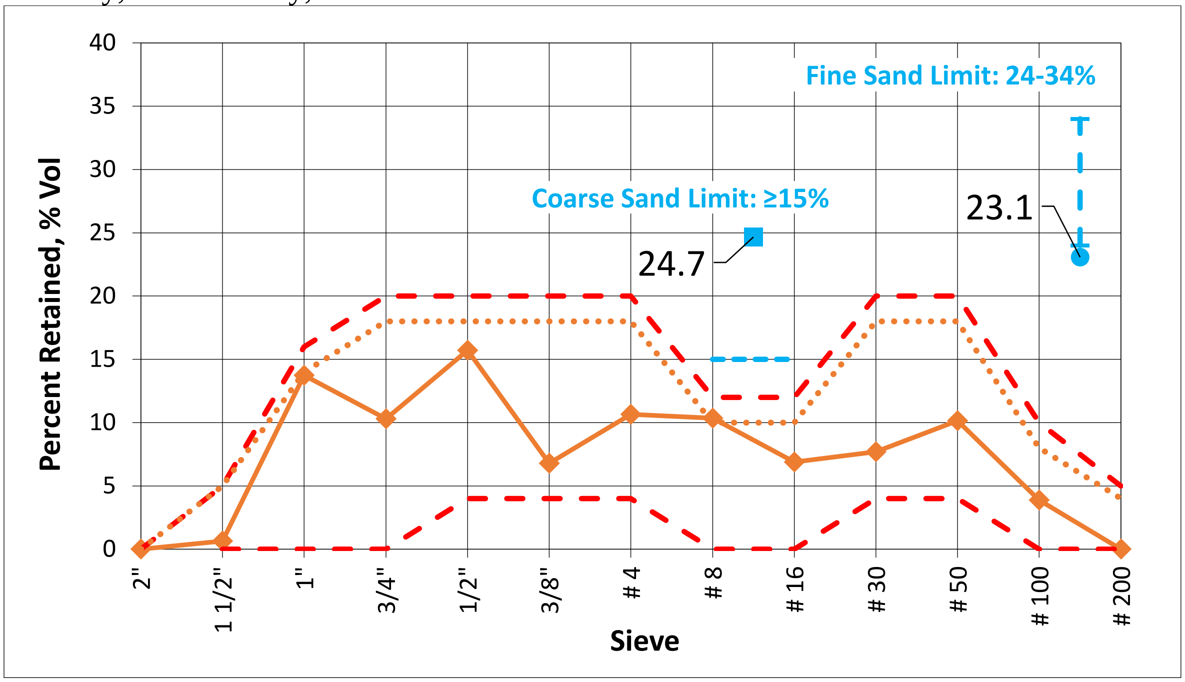


Figure 39: West Waukesha Bypass Tarantula Curve.

Figure 40: West Waukesha Bypass Box Test Photos.

5.1.6 I-39 Rock County

The mixture for I-39 in Rock County is within the Tarantula Curve limits as shown in Figure 41 and the project required the use of the Optimized Aggregate Gradation specifications found in the CMM 870.2.2.3. The Box Test results, shown in Figure 42, for Rock County typically showed good performance with some consolidation issues in some of the box corners. There were edge slumping issues observed in the last few photos. These observations resulted in a Box Test score of 2.1. This means that edge slumping may be a concern in the field. These field measurements showing the variability of the concrete shows there was inconsistency in the quality of the concrete. This could be caused by additional water in the mix or perhaps a change in gradation of the coarse sand for these mixtures. It is not possible to tell from the data collected.

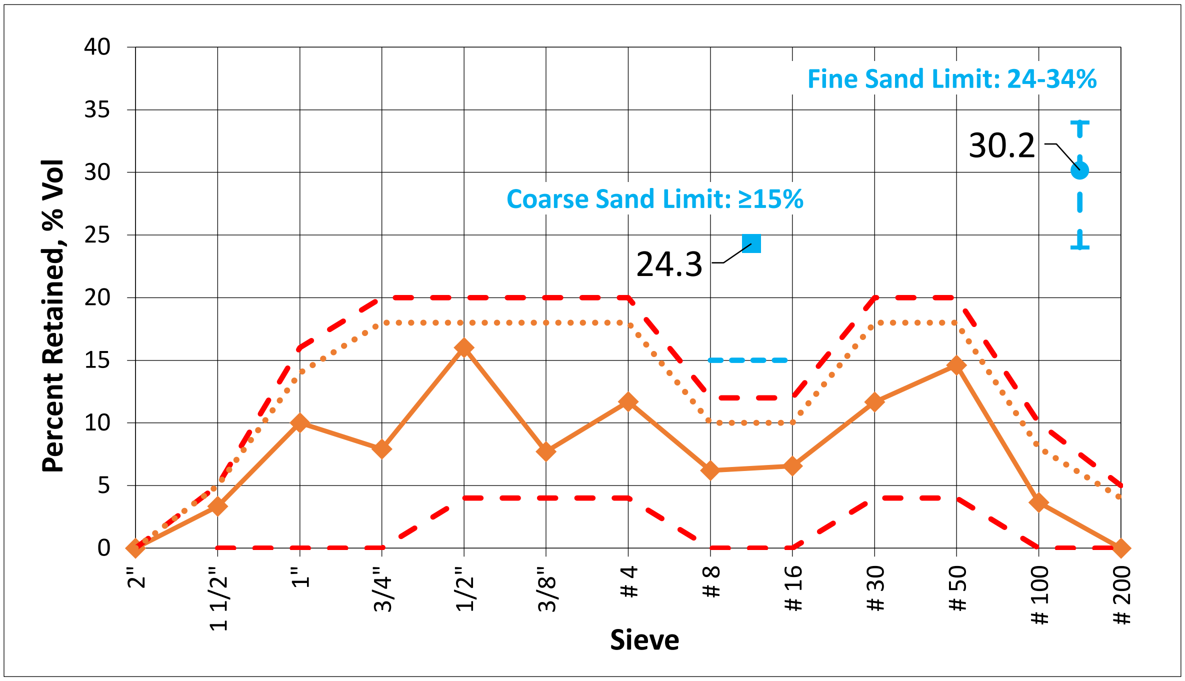


Figure 41: I-39 Rock County Tarantula Curve.

Figure 42: I-39 Rock County Box Test Photos.

5.1.7 I-39 Dane County

This project required the use of and met the Optimized Aggregate Gradation specifications found in the CMM 870.2.2.3 and as shown in Figure 43. The Box Test results, as shown in Figure 44, for the Dane County typically showed good performance. There were a few consolidation issues on some tests, and there were edge slumping issues observed in the last photos. This resulted in a Box Test score of 2.3. This means that edge slumping may be a concern in the field with these mixtures. This shows that there was some inconsistency in the production of the concrete. This could be caused by additional water in the mix or perhaps a change in gradation of the coarse sand for these mixtures. It is not possible to tell from the data collected. It is recommended to use the minimum cementitious content allowed by WisDOT.

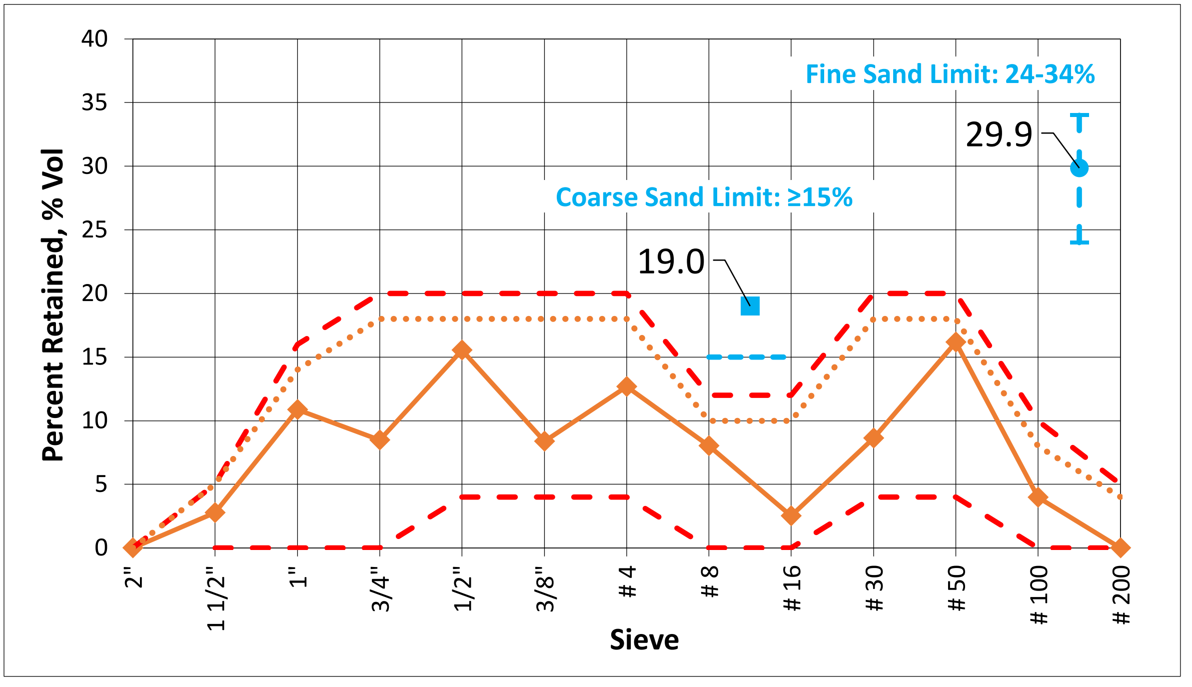


Figure 43: I-39 Dane County Tarantula Curve.

Figure 44: I-39 Dane County Box Test Photos.

5.1.8 Menomonie

Menomonie's mix design, shown in Figure 45, was within the warning band and lower limit of the Tarantula Curve and showed outstanding performance in the Box Test, as shown in Figure 46. This project required the use of the Optimized Aggregate Gradation specifications found in the CMM 870.2.2.3. None of the tests showed consolidation or edge slumping resulting in a Box Test score of 1.3. This means that even as the gradations of the mixture varied, the aggregates were likely within the Tarantula Curve limits. This helps ensure satisfactory performance. Because the mixture is within the Tarantula Curve, it is recommended to use the minimum cementitious content allowed by WisDOT.

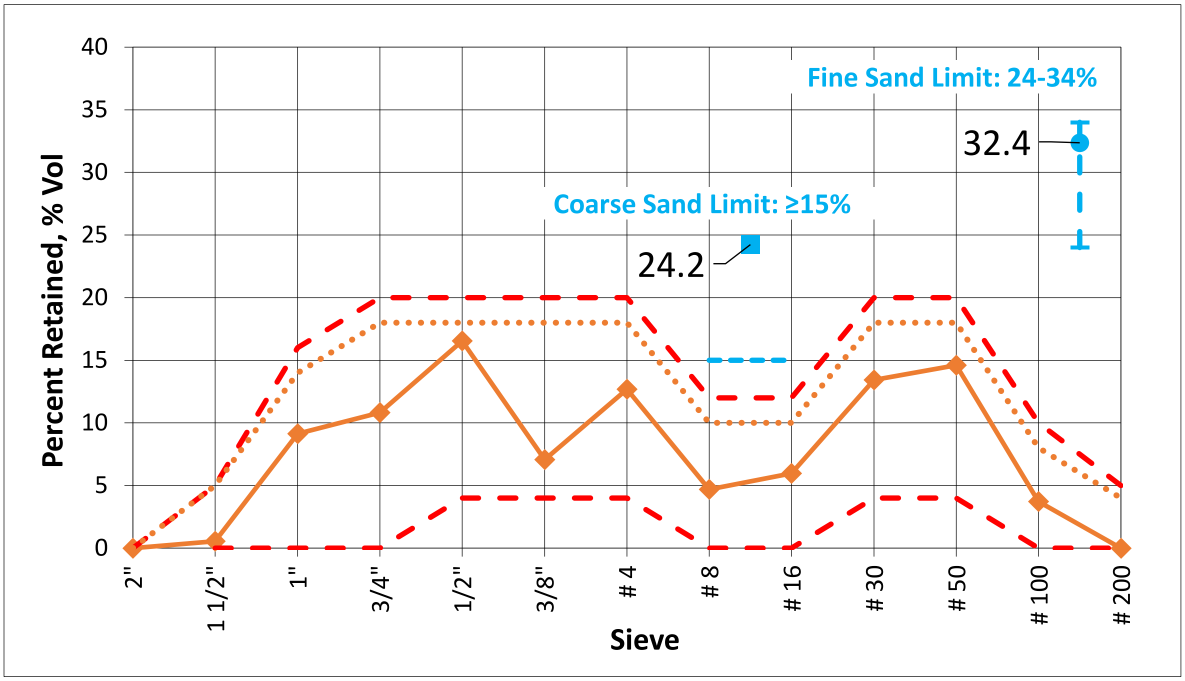


Figure 45: Menomonie Tarantula Curve.

Figure 46: Menomonie Box Test Photos.

5.1.9 STH 29 Shawano

Shawano's STH 29's aggregate gradation is shown in Figure 47. This mixture is near the Tarantula Curve warning band for the ½-in. (12.7 mm) sieve and the #100 sieve. These warning bands do not mean that performance will be poor, but it does mean that as the gradation naturally varies, there becomes a higher chance for performance issues.

This mixture showed middling performance in the Box Test, as shown in Figure 48. There were some surface voids and some edge slumping which resulted in an average Box Test score of 1.83.

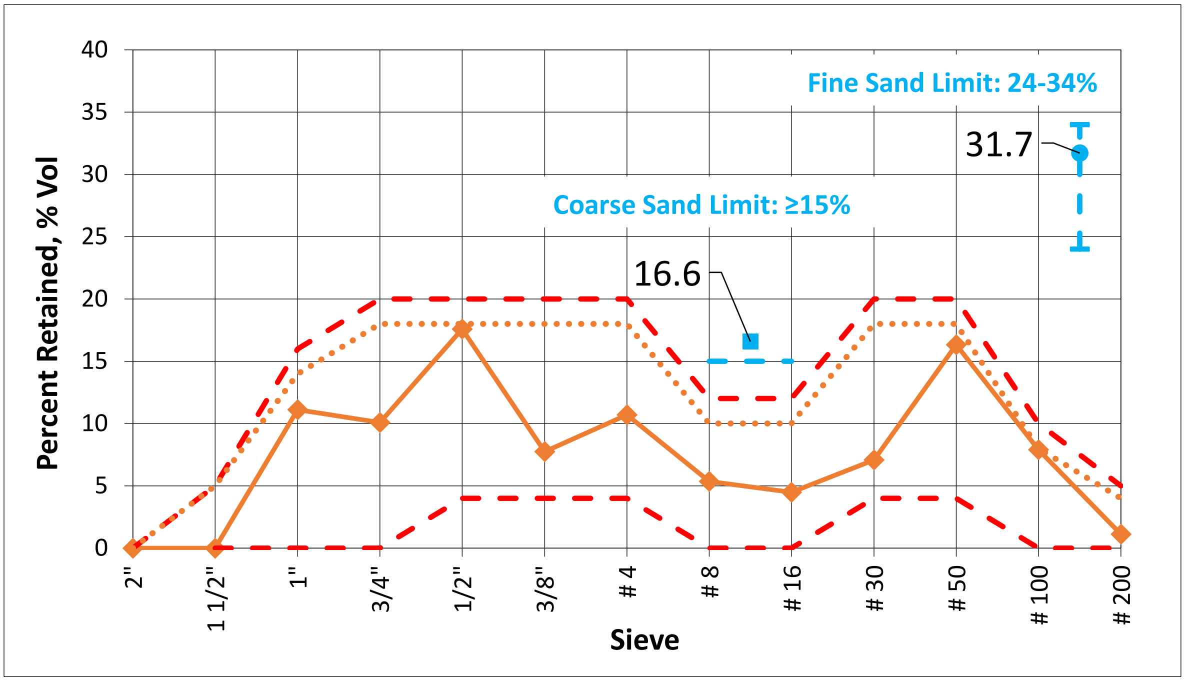


Figure 47: STH 29 Shawano Tarantula Curve.

Figure 48: STH 29 Shawano Box Test Photos.

5.1.10 STH 50 Kenosha

STH 50 Kenosha's aggregate gradation is shown in Figure 49. The mixture was close on the warning band of the #100 sieve. This mixture showed the best performance in the box test, shown in Figure 50, of any mixture investigated with a score of 1.00. The mixture met all performance criteria while using the lowest w/c of 0.38 and no fly ash in the mixture. These results show that further cement may be able to be reduced in this mix with the use of fly ash or with a slight increase in the w/c. This shows the benefits of using the Tarantula Curve in practice and the potential savings in cost and satisfactory performance that is possible with these concretes.

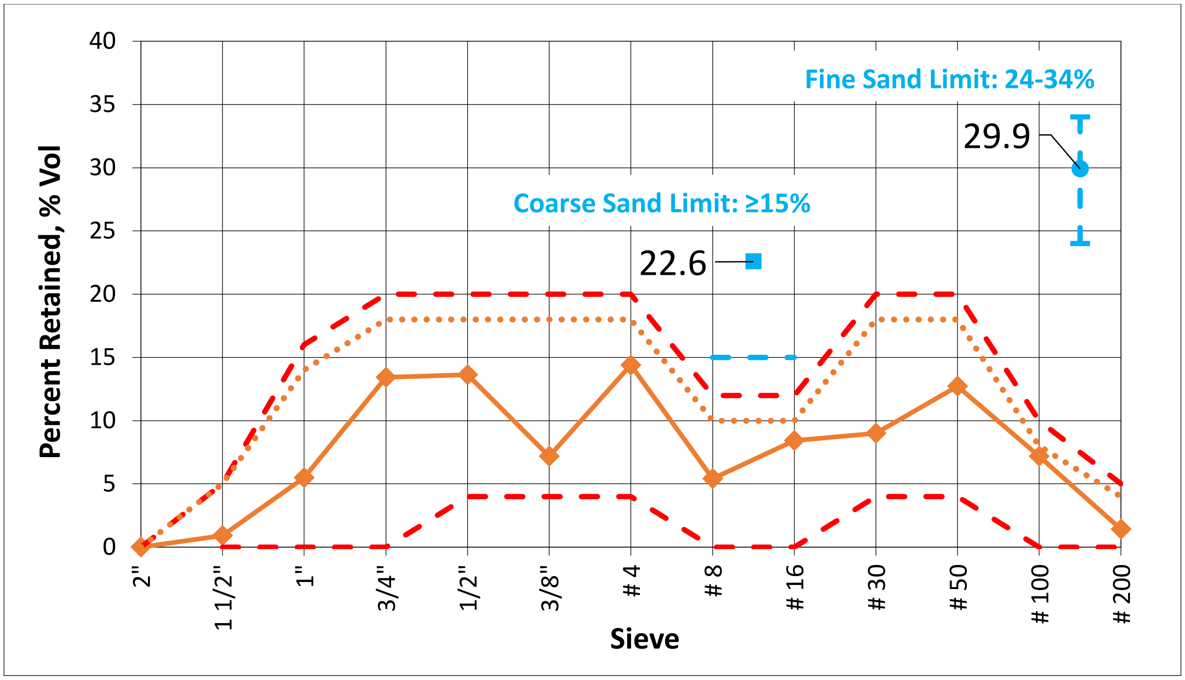


Figure 49: STH 50 Kenosha Tarantula Curve.

Figure 50*: STH 50 Kenosha Box Test Photos.*

5.1.11 STH 23 Fond Du Lac

STH 23 Fond Du Lac's aggregate gradation is shown in Figure 51. This mixture is at the Tarantula Curve warning band for the #16 sieve. This mixture had several Box Tests that showed significant edge slumping which means that the mixture is too workable and then in other tests the mixture had low water content and the performance was poor in the box test – shown in Figure 52. Because of the wide variation in performance, the average performance in the Box Test is 2.08. This variation in the material on the day of sampling could be from a number of items, so it is not clear what is causing the issues. It is clear from the middle set of images that the mixture has the potential to perform well.

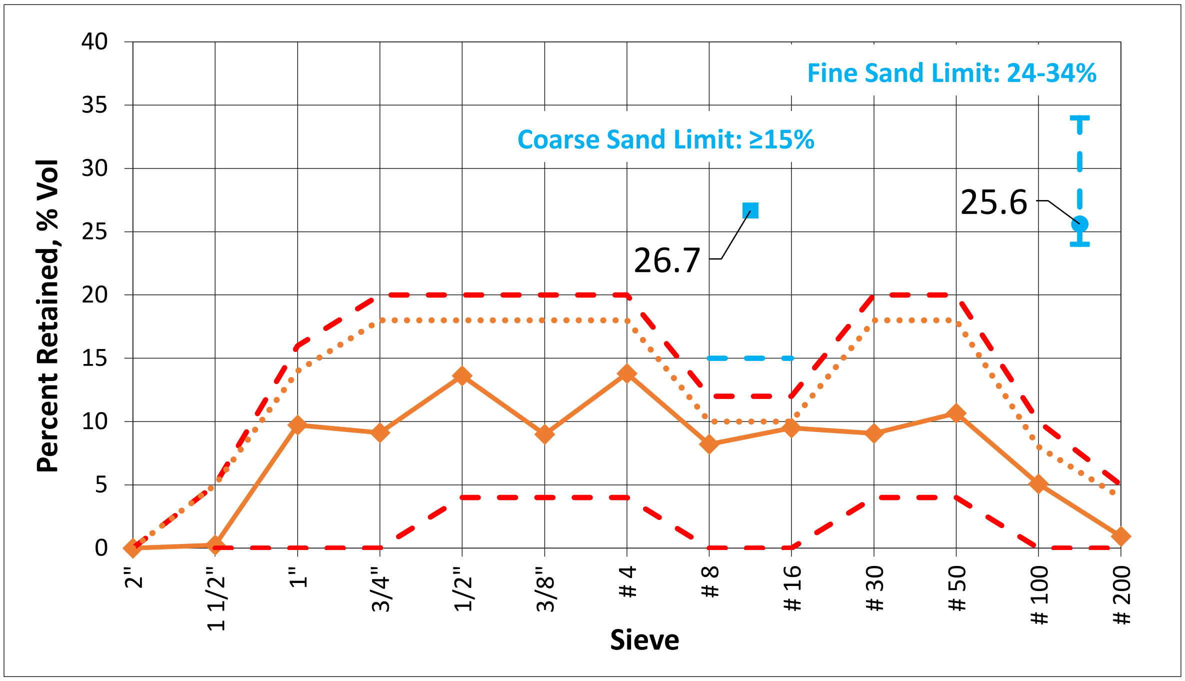


Figure 51: STH 23 Fond Du Lac Tarantula Curve.

Figure 52: STH 23 Fond Du Lac Box Test Photos.

5.1.12 IH 39/90 Dane County

The IH 39/90 Dane Country aggregate gradation is shown in Figure 53. The gradation does not violate the warning band. The second and third row of Box Test images, shown in Figure 54, show that the mixture performed satisfactorily but there was some difference in performance with the first set of mixtures. This inconsistent mixture increased the average Box Test score to 2.08. It should be noted that this mixture was tested on a cold day in early November where the high temperature was 41°F (5°C). Based on the Tarantula Curve, this shows that this mixture has the potential to perform well in the field with a low w/c and binder content, however field conditions resulted in inconsistent production in which we wouldn't recommend pushing the limits any farther.

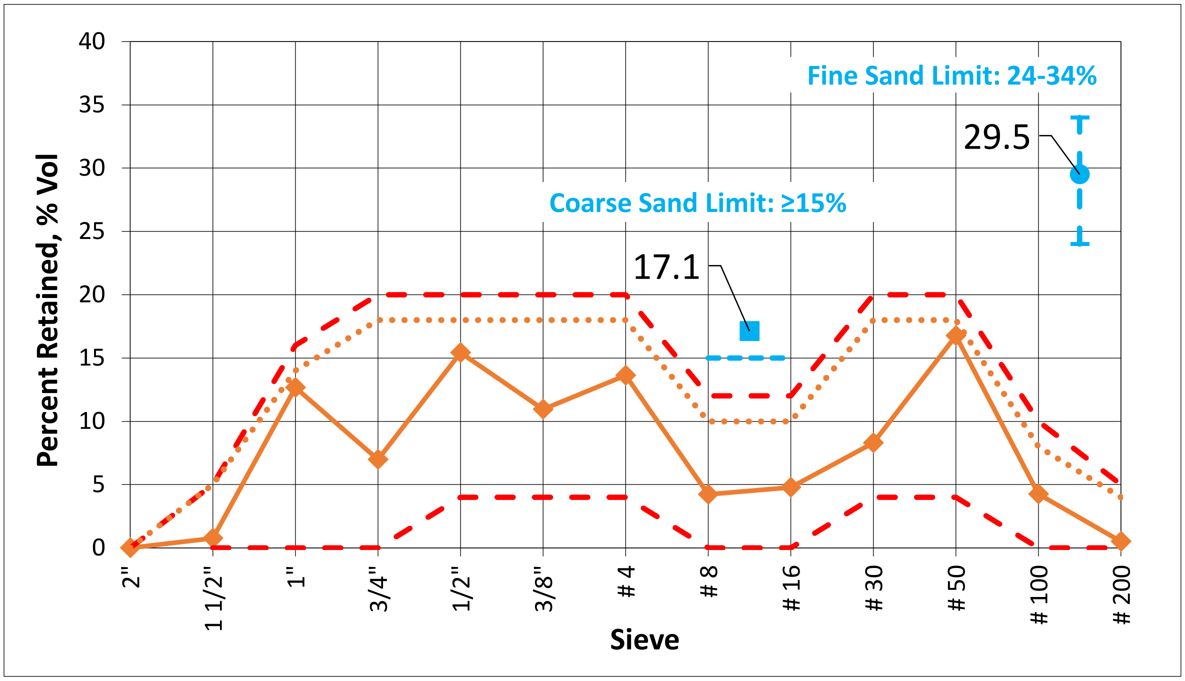


Figure 53: IH 39/90 Dane County Tarantula Curve

Figure 54: IH 39/90 Dane County Box Test Photos

5.1.13 Milwaukee Zoo Interchange

The aggregate gradation for the Milwaukee Zoo Interchange is shown in Figure 55. This mixture did not exceed the Tarantula Curve warning bands for any sieve. This mixture showed very good performance in the Box Test, as shown in Figure 56 with an average Box Test score of 1.50. There was also a minimal amount of edge slumping of the concrete. Only one set of images are shown below because this was a project that was added on at the end to have an opportunity to collect more SAM Meter data. It should be noted that this mixture was tested on a cold day in mid-November where the high temperature was 36°F (2.2°C). This mixture performed well and used a low binder content and w/c. This is another example of how the Tarantula Curve can help create mixtures that perform well in practice and in the Box Test.

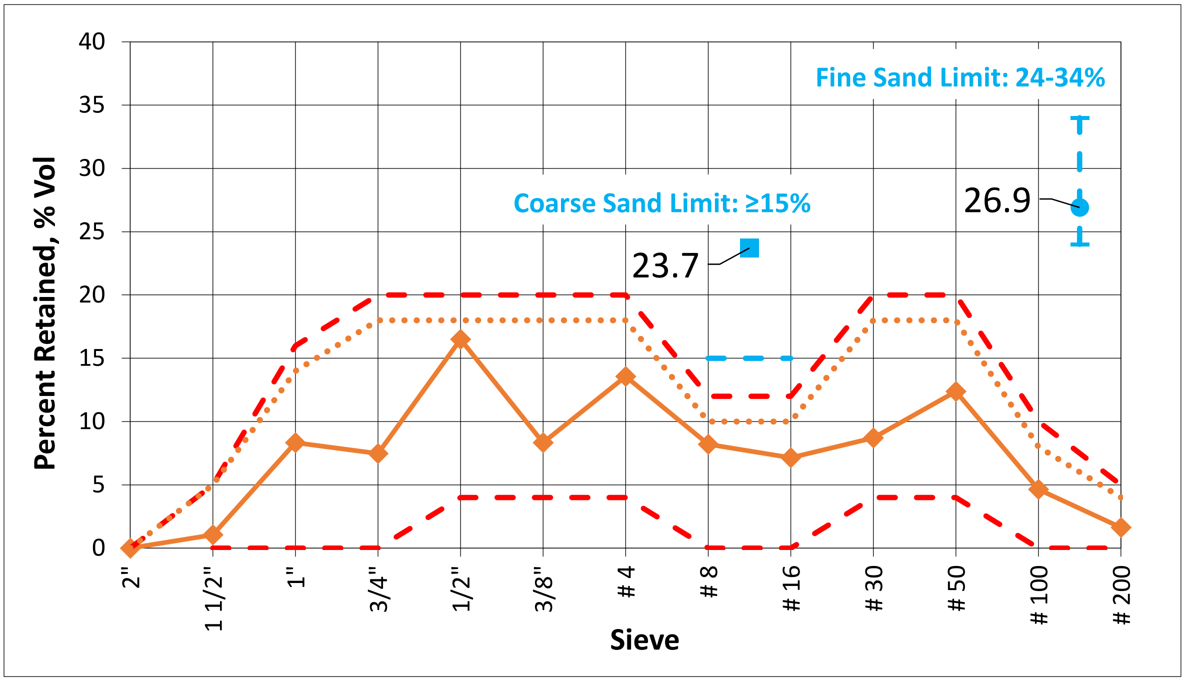


Figure 55: Milwaukee Zoo Interchange Tarantula Curve

Figure 56: Milwaukee Zoo Interchange Box Test Photo

5.2 Compressive and Flexural Strength Analysis

Figure 57 and Figure 58 show the Phase I compressive strength and flexural strength gain over the first 90 days of hydration. The strength gain for both measurements is the largest between 3 and 7 days and then substantially decreases. It is worth noting that the flexural strength of the W. Waukesha Bypass sample dropped at 90 days. This is likely due to a poorly constructed beam, and because only one beam was broken at 90 days, this is the only value that could be reported. This value should not be viewed as a representative strength result.

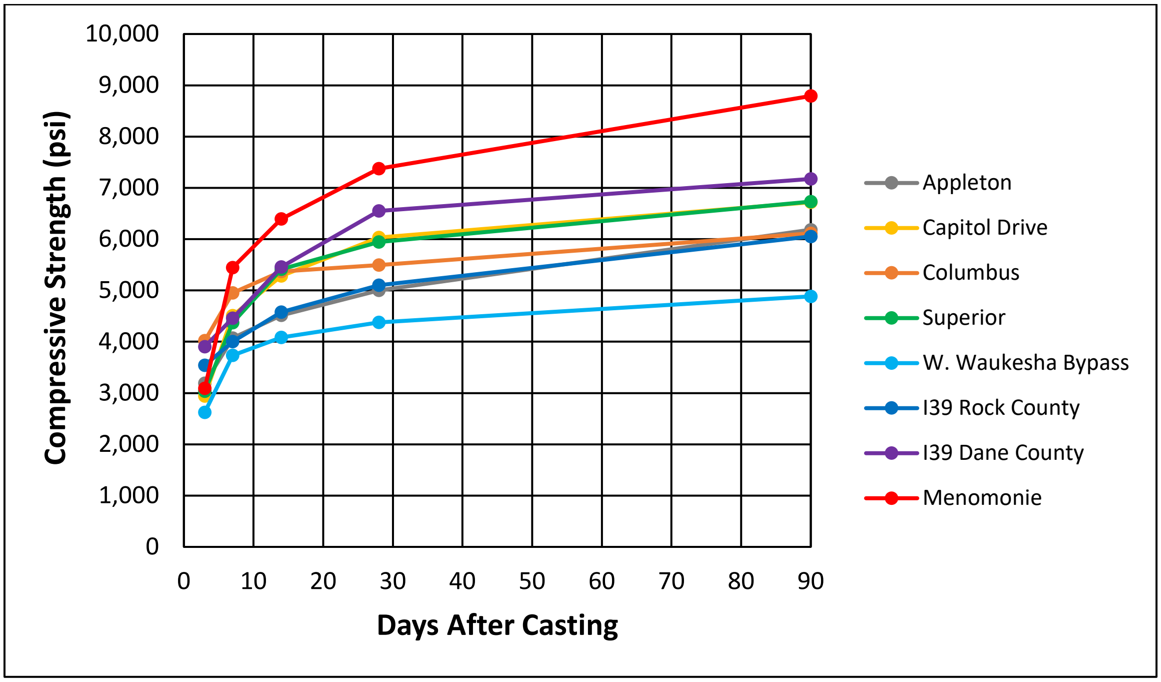


Figure 57: Phase I Compressive Strength development with curing time.

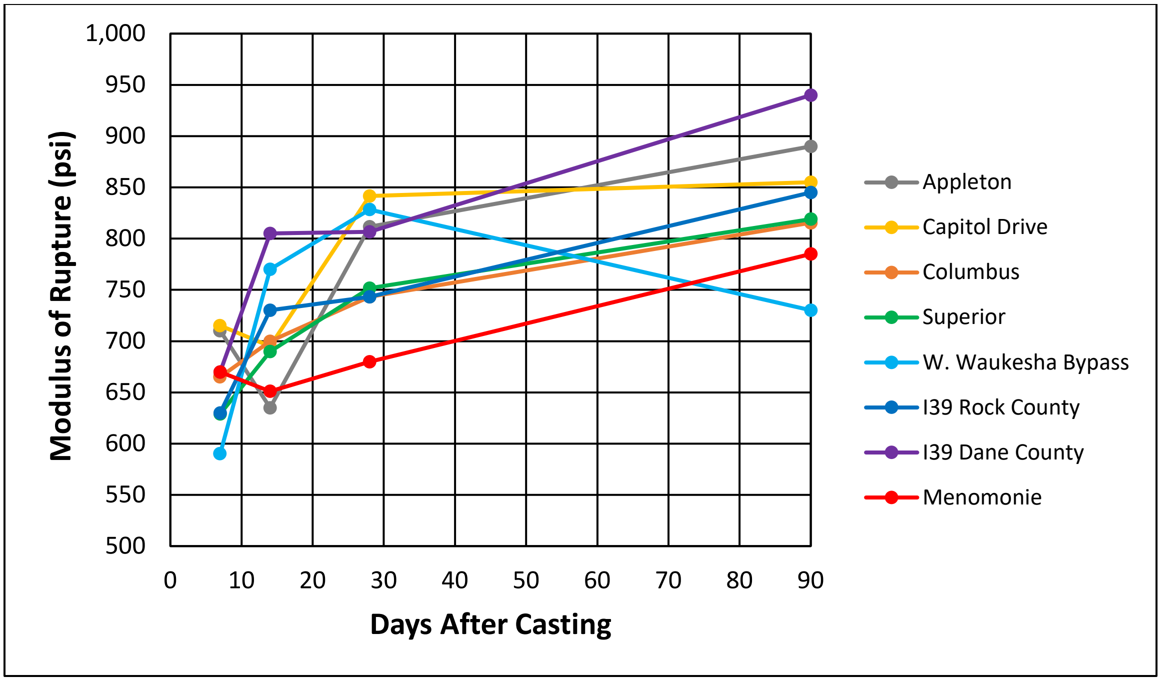


Figure 58: Phase I Modulus of Rupture strength development with curing time.

To better compare the results, Figure 59 uses bar charts to compare both the compressive strength and flexural strength at 28 days organized by aggregate type. This figure is helpful as it shows the relative rank of compressive strength and flexural strength amongst the projects as well as any trends between aggregate types. An important observation from Figure 59 is that the mixtures with the highest compressive strengths did not always have the highest flexural strengths, which is especially apparent with the Menomonie mixture which had the highest compressive strength but the lowest flexural strength among all locations tested.

The equation used to estimate the Modulus of Rupture was derived based on the correlation between compressive and flexural strength of Long-Term Pavement Performance (LTPP) datasets in an FHWA study [43]. It utilizes the power model as traditional methods have used and is best utilized in cases where the compressive strength of the concrete was determined through routine cylinder breaks [43]. When the Phase I data was analyzed alone there was a lower correlation with increasing compressive strength. However, when both Phase I and Phase II datasets were combined, the data better matched the estimated FHWA flexural strength equation, shown below, and increased the correlation as shown in Figure 60. These observations are likely caused by differences in bond strength between the paste and aggregates for the different mixtures. Since concrete pavements are loaded in flexure and the bond between the aggregates and paste is important, then it is logical to continue to test the flexural strength of the concrete.

$$\text{FHWA Estimated Flexural Strength} = 22.7741(f_c')^{0.4082} \tag{9}$$

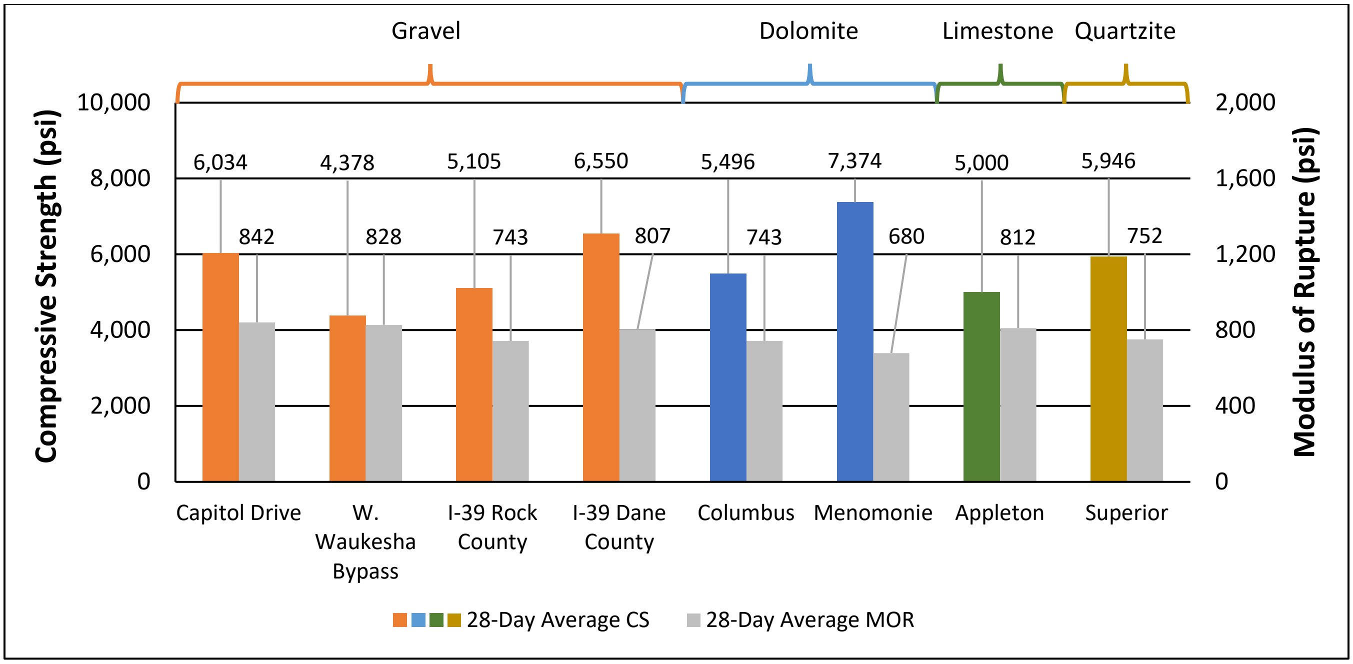


Figure 59: Phase I 28-Day Compressive Strength vs. 28-Day Modulus of Rupture.

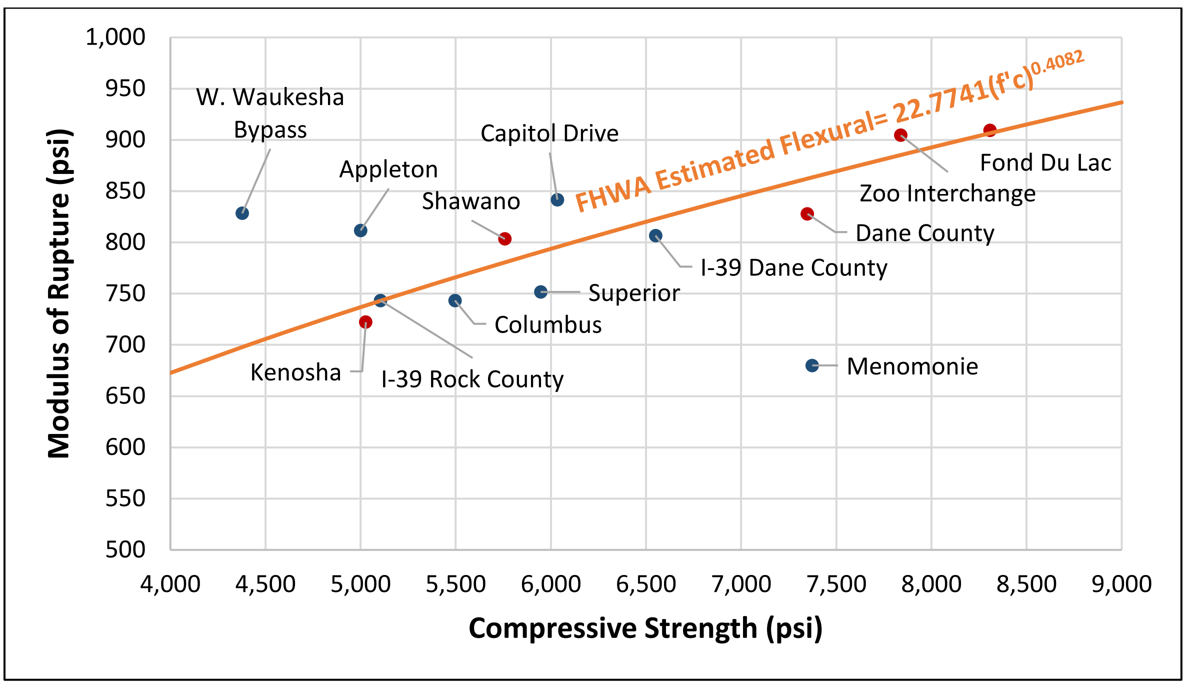


Figure 60: Phase I and II 28-Day Modulus of Rupture vs. Estimated Modulus of Rupture from Compressive Strength. Phase I Projects are Shown with Blue Dots and Phase II Projects are Shown with Red Dots.

5.3 Analysis of Durability Properties

5.3.1 Air Void Analysis

Figure 61 shows the air content and Spacing Factor comparison for the samples investigated with a hardened air void analysis during Phase I. A horizontal dashed line shows a Spacing Factor of 200 µm (0.008 in.). This is the recommended maximum Spacing Factor value as suggested by ACI 201 2R. A vertical line at 5.25% air content is also added to highlight that mixtures at similar air contents can have very different Spacing Factors. In fact, a mixture with 5.25% air content would meet the current WisDOT specifications for air volume but would not provide a satisfactory Spacing Factor.

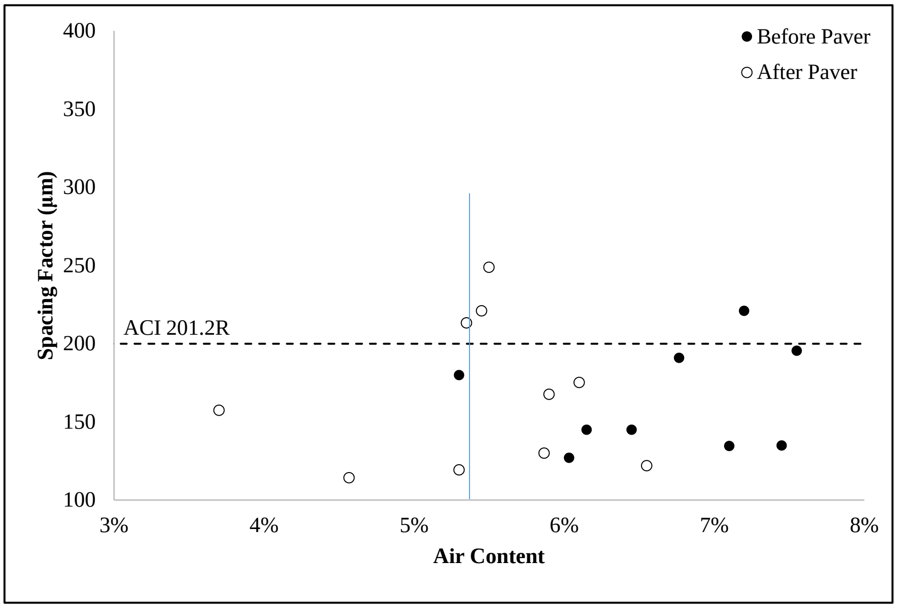


Figure 61: Air content compared to Spacing Factor for the samples investigated with a hardened air void analysis.

Figure 62 shows a comparison of the SAM Number and Spacing Factor. A Spacing Factor of 200 µm (0.008 in.) is shown for comparison. Two vertical lines are shown for the SAM Number. A SAM Number of 0.30 is shown in red and it is recommended as a field limit for freeze thaw durability. The line shown in black is a SAM Number of 0.20 and it is recommended as a target for design, and it is shown in previous research to best correlate to a Spacing Factor of 200 µm (0.008 in.). For example, when the SAM Number is < 0.20 then 85% of the data had a Spacing Factor < 200 µm (0.008 in.).

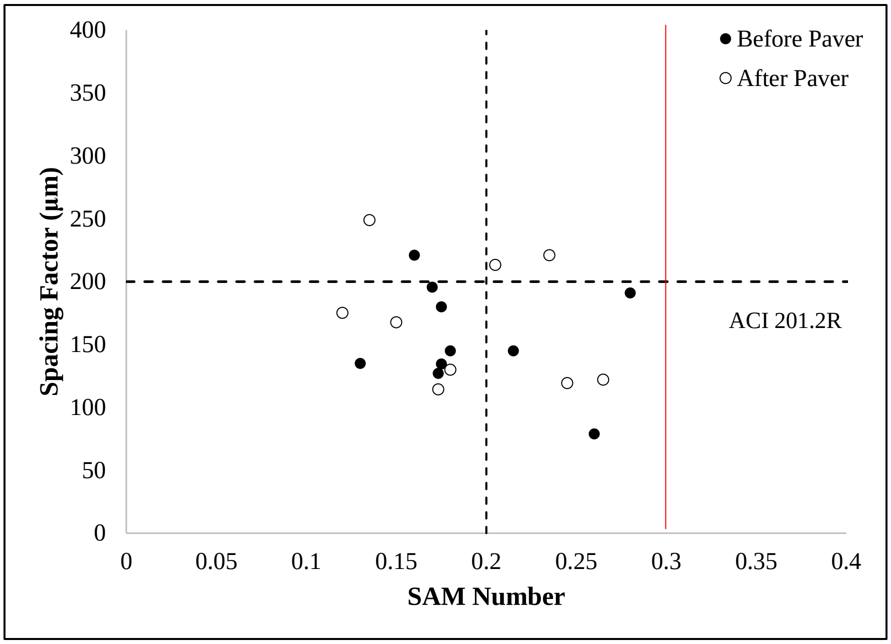


Figure 62: A comparison of the SAM Number and Spacing Factor.

Figure 63 and Figure 64 show the average air content and average SAM Number at the plant, before the paver, and then after being consolidated by the paver. These samples were collected and tested during Phase I of this research project. The green and red fields in Figure 63 illustrate the WisDOT specification limits, 7 ± 1.5%, where points within the green field are within the limits and points within the red field are outside of the limits. Each data point consists of the average of several measurements with the SAM from several different sessions. The reader should be reminded that the values in Figure 61 and Figure 62 are comparisons from individual measurements. The testing shows that the air content typically dropped from the plant until after the paver by as much as 3.5%. This drop was commonly 1% from the plant to the site and then another 2.5% decrease when comparing the concrete before and after the paver. This created air contents that were close to 4% after the paver. Figure 65 verifies these findings with similar trends in air contents, and Figure 66 directly compares the plastic air contents to the hardened air contents.

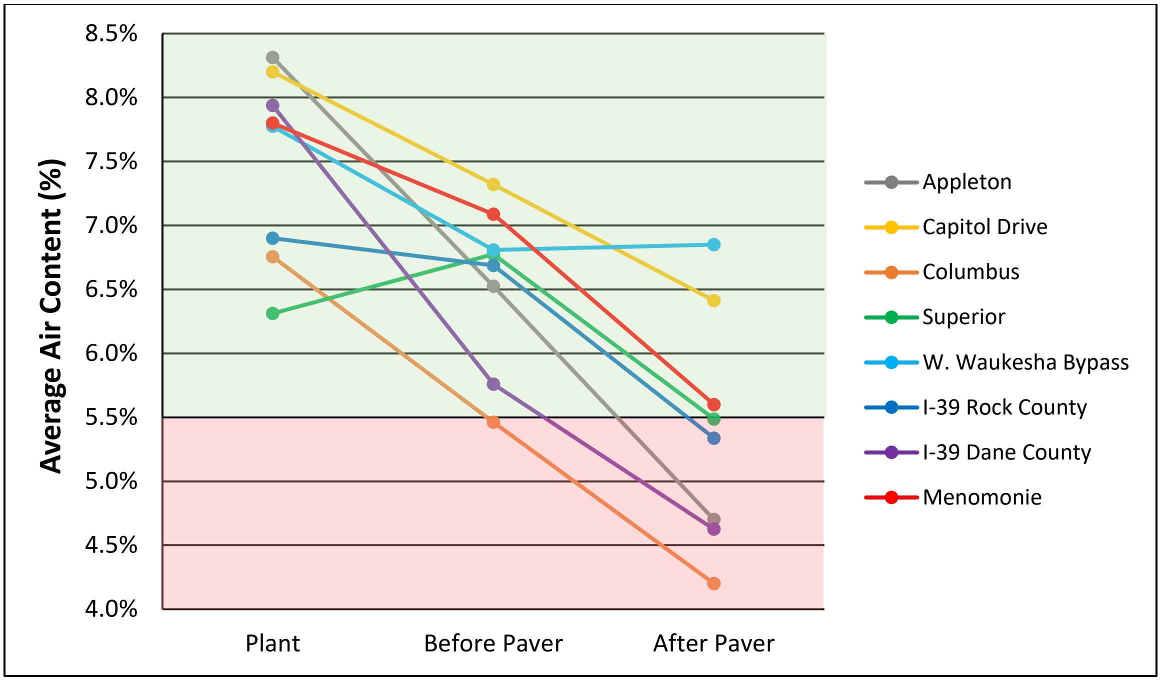


Figure 63: Air content changes throughout production and placement from the plant to before and after the paver.

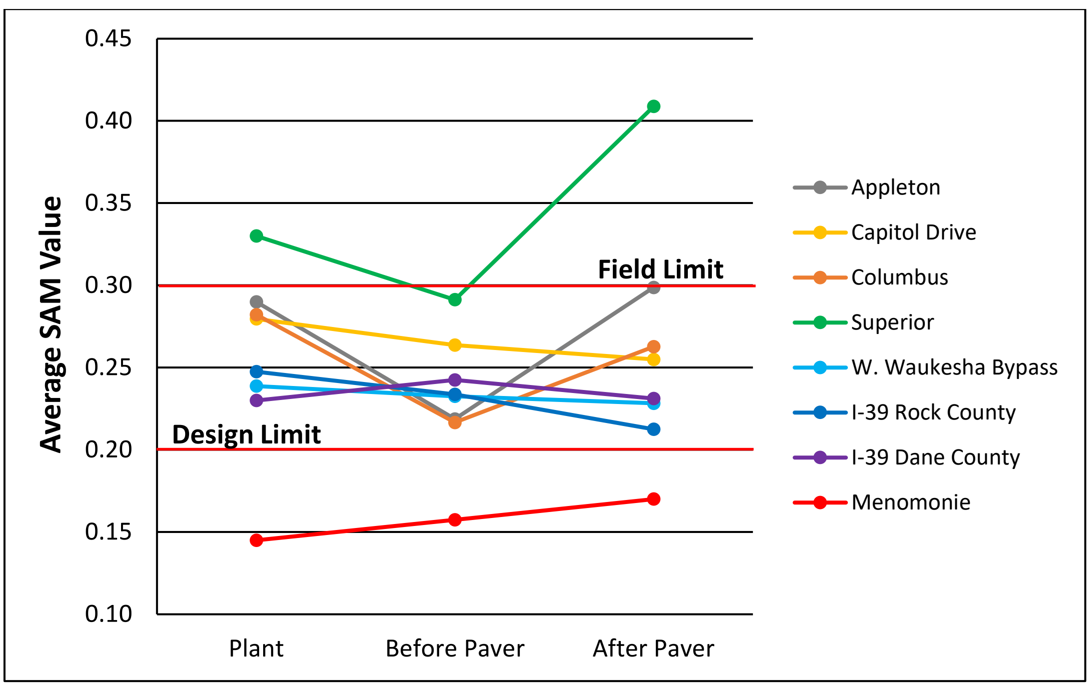


Figure 64: SAM value changes throughout production and placement from the plant to before and after the paver.

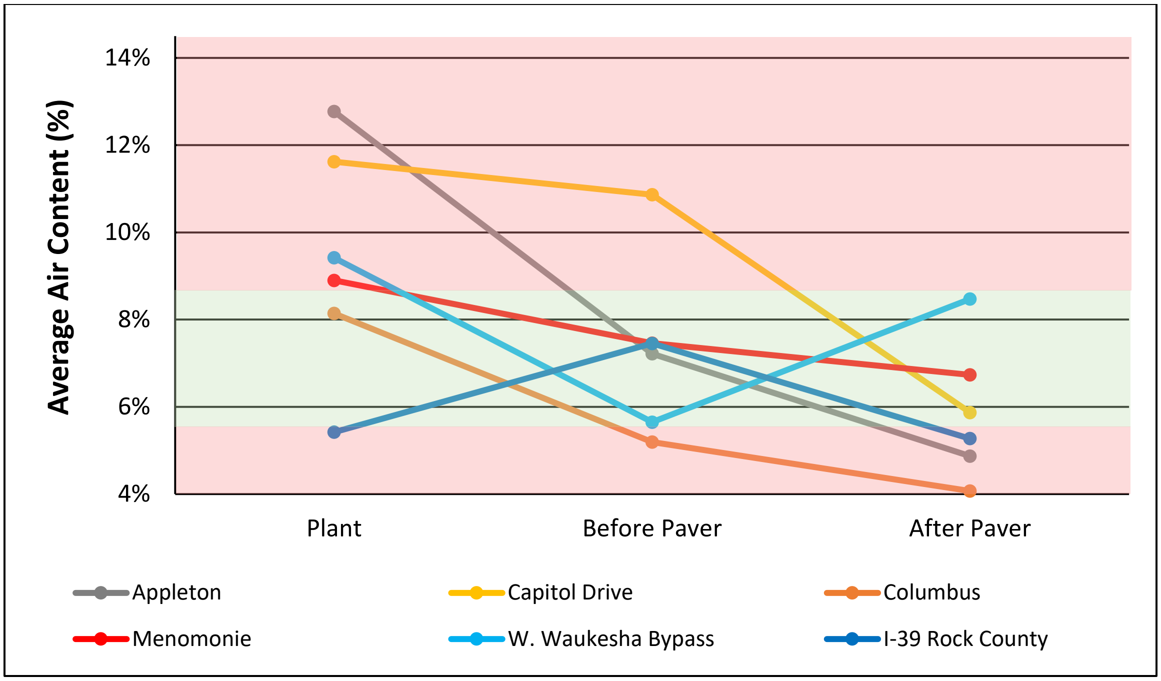
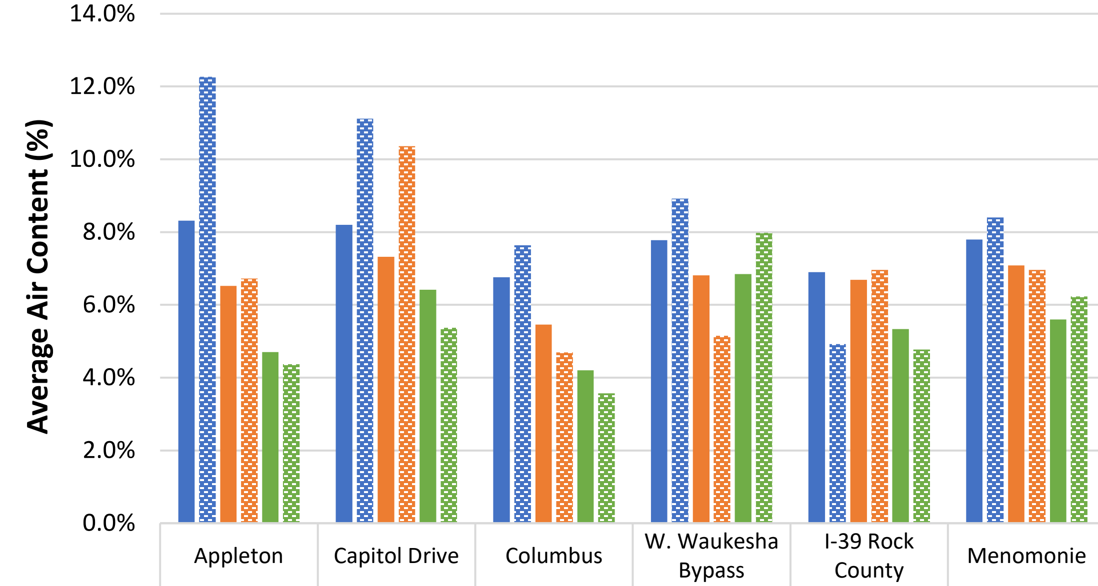


Figure 65: Hardened Air Contents from Samples taken from the Plant, Before, and After the Paver.

Figure 64 shows that there was minimal change in the SAM Numbers when comparing the measurements at the plant, before the paver, and then after the paver for SAM Numbers < 0.28. These mixtures would be expected to have a satisfactory air void distribution of small and well distributed bubbles. However, for the Superior project, SAM Numbers > 0.30 were observed at the plant. This air void system is not recommended for freeze thaw durability. The SAM Numbers decreased to 0.28 for the concrete in front of the paver and then the values increased sharply after the paver to > 0.40. This means that an air void system with poorly distributed bubbles performed differently under vibration than air void systems that were small and well distributed.



	Appleton	Capitol Drive	Columbus	W. Waukesha Bypass	I-39 Rock County	Menomonie
Plant Plastic	8.31%	8.20%	6.76%	7.78%	6.90%	7.80%
Plant Hardened	12.27%	11.12%	7.64%	8.92%	4.92%	8.40%
Before Paver Plastic	6.53%	7.32%	5.46%	6.81%	6.69%	7.09%
Before Paver Hardened	6.72%	10.36%	4.69%	5.15%	6.96%	6.96%
After Paver Plastic	4.70%	6.41%	4.20%	6.85%	5.34%	5.60%
After Paver Hardened	4.37%	5.37%	3.57%	7.98%	4.77%	6.23%

Figure 66: Comparison of Plastic Air to Hardened Air Content.

An important finding from this work is that the air that is lost from the plant to the job site seems to be largely coarse “entrapped” bubbles. This is shown because the air volume is decreasing but the SAM Number is not changing. Further, when the concrete is vibrated by the paver and the SAM Number is < 0.28 or made of mainly fine air void bubbles, there was minimal change in the SAM Number despite a significant change in the volume of air in the concrete. This again shows that the air that was lost was large bubbles. However, when the SAM Number is > 0.28 before the paver then it appears the vibration increases the SAM Number. Again, this shows the vibration removes the large bubbles from the matrix.

This highlights the importance of using the SAM Number to determine the quality of the air void system of the plastic concrete and not relying on the total air volume in the concrete. Based on the data from the field mixtures, if a concrete mixture has a satisfactory SAM Number, either at the plant or before the paver, then the concrete would be expected to have a satisfactory air void system in place. This means that sampling concrete behind the paver would not be necessary. This also means that producers should not focus on air volume but should instead focus on obtaining the correct SAM Number and just ensuring the air volume is > 4%. Implementing this would allow the producers to focus on producing a satisfactory air void spacing and measuring that with the SAM instead of focusing on the volume of air in the mixture.

During Phase I, our team had initially recommended that a SAM Number < 0.20 and air content > 4% be used for the mixture design and evaluated in the lab; and a SAM Number < 0.30 and air content > 4% in the field. Our team additionally proposed an option to have an action limit where the contractor would be required to take action if the SAM Number is between 0.25 and 0.30. This would likely be done by increasing the air content in their mixture.

However, when comparing Phase I to Phase II SAM numbers, it is interesting to note the differences. Shown in Figure 67 below, the Phase II SAM numbers are consistently lower than the Phase I. Both Phase I and Phase II SAM numbers represent material sampled after the paver. These differences could be caused by the varying consolidation, where Phase I was rodding or vibe, and Phase II was using the MinT. However, OSU has shown a closer correlation between the MinT and rodding. Therefore, it is more probable that the lower SAM numbers in Phase II are affected by the use of an Optimized Gradation (Tarantula Curve). In Phase I, three of the projects did not use an Optimized Gradation (Appleton, Columbus and West Waukesha Bypass). More importantly, five out of the eight projects exceeded the Warning Band. The Phase I Menomonie, I-39 Rock County and I-39 Dane County projects were the only project to use an Optimized Gradation *and* remain within the Warning Band. These projects have some of the lowest Phase I SAM numbers, which is more comparable to Phase II. All the Phase II projects used an Optimized Gradation and were all within the Warning Band. Lastly, the Zoo Interchange project was tested during very cold conditions which affected variability as discussed later. These observations prove that a field limit of 0.30 is achievable with an Optimized Gradation mix design.

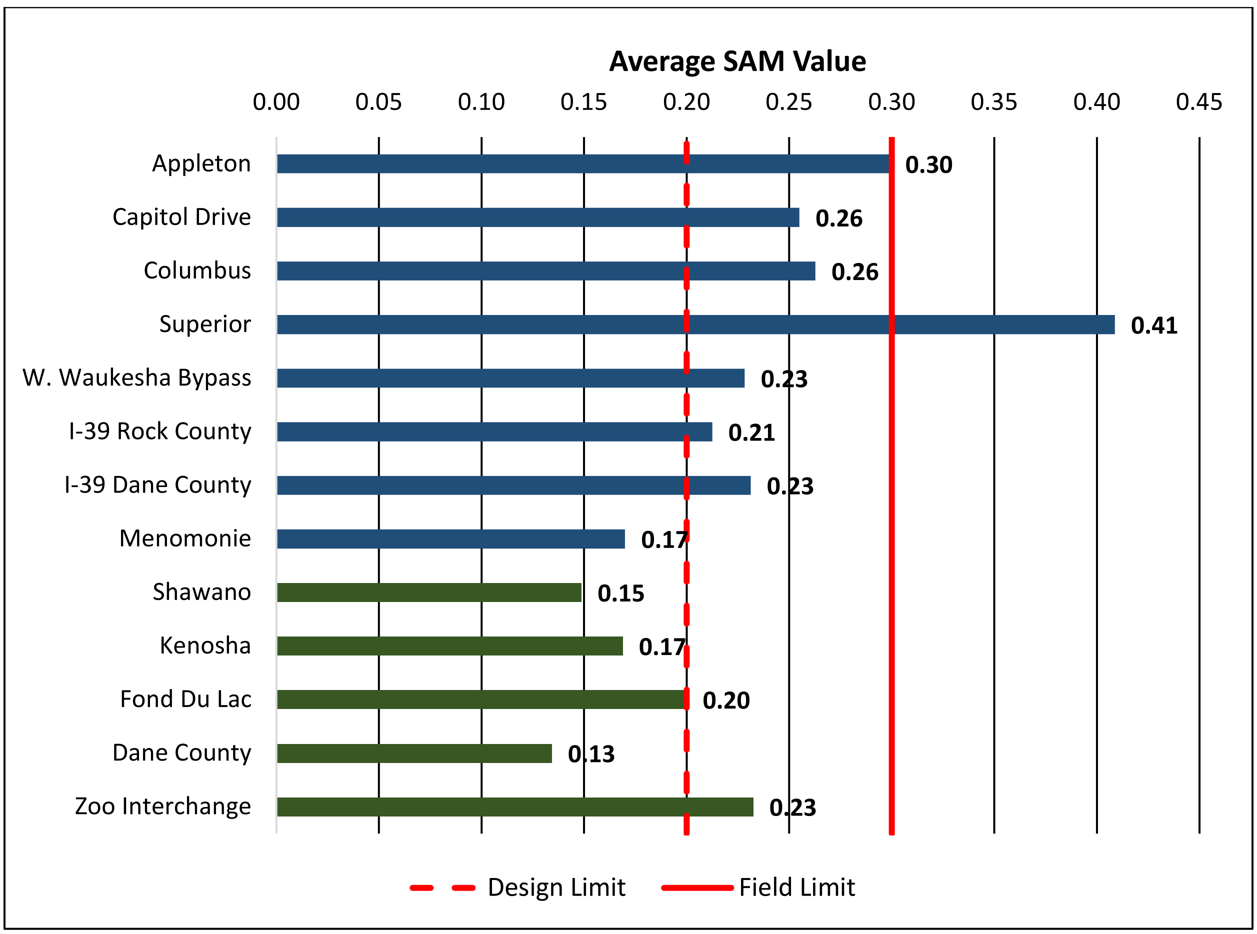


Figure 67: Comparison of Average SAM values from Phase I Projects using Rodding and Battery Vibration Consolidation to Phase II Projects using MinT Consolidation. Phase I Projects are Shown with Blue Bars and Phase II Projects are Shown with Green Bars.

5.3.1.1 SAM Consolidation

During Phase I, the research team was investigating various methods to consolidate for SAM testing. Figure 68 shows the comparison of the SAM Numbers from samples collected during Phase I, and different ways to consolidate the concrete. One set of tests used the battery vibrators that were placed within the unit weight bucket and the others used rodding. The results show that the battery-operated vibrators had a much higher variability or error bars and were not similar for two out of three measurements made on I-39 Dane County and for one out of three measurements for I-39 Rock County. This means that 1/3 of the measurements showed a range of variability. However, none of the rodded tests showed a large amount of variability in the testing. This suggests that an electric vibrator inserted into the unit weight pot should not be used to consolidate the concrete. The two projects with high variability also did not meet the Tarantula Curve limits. It may be possible to allow internal vibration for mixtures within the Tarantula Curve, but the data obtained is not enough to determine this. This trend will be investigated further in Phase II.

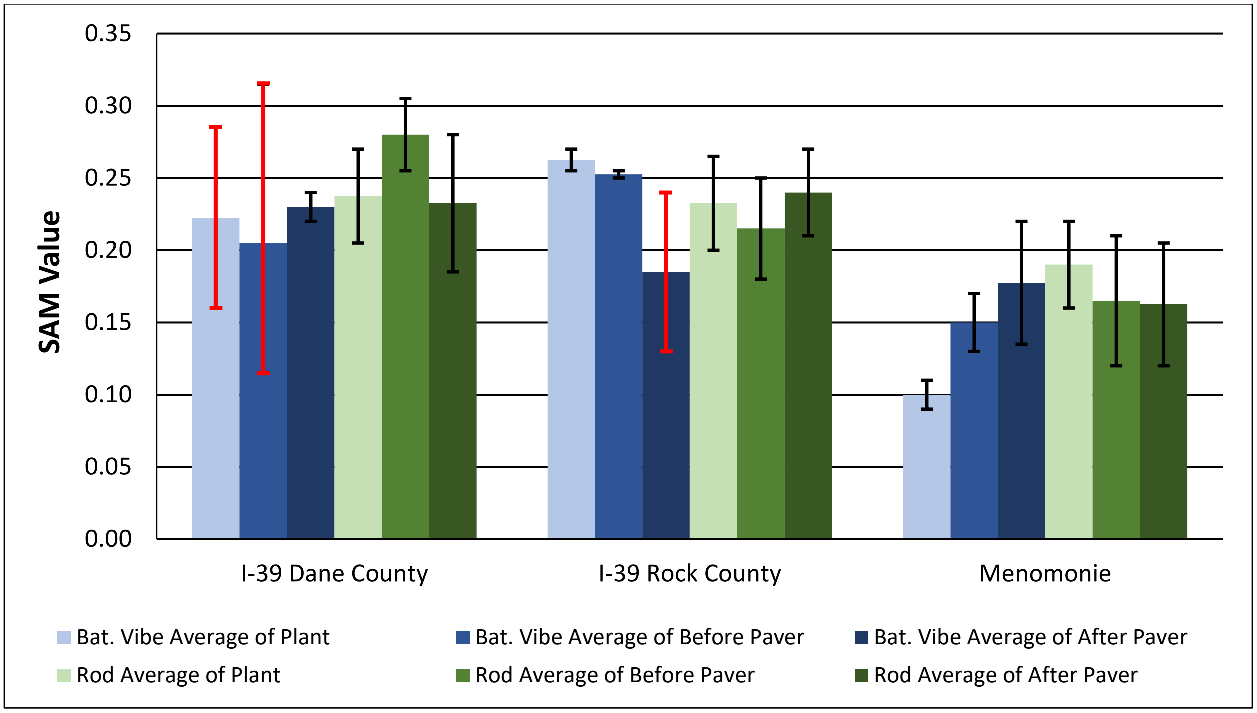


Figure 68: Consolidation method comparison for SAM values by location. Error bars represent the measured low and high SAM values.

Because of the variation exhibited in Phase I, additional consolidation methods were investigated during Phase II. For all field projects during Phase II, samples were consolidated by rodding and using the MinT. Figure 69 on the following page shows the SAM number and standard deviation for both consolidation types. This data is separated by session (morning, noon and afternoon). Please note, we kept the same technicians on each project, and in most instances using the same gauge. One important item to note is the weather. Both the Dane County and Zoo Interchange projects were sampled in the late fall (11/2/2021 and 11/18/2021) where temperatures were 41°F and 36°F (5°C and 2.2°C) respectively.

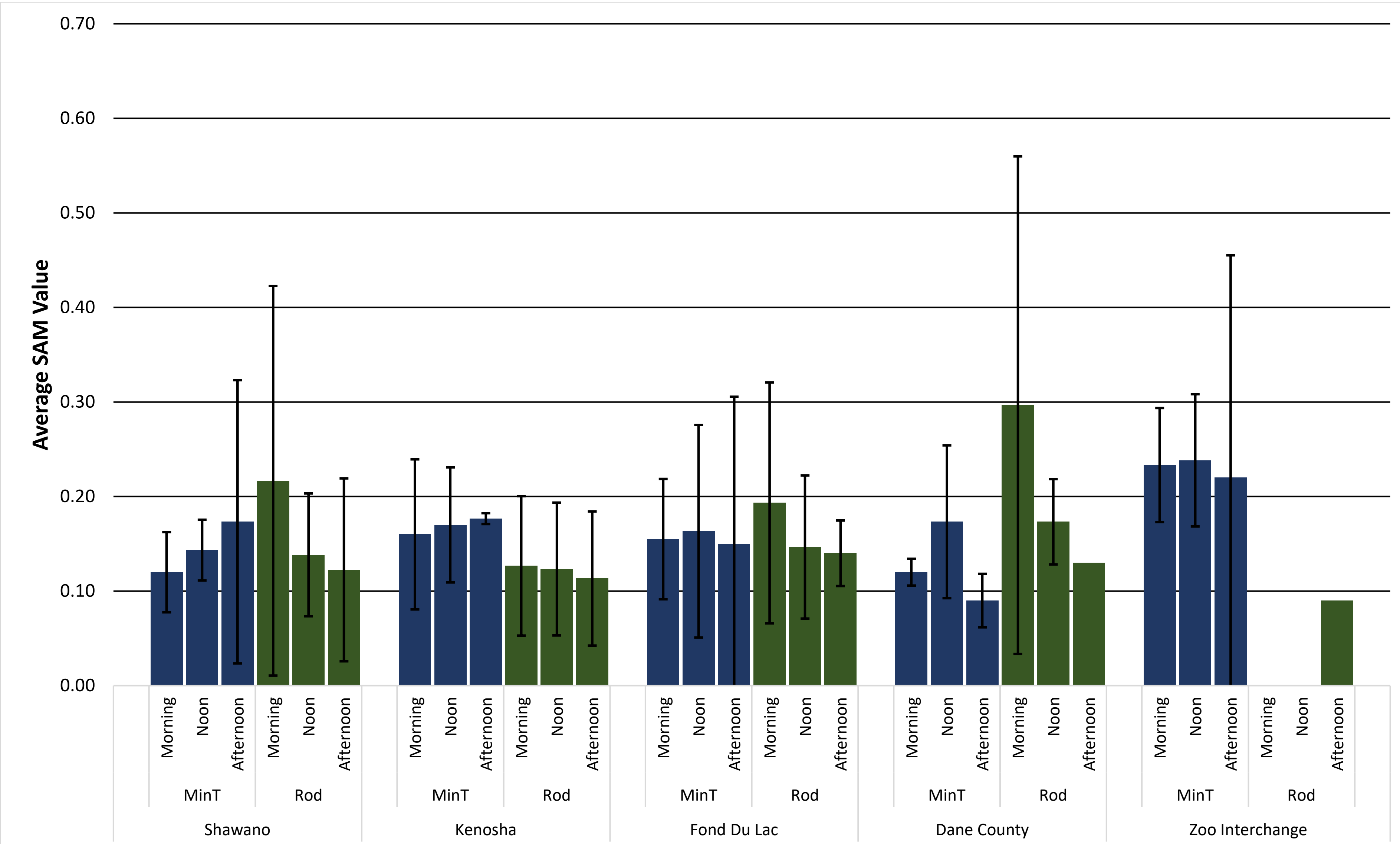


Figure 69: Average SAM Value for Each Location, Consolidation Method, and Session.

On the Dane project we ran into some issues where the SAM testing was extremely variable for both rodding and MinT, and in some instances resulting in errors. Additionally, when using the MinT, there were multiple instances where material had hardened in the lower chamber of the SAM meter and was difficult to remove when cleaning the pot. It was because of these anomalies that the Zoo Interchange project was added to further investigate the MinT. During the Zoo project we focused our efforts on the MinT, with slight variations, and only tested once by rodding. Unfortunately, our efforts were futile to explain the high variability.

We noticed variable SAM values for both the Dane and Zoo projects, which we believe were affected by the ambient temperature. We observed hardened mixture in the lower chamber of the SAM, which could have been attributed to a physically cold mix. Since we did not check the actual mixture temperature during the SAM test, we do not know if mixture temperature was the cause. More likely, cold ambient temperatures affected the mixture production. For instance, producing in cold weather comes with additional obstacles such as possible adjustments to admixture dosage, reduction of water, etc. This theory is more plausible considering the Box Test results were also variable, which coincides with inconsistent mix properties and possibly a low w/c ratio. Therefore, we do not believe this is a testing issue, rather an inconsistent production issue.

Regarding the MinT vs. Rodding, Figure 70 below lists the standard deviations for both forms of consolidation. We did not perform multiple Rodding tests during the Zoo Interchange visit, therefore no comparison between the MinT and Rodding standard deviations exists. For the remaining projects, the MinT resulted in a lower standard deviation on three out of four times.

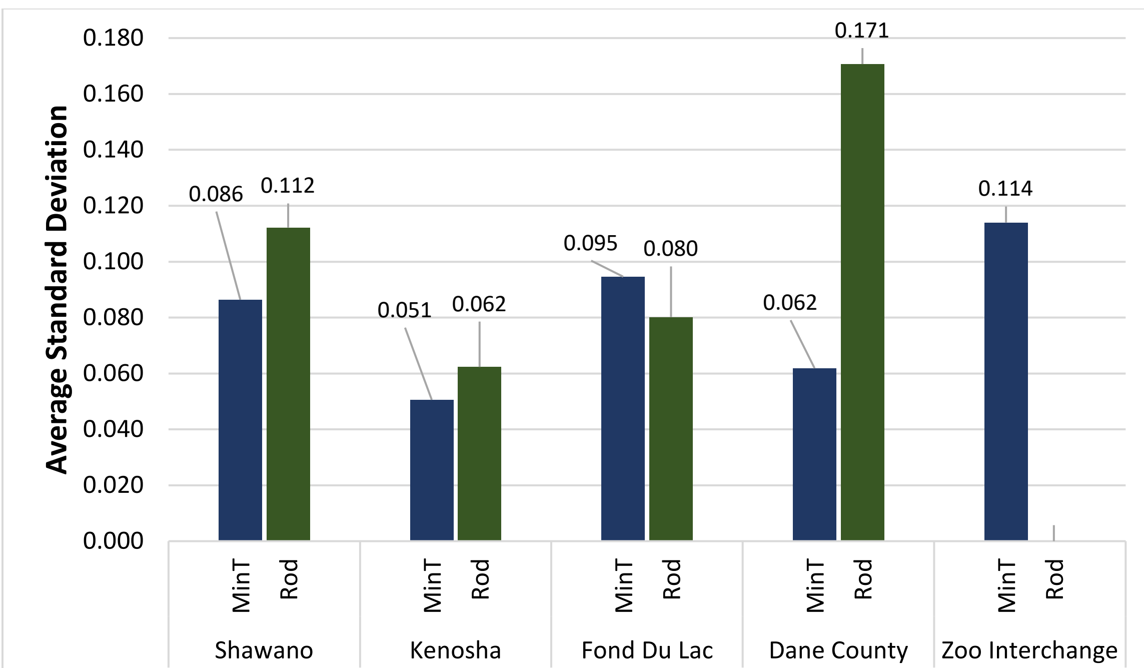


Figure 70: SAM Standard Deviation – MinT vs. Rodding.

5.3.2 Apparent Surface Resistivity, Surface Resistivity and Bulk Resistivity Relationship

Figure 71 illustrates the geometry correction that needs to be applied to the Resipod measurements to convert the probe measurement (apparent surface resistivity) to the actual resistivity of the concrete and enable it to be compared to other measures of resistivity. It can be noticed that as the diameter of the cylinder increases the correction tends toward a value of 1 (becoming closer to the assumption of the semi-infinite half space). As the diameter of the sample decreases the correction becomes larger. As a result, the correction for a 4 in. x 8 in. (101.6 mm x 203.2 mm) cylinder is 1.85 (assuming a 1.5 in. [38.1 mm] probe spacing) and the correction for a 6 in. x 12 in. (152.4 mm x 304.8 mm) cylinder is 1.37 (assuming a 1.5 in. [38.1 mm] probe spacing).

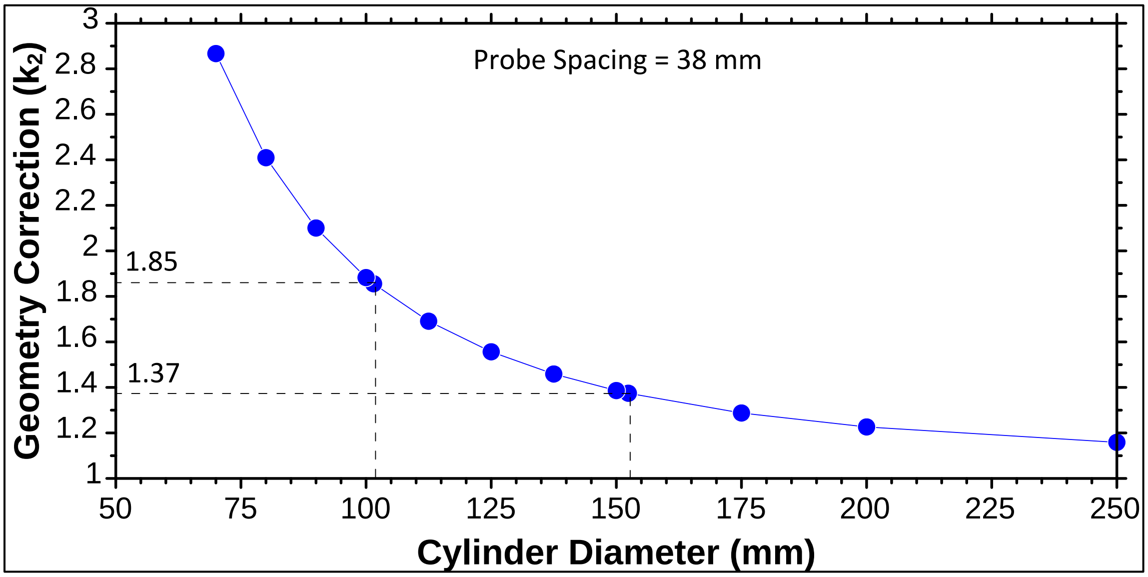


Figure 71: Geometrical Correction for Surface Resistivity Determination

The results are plotted in Figure 72(a) against the bulk resistivity and in Figure 72(b) after the correction factor is applied. It can be noticed that a 'best-fit' correction in Figure 72(a) would have a slope of 1.31 however the data is fit nearly as well with the theoretical value of 1.37. When the correction is applied to yield the actual surface resistivity [Figure 72(b)] the parity plot shows strong correlation between the surface resistivity and the bulk resistivity. The coefficient of variation of the surface resistivity test was 7.25% while the bulk resistivity was 7.0%.

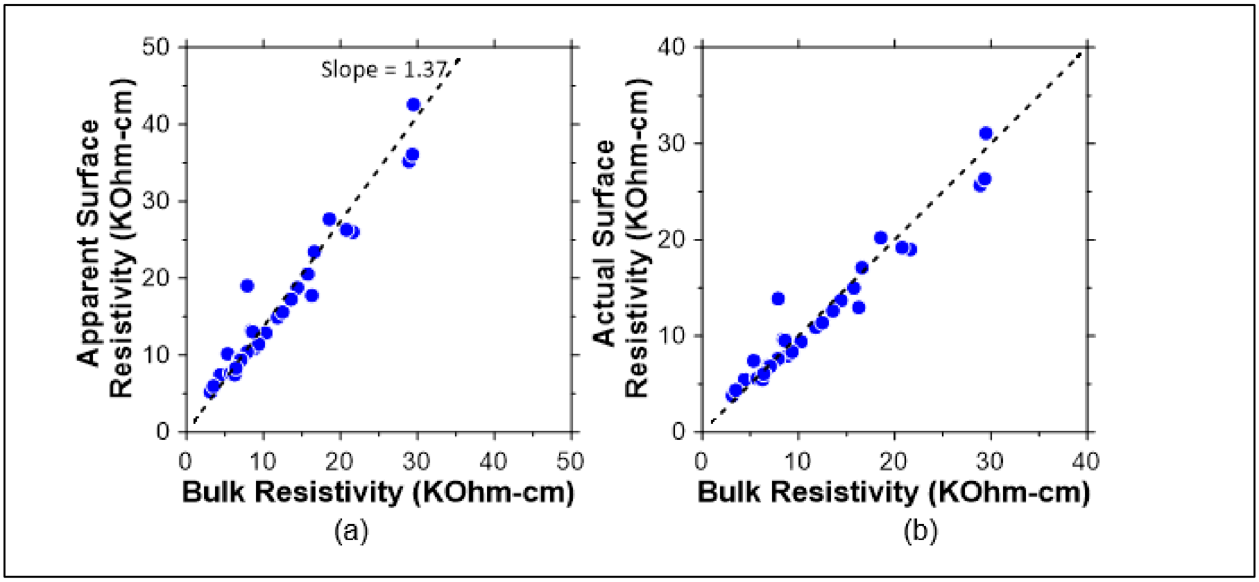


Figure 72: (a) Bulk Resistivity vs. Measured (Apparent) Surface Resistivity. (b) Bulk Resistivity vs. Corrected (Actual) Surface Resistivity.

5.3.3 Surface Resistivity

All data tested per AASHTO T358 – Surface Resistivity, was corrected for geometry and conditioning using the following calculation:

$$\rho=\lambda*\rho_m/k \tag{10}$$

ρ_m = set averaged measured resistivity

λ = 1.1 (the leeching correction factor when conditioned in lime-water tanks)

λ =1 if *not* conditioned in lime water

k = 1.37625 (the probe and sample shape correction – used for only 6-in. [152.4 mm] cylinders and a probe spacing of 1.5-in. [38.1 mm])

The results for the surface resistivity testing for Phase I are shown in Figure 73. The results match what is expected as the resistivity continued to increase over time. Figure 74 shows the resistivity values in terms of the performance levels discussed in AASHTO T358 (listed in Table 2 of this report). It can be noticed that mixtures from Appleton, Superior and Menomonie had values lower than 15 kΩ-cm with Columbus having values lower than 10kΩ-cm. These mixtures (along with Capital which was very near 15kΩ-cm) all were near or had exceeded specification limits on the ½ in. (12.7 mm) aggregate sieve. Figure 74 shows that all the projects are reaching the low permeability value except for Columbus. It is not clear why Columbus did not reach the low resistivity in the same way as the other specimens. Additional testing is needed to better understand this. These results show that surface resistivity is a helpful tool to evaluate the concrete.

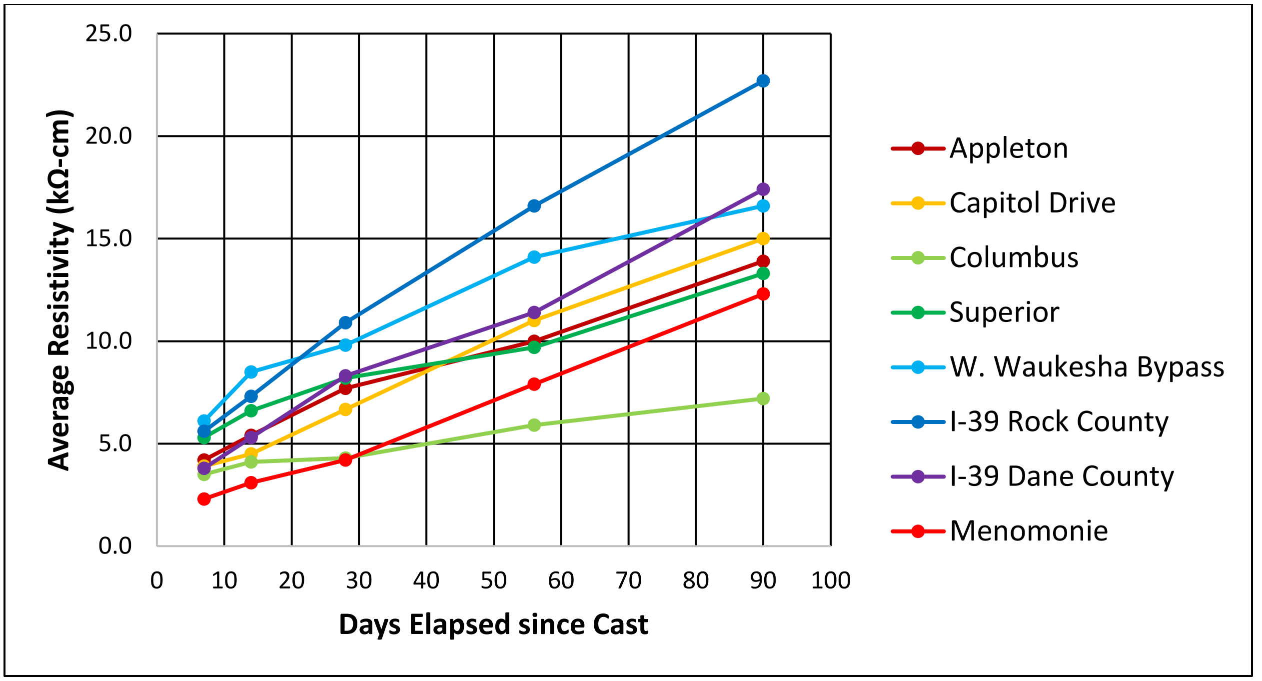


Figure 73: Resistivity vs. Days Elapsed Since Casting using Surface Resistivity.

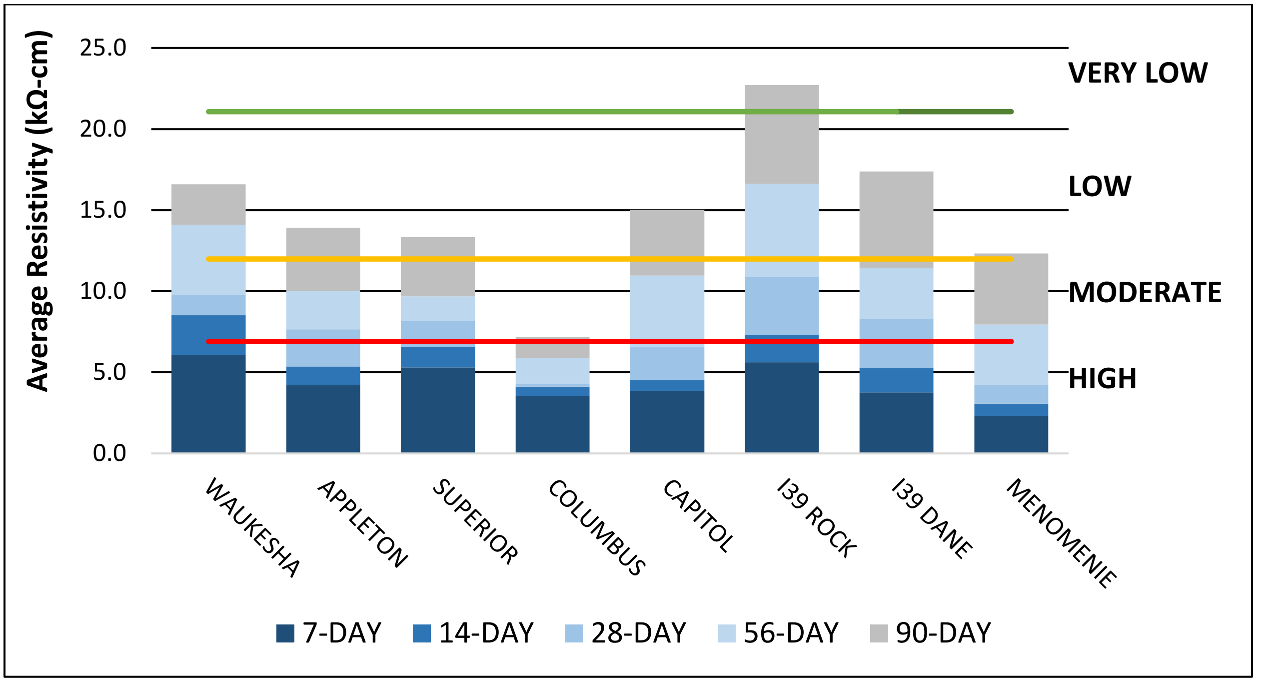


Figure 74: Resistivity Gain by Location using Surface Resistivity.

5.3.3.1 Conditioning Method Evaluation

During Phase II various conditioning methods were evaluated. Initially, only Lime tank and the bucket methods were going to be evaluated. However, after the first field visit, it was decided to add the accelerated and sealed conditioning methods. The accelerated method is where the samples are placed in a high temperature lime tank and the sealed method is where the samples are sealed within plastic bags. Testing methods for the four curing conditions are found in Appendix A1. Figure 75 shows only the 90-day resistivity for the lime tank, bucket method and sealed conditioning methods compared to the 28-day accelerated conditioning method.

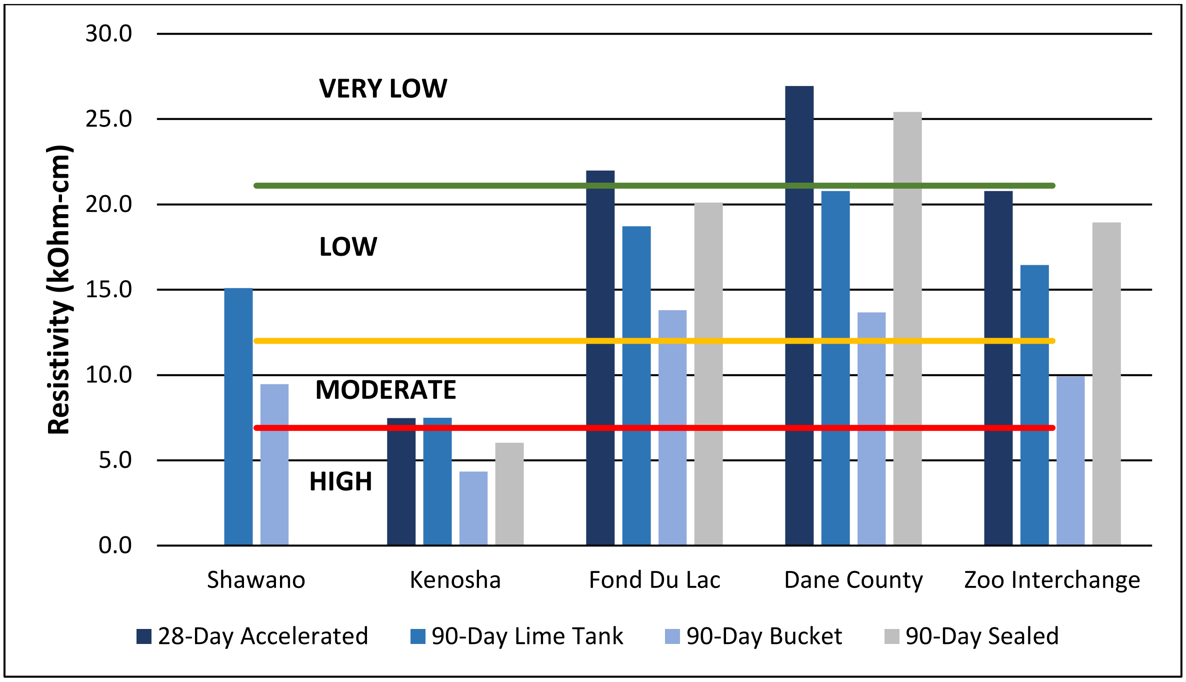


Figure 75: Surface Resistivity 28-Day Accelerated vs. Multiple 90-Day Curing Methods.

The Kenosha mixture did not include any SCMs and the Shawano mixture only included 15% Fly Ash. This points to the value of SCM in improving the resistance of the concrete as observed in many other studies by altering the connectivity of the pore space.

When comparing the conditioning methods, it can be observed that in general the 28-day accelerated curing was higher than the 90-day lime testing (Similar observations were discussed by Bu et al 2012 along with potential corrections). The 90-day bucket test had lower values than the samples stored in lime as they prevented the leaching of ions which increase the pore solution resistivity and therefore the concrete. The sealed samples have a higher resistivity due to self-desiccation which reduces the degree of saturation. These findings are consistent with the findings reported by Spragg et al. 2013 [17].

From a testing standpoint, the 90-day Lime Tank conditioning was the most versatile considering most labs can complete this conditioning without any lab modifications. However, the 90-day testing time is extensive. The 90-day Bucket conditioning method will be difficult for labs to condition a high volume of samples due to the cost of the solution and the extra needed storage space for the individual buckets. The 90-day Sealed conditioning method initially seemed easy using bags and existing lime tanks. However, we found that some of the bags leaked, even when they were double bagged. More specific requirements for types of bags would be needed to properly specify the Bucket conditioning method. Lastly, the 28-day accelerated conditioning method was difficult in that it requires a dedicated tank at the higher temperature. However, this conditioning method, once set up with a separate tank, can allow for a higher volume of samples and has a much shorter testing window of 28-days as opposed to 90-days.

Regardless of the conditioning that is chosen for use, it is important to note that care should be taken in interpreting the pore solution of these materials if further analysis is performed.

5.3.3.2 Conditioning Solutions and Bulk Resistivity Data

The results of the testing are provided in Table 17. The average conductivity for the "LIME" solutions (for 3 samples), assuming the three solutions are replicates was 3.76 mS/cm (0.266 kΩ cm) at 25°C (77°F). The average conductivity for the "CHEM" solutions (for 3 samples), assuming the three solutions are replicates was 168.10 mS/cm (0.006 kΩ cm) at 25°C (77°F).

Table 17: Results of the Solution Testing

Solution ID	Conductivity at 25 °C (mS/cm)	Average conductivity (mS/cm)	Standard deviation
LIME 1	3.62	3.76	0.15
LIME 2	3.92		
LIME 3	3.74		
CHEM 1	167.30	168.10	0.72
CHEM 2	168.60		
CHEM 3	168.50		

We can begin with the saturated lime solution. A saturated lime solution generally has a conductivity of 10.3 mS/cm at 25°C (77°F). While counterintuitive, the addition of ionic species can cause this to decrease as conductive ions (OH-) are swapped for less conductive ions until the concentration becomes large enough that this trend is revered.

According to AASHTO TP-119-22 the cylinders should be stored in a calcium hydroxide saturated pore solution with a resistivity of 0.0128 kΩ cm (in conventional pore solution units 0.128 Ω m) (78.1 mS/cm). The calcium hydroxide-saturated, simulated pore has been proposed in AASHTO TP-119 to be made using 7.6 g/L NaOH (0.19 M); 10.64 g/L KOH (0.19 M); 2 g/L $Ca(OH)_2$. A sample 'recipe' of the solution is provided that is 13,250.0 g water, 102.6 g NaOH, 143.9 g KOH and 27.0 g $Ca(OH)_2$. While the previous ''recipes'' for solution are useful, the key parameter is that when complete resistivity of the simulated pore solution is 0.0128 kΩ cm (78.1 mS/cm). While reagent grade chemicals were originally specified this has been reduced to chemicals with the following purity levels: Calcium Hydroxide ($Ca(OH)_2$) > 97%, Sodium Hydroxide (NaOH) > 96%, Potassium Hydroxide (KOH) > 85%. It should also be noted that since the solution is saturated with lime, lime will precipitate out of the mixture.

This project used recipe consisting of 6 g/L NaOH; 41 g/L KOH; 3 g/L $Ca(OH)_2$. This variation in the salt concentration accounts for the conductivity of the CHEM solution being 0.006 kΩ cm (168.1 mS/cm) rather than the 0.013 kΩ cm (78.1 mS/cm) that is more typical for AASHTO TP-119 testing. As such, if formation factor is determined for these mixtures this alternative resistivity should be used.

The results of the measured bulk resistivities are shown in Table 18.

Table 18: Bulk Resistivity Data

Contract	WHRP - PEM			WHRP - PEM				WHRP - PEM				WHRP - PEM				WHRP - PEM			
Cast Date	9/1/2021			10/12/2021				10/14/2021				11/2/2021				11/23/2021			
Project Name	STH 29			STH 50				STH 50				IH 39				Zoo IC			
Project ID	1053-02-73			1310-10-70				1440-15-71				1007-12-80				1060-33-84			
Cast Location/ID	WH-108	WH-109	WH-110	WH-114	WH-115	WH-116	WH-117	WH-127	WH-128	WH-129	WH-130	WH-137	WH-138	WH-139	WH-140	WH-147	WH-145	WH-144	WH-146
Curing Method	**Lime**	**Lime**	**Bucket**	**Bucket**	**Accelerated**	**Lime**	**Sealed**	**Bucket**	**Accelerated**	**Lime**	**Sealed**	**Bucket**	**Accelerated**	**Lime**	**Sealed**	**Bucket**	**Accelerated**	**Lime**	**Sealed**
7-Day Average	~	~	~	3.34	4.20	4.33	4.80	2.41	3.39	3.20	3.90	2.74	4.88	5.05	9.38	2.85	2.75	3.58	3.69
14-Day Average	~	~	~	2.65	3.81	4.40	4.58	3.55	7.03	4.97	5.88	5.11	9.73	6.63	12.00	3.23	7.98	5.01	5.08
28-Day Average	8.38	7.44	5.33	2.74	7.25	7.00	5.58	5.98	21.60	6.92	8.54	7.27	24.42	9.98	18.94	4.39	21.63	5.01	8.84
56-Day Average	11.82	10.27	5.34	3.12	6.92	5.73	6.25	9.35	28.89	12.48	16.27	8.42	29.35	14.42	20.77	5.60	22.60	12.60	14.30
90-Day Average	15.76	13.58	8.60	3.50	7.88	7.07	6.39	7.88	29.48	16.61	18.56	~	~	~	~	~	~	~	~
7-Day STD DEV	~	~	~	0.09	0.14	0.13	0.18	0.06	0.23	0.07	1.07	0.04	0.25	0.39	0.70	0.08	0.04	0.14	0.36
14-Day STD DEV	~	~	~	0.11	0.06	0.18	0.61	0.50	0.65	0.65	1.26	0.23	0.94	0.33	4.13	0.09	0.47	0.26	0.56
28-Day STD DEV	0.30	0.33	0.25	0.05	0.13	0.19	0.62	0.33	0.48	0.12	1.61	0.40	3.56	0.61	6.73	0.04	0.56	0.26	0.55
56-Day STD DEV	0.87	0.43	0.01	0.08	0.34	0.24	0.28	1.05	0.86	0.01	1.06	2.22	3.50	0.74	2.57	0.24	0.56	0.38	1.71
90-Day STD DEV	0.39	0.40	0.46	0.08	0.37	0.32	0.30	0.81	0.51	0.14	1.75	~	~	~	~	~	~	~	~
7-Day COV	~	~	~	2.83	3.31	3.00	3.65	2.40	6.85	2.07	27.35	1.39	5.05	7.78	7.43	2.63	1.57	3.97	9.80
14-Day COV	~	~	~	4.11	1.52	3.98	13.39	14.08	9.20	13.13	21.48	4.52	9.63	4.95	34.42	2.80	5.88	5.12	11.09
28-Day COV	3.62	4.44	4.70	1.90	1.72	2.70	11.20	5.44	2.22	1.67	18.84	5.44	14.58	6.08	35.52	0.87	2.60	5.12	6.22
56-Day COV	7.34	4.19	0.27	2.45	4.92	4.17	4.51	11.19	2.98	0.12	6.53	26.36	11.93	5.12	12.38	4.29	2.48	3.02	11.96
90-Day COV	2.46	2.95	5.33	2.14	4.68	4.55	4.73	10.25	1.72	0.87	9.41	~	~	~	~	~	~	~	~

It can be noticed that the lowest bulk resistivity is 2.4 kΩ-cm and the highest value is 29.48 kΩ-cm. If we compare the different solutions, we can observe that the samples in the bucket always provide the lowest values (as observed in Phase I). The increase in the bulk resistivity of the samples stored in lime water has been attributed to the leaching of ionic species from the sample, the increase in bulk resistivity in the accelerated samples is due to accelerated leaching and the temperature used, and the increase in the sealed samples is due to the reduction in the saturation level. The samples in the solution, lime water, and accelerated curing have a similar coefficient of variation while the sealed sample had a much higher coefficient of variation (though many of the samples were noted to have been exposed to leaking).

5.3.4 Porosity and Formation factor

Figure 76 illustrates a summary of results from porosity and formation factor measurements. The tested specimens from three different projects show a similar average porosity value (≈16 – 17%). However, the specimens from Rock County show the highest formation factor (i.e., lower permeability) when compared to the tested specimens from Dane County and Menomonie. The specimens from Menomonie have the lowest formation factor. The formation factor results trend similarly with the results from surface electrical bulk resistivity (Figure 74). In addition, a higher variation exists in the formation factor data measured in Menomonie specimens. This variation can be attributed to the poor consolidation of the samples from Menomonie, as illustrated in Figure 77. It is worth noting that while the mix was expected to exhibit good consolidation as was observed in the Box Test in Figure 46, and meeting Optimized Aggregate Gradation specifications shown in Figure 45, there were still issues with consolidation when producing the cylindrical specimen. This is thought to be attributed to different consolidation methods used for the Box Test and to produce cylinders. Additionally, there could have been variations in production material.

According to AASHTO PP 84, Formation Factor values under 520 are associated with high permeability/susceptibility to chloride penetration as shown in Table 3. All three locations tested for Formation Factor exhibited high susceptibility which contrasts with the surface resistivity results which indicate much lower susceptibility to chloride penetration for these projects.

This phenomenon is likely caused by differences in curing conditions between the two tests. Surface resistivity, per AASHTO T 358, recommends using a 100% relative humidity cure room, while the Formation Factor corrects for the pore solution in the sample. The cure room conditions allow samples to leach calcium hydroxide and more importantly alkalis from the pore solution, which will cause the samples to appear more resistive than they really are, thereby explaining the reported differences in chloride ion susceptibility according to AASHTO PP 84. Higher formation factors, which are desirable, can be achieved using larger volumes of reactive supplementary cementitious materials and lower water to cement ratios. While national specifications are still being developed for the formation factors that can be expected from current mixtures, a study of InDOT paving mixtures has resulted in formation factor values typically between 250 and 400, and for MnDOT paving mixtures between 400 and 1,500.

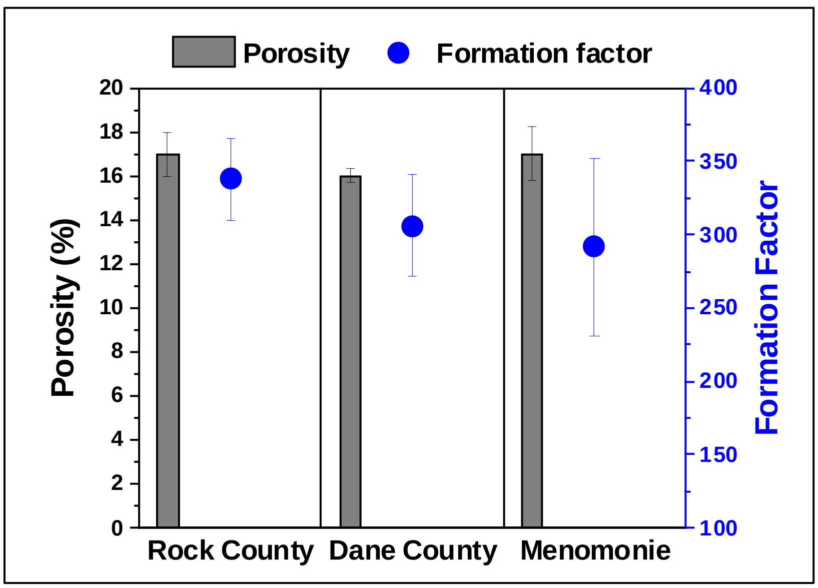


Figure 76: Porosity and formation factor based on bulk electrical resistivity measurement.

Figure 77: Example of specimens from Menomonie with poor consolidation.

5.3.5 Coefficient of Thermal Expansion

Concrete specimens were tested for their coefficients of thermal expansion (CTE). The results are shown below in Figure 78 and Figure 79, and are arranged by aggregate type. While all the coefficients ranged from 4.1 to 6.4 x 10^{-6}/°F (typical observed values for PCC are between 3.3 to 11.2 x 10^{-6}/°F. [44]), it is apparent that the primary aggregate types used in the mix have an influence on the coefficient of thermal expansion. Mixtures containing gravels fell somewhere in the middle range of coefficients while exhibiting more variability between project locations. Dolomite containing mixtures also had similar coefficients of thermal expansion to the gravels, but with what appears to be a slightly lower variability between locations. Limestone exhibited, on average, the lowest coefficient of thermal expansion, and Quartzite the highest. Because lower coefficients of thermal expansion are thought to be desirable for their greater volumetric stability under changing temperatures, this would mean gravels and limestones are more ideal aggregate sources for concrete when temperature susceptibility is of primary concern. Therefore, this data shows the importance of aggregate types to thermal properties of concretes.

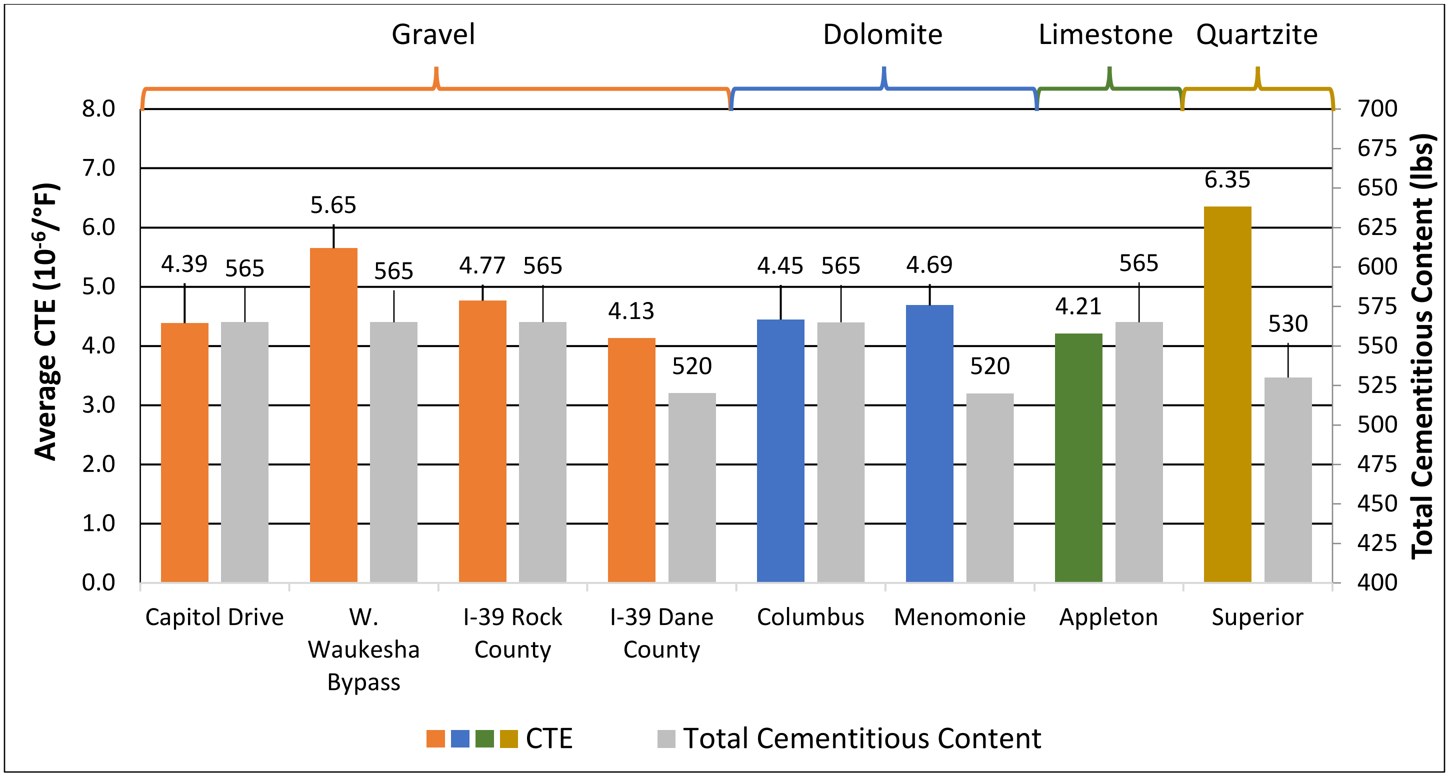


Figure 78: Average Coefficients of Thermal Expansion by Total Cementitious Content and Aggregate Type.

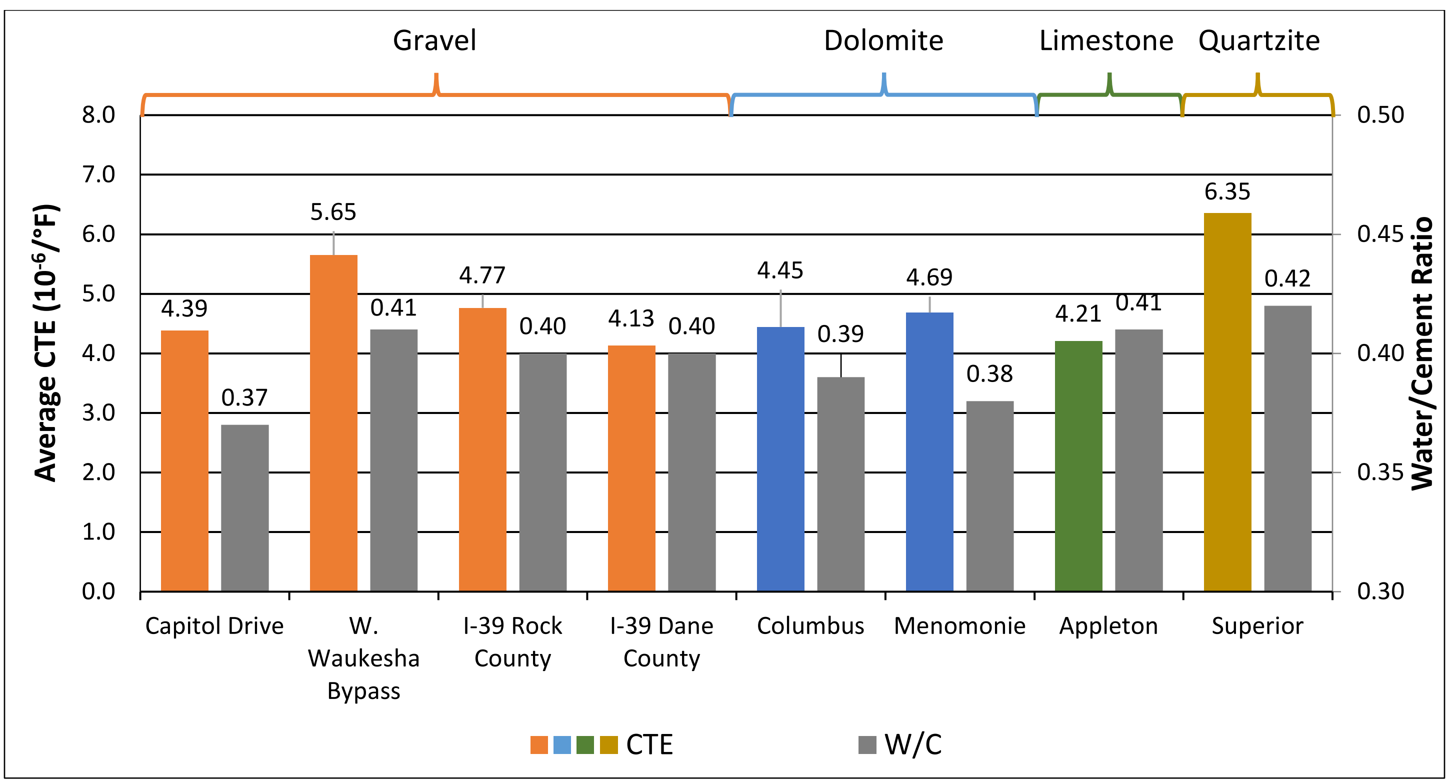


Figure 79: Average Coefficients of Thermal Expansion by w/c Ratio and Aggregate Type.

Average values for coefficients of thermal expansion of PCC with specific aggregate types are shown below in Table 19. Note that the measured CTE values from this study follow similar trends as reported in the FHWA table below with Limestone having the least CTE value, Dolomite following in the middle, and Quartzite showing the largest CTE value.

Table 19: PCC Coefficients of Thermal Expansion based on Aggregate Type from FHWA [43]

PCC CTE based on Aggregate Type	
Aggregate Type	**Average from Data Used in Model (x 10^{-6} in./in./°F)**
Basalt	4.86
Chert	6.90
Diabase	5.13
Dolomite	5.79
Gabbro	5.28
Granite	5.71
Limestone	5.25
Quartzite	6.18
Andesite	5.33
Sandstone	6.33

5.4 Analysis of Workability Properties

The workability of concrete can be described by how easily freshly mixed concrete can be mixed, placed, consolidated and/or finished. The Box Test and V-Kelly Ball tests attempt to quantify the workability of concrete for use in formwork or pavers.

5.4.1 Box Test

While the Box Test is a test to determine workability, our results highlighted a different concern and that is consistency. Figure 80 highlights the mixtures with noticeable variations in the Box Test values. Not every field project is included in Figure 79, just a few examples to show the various consistency. The first four mixtures (Capitol Drive, I39 – Dane County, I39 – Rock County and USH 2 – Superior) are from Phase I, and the last three mixtures (STH 50, STH 23 and I39/90) are from Phase II.

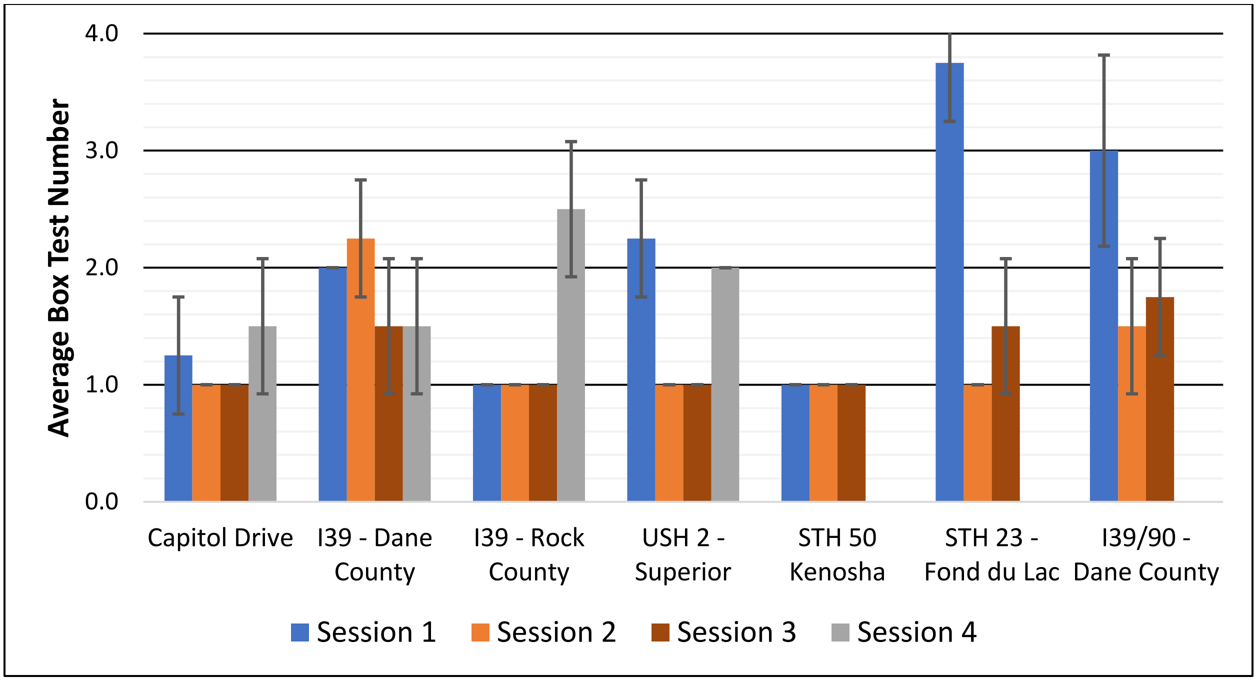


Figure 80: Box Test Number Variability Analysis.

Figure 81 highlights the technician's observations from the field visits, which was an inconsistent mixture from session to session. While consistent mix is beyond the scope of this research, it may explain some of the variable data from session to session. In Figure 80 below, the SAM testing averages and error bars for Kenosha were more consistent than those exhibited for Dane County, which is similar to the variability of the mixture as was observed during the Box test.

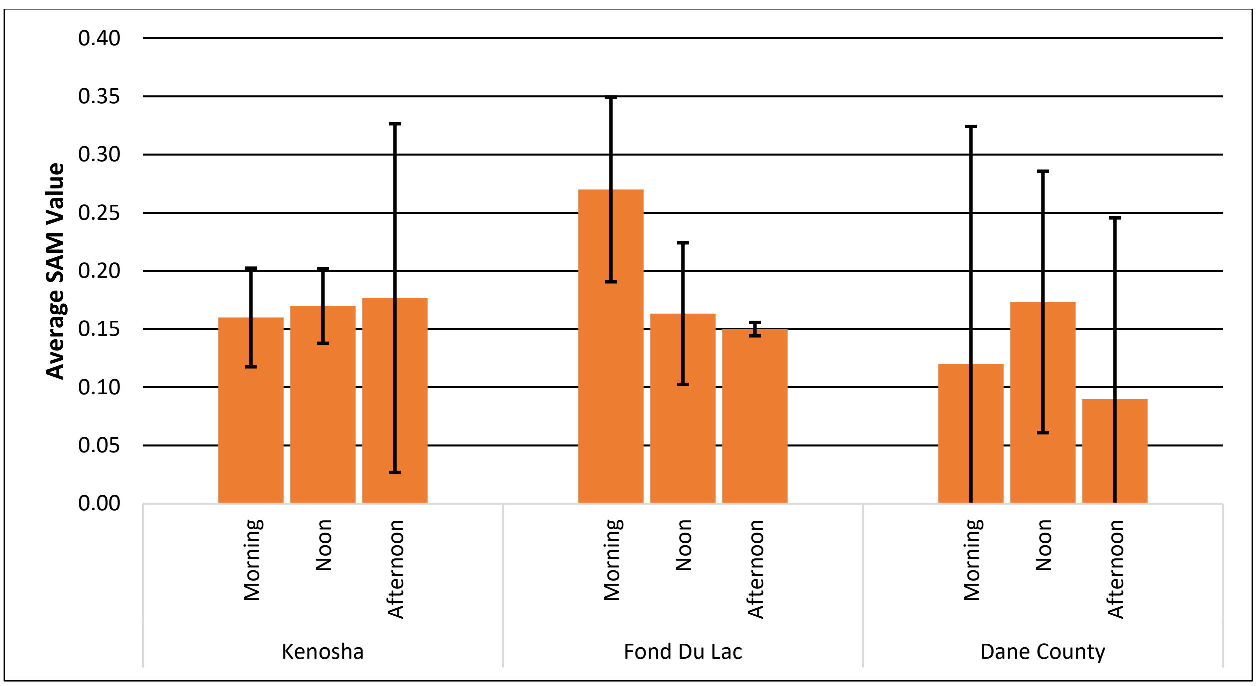


Figure 81: SAM Value Variability Analysis using MinT Consolidation.

5.4.2 Comparing the Box Test and V-Kelly

Figure 82 shows a comparison of the Phase I V-Kelly and the Box Test Number for the values reported in Table 15. The results show that there is general agreement between the two measurements as the lowest V-Kelly measurements did show the highest Box Test values and vice versa, however, this correlation was not as strong as expected. This demonstrates that the V-Kelly can give a rough indication of the plastic concrete's ability to minimize surface voids under vibration as experienced through the slipform paver, but the Box Test is a much better indication of finishability.

It should also be noted that in the field, workability properties varied significantly within certain projects and are demonstrated by the Box Test photos in Figure 42 and Figure 44. Within these figures it is noted that some samples have significant edge slump while others have almost none. These changes can come from a variety of sources including mix adjustments, change in aggregate moistures, etc. This same trend is also shown in Figure 82 as certain projects had consistent V-Kelly results while other projects had much wider ranges.

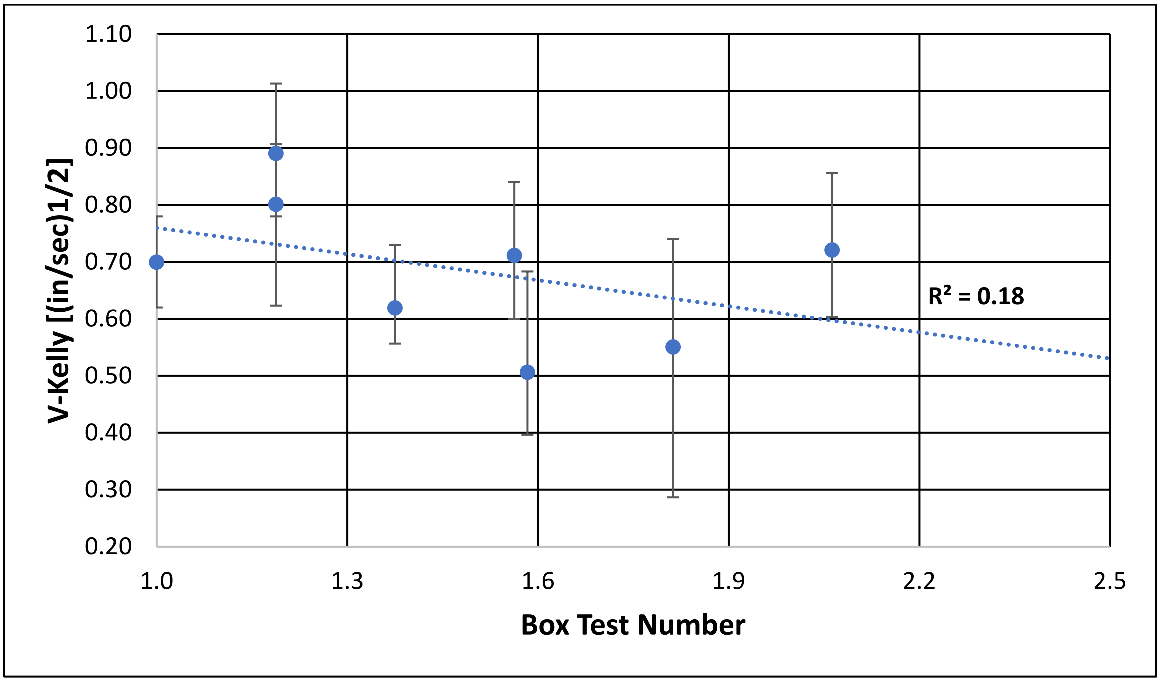


Figure 82: Average Box number vs. Average V-Kelly Index for workability.

*Note: Error bars represent the measured V-Kelly low and high values.

6.0 Summary

This section summarizes the major findings of this study including strength, durability, and workability properties.

6.1 Durability

Several properties of concrete are durability related, these include resistivity (surface and bulk), porosity, coefficient of thermal expansion, and the air void system. Surface and bulk resistivity are very similar in nature and measure the same property. The electrical resistivity (surface and bulk) measured from these tests can be correlated to the susceptibility to water and salt ingress; it can also be used as a reliable check for water content and water/cement ratios. Porosity and air voids are extremely important to freeze/thaw properties of concrete specimens as they allow for the expansion of ice to mitigate expansive stresses induced during freezing which can cause a variety of different failures. The coefficient of thermal expansion is used primarily for volume stability purposes. Concrete, which is a composite material (made up of cement paste and aggregates), can expand and contract with heating and cooling cycles, and the extent to which this occurs is dependent on the composite material properties. The coefficient of thermal expansion is important for factors such as the degree of warping of concrete slabs used on the roadway during normal day-to-day thermal cycling since the slab will have a thermal gradient through its depth which causes the warping.

6.1.1 Air Content

The Phase I results show that total air content is not a good indicator of Spacing Factor as several of the samples with the same air content had widely varying Spacing Factors. When the SAM Number is below the recommended design limit of 0.20, 85% of the mixtures had a Spacing Factor < 200 μm (0.008 in.). Air content testing shows that the air content typically dropped from the plant until after the paver by as much as 3.5%. This drop was commonly 1% from the plant to the site and then another 2.5% decrease when comparing the concrete before and after the paver. This created air contents that were close to 4% after the paver. There was, however, minimal change in the SAM Numbers when comparing the measurements at the plant, before the paver, and then after the paver for SAM Numbers < 0.28. These mixtures would be expected to have a satisfactory air void distribution of small and well distributed bubbles. An important finding is that the air that is lost from the plant to the job site seems to be largely coarse bubbles. This is shown because the air volume is decreasing but the SAM Number is not changing. This shows the importance of using the SAM Number to determine the quality of the air void system of the plastic concrete and not relying on the total air volume in the concrete.

When only looking at specifically the material tested behind the paver, the Phase II SAM Numbers resulted in a lower average (0.18) than Phase I (0.26). This could be contributed to the higher use of optimized gradations within the warning band in Phase II.

Some concerns from Phase II include variability of the mixtures from session to session and the late season / cold weather production. Our data indicated higher standard deviations than expected considering our team of technicians and gauges remained consistent. Observations from the technicians included mixture that seemed to vary from session to session and struggles with testing during the last two projects visited in November where the high temperatures were 41°F and 36°F (5°C to 2.2°C), respectively.

6.1.2 Surface Resistivity

As expected, with increasing curing time, all concrete specimens exhibited increased surface resistivity. A range is suggested by the AASHTO T 358 in Table 2. It shows that all the projects reached the low permeability values except for Columbus, Shawano (for bucket conditioning only) and Kenosha. Kenosha's mixture contained 0% SCMs and Shawano's mixture contained only 15% Fly Ash. It is unclear why Columbus did not reach the low surface resistivity in the same way as the other specimens.

When comparing the conditioning methods, it can be observed that in general the 28-day accelerated curing was higher than the 90-day lime testing. The 90-day bucket test had lower values than the samples stored in lime as they prevented the leaching of ions which increase the pore solution resistivity and therefore the concrete. The sealed samples have a higher resistivity due to self-desiccation which reduces the degree of saturation.

From a testing / lab standpoint, each method has their pros and cons regarding test duration, additional equipment, or supplies (chemicals or bags). The one benefit that outweighs the others is to receive test results 62 days sooner when using the accelerated conditioning method. However, the data showed that the accelerated conditioning method resulted in the highest resistivity values. This needs to be considered when determining an appropriate minimum (design or field) requirement for resistivity.

6.1.3 Porosity and Formation Factor

The tested specimens from three different projects show a similar average porosity value (≈16 – 17%). However, the specimens from Rock County show the highest formation factor (i.e., lower permeability) when compared to the tested specimens from Dane County and Menomonie. The specimens from Menomonie have the lowest formation factor likely due to poor sample consolidation.

6.1.4 Coefficient of Thermal Expansion

The coefficient of thermal expansion/contraction describes how much concrete will expand or contract under changing thermal conditions. Mixtures containing gravels fell somewhere in the middle range of coefficients while exhibiting more variability between project locations. Dolomite containing mixtures also had similar coefficients of thermal expansion to the gravels, but with what appears to be a slightly lower variability between locations. Limestone exhibited, on average, exhibited the lowest coefficient of thermal expansion, and Quartzite the highest.

6.2 Strength

Concrete cylinders and beams were broken to determine the compressive and flexural strength respectively. As has been well established for concrete, primary strength gain occurs within the first 7 – 14 days, and then increases at diminishing rate with time beyond this time. Even though theoretical relationships between compressive and flexural strength have been developed, the mixtures tested in this study with the highest compressive strengths did not always have the highest flexural strengths. These observations are potentially caused by differences in bond strength between the paste and aggregates for the different mixtures. Since concrete pavements are loaded in flexure and the bond between the aggregates and paste is important to observed flexural strength then it is logical to continue to test the flexural strength of the concrete. It is also logical to continue compressive tests for applications in which the concrete is subjected to a compressive load.

6.3 Workability

Both the Vibrating Kelly Ball (V-Kelly) and Box Test attempt to quantify the workability of a concrete mixture. The V-Kelly test uses a quantitative measurement by calculating an index based on the rate of penetration that is measured over 30 seconds. The Box Test is a subjective quantification of workability based on the presence of visible surface voids after plastic concrete has been poured into a box frame and consolidated via vibration. Both tests are subject to variability. The V-Kelly test demonstrated moderate variability even though the test is much more controlled than the Box Test. The Box Test, due to being subjectively measured, has potential for bias and variability depending on the person performing the test. Despite the variability in these two tests, there was general agreement between the results.

One of the primary purposes of using the optimized gradation (Tarantula Curve) is to maintain workability with little to no loss in performance. Having too much or too little sand in a mixture is a very important detail that often is overlooked during mix design, and the aggregate gradations directly affect segregation, ability to finish, edge slumping, and many other mixture issues. The Tarantula Curve was designed to place limits on the different aggregate sizes in order to maintain a workable mixture, and in general the Box Test results agreed with whether the mix design was within the Tarantula Curve limits. For Phase I, mixtures that were near or past the warning band but within the upper/lower limits tended to exhibit more surface voids or edge slumping and therefore lower workability and formability. Mixtures that were well within the limitations set by the Tarantula Curves exhibited good workability and formability with few surface defects. Phase II mixtures were all within the Tarantula Curve, having only two mixtures exceed the warning band. However, Box Tests were inconsistent from session to session due to field production variability.

7.0 Recommendations

This section summarizes the recommendations of the research team for consideration by WisDOT.

PEM Test	Current WisDOT Practice	Proposed PEM Recommendation	Specification
Optimized Aggregate Gradation	Optional Spec.	Implement a warning band. Fine/Coarse Limits	CMM 8-70.2.2.3
Compressive Strength	28-day	No Change	AASHTO T 22
Flexural Strength	28-day	No Change See sec. 4.2	AASHTO T 97
Surface Resistivity	N/A	Consider as a design parameter and collect data for future consideration of implementation. Recommend "accelerated moist-curing" conditions in T 358-21 Section 8.1.1.	AASHTO T 358
Slump	WisDOT Section 415.2.1(3) Slip-Formed 2.5 in. (63.5 mm) or less	Consider removing slump from the requirements for paving concrete.	AASHTO T119
Coefficient of Thermal Expansion	N/A	Consider as a design parameter and collect data for future consideration of implementation.	AASHTO T 336
Air Content	Std. Spec 501.3.2.4.2: 7% ± 1.5%	If the SAM number is acceptable, then allow Air Content to a minimum of 4%	AASHTO T 152
Super Air Meter	CMM 8-70 Attachment 4: No contractual specification limits, an acceptable SAM number is ≤0.25. Acceptable SAM numbers typically require a minimum of 4.0% air. Failing SAM numbers typically do not occur above 8.0% air.	Design Limit: 0.20 Field Limit: 0.30	AASHTO TP 118
Hardened Air Voids	CMM 8-70.5.2.4: The total air content equals or exceeds the lower control limit for the in-place concrete item. AND/OR The spacing factor is less than or equal to 0.008 in. (0.200 mm).	To be used for dispute resolution or future research.	ASTM C457
Vibrating Kelly Ball	N/A	Do not recommend incorporation into WisDOT specifications.	AASHTO TP 129

Box Test	HTCP PCC II	Recommend during Trial Batches, to replace the Slump Test with the Box Test and require <2.0. Consider using the Box Test during production to help identify inconsistency.	AASHTO TP 137

7.1 Tarantula Curve

WisDOT allows for 1.5 in. (38.1 mm) maximum aggregate size in its mixtures. The original Tarantula Curve did not have recommendations for 1.5 in. (38.1 mm) diameter aggregates. Based on conversations with WisDOT, this limit was added based on previous experience and more research data should be generated to support this limit. Previous testing done during the development of the Tarantula Curve showed that the use of aggregates greater than 1 in. (38.1 mm) in diameter did not impact the workability of the concrete and that many different aggregate combinations could be used to produce satisfactory concrete mixtures for slipformed pavement applications [37]. The results from Phase II of this study confirmed this research demonstrating that there is no difference in the Box Test, strength, resistivity, or shrinkage of mixtures with the same paste content and w/c but different amounts of large aggregate in the mixture. Based on this work, the WisDOT requirements of requiring a minimum amount of coarse aggregate retained on the 1.5 in. sieve is not improving the performance of the concrete. Instead, it is recommended that the only requirement is that the aggregate gradation meets the Tarantula Curve limits.

It is recommended to require optimized gradation for all WisDOT pavement mixtures where long-term performance is important. As part of using the optimized gradations/Tarantula Curves. It would be good in future research to investigate the use of a warning band to be used for mixture design purposes and allows for variability during production with the goal to maintain a gradation that is within Tarantula Curve limits throughout production. The suggested warning band is listed below:

Table 20: WisDOT Optimized Gradation with Recommended Warning Bands.

SIEVE SIZES	PERCENT RETAINED[3]	RECOMMENDED WARNING BAND
2 in. (50.8 mm)	0	0
1 1/2 in. (38.1 mm)	$\leq$ 5	5
1 in. (25.4 mm)	$\leq$16	14
3/4 in. (19 mm)	$\leq$ 20	18
1/2 in. (12.7 mm)	4-20	18
3/8 in. (9.5 mm)	4-20	18
No. 4	4-20	18
No. 8[1]	$\leq$12	10
No. 16[1]	$\leq$12	10
No. 30[1] [2]	4-20	18
No. 50 [2]	4-20	18
No. 100 [2]	$\leq$10	8
No. 200 [2]	$\leq$ 5	4

[1] Minimum of 15% retained on the sum of the #8, #16, and #30 sieves.

[2] Conform to 24-34% retained of fine sand on the #30-200 sieves.

[3] 2022 WisDOT Standard Specification Table 501-4 Optimized Aggregate Gradation.

7.2 Air Void System

Based on the results shown in Figure 64, it is recommended that a SAM Number < 0.20 and air content > 4% be used for the mixture design and evaluated in the lab with the intention of keeping the design far away from the limit. However, in the field it is recommended to use a SAM Number < 0.30 and air content > 4%. If the SAM Number is < 0.30 then consolidation from the paver is not expected to change the quality of the air void system and so testing is not required after the concrete paver.

Based on the results shown in Figure 68 for Phase I, consolidation of concrete by inserting a battery powered vibrator is shown to increase the variability of the SAM results when the mixtures did not meet the Tarantula Curve limits. During Phase II, consolidation was performed using the MinT and rodding. Figure 69 shows the SAM Number for each project, session and consolidation.

Lab data has shown that using the MinT will reduce the variability of SAM Meter testing. The field data exhibited higher variability than the lab data for both MinT and rodding, however the MinT still produced a lower standard deviation than the rodding. At this time, it is recommended to use a MinT to consolidate the concrete for two reasons:

1. The standard deviation for MinT was less than the standard deviations for rodding in the field and lab.
2. The MinT is mechanical method that should reduce human error, which will be a benefit for new technicians running this test.

7.3 Strength Properties

During Phase I, mixtures with higher compressive strengths didn't always have higher flexural strength. However, during Phase II, when more data was added, there was a much higher correlation between compressive and flexural strength, as was observed in Figure 59. Therefore, we recommend that WisDOT continue requiring strength testing depending on the application of the concrete. Compression testing is recommended in applications where concrete will be supporting compressive loads such as in structural columns for bridges, while flexural testing is recommended for applications requiring flexural strength such as roadways.

7.4 Workability

While the Vibrating Kelly Ball (V-Kelly) and Box Test both attempt to quantify workability, are loosely correlated with each other with large variability and bias as shown in Figure 82. The V-Kelly procedure recommends 0.8 - 1.2 $(in/sec)^{1/2}$, however the data in this study showed good box values can be achieved with V-Kelly values as low as 0.7 $(in/sec)^{1/2}$.

From a contractor's perspective, the Box Test and the V-Kelly may be valuable tool, regarding the workability or ability to finish their mixes before construction as to avoid these types of issues during production. The V-Kelly test may give more insight as to how well a mix will move through a paver, while the Box Test gives a better indication of the ability to finish and potential for issues like edge slumping or large surface voids.

We do not recommend WisDOT specify or attempt to standardize the V-Kelly test as a performance qualifier. However, we recommend replacing the Slump test with the Box Test (<2.0) during Field Trials. Additionally, the Box Test proved useful to help quantify workability and identify changes in consistency from lot to lot. For this reason, WisDOT may want to consider using the Box Test during production.

7.5 Resistivity

The following recommendations can be made:

1) WisDOT should select one sample geometry for testing. Bulk and surface resistivity measure resistivity. Bulk resistivity measures a larger volume of the specimen and is less sensitive to surface effects. Either method could be used as it has been shown that with appropriate geometric correction factors both provide comparable results. Since WisDOT uses primarily 6 in. x 12-in. (152.4 mm x 304.8 mm) cylinders and Surface Resistivity probes, it is recommended to continue with these practices.
2) WisDOT should require resistivity to be the reported value (with a correction of 1.37 for 6 in. x 12 in. [152.4 mm x 304.8 mm] cylinders with 1.5 in. [38.1 mm] probe spacing) when using AASHTO T358.
3) WisDOT has adopted the conditioning of samples according to AASHTO T 358-21 Section 8.1.1, accelerated curing. However, care should be taken in interpreting the pore solution of these materials if further analysis is performed.
4) WisDOT should continue to collect resistivity data, especially because results can identify mixture concerns as seen with Columbus and Menomonie as shown in Figure 74.
5) At the current time we do not recommend enacting a specification for payment regarding resistivity until the more data is gathered using the above recommendations.

While suggesting a resistivity target was out of the scope of this research, there is however an existing WisDOT specification that has a requirement for chloride penetration. The High-Performance Concrete (HPC) Masonry Structures, Item SPV.0035 requires a maximum of 1,500 coulombs per the AASHTO T277 Rapid Chloride Permeability Test (RCPT). It is possible to take 1,500 coulombs requirement and convert it into an equivalent surface resistivity using the following interpolation:

Values Used from AASHTO PP84-17: Table X2.1

ASTM C1202 Classification	**RCPT: Charge Passed (Coulombs), x**	**Surface Resistivity: ρ (kΩ-cm), y**
High	RCPT ≥ 4000	ρ < 5.2
Moderate	2000 ≤ RCPT < 4000	5.2 ≤ ρ < 10.4
Low	1000 ≤ RCPT < 2000	10.4 ≤ ρ < 20.8
Very Low	100 ≤ RCPT < 1000	20.8 ≤ ρ < 207

Variables:

x = 1,500 (input to be converted to kΩ-cm) [Coulombs]
y = Equivalent Surface Resistivity (variable being solved for) [kΩ-cm]
b = y intercept = 10.4 kΩ-cm

Interpolation Bounds:

x_1 = 1,000 Coulombs
y_1 = 10.4 kΩ-cm
x_2 = 2,000 Coulombs
y_2 = 20.8 kΩ-cm

Equations:

$$\text{Linear Interpolation Equation: } y = b + (x-x_1)(dy/dx) \tag{11}$$

$$\text{Where: } dy/dx = [(y_2-y_1)/(x_2-x_1)] \tag{12}$$

Calculation:

$$y = 10.4 + (1{,}500-1{,}000)*[(20.8-10.4)/(2{,}000-1{,}000)] = 15.6 \text{ k}\Omega\text{-cm} \tag{13}$$

The following graphs, shown in Figure 83, display the surface resistivity values from the Phase I and Phase II field projects (respectively) and the equivalent resistivity of 15.6 kΩ-cm. This data is for information only and intended to compare existing specifications with in-place field mixtures.

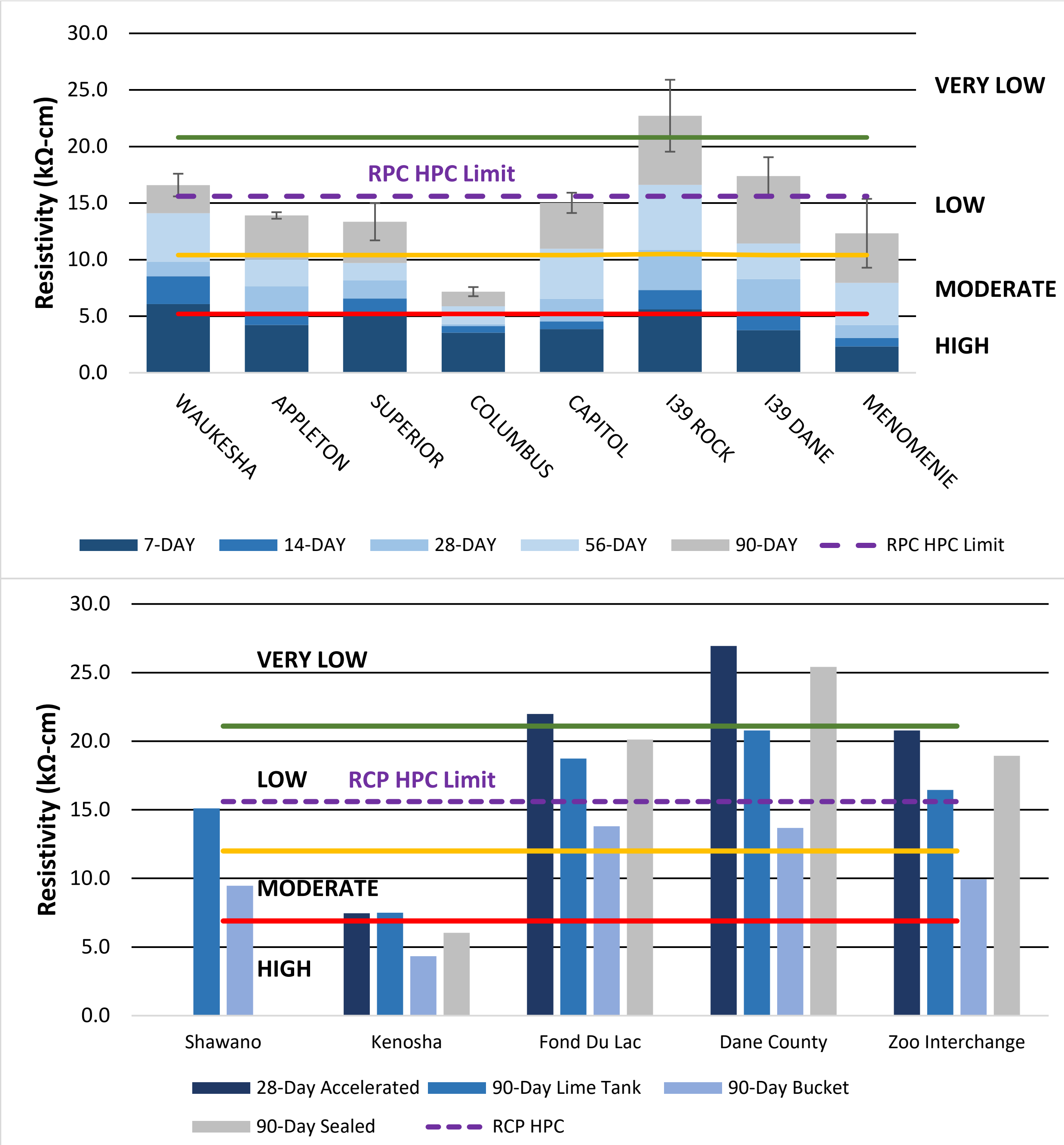


Figure 83: Phase I and II Resistivities Compared to the Rapid Chloride Penetration High Performance Concrete Equivalent Surface Resistivity Specification.

7.6 Hardened Air Voids

As was shown in Figure 63, air content decreases as it moves from the plant, to before the paver, and then finally after the paver. This isn't necessarily a problem because as the SAM data showed in Figure 64, we can expect there to still be a sufficient air void network. What is happening is that the larger coarse air bubbles are being removed from the mix, and the small, entrained air network remains.

7.7 Coefficient of Thermal Expansion

Due to WisDOT's widespread use of jointed-plain concrete pavements, the coefficient of thermal expansion may be of interest to WisDOT. In applications where there is freedom for movement such as in a jointed concrete roadway, thermal expansion and contraction can cause issues like concave/convex warping. Therefore, we recommend that WisDOT consider collecting more data regarding the coefficient of thermal expansion for further analysis and perhaps subsequent standardization.

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Appendix

A1 Testing Sessions

Phase I

Hardened Concrete Testing				
Test	**Session**	**Sample Day**	**Sample Location**	**# of Samples**
Compressive Strength	2	D1 - EVE	Before Paver	10
Flexural Strength	1	D1 - MORN	Before Paver	6
Surface Resistivity	1	D1 - MORN	Before Paver	3
	2	D1 - EVE	Before Paver	3
	3	D2 - MORN	Before Paver	3
Resistivity, Porosity and Pore Solution Resistivity	1	D1 - MORN	Before Paver	2
Hardened Air	1	D1 - MORN	Plant, Before & After Paver	3
	2	D1 - EVE	Plant, Before & After Paver	3
	3	D-2 MORN	Plant, Before & After Paver	3
	4	D-2 EVE	Plant, Before & After Paver	3
Coef of Thermal Expansion	1	D1 - MORN	Before Paver	1
	3	D2 - MORN	Before Paver	1

Plastic Testing				
Test	**Session**	**Sample Day**	**Sample Location**	**# of Samples**
Super Air Meter	1	D-1 MORN	Plant, Before & After Paver	3 (w/2 rep)
	2	D-1 EVE	Plant, Before & After Paver	3 (w/2 rep)
	3	D-2 MORN	Plant, Before & After Paver	3 (w/2 rep)
	4	D-2 EVE	Plant, Before & After Paver	3 (w/2 rep)
Box Test	1	D-1 MORN	Plant	1
	2	D-1 EVE	Plant	1
	3	D-2 MORN	Plant	1
	4	D-2 EVE	Plant	1
V-Kelly Ball	1	D-1 MORN	Plant	1
	2	D-1 EVE	Plant	1
	3	D-2 MORN	Plant	1
	4	D-2 EVE	Plant	1

Phase II

Test	Session	Sample Location	Sample Unit	Number of Replicates	Number of Projects	Number of Curing Conditions	Curing Conditions	Total Cylinders	Total Beams	Total Tests
Compression (AASHTO T 23 & ASTM C39)	Morning	Before Paver	6" x 12" Cylinders	3	4	1	28-Day Lime Water Bath	12	-	36
	Noon			3	4	1		12	-	
	Afternoon			3	4	1		12	-	
Flexural Testing (AASHTO T 23 & T 97)	Morning		6" x 6" x 21" Beams	3	4	1	28-Day Lime Water Bath	-	12	36
	Noon			3	4	1		-	12	
	Afternoon			3	4	1		-	12	
Box Test	Morning		12" x 12" x 12" Box of Concrete (1 ft^3)	1	4	-	-	-	-	12
	Noon			1	4	-		-	-	
	Afternoon			1	4	-		-	-	
Super Air Meter* (AASHTO TP 118)	Morning		0.25 ft^3 of Concrete	6	4	-	-	-	-	72
	Noon			6	4	-		-	-	
	Afternoon			6	4	-		-	-	
Surface Resistivity / Bulk Resistivity / Formation Factor (AASHTO T 358 / TP 119)	Morning		6" x 12" Cylinders	0	4	1	28, 56, 90-Day Lime Water Bucket	0	-	12
	Noon			3	4	1		12	-	
	Afternoon			0	4	1		0	-	
	*For the SAM test, 2 replicates will be for MinT consolidation, 2 for rodding, and 2 for battery vibration for each session.									

Phase II revised Resistivity Sample conditioning

	Bucket Curing	Accelerated Curing	Lime Curing	Sealed Sample
Standard	AASHTO TP119-21 Electrical Resistivity of a Concrete Cylinder Testing in a Uniaxial Resistance Test	AASHTO T358-21 Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration	AASHTO R39-21 Making and Curing Concrete Test Specimens in the Laboratory	AASHTO TP119-21 Electrical Resistivity of a Concrete Cylinder Testing in a Uniaxial Resistance Test
Section	10.2	8.1.1	8.3	10.3
Description	Option A – Immersion of Specimens in a single 5-gal bucket with Calcium Hydroxide-Saturated 7.6 g/L NaOH (0.19 M); 10.64 g/L KOH (0.19 M); 2 g/L $Ca(OH)_2$	Accelerated Moist Curing – 7 days cured according to R39, then immerse the specimens for 21 days in lime-saturated water at 100 ± 3.5°F	All specimens shall be immersed in a lime saturated water to prevent leaching, at 73.5 ± 3.5°F, from the time of molding until the moment of test.	Option B – Seal Samples by maintaining them in the cylinder molds or demolding them and placing them in two 6-mil double sealed bags. Maintain 73± 4°F
Number of Cylinders	3	3	3	3
Curing Days	7,14,28,56,90	7,14,28,56,90	7,14,28,56,90	7,14,28,56,90