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THE PERFORMANCE OF CEMENT-LIMITING AND HIGH-VOLUME RECYCLED MATERIAL MIX DESIGNS FOR USE IN CONCRETE PAVEMENTS THROUGH LABORATORY AND FIELD IMPLEMENTATION TESTING

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THE PERFORMANCE OF CEMENT-LIMITING AND HIGH-VOLUME RECYCLED MATERIAL MIX DESIGNS FOR USE IN CONCRETE PAVEMENTS THROUGH LABORATORY AND FIELD IMPLEMENTATION TESTING

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BY

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Abstract

Conventional methods of replacing roadways are beginning to negatively impact the environment. The continued use of high volumes of cement will continue to exacerbate the increasing levels of carbon dioxide. In addition to cement concerns, removing existing deteriorated pavement will continue to stress already overburdened landfills. In an attempt to alleviate these issues, engineers have been investigating methods to increase the environmental sustainability of concrete pavements. One method being investigated to reduce the overall cement content, and another is to incorporate construction waste in the form of recycled concrete aggregate (RCA) and fly ash. The first method, Cement-Limiting, investigated three different aggregate optimization techniques (Coarseness/Workability, Percent Retained, and Power 45) to reduce the amount of void space in a concrete mix, allowing a reduction in the cementitious material. The second method, High-Volume Recycled Material, investigated using up to 100% RCA replacement of natural aggregate and up to 75% fly ash replacement of cementitious material.

To test the performance of both of these mix designs, multiple ASTM laboratory tests were performed on the fresh and hardened concrete, as well as full-scale, instrumented field implementations. These results were monitored and compared to the Oklahoma Department of Transportation (ODOT) Specifications, as well as to commonly used concrete mix designs. Analysis of the results indicate that the Cement-Limiting mix design adequately performed or surpassed ODOT Specifications the Class A standard pavement concrete mix and the, while the High-Volume Recycled Material mix design performed up to the ODOT Specifications, but not to Class A mix.

1. Introduction

1.1. Background and Justification

Transportation is one of the most influential industries to the United States economy. The transportation industry encompasses a variety of components ranging from commercial and public entities in air, sea, and land. The land transportation system of the United States connects 118.7 million households, 7.4 million business establishments, and 89 thousand governmental buildings with one another through 4 million miles of roads (U.S. Department of Transportation 2014). This research shows the impact the transportation industry has on people's daily lives in the United States. In addition, many people utilize the system for commercial ventures. In 2012, the United States freight system moved 53.9 million tons of goods worth \$47.5 billion each day (U.S. Department of Transportation 2014). This pronounced use and importance requires constant reinvestment to maintain a productive and useful transportation industry. In total, the public and private sectors spent \$119 billion on transportation construction in 2012, two-thirds of which was on highway infrastructure (U.S. Department of Transportation 2014).

For roadway projects, most of this cost is directly related to the cost of materials, such as concrete or asphalt for pavement construction. Material costs can make up more than 50% of an overall construction project. Many roadway projects are using concrete pavement over asphalt pavement due to the increasing price of asphalt, manageability, and design life of concrete. As concrete becomes more and more common, it is starting to catch up with other areas of construction as concrete is the most commonly used construction material throughout the United States. (Yang, Hao and Wang 2010). The main driving force behind concrete’s prominent use is its high strength, ease of production, and variety of applications. Basic concrete mix designs are relatively simple, composed of only four ingredients; fine aggregates, coarse aggregates, water, and portland cement. The first use of concrete pavement in the U.S. dates back over 100 years, to 1891 in Bellefontaine Ohio, where, this pavement section is still in use today (Goonan 2019). One of the major benefits of concrete pavements is its outstanding ability to withstand a variety of extreme scenarios, a relatively long design life, and only requiring a few ingredients. Mixtures can become more complex by including additional fibrous and/or powderous admixtures to increase the strength, workability, and/or hardening characteristics. With concrete’s numerous benefits it performs exceptionally as a construction material, especially a pavement material.

Many concrete pavements have a design life between 30 to 50 years, after which the pavement restoration process could vary. Some pavement sections need top layer grinding and resurfacing, or crack sealing and spalling replacement, while others require full depth replacement. President Eisenhower signed the Federal Aid Highway Act in 1956; which directed the construction of 40,000 miles of roadway throughout the country (Weingroff 2015). Today those roadways are well past or at their maximum design life and are going to require rehabilitation. Using concrete to rehabilitate degraded pavements has multiple benefits, however, greenhouse gas emissions and construction waste are two major environmental concerns.

1.1.1. Greenhouse Gas Emissions from Cement Production

In 2001, the United States was the world's third largest producer of cement at 90 million metric tons (MMt) and imported an additional 25 MMt (Hanle). This is expected to continue to rise in the future. In short, the process of creating cement is super heating raw materials to around 2732 °F and grinding the materials to a very fine powder. A variety of fuels are used, the most common being coal, which makes up 71%, followed by petroleum coke at 12%, 9% from liquid and solid waste fuels, and the remainder is from natural gas or a coke fuel mix (Hanle). Consequences from this intense heating and breaking down of raw materials to produce cement results in 5.5 million btu per ton of cement produced (Hanle). This significant energy use expresses the energy requirements, which causes a release in greenhouse gas emission of a sizable scale. It was estimated that in 1999, the U.S. cement industry emitted 22.3 MMt of CO_2 in to the atmosphere (Ernst and Christina 2004). These emissions are only from the United States. Worldwide production of cement results in a significant contribution to global greenhouse gas emissions. Limiting the amount of cement used in typical concrete mixes will decrease the nationwide production of cement, therefore decreasing the input energy required and reducing the greenhouse gases produced. A concern for reducing the cement is maintaining adequate strength parameters to meet performance and safety requirements. One method that counteracts the effects of reducing the cement content is aggregate optimization. Aggregate optimization utilizes the varying size and shape of various aggregates to fill void spaces, requiring less cement mortar.

1.1.2. Implications of Construction Waste and Natural Aggregates

With the passing of the Federal Highway Act, many existing roadways are going to have to be reconstructed in the near future. The United States Geological Society estimates 1,500 million metric tons of natural aggregates and 48 million metric tons of cement are in the national highway system today (Goonan 2009). Replacing this existing system requires the production of new concrete and will generate large amounts of construction waste. Over 2 billion tons of concrete is produced yearly throughout the world, incorporating the cement previously discussed, water, and the raw aggregate materials. By 2020, aggregate production is expected to increase to more than 2.5 billion tons per year (Federal Highway Administration 2004). This continued production will strain the existing supplies of aggregates and push suppliers to discover additional sources. Construction waste produced from building demolition alone is estimated at 123 million tons (Federal Highway Administration 2004), a value that will increase significantly when including reconstruction of the nation's highway system. Instead of sending this waste to a land fill, some state agencies are recycling the concrete for aggregate. This aggregate is call recycled concrete aggregate (RCA). This has led many to a Federal Highway Association (FHWA) survey on State Agencies' use of RCA as an aggregate (Figure 1.1), aggregate for base (Figure 1.2), and aggregate for concrete (Figure 1.3). These Figures show that many states are open to using RCA, however they are reluctant to implement RCA in concrete. Potentially utilizing RCA in concrete pavements will decrease the use of natural aggregates and the amount of construction waste from roadway construction.

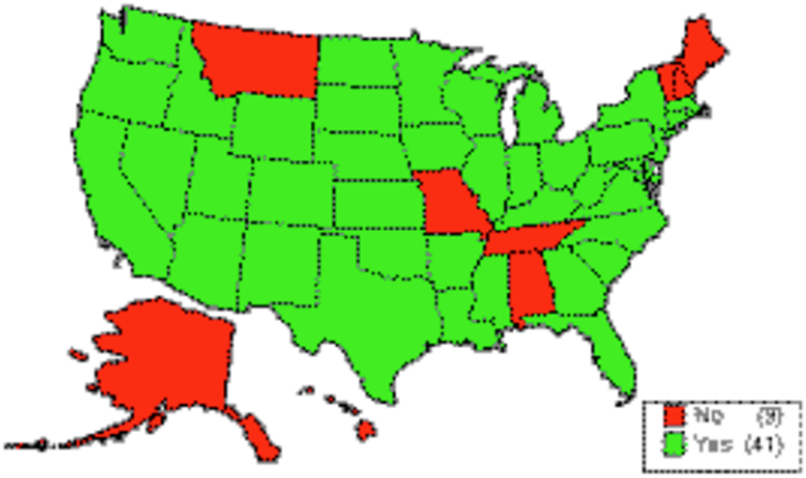


Figure 1.1 - States Using RCA as Aggregate (Federal Highway Administration 2004)

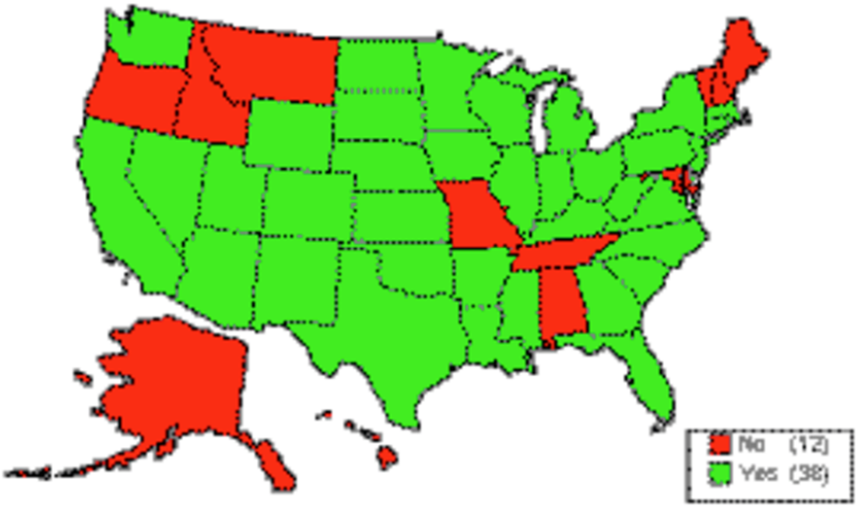


Figure 1.2 - States Using RCA as Aggregate Base (Federal Highway Administration 2004)

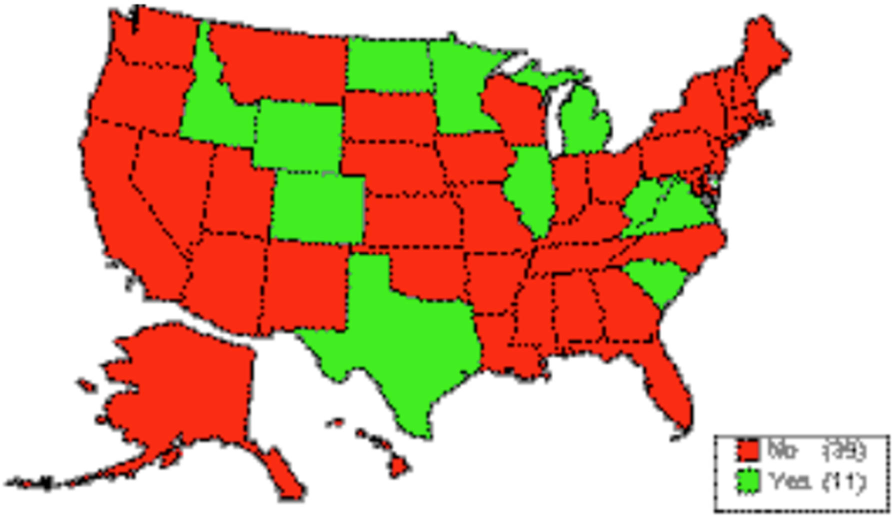


Figure 1.3 - States Using RCA as Aggregate in Concrete (Federal Highway Administration 2004)

1.2. Project Scope

This research incorporates two methods of reducing the cost and ecological footprint of concrete pavements while increasing their sustainability. The first method is to investigate the response to limiting the overall cement content, and the second is to incorporate large volumes of recycled materials into the mix design.

1.3. Cement-Limiting Concrete

Typical concrete mixes, as previously stated, can involve significantly large amounts of CO_2 emissions to develop. This is primarily due to the heating of raw materials to almost 3,000 °F to create the cement. Developing concrete pavement mix designs that have a reduced amount of cement would have a positive impact on the environment and increase the sustainability of concrete. The 2009 Oklahoma Department of Transportation (ODOT) Specifications (Oklahoma Department of

Transporation 2009) states the minimum cement content for a Class A (Pavement) Mix is 517 lb/yd^3,with a special provision 701.14 stating that for the use of aggregate optimization, the cement content can be reduced to 470 lb/yd^3. The amount of cementitious material in a concrete mix design is important to the overall cost. Also, decreasing the cement content will decrease the possibilities of shrinkage cracking from thermal and drying behaviors. To compensate for any potential strength limitations through decreasing the cement content, multiple methods of aggregate optimization will be investigated. Aggregate optimization is the method of efficiently packing aggregates to limit the void space and effectively use the cement to bind the aggregates together. Thus, limiting the cement content, and investigating different methods of aggregate optimization, could lend to a potential concrete pavement mix design that is cheaper, environmentally more sustainable, and has better performance.

1.4. High-Volume Recycled Material Concrete

Reusing existing concrete material has the potential to significantly decrease the cost of a project and increase the sustainability of the proposed concrete. The cost savings are in the material transportation of the new aggregate material in the concrete mix, and transportation of this removed material and new material to and from the site, which are not involved RCA implementation. Again, not only would this save in the overall project cost, reusing this material would increase the sustainability of concrete by limiting the amount of construction waste in landfills and CO_2 emissions. The ODOT 2009 Specification does not allow the use of RCA in a concrete mix design and allows only a maximum fly ash replacement of 20%. A concern with using RCA is the decrease in compressive strength when compared to concrete with virgin materials and its increased irregularity, which can lead to a “harsh” mix. However, introducing another recycled material, such as fly ash, can positivity impact the compressive strength and workability of the concrete. Supplementing the total amount of cement with a percentage of fly ash will counteract the effects of less cement as well as increase the use and amount of recycled materials. Incorporating both RCA and fly ash into a concrete pavement mix design could result in a concrete pavement mix design that is cheaper, environmentally more sustainable, and has better performance.

1.5. Objectives and Goals

1.5.1. Objectives

To evaluate the response and performance of both the Cement-Limiting and High-Volume Recycled Materials Concrete pavements, multiple objectives will be investigated:

- Evaluate and characterize the aggregates based on key engineering properties necessary for developing accurate concrete mix designs.
- Optimize aggregate skeleton and characteristics based on packing density to ensure adequate rheology, stability and mechanical properties (Cement-Limiting Only).
- Maximize the use of recycled materials, to at least 50% of the mass of solids (High-Volume Recycled Only).
- Evaluate the key engineering properties, and durability through laboratory testing of proposed concrete mix designs for transportation roadways.

- Perform field implementation and in-situ testing to establish performance guidelines for casting and placement of potential environmentally sustainable concrete mix designs for pavement construction.

1.5.2. Goals

The goal of this research is to inform people of environmental impacts of typical concrete; and develop multiple environmentally sustainable mix designs that perform above the minimum ODOT Specifications that are more environmentally sustainable.

1.6. Outline

This thesis will contain two investigations on the performance of Cement-Limiting Concrete and High-Volume Recycled Material Concrete discussed simultaneously.

- Section 1 contains an explanation of the implications and relevance of transportation and concrete, limitations of concrete, and Cement-Limiting Concrete and High-Volume Recycled Materials Concrete methods for improving the sustainability of concrete.
- Section 2 discusses previous research on the properties of RCA, aggregate optimization methods, RCA and fly ash concrete performance, and aggregate optimized concrete performance.
- Section 3 details the ODOT specifications and requirements, starting with the aggregate properties, and then detailing the fresh and hardened concrete properties required by ODOT.

- Section 4 details the properties and testing behind the aggregates used in the concrete mix designs. Also, this section presents the fresh and hardened concrete tests that will be performed throughout the research.
- Section 5 outlines the investigation of the mix designs for the Cement-Limiting Concrete from the aggregate properties, to the multiple mix designs, and the laboratory results.
- Section 6 outlines the investigation of the mix designs for the High-Volume Recycled Materials Concrete from the aggregate properties, to the multiple mix designs, and the laboratory results.
- Section 7 explains the preparations for field implementation. Included in the explanation are the construction methods, instrumentation placement, and the experimental mix designs.
- Section 8 presents the results from the field implementation panels. Three responses are investigated for the ODOT standard mix design, the Cement-Limiting mix design, and High-Volume Recycled Materials Concrete mix design. Both the laboratory testing and in-situ strain data will be presented and discussed for each mix design.
- Section 9 summarizes the findings, conclusions, and additional research recommendations from this study.

2. Literature Review

2.1. Recycled Concrete Aggregate Properties

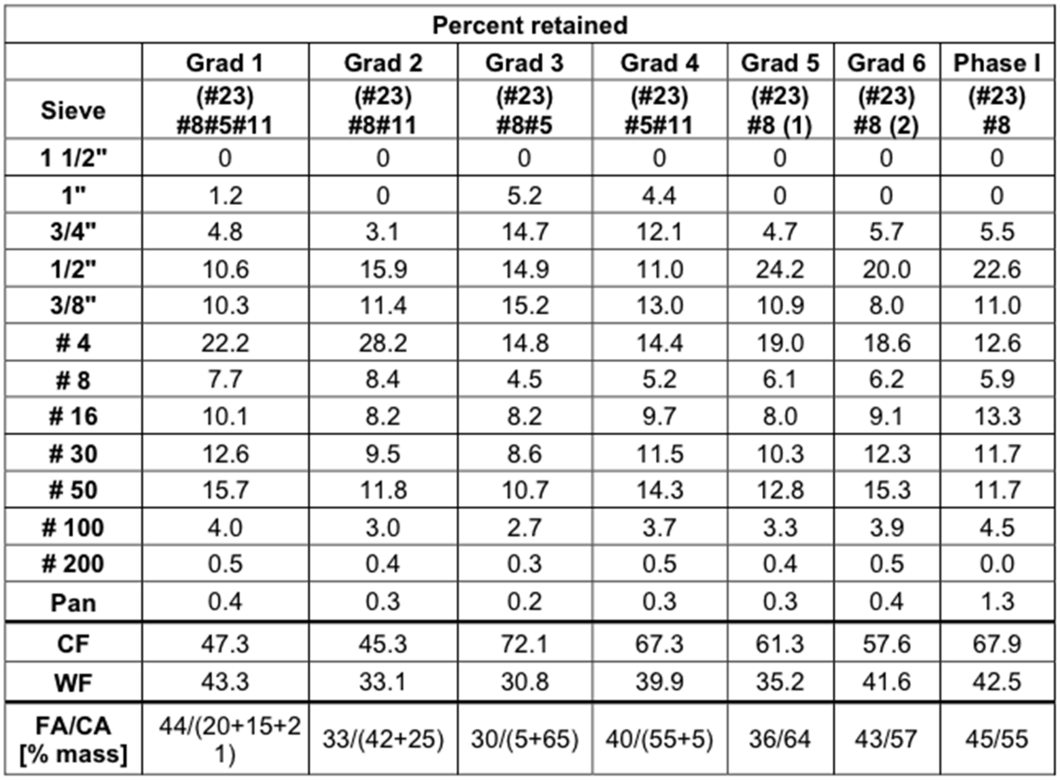
The first area to begin investigating is how natural aggregate (NA) properties compare to recycled concrete aggregate (RCA) properties. Yang et al. (2009) began comparing RCA and NA leading to two interesting findings. The first finding indicates the water absorption rate for RCA is 80.8% higher than NA. Having a higher rate of absorption greatly influences the performance of concrete, and should strongly influence the mix design. Second, RCA had a lower density at 2640.2 kg/m^3 compared to 2728.1 kg/m^3 of NA, due to the attached mortar to the aggregate particles. In the next study, Mas et al. (2011) also looked at the water absorption rates for NA and RCA. Mas et al. (2011) found that the RCA absorption rate was of 7.5%, which was significantly higher than NA, which had absorption rates around 4%. This finding agreed with the findings of Yang, Hao and Wang (2009) which were previously discussed.

2.2. Aggregate Optimization

Multiple studies have looked into varying aggregate optimization methods for concrete mix designs. Abdulkareem (2012) looked at the response to the Power 45, Percent Retained method, and Coarseness Chart method with two aggregates and three aggregate mix designs. First, looking at the Power 45 maximum density line showed that the ideal mix with two aggregates tended to have a higher percentage passing for the large sieve sizes and lower percent passing on the smaller sieve sizes. The percent of aggregates of the two aggregate mix is made up of 58% coarse aggregate and 42% fine aggregate. A mix design with three aggregates tended to be an overall rocky mix. The percent of aggregates of the three aggregate mix is made up of 48% and 14% coarse aggregates and 38% fine aggregate. The Percent Retained method showed that the two aggregate mix resulted in two consecutive valleys due to the large percent retained difference between consecutive sieves. However, the "two aggregate mix achieved good concrete properties" (Abdulkareem 2012). The three aggregate mix had only one small peak and was acceptable by the chart standards. For the Coarseness Chart, the results from the three aggregate mix fell in the boundary lines, but towards the rocky mix response. This followed the same characteristics as the Power 45 curve. The two aggregate mix fell on the boundary line of a gap-graded mix. Using these results, Abdulkareem (2012) concludes "optimized gradations with choosing the appropriate cement content, water cement ratio, chemical admixture has led to good mixture design that reduced the amount of cement binder, acquired good compressive strength, with a suitable workability for pavement works."

The second study performed by Rudy (2009) looked at the response of multiple gradations involving a combination of three coarse aggregates and one fine aggregates, one with four total aggregates, three with three total aggregates, and two with two total aggregates. The experimental gradations are outlined in Table 2.1.

Table 2.1 - Experimental Gradations (Rudy 2009)



Percent retained							
	Grad 1	Grad 2	Grad 3	Grad 4	Grad 5	Grad 6	Phase I
Sieve	(#23) #8#5#11	(#23) #8#11	(#23) #8#5	(#23) #5#11	(#23) #8 (1)	(#23) #8 (2)	(#23) #8
1 1/2"	0	0	0	0	0	0	0
1"	1.2	0	5.2	4.4	0	0	0
3/4"	4.8	3.1	14.7	12.1	4.7	5.7	5.5
1/2"	10.6	15.9	14.9	11.0	24.2	20.0	22.6
3/8"	10.3	11.4	15.2	13.0	10.9	8.0	11.0
# 4	22.2	28.2	14.8	14.4	19.0	18.6	12.6
# 8	7.7	8.4	4.5	5.2	6.1	6.2	5.9
# 16	10.1	8.2	8.2	9.7	8.0	9.1	13.3
# 30	12.6	9.5	8.6	11.5	10.3	12.3	11.7
# 50	15.7	11.8	10.7	14.3	12.8	15.3	11.7
# 100	4.0	3.0	2.7	3.7	3.3	3.9	4.5
# 200	0.5	0.4	0.3	0.5	0.4	0.5	0.0
Pan	0.4	0.3	0.2	0.3	0.3	0.4	1.3
CF	47.3	45.3	72.1	67.3	61.3	57.6	67.9
WF	43.3	33.1	30.8	39.9	35.2	41.6	42.5
FA/CA [% mass]	44/(20+15+21)	33/(42+25)	30/(5+65)	40/(55+5)	36/64	43/57	45/55

Figure 2.1 shows where the mixes fall on the Coarseness Chart and the properties each mix is expect to have. First, these gradations were plotted on the Percent Retained Chart. All six gradations showed a “shortage of particles retained on sieve #8. This is due to using sand with a constant gradation”. Another trend showed that “significant differences exist in the amount of aggregate retained on 3⁄4”, 1⁄2” and No.4 sieves”. Gradations #4 and #6 best meet the requirements of the Percent Retained Chart. These gradations are taken and plotted on the Power 45 curve. Here the gradations were separated into two groups by nominal maximum size (NMS).

The gradations were separated by gradations 2, 5, and 6 with NMS of 0.75 inch and 1, 3, and 4 with a NMS 1 inch. The results showed “gradation No. 6 can be considered to be best optimized for 0.75 inch (NMS) and gradation No.4 can be considered as best optimized for NMS of 1 in." The conclusion from Rudy (2009) shows a stronger correlation between the Percent Retained Chart and Power 45 curves than the Coarseness Factor Chart.

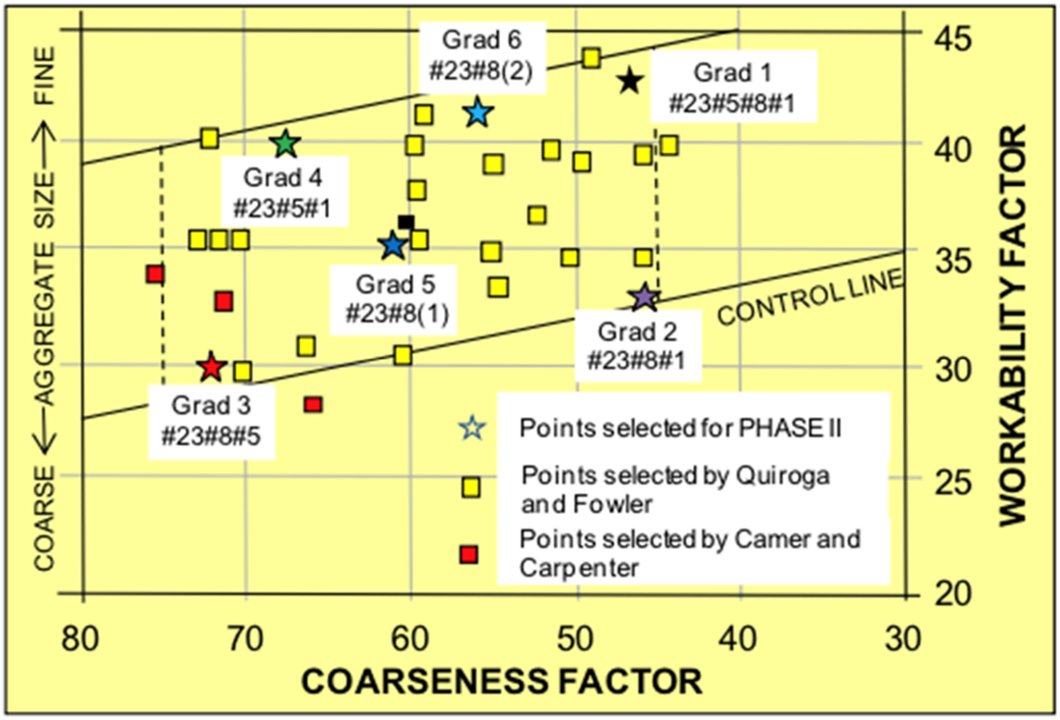


Figure 2.1 - Coarseness and Workability Chart (Abdulkareem 2012)

2.3. Recycled Concrete Aggregate and Fly Ash Concrete Performance

The next topic compares previous research to the performance of recycled concrete. All of the studies discuss the use of increasing amounts of recycled aggregates in multiple concrete mix designs. Yang et al. (2010) developed five different mix designs with increasing rates of RCA: 0%, 30%, 50%, 70%, and 100%.

The characteristics of fresh concrete were analyzed and compared in Table 2.2. The results indicate as the percentage of RAC increases, both the slump value and the compressive elastic modulus decreased. However, the apparent density had a small decrease as RCA amounts increase. After the specimens cured for 7 days, 14 days, and 28 days, the compressive strength was tested and recorded. The study found that the optimal RCA concrete mix design contains less than 50% RCA. “The RCA replacement ratio has a remarkable influence on the compressive strength of RCA concrete; nevertheless, the compressive strength of recycled aggregate concrete can achieve the target compressive strength by choosing a replacement ratio of recycled aggregate” (Yang et al. 2009).

Table 2.2 - Experimental Concrete Properties (Mas et al. 2011)

Number of specimen	**W/C**	**Proportion of concrete mixtures W:C:S:G:RA**(kg/m^3)	**Slump value** (mm)	**Apparent density** (kg/m^3)	**Compressive elastic modulus** (GPa)
RAC0	0.56	185:330:754:1131:0	65	2412	31.8
RAC30	0.56	185:330:754:792:339	59	2409	28.2
RAC50	0.56	185:330:754:565:565	45	2406	25.6
RAC70	0.56	185:330:754:339:792	33	2405	19.8
RAC100	0.56	185:330:754: 0: 1131	20	2397	18.2

Again looking at compressive strength, Mas et al. (2011) created three series of concrete mix designs based on their slump value. Series I, a slump value: 6–9 cm; for Series III, a slump value: 10–15 cm; for Series III, a slump value: 0–2 cm. Within each series, three mixes were used with an increasing amount of RCA from 0%, to 20%, and 40%. Mas et al. (2011) found that Series III had a “very high compressive strength compared with the other series studied, it also presented the highest percentage of reduction when incorporating RCA” (Mas et al. 2011).

The two previous studies discussed concrete mix designs that only observed the effects of RCA in concrete performance. In Limbachiya et al. (2000) studied the effects of RCA combined with fly ash (FA) for multiple mix designs. The mixes used varying percentages of RCA, starting with 0% and increasing to 30%, 50%, and 100%, and some mixes also included 30% fly ash replacement of cement. They (Limbachiya et al. 2000) arrived at three primary conclusions. First, the compressive strength of both concrete types, portland cement (PC) and portland cement with fly ash (PCFA), decreases as the replacement level of NA by RCA increases. Second, the high initial moisture levels may lower the early-age compressive strength, but it would be beneficial for a continuous wet condition and long-term cement hydration, especially for PCFA concrete. Third, with 30% coarse RCA, as NA substitute, could be considered as the optimum content, as beyond this level causes a negative effect on compressive and flexural strengths.

These studies provide great insight into multiple aspects of RCA and FA concrete. Beginning with the aggregate properties found in Yang et al. (2009) and Mas et al. (2011), both indicate that increased water absorption for RCA needs to be accounted for in the concrete mix design. The RCA does affect the compressive strength when compared to higher strength concrete, however for lower strength concrete, RCA substitution has adequate strength, especially while incorporating fly ash.

3. ODOT 2009 Specifications

3.1. Introduction

In the state of Oklahoma, the Oklahoma Department of Transportation (ODOT) is the governing body of transportation throughout the state. For design and construction, ODOT provides a standard set of specifications which describes the construction requirements for all ODOT and many other transportation projects throughout the state (Oklahoma Department of Transportation 2009). The fresh and hardened properties of the Cement-Limiting and High-Volume Recycled mix designs were compared to the requirements of the Specifications.

3.2. Oklahoma Department of Transportation Specifications

The ODOT 2009 Specifications is the latest and most updated version. These Specifications detail the requirements for soils, aggregates, materials, and multiple other aspects. Section 701 outlines all the variations to portland cement concrete mix designs and the requirements for fresh and hardened concrete properties. Due to concrete's wide variety of uses, the Specifications describe multiple classes of concrete. These classes are based on the application of the concrete. Class A is designated for concrete pavements. Since aggregates are vitally important, the same section outlines all the requirements for both coarse and fine aggregates.

3.3. Class A Aggregate Properties

The Class A mix follows the trend of using only two aggregates, a coarse and a fine aggregate. The requirements vary depending on if the aggregate is fine or coarse. For the coarse aggregate, the mix recommends using No. 57 rock. Table 3.1 provides the upper and lower bound gradation limits of the No. 57 rock, with a gradation chart shown in Figure 3.1. In addition to the gradations for the coarse aggregate, a LA Abrasion test requirement is a maximum loss of 40%. Looking at the fine aggregate, the two requirements are gradation data, which is provided in Table 3.2 and in a gradation chart shown in Figure 3.2, and the fineness modulus. The fineness modulus is expected to be between 2.3 and 3.1. Along with the aggregate properties, the actual concrete must meet requirements as well.

Table 3.1 - ODOT Coarse Aggregate Gradation Bounds (Oklahoma Department of Transportation 2009)

Sieve Size/#	Sieve Opening (in)	Percent Passing (Lower Bound)	Percent Passing (Upper Bound)
1.5"	1.48	100%	100%
1"	0.98	95%	100%
3/4"	0.37	95%	100%
3/8"	0.19	25%	60%
#4	0.09	25%	60%
#8	0.08	0%	5%
#16	0.05	0%	5%
#30	0.02	0%	5%
#50	0.01	0%	5%
#100	0.01	0%	5%
#200	0.00	0%	2%

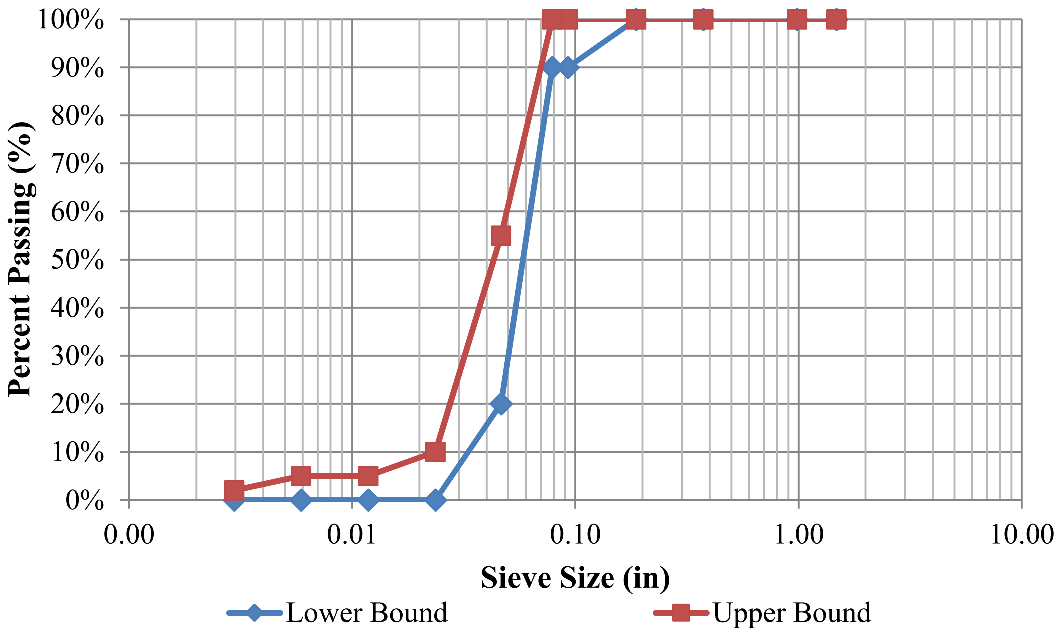


Figure 3.1 - ODOT Coarse Aggregate Gradation Bounds (Oklahoma Department of Transportation 2009)

Table 3.2 - ODOT Fine Aggregate Gradation Bounds (Oklahoma Department of Transportation)

Sieve Size/#	Sieve Opening (in)	Percent Passing (Lower Bound)	Percent Passing (Upper Bound)
3/8"	0.19	100%	100%
#4	0.09	95%	100%
#8	0.08	80%	100%
#16	0.05	50%	85%
#30	0.02	25%	60%
#50	0.01	5%	30%
#100	0.01	0%	10%
#200	0.00	0%	3%

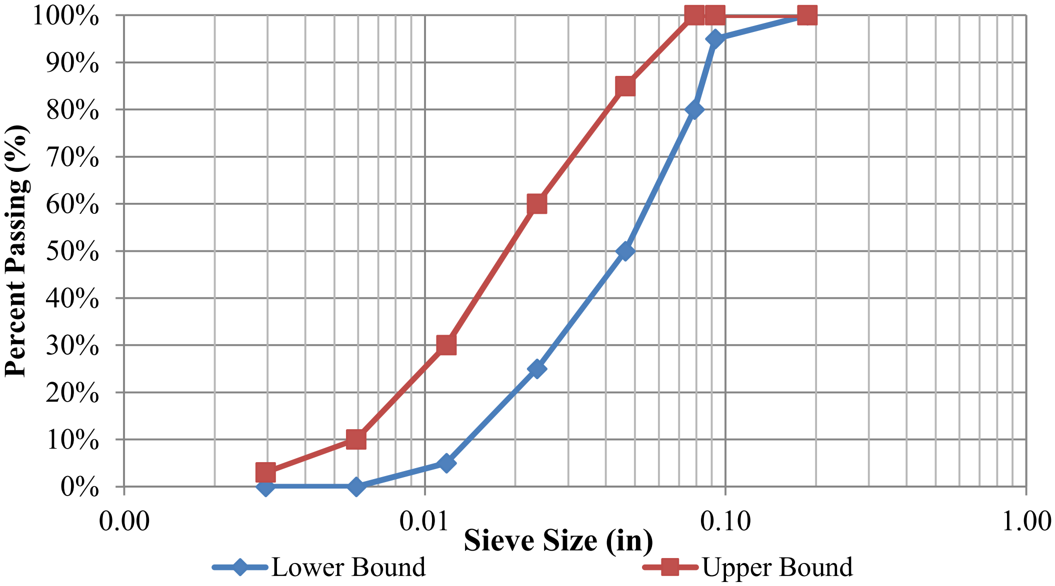


Figure 3.2 - ODOT Fine Aggregate Gradation Chart (Oklahoma Department of Transportation 2009)

3.4. Fly Ash Cement Substitution

For Class A concrete, a portion of the cement can be replaced with fly ash. The Specifications allow a 20% replacement of fly ash for portland cement. This substitution is based on a one to one weight replacement.

3.5. Air Entraining and High Range Water Reducer Admixtures

The Specifications allow the use of a variety of admixtures depending on the concrete's application. Two admixtures used in Class A mix designs are air entrainer and high range water reducer (HRWR). For both admixtures, the Specifications state their use should be in accordance with AASHTO M 154 (air entrainer) and AASHTO M 194 (HRWR).

3.6. Class A Concrete Properties

Beyond the aggregate properties, the Specifications provide requirements for both fresh and hardened concrete. The fresh properties a Class A mix are expected to meet are: a minimum cement content, water to cement ratio, slump, and air content. After curing, the concrete is required to meet a minimum compressive strength. All of these concrete requirements are provided in Table 3.3. These fine and coarse aggregate properties, along with the concrete properties, are the requirements that make up the standard pavement mix for ODOT and the reference mix for comparison of the research mixes.

Table 3.3 - ODOT Requirements for Class A Concrete (Oklahoma Department of Transportation 2009)

Class of Concrete	Minimum Cement Content	Air Content	Water/Cement Ratio	Slump	Minimum 28-Day Compressive Strength
	(lb/yd)	**%**	**lb/lb**	**in**	**psi**
A	517	6 +/- 1.5	.25-.48	2 +/- 1	3,000

4. Aggregate Properties and Concrete Testing

4.1. Mix Design Aggregates

All of the aggregates used throughout the mix designs were tested accordingly to American Society of Material Testing (ASTM) standards. A total of four aggregates were used in the concrete mix designs, #57 Rock, River Sand, 3/8" Chip rock, and recycled concrete aggregate. All of these materials are regularly available at local material suppliers in the Oklahoma City metro area. The aggregates used in this research came from either Dolese Bros. Co. or Metro Materials. Dolese Bros. Co. is a local full service construction supply and material operations company spanning two states. Dolese provided the #57 Rock, River Sand, and 3/8" Chip rock. Metro Materials is a local material supplying company in Norman, Oklahoma that supplies multiple types of rock, sand, and mulch. Metro Materials provided the recycled concrete aggregate. All of these materials were subjected to multiple tests to ensure they meet appropriate standards and are suitable in concrete.

4.2. Aggregate Testing

A total of six different tests were run on these aggregates. All tests conformed to the ASTM methods and procedures. Some of the properties are to specifically measure the characteristics of the aggregates, while others are required to accurately develop mix designs. The tests include sieve analyses, fine and coarse aggregate absorptions, durability, fineness, and specific gravities. Table 4.1 presents all the tests and corresponding ASTMs, and a brief summary of the importance of the test. After the aggregate testing was completed, the results were used in calculations of the Cement-Limiting and High-Volume Recycled mix designs.

Table 4.1 ASTM Fine and Coarse Aggregate Characterization Tests

Property	Test Method	Test Description
Dry Rodded Unit Weights	ASTM C 29	Test Methods for Bulk Density and Voids in Aggregates
Density & Absorption	ASTM C 127	Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate.
Density & Absorption	ASTM C 128	Test Method for Density, Relative Density (Specific Gravity), and Absorption of Fine Aggregate.
Abrasion Resistance	ASTM C 131	Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in Los Angeles Machine
Sieve Analysis	ASTM C 136	Test Method for Sieve Analysis of Fine and Coarse Aggregates Fineness Modulus

4.3. Aggregate Testing Results

As mentioned previously, due to the variety of mix designs, multiple aggregates were tested. These tests were conducted at either Fears Structural Laboratory or Broce Civil Engineering Materials Laboratory located on the University of Oklahoma campus. These results were compared to the ODOT Specifications for the #57 Rock and the River Sand. For the #57 Rock, as previously mentioned, the maximum allowable loss from the LA Abrasion test is 40%, the loss for this rock was 23.6%. Table 4.2 shows the gradation values for the #57 Rock compared to the bounds set by the ODOT Specification, and Figure 4.1 shows the sieve analysis with the ODOT bounds. The rock tested was outside the lower bounds for the 3/4” and 3/8” sieves. This indicated that the coarse aggregate contained more particles smaller than the 3/8” sieve than the ODOT specified #57 Rock. The absorption and the specific gravity for the #57 Rock were 0.86% and 2.67, respectively. Lastly, the dry rodded unit weight (DRUW) was calculated as 102 lb/ft^3.

The River Sand, with a fineness modulus 2.63 met the fineness modulus range of 2.3 to 3.1 in the Specifications. Table 4.3 shows the gradation values for the River Sand compared to the bounds set by ODOT Specification, and Figure 4.2 shows the sieve analysis with the ODOT bounds. The River Sand gradation fit right in between the set ODOT bounds. The River Sand had absorption of 0.70% and a specific gravity of 2.51.

Table 4.4 shows the gradation values for the 3/8" Chip aggregate, and Figure 4.3 shows the sieve analysis. The absorption and specific gravity for the 3/8" Chip was 1.01% and 2.67, respectively. Lastly, the DRUW was calculated as 104 lb/ft^3.

The RCA was used as a substitute for the #57 Rock and to supplement the River Sand in the High-Volume Recycled mix designs. Due to the nature of RCA, both fine and coarse aggregate tests were conducted. Table 4.5 shows the gradation values for the RCA, and Figure 4.4 shows the sieve analysis. The coarse absorption was 4.47% and fine absorption was 6.48%. Both of these absorption rates were significantly higher than that of the #57 Rock and River Sand. The specific gravities were 1.92 and 2.01 for the coarse and fine RCA particles, respectively. The dry rodded unit weight was 92.5 lb/ft^3 and the LA Abrasion was 43.6%. Lastly, the dry rodded unit weight (DRUW) was calculated as 92.5 lb/ft^3. Table 4.6 outlines the results for all the aggregate testing.

Table 4.2 - #57 Rock Gradations with ODOT Upper and Lower Bounds

Sieve Size/ #	Sieve Opening (in)	Percent Passing (Lower Bound)	Tested #57 Percent Passing	Percent Passing (Upper Bound)
1.5"	1.500	100%	100.00%	100%
1"	1.000	95%	99.25%	100%
3/4"	0.750	95%	79.28%	100%
3/8"	0.375	25%	10.11%	60%
#4	0.187	0%	1.27%	10%
#8	0.093	0%	0.53%	5%
#16	0.047	0%	0.36%	5%
#30	0.024	0%	0.26%	5%
#50	0.012	0%	0.17%	5%
#100	0.006	0%	0.09%	5%
#200	0.003	0%	0.04%	2%

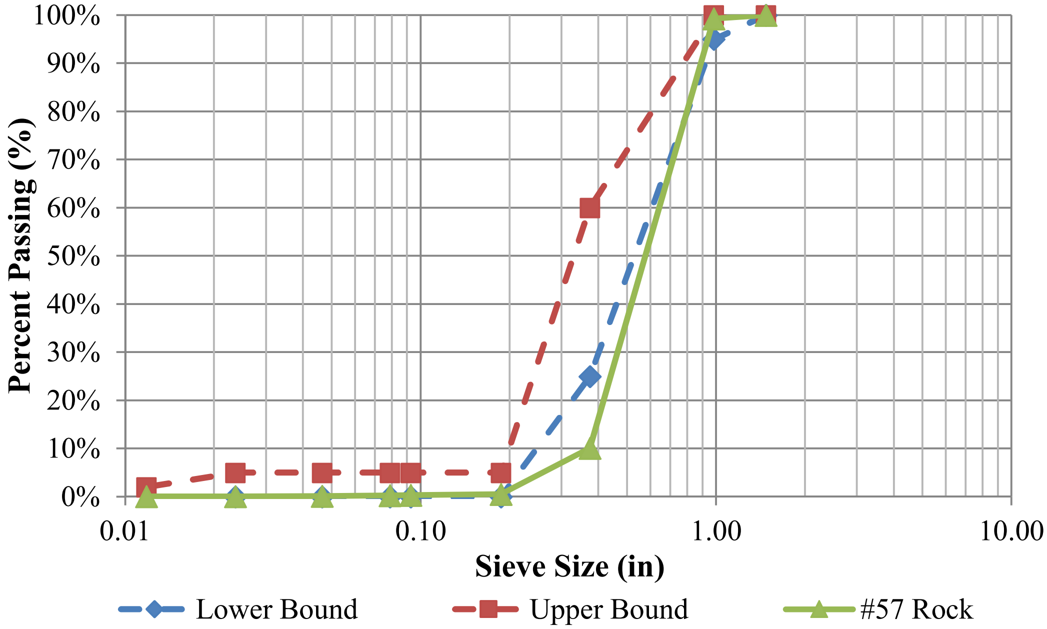


Figure 4.1 - #57 Rock Gradation with ODOT Upper and Lower Bounds

Table 4.3 - River Sand Gradation with ODOT Upper and Lower Bounds

Sieve Size/#	Sieve Opening (in)	Percent Passing (Lower Bound)	River Sand Percent Passing	Percent Passing (Upper Bound)
3/8"	0.375	100%	100%	100%
#4	0.187	95%	99%	100%
#8	0.093	80%	95%	100%
#16	0.047	50%	80%	85%
#30	0.024	25%	47%	60%
#50	0.012	5%	14%	30%
#100	0.006	0%	2%	10%
#200	0.003	0%	0%	3%
Pan	-	0%	0%	0%

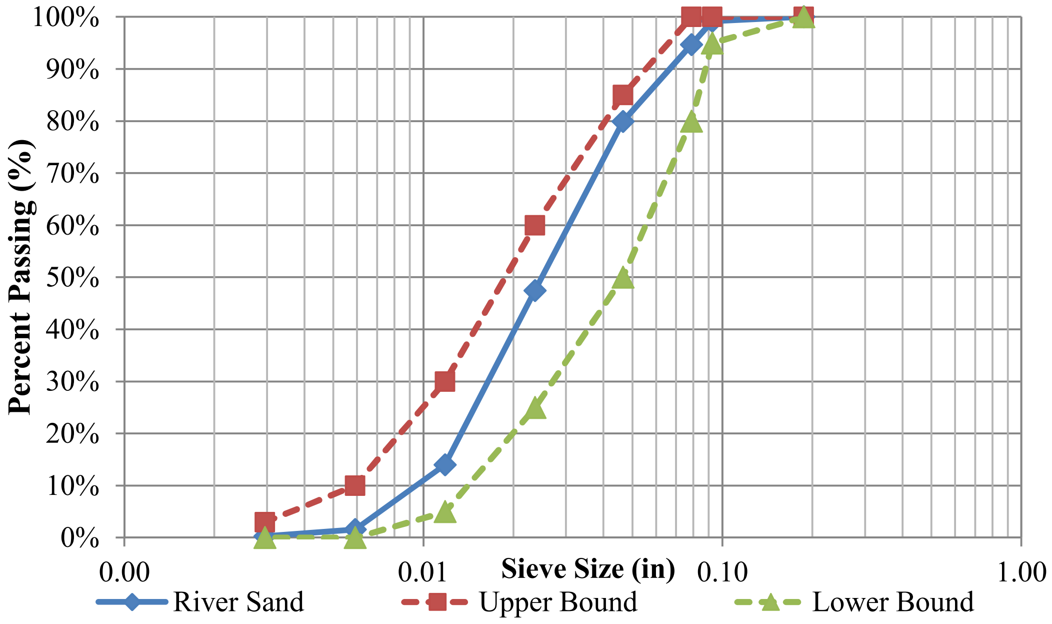


Figure 4.2 - River Sand Gradation with ODOT Upper and Lower Bounds

Table 4.4 - 3/8" Chip Gradation Data

Sieve Size/#	Sieve Opening (in)	Individual Weight Retained (lb)	Cumulative Weight Retained (lb)	Percent Retained	Percent Passing
1.5"	1.500	0.00	0.00	0.00%	100.00%
1"	1.000	0.00	0.00	0.00%	100.00%
3/4"	0.750	0.00	0.00	0.00%	100.00%
3/8"	0.375	0.64	0.64	6.00%	94.00%
#4	0.187	9.03	9.68	87.00%	13.00%
#8	0.093	1.05	10.73	96.00%	4.00%
#16	0.047	0.16	10.89	97.00%	3.00%
#30	0.024	0.09	10.98	98.00%	2.00%
#50	0.012	0.04	11.02	99.00%	1.00%
#100	0.006	0.06	11.08	99.00%	1.00%
#200	0.003	0.07	11.15	100.00%	0.00%
Pan	-	0.04	11.19	100.00%	0.00%

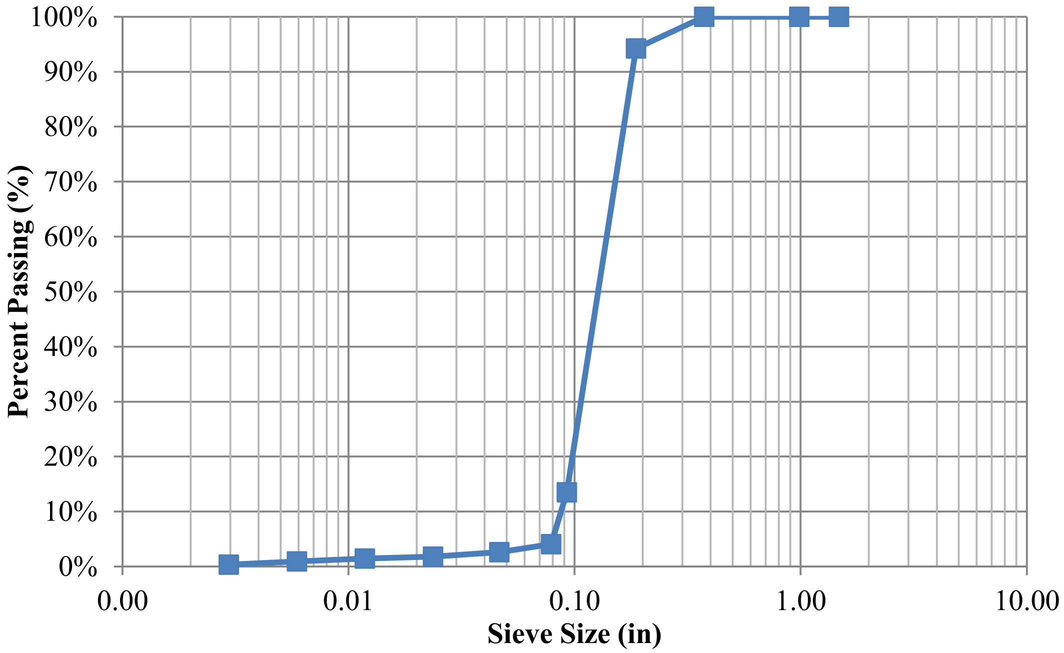


Figure 4.3 - 3/8" Chip Gradation Chart

Table 4.5 - Recycled Concrete Aggregate Gradation Data

Sieve Size/#	Sieve Opening (in)	Individual Weight Retained (lb)	Cumulative Weight Retained (lb)	Percent Retained	Percent Passing
1.5"	1.500	0.00	0.00	0.00%	100.00%
1"	1.000	1.69	1.69	9.00%	91.00%
3/4"	0.750	3.05	4.73	25.00%	75.00%
3/8"	0.375	6.00	10.73	57.00%	43.00%
#4	0.187	3.30	14.03	74.00%	26.00%
#8	0.093	1.53	15.57	82.00%	18.00%
#16	0.047	0.91	16.48	87.00%	13.00%
#30	0.024	0.79	17.26	91.00%	9.00%
#50	0.012	0.76	18.03	95.00%	5.00%
#100	0.006	0.59	18.62	98.00%	2.00%
#200	0.003	0.27	18.89	100.00%	0.00%
Pan	-	0.06	18.95	100.00%	0.00%

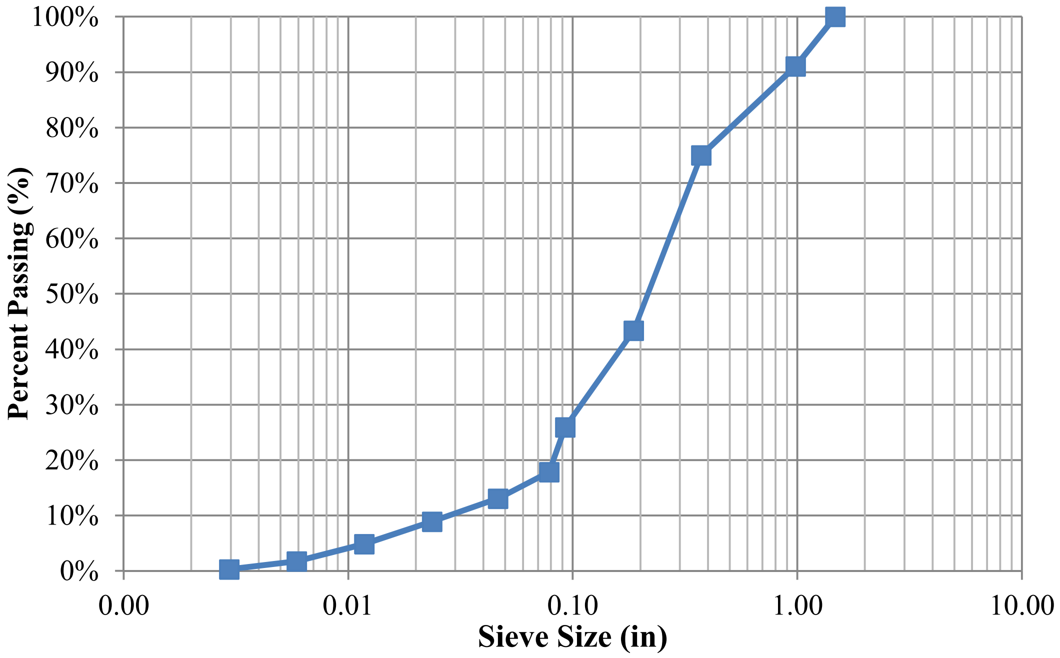


Figure 4.4 - Recycled Concrete Aggregate Gradation Chart

Table 4.6 - Fine and Coarse Aggregate Testing Results

#57 Rock Aggregate	
Absorption	0.860%
Specific Gravity	2.67
DRUW (lb/ft^3)	102
LA Abrasion	26.5%
River Sand	
Absorption	0.700%
Specific Gravity	2.51
Fineness Modulus	2.63
3/8" Chip Aggregate	
Absorption	1.010%
Specific Gravity	2.67
DRUW (lb/ft^3)	104
Recycled Concrete Aggregates	
Absorption (Coarse)	4.47%
Absorption (Fine)	6.48%
Specific Gravity (Coarse)	1.923
Specific Gravity (Fine)	2.01
DRUW (lb/ft^3)	92.5
LA Abrasion	43.6%

4.4. Concrete Laboratory Testing

Since multiple mix designs were conducted, a variety of ASTM testing methods were used to compare the fresh and hardened properties. In determining the appropriate mix designs to incorporate in to the full-scale test sections, the slump (ASTM C 143) was performed on the fresh concrete and compressive strength (ASTM C 39) were taken at 7, 14, and 28 days for the Cement-Limiting mix designs and 1,3,7,14, and 28 days for the High-Volume Recycled mix designs. These results were compared to the ODOT Specifications requirements that were previously mentioned. Once the Cement-Limiting and High-Volume Recycled Concrete mix designs were selected, and performed to the required Specifications, additional tests were conducted upon pouring the full-scale sections.

In addition to previous test on the full-scale sections; on the fresh concrete unit weight (ASTM C 138), slump (ASTM C 143), and air content (ASTM C 231) were performed. During the curing and hardening process compressive strengths (ASTM C 39) were taken at 1, 3, 7, 14, 28 days, and tensile strength (ASTM C496), flexural strength (ASTM C 78), and modulus of elasticity (ASTM C 469) were taken at 28 days. Table 4.7 outlines the tests performed on the concrete and the time at which the test was performed.

Table 4.7 - Concrete Property Tests

Property	Test Method	Test Description	Time
Fresh Concrete Property Tests			
Unit Weight	ASTM C 138	Test Method for Density (Unit Weight)	Time of pour
Slump	ASTM C 143	Test Method for Slump of Hydraulic-Cement Concrete	Time of pour
Air Content	ASTM C 231	Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method	Time of pour
Hardened Concrete Property Tests			
Compressive Strength	ASTM C 39	Test Methods for Compressive Strength of Cylindrical Concrete Specimen	1, 3, 7, 14, and 28 Days
Tensile Strength	ASTM C 496	Test Methods for Tensile Strength of Cylindrical Concrete Specimens	28 Days
Flexural Strength	ASTM C 78	Test Method for Flexural Strength of Concrete	28 Days
Modulus of Elasticity	ASTM C 469	Test Method for Static Modulus of Elasticity	28 Days

5. Laboratory Cement-Limiting Concrete

5.1. Introduction

The first experimental mix design investigated was the Cement-Limiting concrete. In this mix the cement content was reduced below the limit set at 517 lb/yd^3 for a regular Class A mix, or 470 lb/yd^3 if using aggregate optimization. Due to the environmentally taxing requirements in manufacturing cement, aggregate optimization methods were used to reduce the maximum amount of cement in potential mix designs. ODOT Specifications special provision 701.14 outlines the additional requirements for optimized gradation concrete mix designs for concrete pavements. In addition to the previously discussed requirements, the mix design is required to meet Area II in the Coarseness/Workability Chart, Figure 5.1, and reach a flexural strength of 700 psi. The Coarseness/Workability Chart is the primary method for aggregate optimization with Power 45 and Percent Retained as secondary methods to supplement the results of the Coarseness/Workability Chart. For this research, all three aggregate optimization methods were investigated separately to arrive at three potential aggregate distributions, then a blended approach of all three optimization methods was used to develop a single optimized aggregate distribution. The properties and response of four mix designs based on the results of each of the four different approaches were then compared.

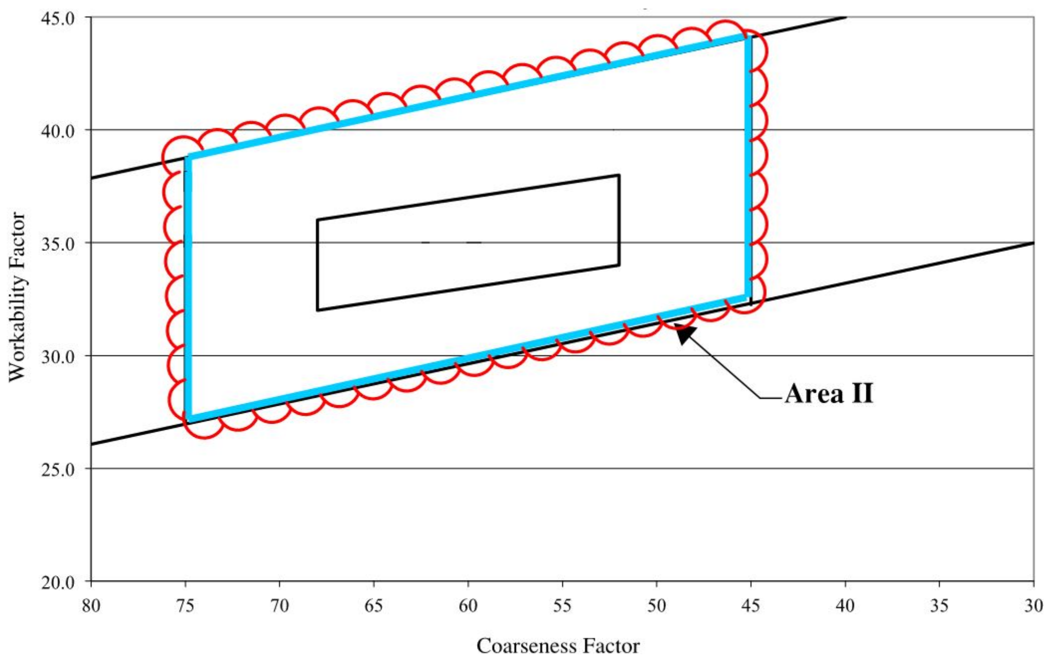


Figure 5.1 - Coarseness/Workability Chart

5.2. Aggregate Optimization Methods and Gradations

For the aggregate optimizations, two coarse aggregates, #57 Rock and 3/8" Chip, and one fine aggregate, River Sand, were used in all the mix designs. To determine how each optimization method performed in a controlled scenario, each individual method was optimized and a mix was conducted in the laboratory. Each gradation investigation began with a 56/44 split of coarse to fine aggregate.

5.2.1. Coarseness and Workability Method

This "model" gradation was selected based on multiple trial and error attempts to graphically center the gradation in the center of Area II. These attempts were individually evaluated to see how each factor was calculated and plotted on the Coarseness/Workability chart. The final selection was chosen based on the closest to a coarseness factor of 61, workability factor of 35, and plot location at the center of Area II on a Coarseness/Workability Chart (Shilstone).

The final Coarseness/Workability was selected based on a coarseness factor calculation of 61.1, workability factor of 35.1, and a plot location shown in Figure 5.3. The Coarseness/Workability mix design contained an aggregate percentage breakdown of 32% #57 Rock, 16% 3/8" Chip, and 52% River Sand. This proportion resulted in a Workability factor of 35.1 and Coarseness factor of 61.1. Table 5.1 shows the individual and combined gradation data, and Figure 5.2 shows the gradation chart of the combined results. Figure 5.3 shows the placement of the design gradation on the Coarseness/Workability chart.

Table 5.1 - Coarseness/Workability Gradation Data

Sieve Size	Sieve Size (in)	# 57 Rock % Passing	3/8" Chip % Passing	River Sand % Passing	Combined % Passing	Combined Cumulative % Retained
1.5"	1.500	100.00	100.00	100.00	100.00%	0.00%
1"	1.000	84.36	100.00	100.00	95.00%	5.00%
3/4"	0.750	58.38	100.00	100.00	86.70%	13.30%
3/8"	0.375	8.16	93.05	100.00	69.50%	30.50%
#4	0.187	1.33	14.66	99.18	54.30%	45.70%
#8	0.093	0.62	4.07	94.67	50.10%	49.90%
#16	0.047	0.43	2.40	79.88	42.10%	57.90%
#30	0.024	0.30	1.60	47.39	25.00%	75.00%
#50	0.012	0.19	1.24	13.97	7.50%	92.50%
#100	0.006	0.10	0.71	1.58	0.10%	99.90%
#200	0.003	0.03	0.32	0.27	0.20%	99.80%
Pan	0.000	0.00	0.00	0.00	0.00%	100.00%
Percent of Total Agg.		32.00%	16.00%	52.00%	100.00%	

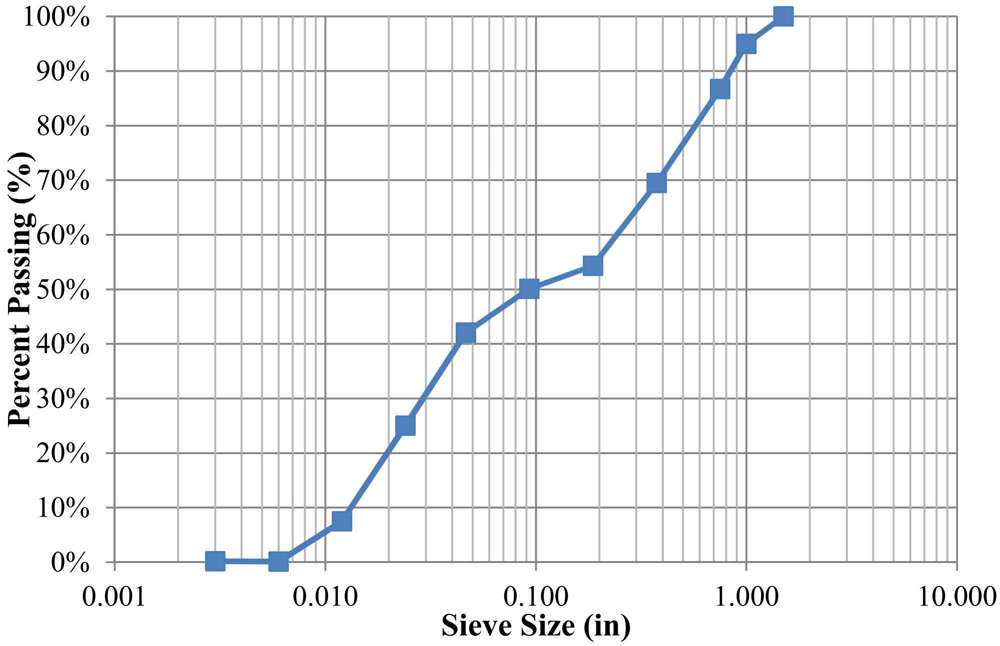


Figure 5.2 - Coarseness/Workability Gradation Chart

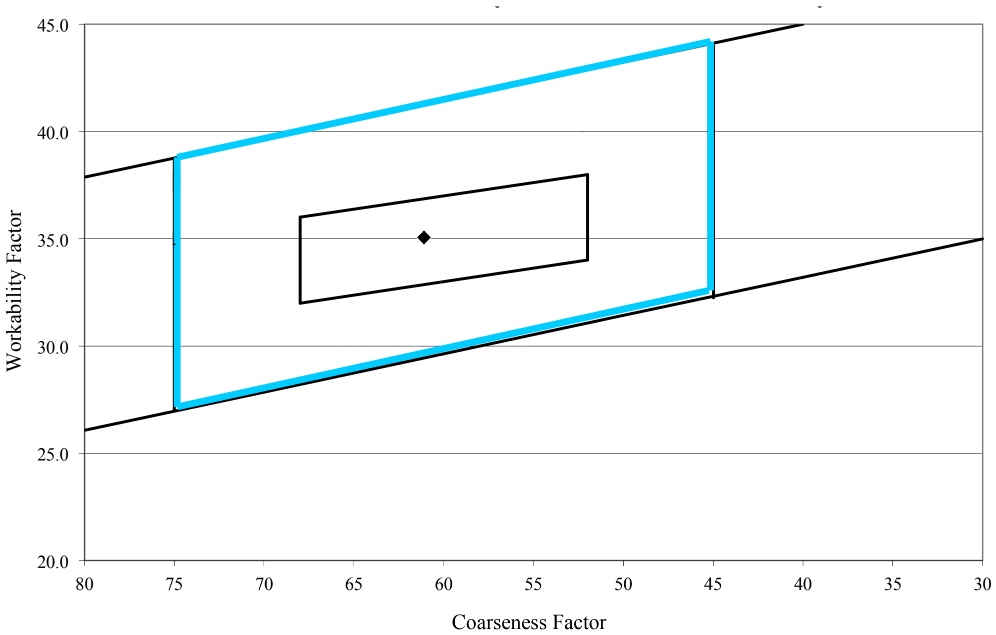


Figure 5.3 - Coarseness/Workability Chart Analysis

5.2.2. Percent Retained Method

The Percent Retained method was the next aggregate optimization method employed in the research study. Figure 5.4 shows a blank Percent Retained method chart with the upper and lower bounds. The goal behind the Percent Retained method is to maintain the percent retained data of the combined aggregate gradation in between the two bounds and limit the number of peaks and valleys between each sieve size. These peaks and valleys in the gradation are affected by the nature of the aggregate size and the overall percent amount of the aggregate used. The final gradation was selected based on multiple trial and error attempts to graphically create the best fit into the Percent Retained Chart. These attempts were individually evaluated to see how different aggregate percentage breakdowns plotted on a Percent Retained Chart and their corresponding peaks and valleys. The selected Percent Retained mix design contained an aggregate percentage breakdown of 30% #57 Rock, 23% 3/8" Chip, and 47% River Sand. Table 5.2 shows the individual and combined gradation data, and Figure 5.5 shows the gradation chart of the combined results. Figure 5.6 shows the placement of the design gradation on a Percent Retained chart.

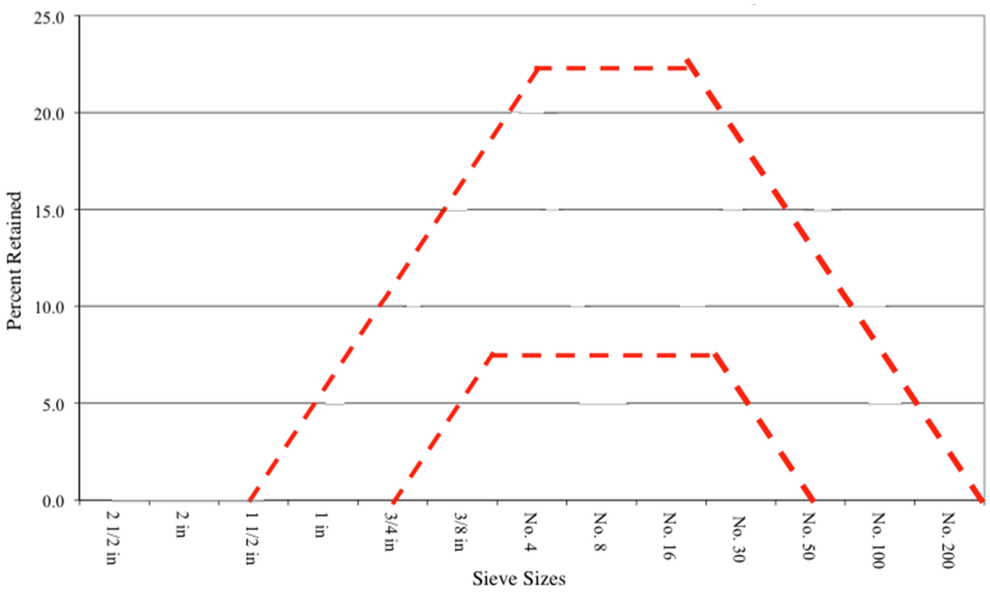


Figure 5.4 - Percent Retained Method Chart

Table 5.2 - Percent Retained Gradation Data

Sieve Size	Sieve Size (in)	# 57 Rock % Passing	3/8" Chip % Passing	River Sand % Passing	Combined % Passing	Combined Cumulative % Retained
1.5"	1.500	100.00	100.00	100.00	100.00%	0.00%
1"	1.000	84.36	100.00	100.00	95.31%	4.69%
3/4"	0.750	58.38	100.00	100.00	87.51%	12.49%
3/8"	0.375	8.16	93.05	100.00	70.85%	29.15%
#4	0.187	1.33	14.66	99.18	50.39%	49.61%
#8	0.093	0.62	4.07	94.67	45.62%	54.38%
#16	0.047	0.43	2.40	79.88	38.22%	61.78%
#30	0.024	0.30	1.60	47.39	22.73%	77.27%
#50	0.012	0.19	1.24	13.97	6.91%	93.09%
#100	0.006	0.10	0.71	1.58	0.94%	99.06%
#200	0.003	0.03	0.32	0.27	0.21%	99.79%
Pan	0.000	0.00	0.00	0.00	0.00%	100.00%
Percent of Total Agg.		30.00%	23.00%	47.00%	100.00%	

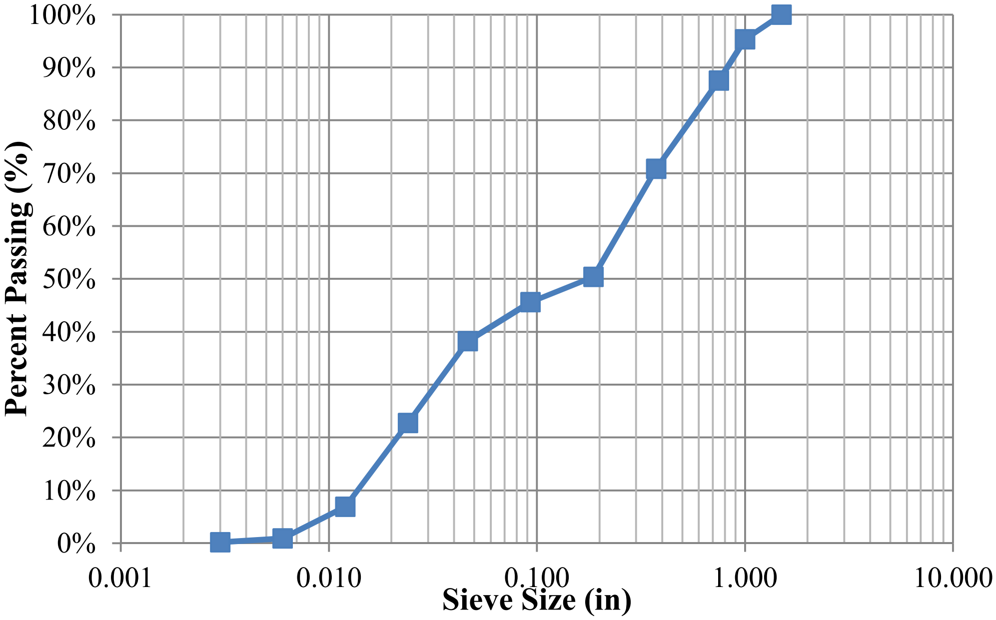


Figure 5.5 - Percent Retained Gradation Chart

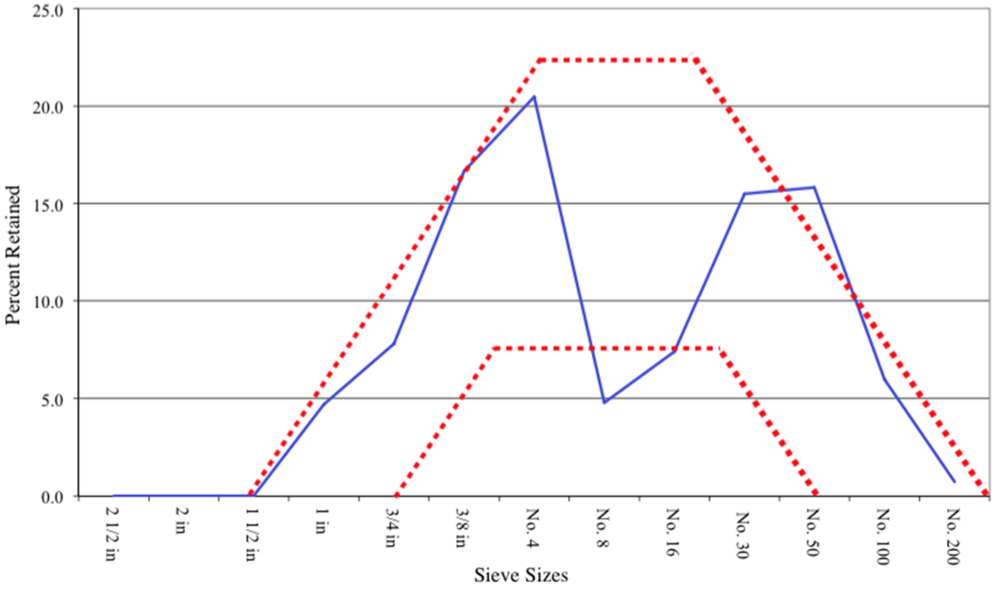


Figure 5.6 - Percent Retained Chart

5.2.3. The Power 45 Method

The Power 45 method was the last aggregate optimization method investigated in the research study. Initial combinations of aggregates will be based on traditional methods of maximizing packing density. These methods are based on the Fuller optimization curve, described by Eq. (1), which provides the ideal particle size distribution to achieve the maximum density of the aggregate materials within a concrete mixture (Fuller and Thompson, 1907).

$$P = (d_i/D_{max})^q$$

Eqn. 5-1

where P is the cumulative passing % at a specific diameter d_i; d_i is the particle diameter under consideration; D_{max} is the nominal maximum particle diameter; and q is the packing exponent.

The Fuller curve is based on a packing exponent of 0.50. Talbot and Richart (1923) developed Eq. (5-1) based on Fuller's curve but suggested a packing exponent of 0.45. Their relationship is commonly referred to as the "0.45 power curve." Andreasen and Andersen (1930). Figure 5.7 shows a blank Power 45 chart with the upper and lower bounds. The final gradation was selected based on multiple trial and error attempts in Microsoft Excel. These attempts were individually evaluated to see how different aggregate percentage breakdowns plotted on a Power 45 Chart. The selected Power 45 mix design contained an aggregate percentage breakdown of 35% #57 Rock, 25% 3/8" Chip, and 40% River Sand. Table 5.3 shows the individual and combined gradation data, and

Figure 5.8 shows the gradation chart of the combined results. Figure 5.9 shows the placement of the design gradation on the Power 45 chart.

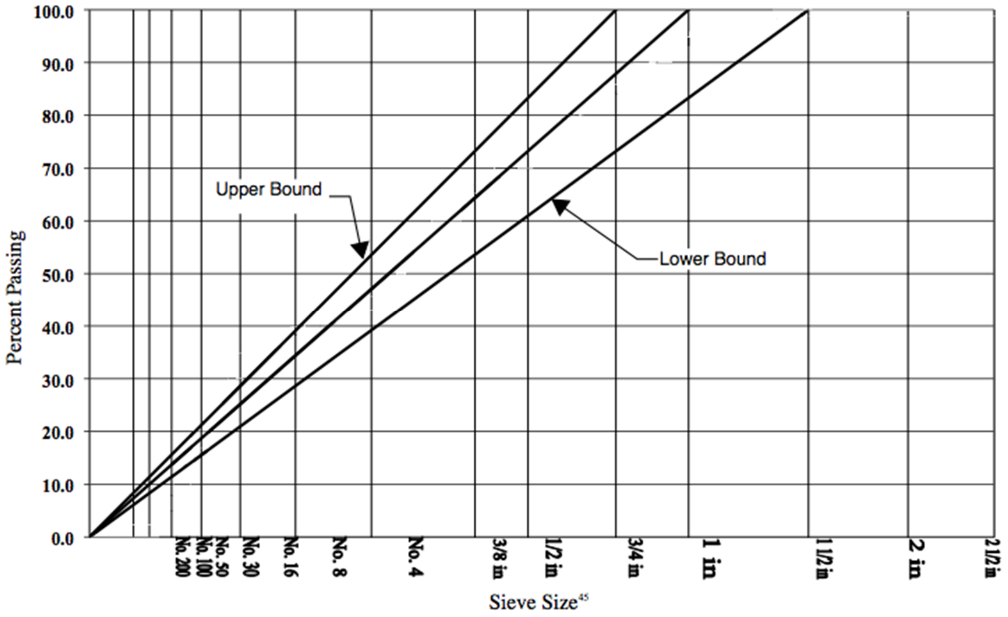


Figure 5.7 - Blank Power 45 Chart

Table 5.3 - Power 45 Gradation Data

Sieve Size	**Sieve Size (in)**	**# 57 Rock % Passing**	**3/8" Chip % Passing**	**River Sand % Passing**	**Combined % Passing**	**Combined Cumulative % Retained**
1.5"	1.500	100.00	100.00	100.00	100.00%	0.00%
1"	1.000	84.36	100.00	100.00	94.53%	5.47%
3/4"	0.750	58.38	100.00	100.00	85.43%	14.57%
3/8"	0.375	8.16	93.05	100.00	66.12%	33.88%
#4	0.187	1.33	14.66	99.18	43.80%	56.20%
#8	0.093	0.62	4.07	94.67	39.10%	60.90%
#16	0.047	0.43	2.40	79.88	32.70%	67.30%
#30	0.024	0.30	1.60	47.39	19.46%	80.54%
#50	0.012	0.19	1.24	13.97	5.96%	94.04%
#100	0.006	0.10	0.71	1.58	0.84%	99.16%
#200	0.003	0.03	0.32	0.27	0.20%	99.80%
Pan	0.000	0.00	0.00	0.00	0.00%	100.00%
Percent of Total Agg.		35.00%	25.00%	40.00%	100.00%	

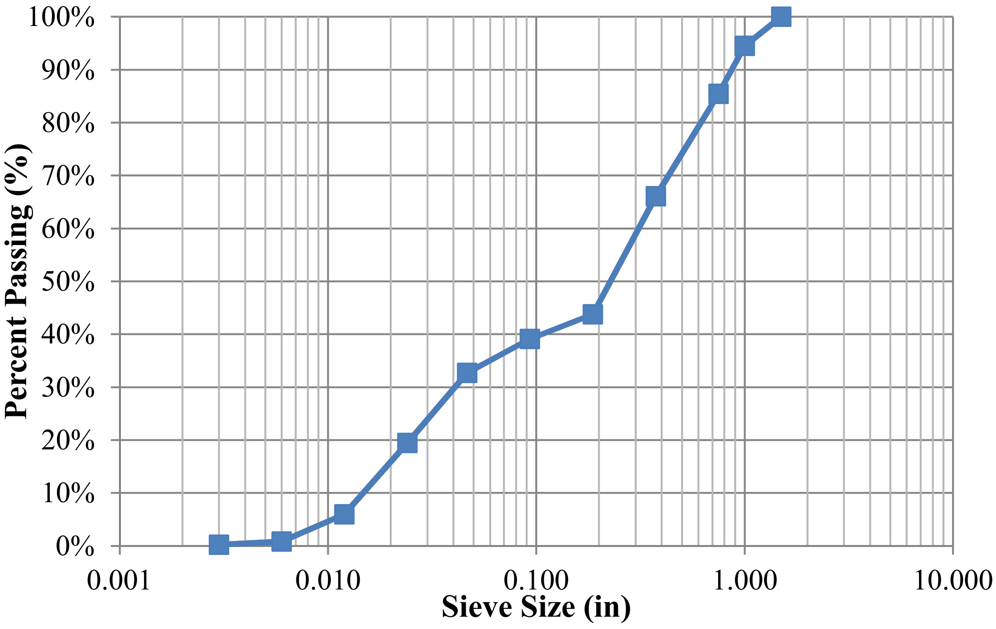


Figure 5.8 - Power 45 Gradation Chart

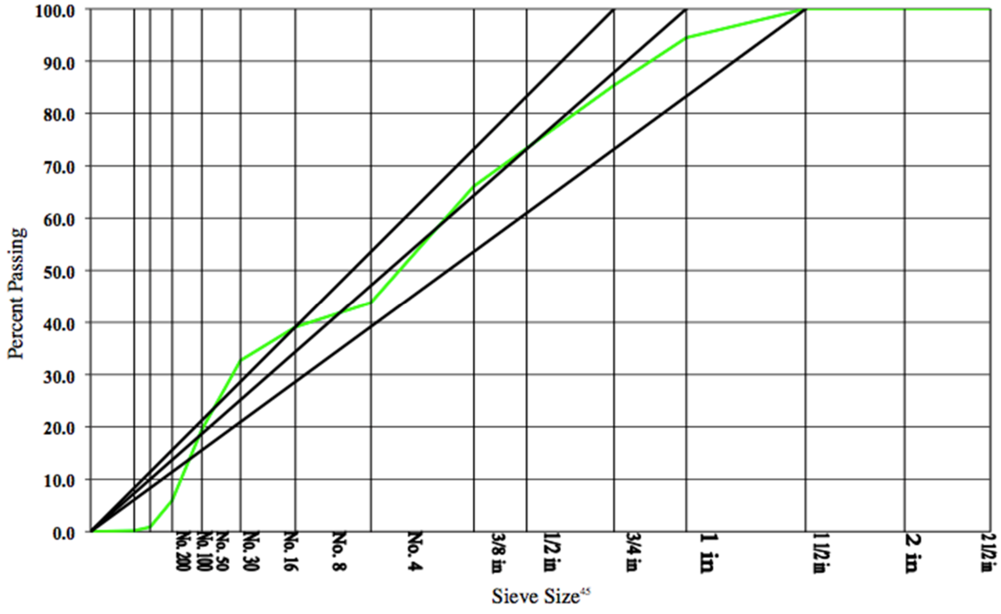


Figure 5.9 - Power 45 Chart Analysis

5.3. Aggregate Optimization Method Comparison

The three aggregate optimization methods produced three different percent aggregate breakdowns. Comparing each of the different aggregate optimization methods, we see how the aggregate percentages were affected (Table 5.5). Using the 56% 44% coarse/fine aggregate split as a basis, the Power 45 method was the only method to incorporate less fine aggregate. This was due to the #50 sieve that retains 50% of the sand, which causes a rise in the Power 45 curve towards the end of the chart. The approach to bring this back within the bounds is to decrease the percentage of River Sand. Table 5.4 shows the combined gradation data for all three aggregate optimization methods, followed by Figure 5.10 which shows the gradation chart containing the results of each method. Figure 5.10 also shows the decrease in the percent passing with the between 3/8” sieve to #30 with the Power 45 Method having the highest percent passing followed by Percent Retained Method and the Coarseness/Workability Method. This comes down to the amount of 3/8” Chip and River Sand. For all of these aggregate distributions, an identical concrete mix design was used to evaluate the effect of each optimization on the resulting fresh and hardened concrete properties, which will help determine the “best” aggregate optimized mix design.

Table 5.4 - Percent Fine and Coarse Aggregate

	Percent #57 Rock	**Percent 3/8" Chip**	**Percent River Sand**
Coarseness/Workability Method	32	16	52
Percent Retained Method	30	23	47
Power 45 Method	35	25	40

Table 5.5 - Combined Percent Passing for Each Aggregate Optimized Method

Sieve Size	Sieve Size (in)	Coarse/Work Combined % Passing	% Retained Combined % Passing	Power 45 Combined % Passing
1.5"	1.500	100%	100%	100%
1"	1.000	95%	95%	95%
3/4"	0.750	87%	88%	85%
3/8"	0.375	70%	71%	66%
#4	0.187	54%	50%	44%
#8	0.093	50%	46%	39%
#16	0.047	42%	38%	33%
#30	0.024	25%	23%	19%
#50	0.012	8%	7%	6%
#100	0.006	0%	1%	1%
#200	0.003	0%	0%	0%
Pan	0.000	0%	0%	0%

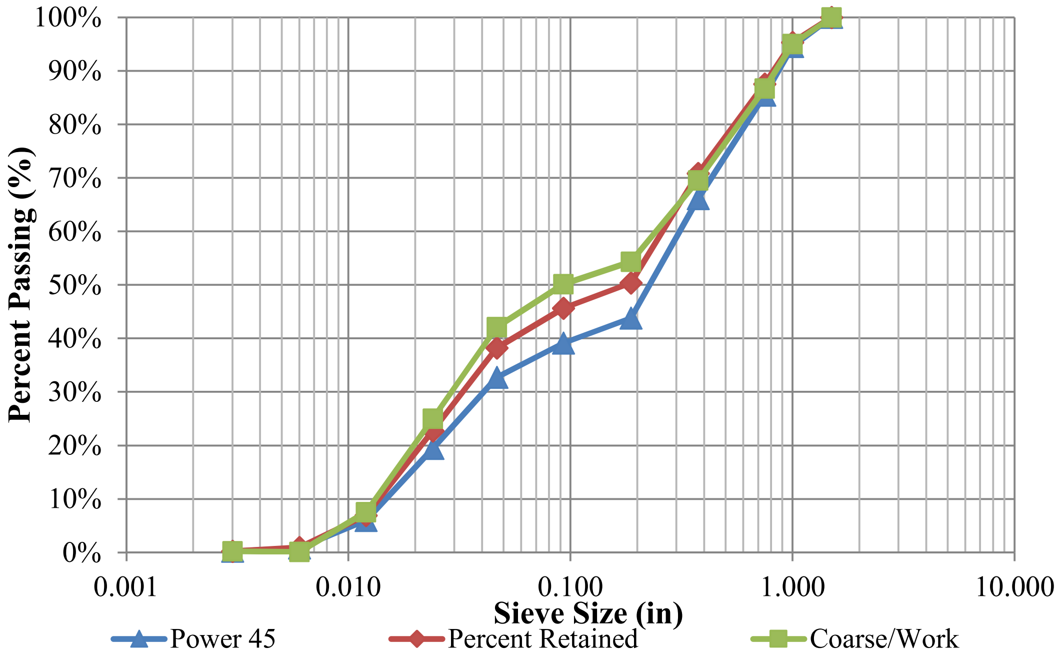


Figure 5.10 - All Individual Aggregate Optimization Method Gradation Chart

5.4. Concrete Mix Designs Properties

Each of the aggregate optimization mix designs contained the percent aggregate as stated previously. The cementitious material was reduced 10% to 423 lb/yd^3 to create a 4.5 sack mix, which incorporated 20% Class C Fly Ash. The addition of fly ash reduced the total amount of cement to 338.4 lb/yd^3. The water-to-cement ratio (w/c) was set at 0.48, the amount of air entraining admixture was 0.67 ounce per hundredweight (oz/cwt), and water reducer was not included in the mix design. Table 5.6 outlines the requirements for the Cement-Limiting mix design. The total weight of aggregates was identical for each mix design. This allowed the investigation into the effect of each optimization method on the fresh and hardened properties of an identical concrete mix.

Table 5.6 - Cement-Limiting Concrete Mix Design Properties

Cementitious Material, lb/yd^3	423.0
w/c Ratio	0.48
Fly Ash, %	20%
Fly Ash, lb/yd^3	84.6
Cement, lb/yd^3	338.4
Water, lb/yd^3	203.0
Master Builders AE-90 (air entrainer), oz/cwt	0.67
Glenium 7500 (water reducer), oz/cwt	0.00

5.5. Testing Methods and Fresh Concrete Properties

For the Cement-Limiting mix designs, a total of 2.5 ft^3 was batched for each of the three aggregate optimization methods. This provided a sufficient amount of concrete to perform a slump test and prepare 12 total 4”x8” cylinders for compressive strength testing at 7, 14, and 28 days, with 3 additional. For the compressive strength tests the cylinders were ground to create a smooth area to remove all stress concentrations. Table 5.7 outlines the slump test results for all three optimization methods. Only the Coarseness/Workability method was outside the required ODOT Specification. These results also show a correlation in that a lower percentage of fine aggregates may cause an increase in slump but also a less cohesive mix that was rocky and difficult to consolidate and finish.

Table 5.7 - Fresh Concrete Properties for Cement-Limiting Laboratory Mix Designs

	Slump (in.)
Coarseness/Workability Method	0.75
Percent Retained Method	1.0
Power 45 Method	1.5

5.6. Hardened Concrete Results

With the goal of reaching the ODOT specifications and determining the characteristics of each of the aggregate optimization methods, the compressive strengths were taken at 7, 14, and 28 days. The Percent Retained and Power 45 methods reached the 3,000 psi limit at 7 days and continued to gain strength, reaching approximately 4,500 psi for the Power 45 and the highest overall strength of 4,900 psi for the Percent Retained method. The Coarseness/Workability method reached approximately 2,150 psi at 7 days, increased to 2,400 psi at 14 days, and increased to 2,850 psi at 28 days, never reaching the ODOT specified goal of 3,000 psi. Figure 5.11 shows a graphical representation of the compressive strengths over the 28 day test period. Table 5.8 outlines the compressive strength test results for all three aggregate optimization methods.

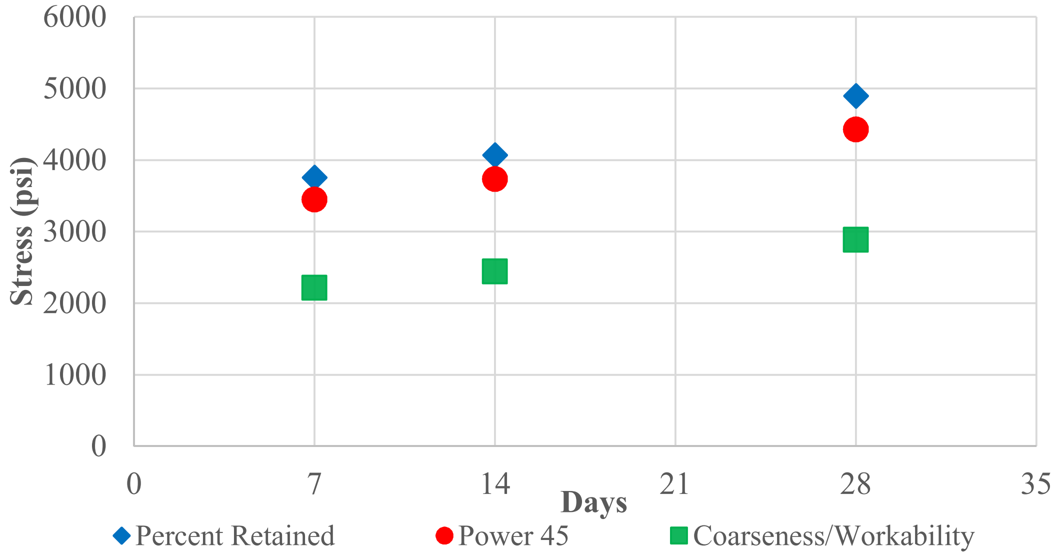


Figure 5.11 - Cement-Limiting Concrete Compressive Strengths

Table 5.8 - Cement Limiting Laboratory Concrete Compressive Strengths

	Percent Retained	
	Load (lb)	Stress (psi)
7 Day	47230	3760
14 Day	51160	4070
28 Day	61560	4900
	Power 45	
	Load (lb)	Stress (psi)
7 Day	43320	3450
14 Day	46940	3740
28 Day	56620	4430
	Coarseness/Workability	
	Load (lb)	Stress (psi)
7 Day	27830	2220
14 Day	30680	2440
28 Day	36240	2880

5.7. Laboratory Discussion

The aggregate gradation data played a role in both the fresh properties and the compressive strength results for all three aggregate optimization methods in the Cement-Limiting concrete. The percent fines ranged from 40% - 52%, with the Power 45 method at the low end and the Coarseness/Workability method at the high end. During batching, the visual cues and texture of the fresh concrete properties varied, especially for the Power 45 and the Coarseness/Workability mixes. Due to the lower amount of fine aggregate, the Power 45 mix was “harsh”, which resulted in difficulty in consolidating and finishing the slump test and test specimens. On the other hand, the higher amount of fine aggregate in the Coarseness/Workability mix lead to a high workability and cohesiveness, which made preforming the slump test and preparing the test specimens much easier. This varied from the slump test results. The Coarseness/Workability mix lowered the slump values by 50%, which is one measure of the workability for fresh concrete. The increased fine aggregate material allowed the fresh mix to “compact” better around the coarse aggregates.

However, beyond the fresh characteristics of the mixes, the hardened properties also show an overall trend, that as the percent fine aggregates decreases below 50%, the compressive strengths increases. This can only be seen in the Coarseness/Workability method, which has the largest percentage fine aggregates, and the Power 45 method and the Percent Retained method, which has the least percentage of fine aggregates. The difference in 28 day strengths of the Power 45 and Percent Retained were similar, however here was a significant decrease in strength comparing the these two mixes and the Coarseness/Workability method. This decrease in strength could be related to the increased amount of fine aggregates that decreases the percentage of coarse aggregate, which provides a stronger compression area due to the higher energy required to facture the coarse aggregates. Even though these two methods have only 7% or greater River Sand, the Percent Retained Method had relatively similar strengths compared to the Power 45 method. However, the Coarseness/Workability method goes beyond the 50% coarse and fine aggregate breakdown. Once the percent fine aggregates goes beyond 50% of the total aggregate, this resulted in more than a 1,000 psi difference.

Comparing the compressive strength results and the slump tests results again we see a comparable trend. As the amount of fine aggregates increases, the slump decreases as does the compressive strength as seen in the Coarseness/Workability mix. The increased fine aggregates allowed the smaller particles to compact easier in the void space of the coarse aggregates. However, this compaction did not translate into higher strengths since the Coarseness/Workability method recorded the lowest compressive strengths. The opposite was true for the Power 45, which had a higher slump and higher compressive strengths.

6. Laboratory High-Volume Recycled Concrete

6.1. Introduction

The High-Volume Recycled Material concrete was the second experimental mix design investigated. As previously mentioned, RCA and fly ash are the major recycled materials used. The High-Volume Recycled Material mix was held to the same requirements set by the ODOT Specification for Class A concrete. The ODOT Specifications outline the allowed application and use of both of these materials for construction. RCA is allowed as aggregate base, but not for any concrete application. Fly ash, however, is allowed as a sub-base stabilizer and can be incorporated into concrete as a partial cement substitution. To develop a well performing mix design that maximizes the amount of recycled material, the “natural” materials were slowly replaced with the recycled materials.

6.2. Recycled Material Replacement

Due to the nature of RCA, which consists of a mix of uniformly graded material, especially when compared to typical coarse aggregates, care must be taken when introducing the material into a potential concrete mix. Due to this variance, RCA cannot be incorporated as a direct replacement for the coarse aggregates. For this project, the RCA was substituted using statistical methods based on the individual material’s gradation properties. Using the RCA and #57 Rock gradations previously presented, the amount of fine aggregates were for each material was calculated. These percentages were then used to calculate the ratio, or percentage, that RCA contributes when compared to the #57. Eqn. 6.1 shows how this ratio was calculated for the percent passing for the two coarse aggregates. This ratio was then used to multiple by the percentage of the overall mix that is made up of RCA to determine the amount of fines the RCA contributed to the mix. RCA contributed to 50% of the fine aggregates required in high percentage RCA mix designs. For the fly ash replacement, the nature of both fly ash and cement allow for straight percentage replacement of the overall cementious material, although there are implications on rate of strength gain for high percentages of replacement. To reduce the potential loss in the rate of strength gain, supplemental calcium hydroxide was added when the substitution rate reached >50%. Hydrated lime was introduced at a straight percentage replacement of the cementious material in order to provide sufficient calcium hydroxide.

$$Percent\ of\ Fines\ in\ RCA = \frac{\left((\%\ Passing\ \#4\ Sieve\ of\ RCA) - (\%\ Passing\ \#4\ Sieve\ of\ \#57)\right)}{\left(100 - (\%\ Passing\ \#4\ Sieve\ of\ RCA) - (\%\ Passing\ \#4\ Sieve\ of\ \#57)\right)}$$

Eqn. 6.1 – Percent of Fines in RCA

6.3. Concrete Mix Designs Properties

As briefly mentioned before, the recycled materials were incorporated into the overall mix design as a percentage of the total overall materials. The first mix design studied was an ODOT Class A mix that met all the specification requirements and included no RCA or fly ash. This mix served as a baseline and allowed for the characterization of the fresh and hardened properties of concrete without the introduction of recycled material. Then the introduction of RCA was implemented at a rate of 25% replacement of the natural coarse aggregate till a complete 100% replacement was reached. All the fresh and hardened properties were recorded and categorized each of the 0%, 25%, 50%, 75%, and 100% replacement mixes. The fly ash replacement was introduced after the introduction of all the RCA mixes. No laboratory mixes were performed with introduction of fly ash and only natural material, but always included RCA. The fly ash was implemented at a similar 25% replacement rate as the RCA, for a 25%, 50%, and 75%. Fly ash, however, was limited at a maximum 75% replacement of the cement due to the inherent limitations of fly ash. In addition to the fly ash replacement, on two mix designs, the 75% and eventually the 65% cement replacement, hydrated lime was introduced into the cementitious material. The lime replaced 10% of the fly ash replacement. This was introduced to supplement and increase the initial curing process of that was limited due to the High-Volumes of fly ash. In addition to the recycled material replacement, the water cement ratio varied due to the properties of both RCA and fly ash. The ODOT Specifications provide the allowable range of 0.44 - 0.48 (Table 3.3). When starting with the RCA only batches, the highest allowable w/c ratio of 0.48 was used. This step was done to ensure that the mixes would adequately perform to the ODOT Specification requirements, particularly slump. This factor did change with the introduction of fly ash. With the increased amounts of fly ash, the w/c ratio was decreased .44 to account for the increased flowable properties of the fly ash mixes. Table 6.1 outlines all the common ingredients used in each of the High-Volume Recycled Concrete mixes. All of these properties conform to the requirements previously discussed in the ODOT Specifications. The minimum amount of cementitious material was set at 517 lb/yd^3, with the cement amount varying depending on the fly ash content. Again, the w/cm will vary depending on the total amount of fly ash, from 0.44 - 0.48, and the air entraining admixture was selected as 0.67 oz/cwt. Table 6.1 outlines the requirements for the High-Volume Recycled mix design. All the other components of the mix design remained the same expect for the varying amounts of fly ash, RCA, and w/cm. This allowed an investigation into the response of the optimization methods.

Table 6.1 - High-Volume Recycled Concrete Properties

Cementitious Material, lb/yd^3	517
w/cm	0.44-0.48
Fly Ash	0% - 75%
Amount of RCA Replacement	0% - 100%
Master Builders AE-90 (air entrainer), oz/cwt	0.67

6.4. Testing Methods and Fresh Concrete Properties

For the High-Volume Recycled mix designs, a total of 2.5 ft^3 was batched for all the mixes with varying amounts of RCA and fly ash. This provided enough concrete to perform the slump test and prepare cylinders for the testing frequency previously discussed. For compression strengths test, 4”x8” cylinders were used and pads were used to remove any stress concentrations. For Figure 6.1 and Figure 6.2 outline the slump tests results for each of the trial mix designs. Figure 6.1 compares the effect of varying the amount of RCA. Figure 6.2 compares the effect of varying the amount of fly ash for mixes incorporating 100% RCA replacement. The trends in each figure are consistent with what was expected. First, looking at the RCA replacement only, Figure 6.1, with an increase in RCA, there is a decrease in slump. This behavior is most likely related to the increased angularity of the RCA aggregate, particularly the fine aggregates, and the increased absorption rates of RCA. The second trend is that the slump increases with additional fly ash due to the spherical nature of fly ash particles versus cement particles (Figure 6.2). Fly ash tends to have smaller spherical particle compared to cement, which in general has larger, more angular particles due to the cement grinding process.

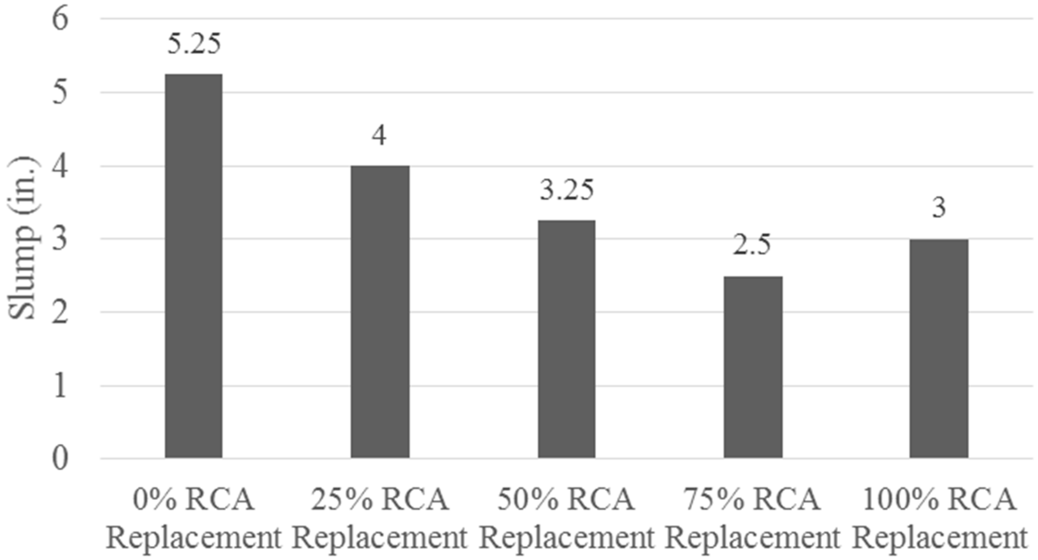


Figure 6.1 - Slump Results for RCA Replacement

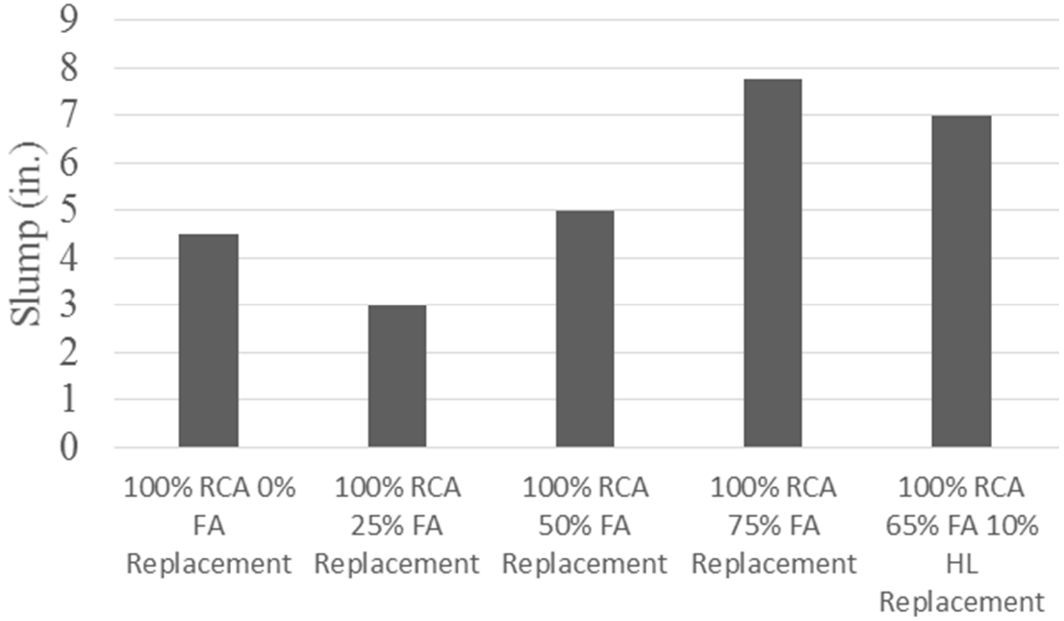


Figure 6.2 - Slump Results for RCA and FA Replacement

6.5. Hardened Concrete Results

With the compressive strengths recorded at 1, 3, 7, 14, and 28 days, the results were compared to the ODOT specified 3,000 psi minimum value. Figure 6.3 is a plot of the compressive strength results for each mix at each of the testing times. With the introduction of RCA, there are consistent results with all the values falling within an approximate 1,000 psi window for all testing days. Within this window, the order of highest strength does vary. However, from day 3 to day 28, the highest performing mix was the 50% RCA replacement. The 25% RCA replacement mix had the highest strength at 1 day, but did not gain as much strength as the other mixes and was the weakest at 14 and 28 days. The 0%, 75%, and 100% RCA replacement mixes had the lowest strengths for 1, 3, and 7 days but were in the middle of the 25% and 50% RCA replacement mixes at 14 and 28 days. All mixes surpassed the 3,000 psi requirement in the ODOT Specifications as shown in Table 6.2.

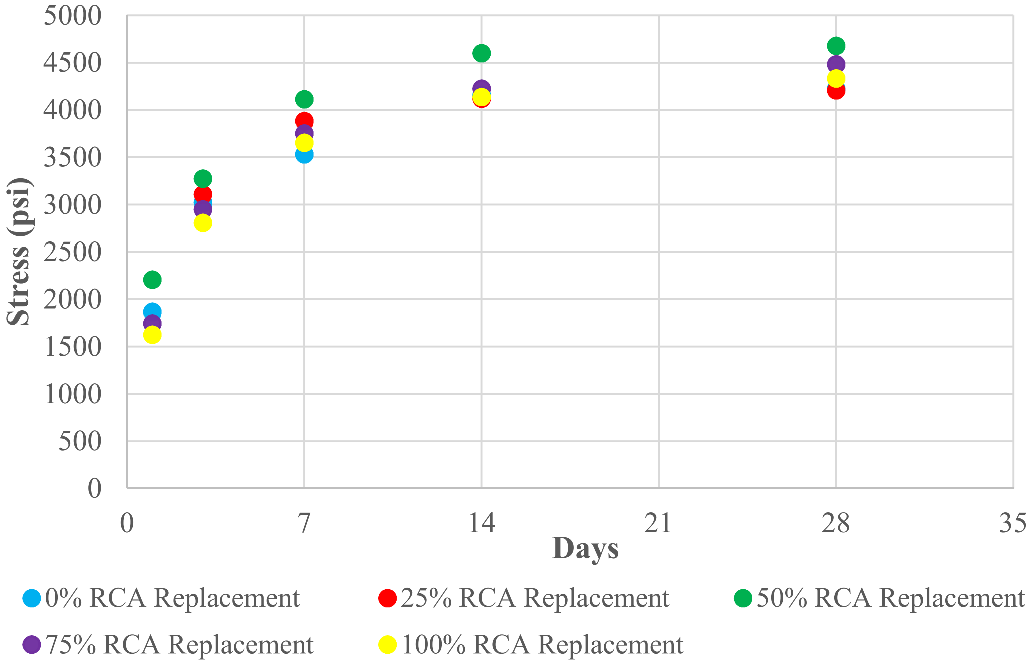


Figure 6.3 - RCA Replacement Compressive Strengths

Table 6.2 – Average RCA Replacement Compressive Strengths

	0% RCA Replacement	
	Load (lb)	Stress (psi)
1 Day	23500	1870
3 Day	37990	3020
7 Day	44370	3530
14 Day	52250	4160
28 Day	53050	4220
	25% RCA Replacement	
	Load (lb)	Stress (psi)
1 Day	25160	2000
3 Day	39050	3110
7 Day	48830	3890
14 Day	51800	4120
28 Day	52840	4210
	50% RCA Replacement	
	Load (lb)	Stress (psi)
1 Day	27740	2210
3 Day	41150	3270
7 Day	51720	4120
14 Day	57810	4600
28 Day	58790	4680
	75% RCA Replacement	
	Load (lb)	Stress (psi)
1 Day	21900	1740
3 Day	37080	2950
7 Day	47140	3750
14 Day	53090	4220
28 Day	56320	4480

Table 6.2 Cont. – Average RCA Replacement Compressive Strengths

	100% RCA Replacement	
	Load (lb)	Stress (psi)
1 Day	20800	1630
3 Day	35290	2810
7 Day	45930	3660
14 Day	51990	4140
28 Day	54450	4330

Implementing fly ash led to a larger variance in the compressive strength test results, as shown in Figure 6.4. These results show a range of approximately 2,500 psi between the lowest and highest values. The 100% RCA and 75% FA results were extremely low and not worth plotting. To maximize the recycled material hydrated lime (HL) was introduced for one mix of 65% replacement. This mix to reached the baseline 3,000 psi while maximizing the amount of recycled material was the 100% RCA 65% FA 10% HL with final strengths ranging between 2,900 and 3,810psi. The other three mixes were within an approximately 750 psi window for 7, 14, and 28 days, with all finishing significantly above 3,000 psi. Table 6.3 shows the 28 day strengths for each of the RCA and fly ash replacement mixes.

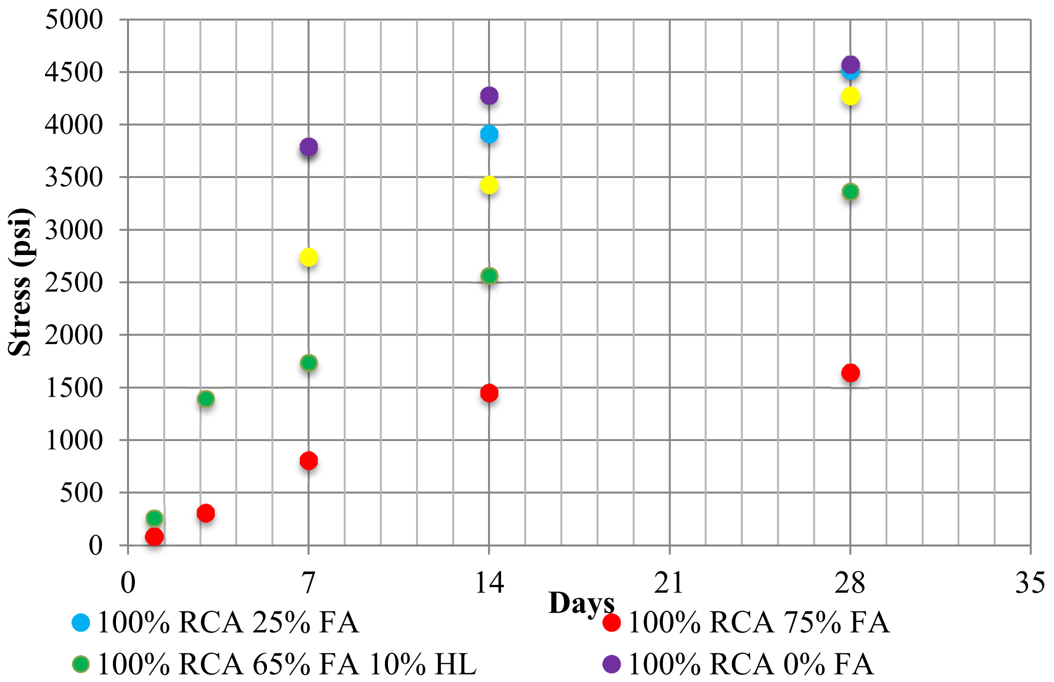


Figure 6.4 - Average RCA and FA Replacement Compressive Strength Results

Table 6.3 - Average RCA and FA Replacement Compressive Strengths

	100% RCA 25% FA	
	Load (lb)	Stress (psi)
7 Day	47660	3790
14 Day	49210	3920
28 Day	56730	4510
	100% RCA 50% FA	
	Load (lb)	Stress (psi)
7 Day	39190	3120
14 Day	49640	3950
28 Day	55720	4430
	100% RCA 75% FA	
	Load (lb)	Stress (psi)
1 Day	1050	80
3 Day	3830	300
7 Day	10160	810
14 Day	18220	1450
28 Day	20610	1640

Table 6.3 Cont. - Average RCA and FA Replacement Compressive Strengths

	100% RCA 65% FA 10% HL	
	Load (lb)	Stress (psi)
1 Day	3290	260
3 Day	15530	1400
7 Day	21830	1740
14 Day	32230	2560
28 Day	42280	3360

6.6. Laboratory Discussion

Beginning with the batching of the High-Volume Recycled concrete, there are unique characteristics that play a role in the behavior of the mix. The moisture meter, which was used to determine the moisture content for the aggregates prior to batching proved insufficient when used on the RCA. This is primarily due to high variability in fine and coarse particles among different samples of RCA. The absorption rates of RCA are 4 times higher for the fine aggregates compared to the River Sand and 6 times higher for the coarse particles when compared to the #57 Rock. Furthermore, the absorption rates required the highest w/cm allowed by the ODOT Specifications. This also effected the performance and properties of the fresh concrete mix.

After batching, the fresh concrete and workability trends were correlated with the slump tests. As the RCA replacement increased, the "harshness" increased and the workability decreased. This was due to the increased angularity of the RCA aggregates compared to the natural material, particularly the fine aggregate. Higher variability in the RCA aggregates also increased the variability and performance of the mix designs. This resulted in decreased slumps and increased the difficulty in preparing consistent concrete mixes.

With the introduction of fly ash, the slump steadily increased as the fly ash content increased. This was also what was expected because of the natural structure of the fly ash particles compared to the cement particles. Fly ash is a smaller, spherical particle that increases the flowability whereas cement is in general a larger, more angular particle.

At 28 days the compressive strength for the RCA replacement increased to the 50% RCA replacement level, then proceeded to decrease for the 75% and 100% replacement. The decrease was not significant from the 50% replacement to the 75% and 100% replacements, 4330 psi to 4680 psi, because the overall compressive strengths for the 75% and 100% were still higher than the 25% RCA replacement mix at 4210 psi. Each of the mixes performed higher than the ODOT specifications, with a 28 day compressive strength range of 4,200 psi to 4,680 psi. One reason for the 50% RCA replacement having a higher compressive strength is the 50/50 split resulted in a self-imposed aggregate optimization. The smaller maximum nominal size of the #57 Rock limited the void space of the larger maximum nominal size RCA. With the introduction of fly ash, the compressive strength was significantly affected with the fly ash replacement beyond 50%. At the lower 0%, 25%, and 50% fly ash replacements, the compressive strengths were approximately 4,500 psi, which was similar to the previous RCA replacement mixes. Once the fly ash replacement was above 50%, the compressive strength decreased significantly. The 75% fly ash did not reach any significant strength due to the slowed reactivity of the fly ash from the lack of calcium hydroxide from the cement. To help with the lack of calcium hydroxide, hydrated lime was introduced into the mix, with a 10% replacement of the cementious material. For the 65% fly ash replacement and 10% hydrated lime mix, the compressive strength reached the ODOT required 3,000 psi.

7. Field Implementation

7.1. Introduction

To provide additional information on the Cement-Limiting and High-Volume Recycled mix designs, multiple full scale pavement sections were constructed. These sections were to go beyond the information collected from the laboratory testing on the previously discussed mix designs. Laboratory mixes provide insight into controlled small batch responses, but full scale sections provide insight into real life applications. The field implementation provides additional information into the fresh properties from a full slab preparation as well as additional laboratory testing of multiple cubic yard mixes. For these field implementations, an existing road surface and base were removed and then prepared for the experimental concrete test sections.

7.2. Set Up

These full scale test sections were constructed at the Donald J Fears Structural Engineering Laboratory on the University of Oklahoma campus. An existing roadway and base on the west side of the facility was removed for placement of these pavement sections, as shown in Figure 7.1. A total of eight sections were constructed, with one section constructed at a time. Each section was 11 ft. wide, by 16 ft. long, and 8 in. thick placed over an 8 in. thick base of crushed rock aggregate, as shown in Figure 7.2. Four panels were constructed using an ODOT Class A mix, two panels were constructed with the Cement-Limiting Concrete mix, and two panels were used for the construction of the High-Volume Recycled Concrete mix. Multiple panels were constructed for each mix to provide additional results and understanding.

Figure 7.1 - Existing Asphalt Pavement

Beginning at the end of the radii of the existing asphalt roadway, two Dolese ODOT Class A concrete panels were placed. These panels were not instrumented. The rest of the panels were constructed moving north, constructing the western panel first then constructing the eastern panel. The eastern panels were the only panels that were tested and instrumented. Discussion on the testing instrumentation is covered in this section. Again, Dolese ODOT Class A was used in the next two panels. Then the experimental mix designs were used beginning with the Cement-Limiting mix designs for two panels and ending with two High-Volume Recycled Material mix designs. Figure 7.2 shows the overall set up of the field implementation.

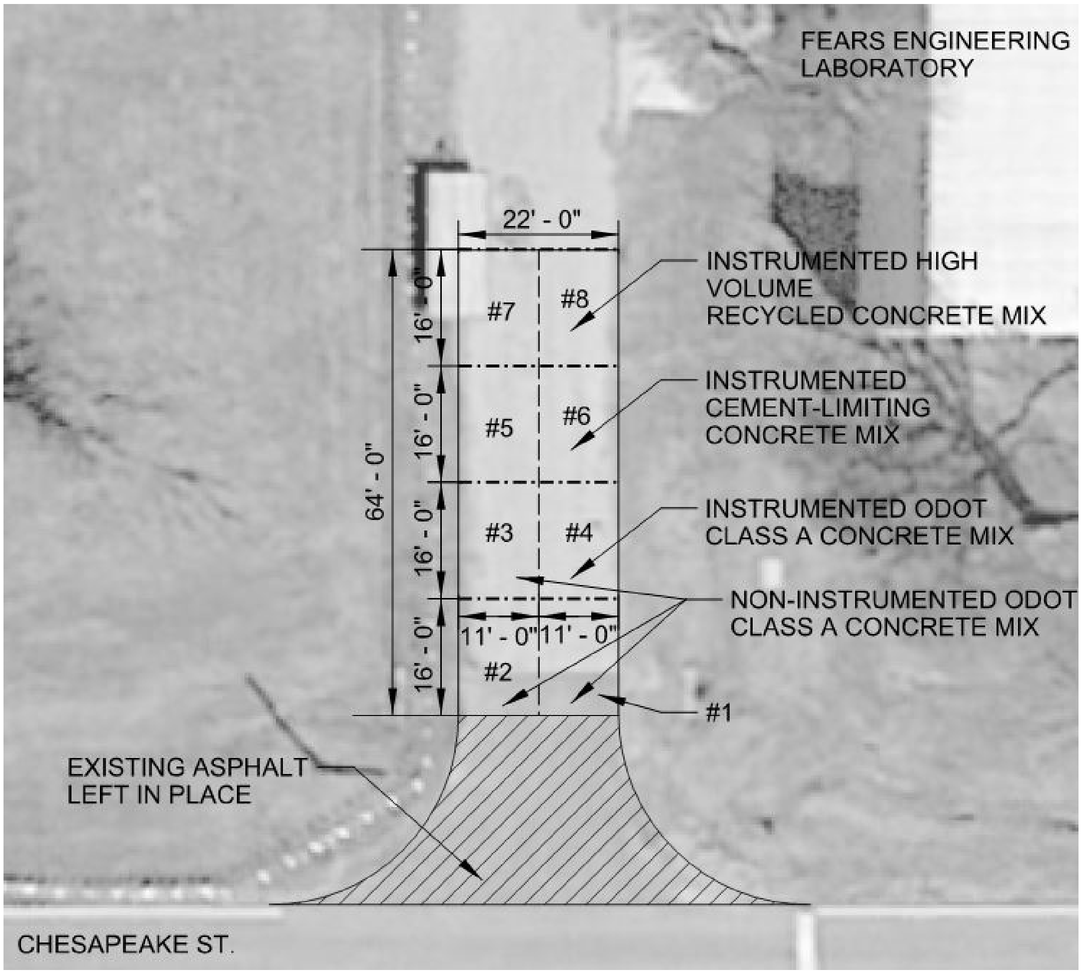


Figure 7.2 - Field Implementation set up Overview

Each panel incorporated both longitudinal and transverse dowel bars in accordance with standard ODOT highway pavement details. For the longitudinal joints 2 ft.-6 in.-long deformed #4 bars were placed at 2 ft.-6 in. center-on-center beginning 6 in. from the previous panel. Using these dimensions, a total of 7 bars were placed in each panel. On the transverse joints, 1 in. diameter smooth dowel bars were placed at 1 ft. center-on-center beginning at 6 in. from the longitudinal joint. Figure 7.3 shows the placements of the longitudinal and transverse dowel bars.

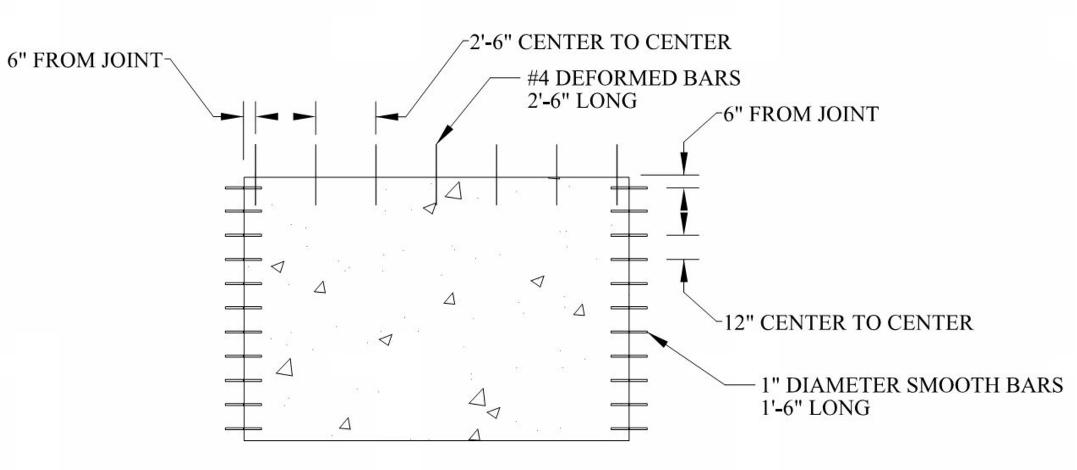


Figure 7.3 - Dowel Bar Placement in all Concrete Panels

7.3. Instrumentation Plan

To gain additional knowledge about the experimental mix designs and how the results compare to an ODOT standard mix, Geokon 4200 series vibrating wire strain gages (VWSG) were embedded into the concrete panels at multiple locations and varying depths. The gages were placed at three different locations, two were directly under the wheel load of a traveling vehicle and one was in the direct center of the panel. Each set was called a tree and given a number between 1 and 3, increasing as one moves north on the panel. It was assumed that the average vehicle wheel loading was 6 ft. on center, causing the two trees of VWSG in the wheel path to be placed 2 ft.-6 in. from the longitudinal joint and 5 ft. from the transverse joints. The center tree of VWSG were placed directly in the center at 5 ft.-6 in. from the longitudinal joint or outer edge of the panel and 8 ft. from the transverse joints. Figure 7.4 describes the placement of each tree of VWSG and their placement in the concrete panel.

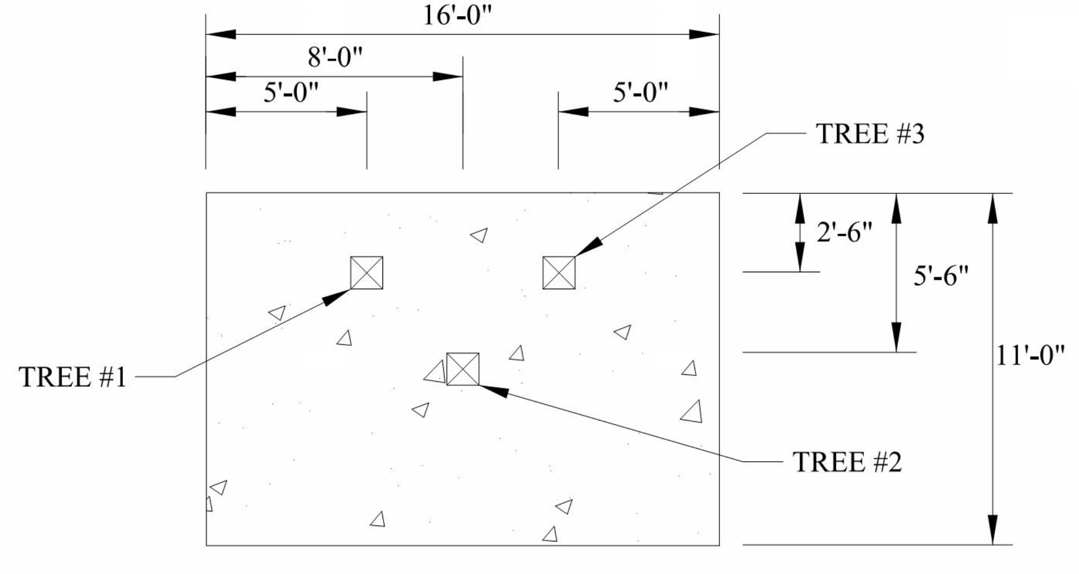


Figure 7.4 - VWSG Tree Placement in Concrete Panel

A total of 8 VWSG were placed in each test panel spread throughout the three locations. The frame of each tree was constructed of four #2 bars to hold the gages at the appropriate depths, a frame of #4 bars to provide location of the ground surface and additional strength in case of contact during construction, and #4 bars placed vertical to tie all the frames together and provide vertical support in case of contact during construction.

Tree #1 contained four VWSG placed at 2 in., 4 in., and 6 in. from the ground surface. One VWSG was placed at each depth parallel with the direction of traffic and one VWSG was placed at 2 in" perpendicular to traffic. Figure 7.5 shows the dimensions of Tree #1 with the locations of each VWSG if one was looking at the tree in the direction of traffic. Figure 7.6 shows the dimensions of Tree #1 with the locations of each VWSG if one was looking at the tree perpendicular to traffic. Trees #2 and #3 contained one VWSG placed at 2 in. and at 6 in. and were placed in the direction of traffic. Figure 7.7 shows the dimensions of Trees #2 and #3 with the locations of each VWSG if one was looking at the tree in the direction of traffic. Figure 7.8 shows the dimensions of Trees #2 and #3 with the locations of each VWSG if one was looking at the tree perpendicular to traffic. Figure 7.9 shows a picture example of a Tree wired with the gages and that was placed in to panel. Figure 7.10 a picture of the three Trees before they were covered in concrete.

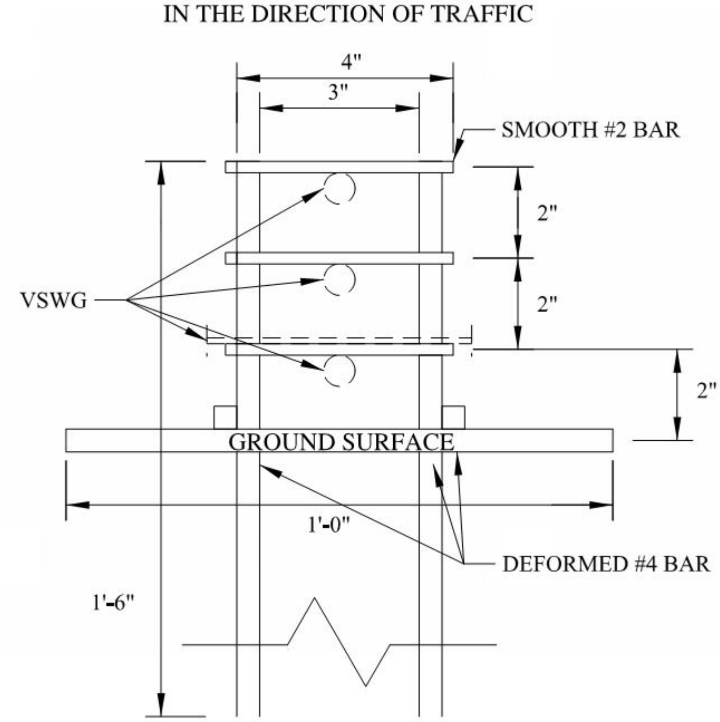


Figure 7.5 - Dimensions of VWSG Tree #1 in the Direction of Traffic

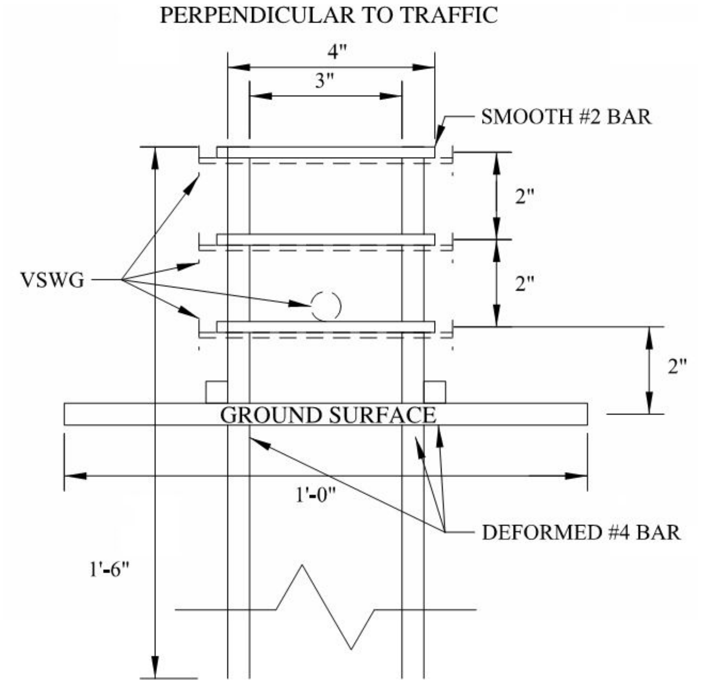


Figure 7.6 - Dimensions of VWSG Tree #1 Perpendicular to Traffic

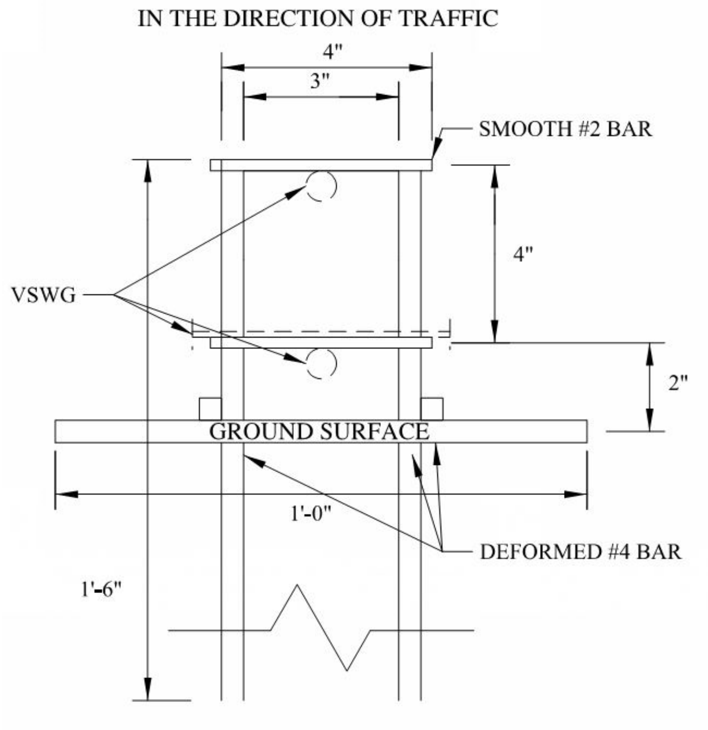


Figure 7.7 - Dimensions of VWSG Tree #2 and #3 in the Direction of Traffic

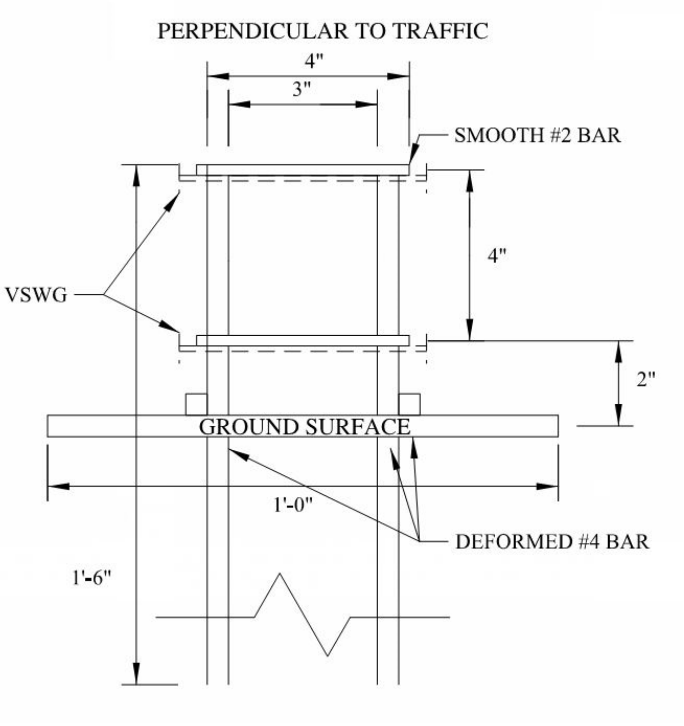


Figure 7.8 - Dimensions of Tree#2 and #3 Perpendicular to Traffic

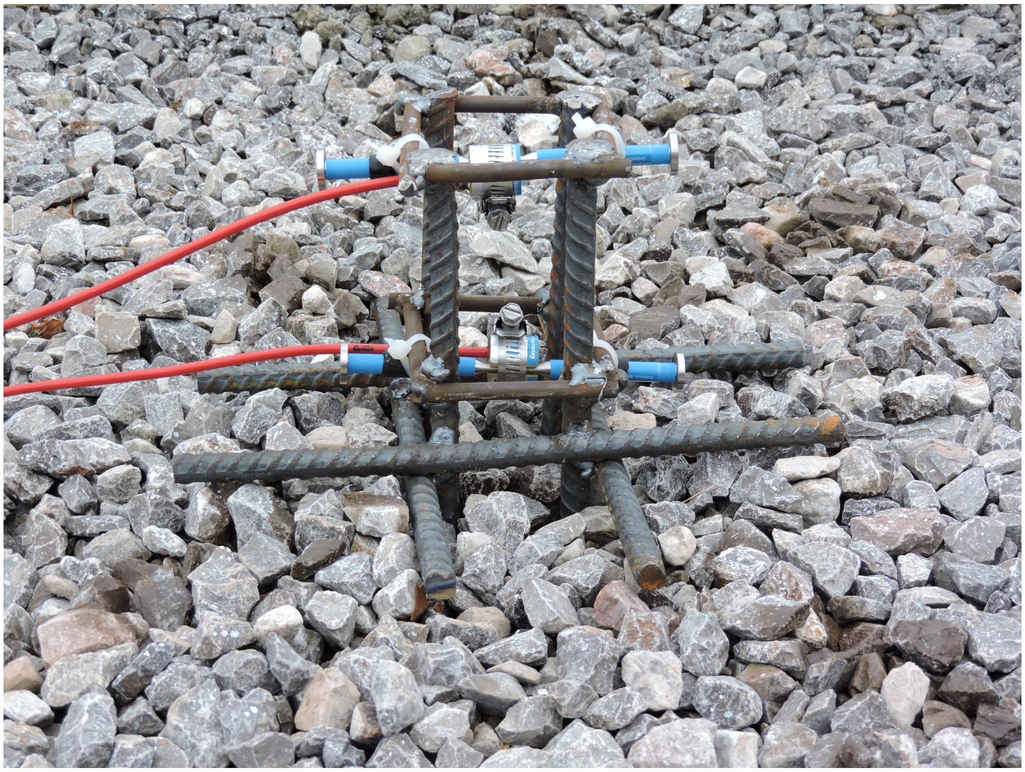


Figure 7.9 - Example and Placement of VSWG Tree

Figure 7.10 - VSWG Tree placement of in an Instrumented Panel

7.4. Field Implementation Mix Designs

7.4.1. ODOT Class A Mix Design

Panels 1 through 4 were constructed using Dolese's ODOT Class A mix. This mix design is frequently used by Dolese throughout the state on almost all standard concrete roadways. The cementitious material was set at the lowest allowed by ODOT Specification at 517 lb/yd^3, with 20% of the cementitious material consisting of fly ash. The water to cement ratio was set at 0.42. The aggregate split was 44% fine aggregate and 56% coarse aggregate. The mix design also incorporated 0.76 oz/cwt of air entraining admixture and 3 oz./cwt of water reducer. Table 7.1 outlines all the components of the ODOT Class A mix design, and Table 7.2 contains the actual mix design weights for 1 yd^3. This mix design was used as the baseline to measure the properties and characteristics of the Cement-Limiting and High-Volume Recycled Material experimental mix designs .

Table 7.1 - Field Implementation ODOT Class A Mix Design

Cementitious Amount, lb/yd^3	517
w/cm	0.42
Fly Ash	20%
Amount of Fine Aggregate	44%
Amount of #57	56%
Master Builders AE-90, oz/cwt	0.76
Glenium 7500, oz/cwt	3.0

Table 7.2 - Class A Mix Design Weights for 1 yd^3

Cement, lb	414
Fly Ash, lb	103
w/cm	0.42
River Sand, lb	1400
3/8” Chip, lb	0
#57 Rock, lb	1583
Air Entrainer, mL	116
Water Reducer, mL	459

7.4.2. Cement-Limiting Concrete Mix Design

The Cement-Limiting concrete mix design was incorporated into Panels 5 and 6. The laboratory results from the three different aggregate optimization methods were used as a starting point for developing the field implemented Cement-Limited percent breakdown. For the percent of fine aggregate, due to the significant strength loss when the fine aggregate percent was greater than 50%, the fine aggregate was limited to 47.7%. The fine aggregate percentage still had to remain as close to 50% as possible to keep the coarseness/workability inside Area II. This lead to extending beyond the upper bound on the Power 45 chart. The percent of 3/8 in. Chip in the mix design was 20%. This percentage is lower than any of the values from the individual aggregate optimization methods. The lower percentage was needed to keep the middle of the Power 45 curve inbetween the bounds, and limit the first peak on the Percent Retained Chart. The percent of #57 Rock in the mix design was 32.3%. This was towards the upper end of the percent #57 from the previous three aggregate optimization methods. The higher percentage of #57 Rock will increase the compressive strength following the trends from previous data. This percentage keeps a more even split on the bounds of the Percent Retained Chart and Power 45 Chart. Table 7.3 shows the percent aggregate breakdown for the combined Cement-Limiting mix design.

Table 7.3 – Field Implementation Cement-Limiting Concrete Aggregate Percentage Breakdown

Amount of Fine Aggregate (by volume)	47.7%
Amount of #67 (by volume)	32.3%
Amount of 3/8" Chip (by volume)	20.0%

For the overall mix design, the cementitious material amount was 423 lb./yd^3 following the previous mix designs, which is one sack less than the Class A, and a half sack less than the ODOT Specifications allow for aggregate optimization. The cementious material contained 20% fly ash. The w/cm material ratio was set at 0.48, with 0.67 oz./cwt air entraining admixture and 0 oz./cwt of water reducer. Table 7.4 shows the breakdown of the Cement-Limiting mix design used in the field implementation, and Table 7.5 contains the actual mix design weights for 1 yd^3. The performance of this mix design will be compared to the Class A pavement sections, but also the laboratory design on the three separate Cement-Limiting mixes.

Table 7.4 - Field Implementation Cement-Limiting Concrete Mix Design

Cementitious Amount, lb/yd^3	423
w/cm	0.48
Fly Ash, %	20.0%
Master Builders AE-90, oz/cwt	0.67
Glenium 7500, oz/cwt	0

Table 7.5 - Cement-Limiting Mix Design Weights for 1 yd3

Cement, lb	338
Fly Ash, lb	85
w/cm	0.48
River Sand, lb	1563
3/8” Chip, lb	585
#57 Rock, lb	945
Air Entrainer, mL	83
Water Reducer, mL	0

7.4.3. High-Volume Recycled Material Mix Design

Panels 7 and 8 were constructed using the High-Volume Recycled Material mix design. The laboratory mix designs results previously discussed were used as a starting point for the field implementation mix design. The main concern with this mix design was having the concrete obtain an early enough strength. For this reason, the mix design with 100% RCA and 50% fly ash was selected over the higher fly ash replacement mixes. The cementitious material was set at 517 lb./yd^3, with 50% consisting of fly ash. The coarse and fine aggregate split was set at 60/40, respectively. The water to cement ratio was set at 0.44, with 0.67 oz./cwt of air entraining admixture and 0 oz./cwt of water reducer. Table 7.6 outlines the field implementation mix design for the High-Volume Recycled Material, and Table 7.7 contains the actual mix design weights for 1 yd^3. The performance of this mix design will be compared to the Class A pavement sections, but also the laboratory results of the other High-Volume Recycled Material mix designs.

Table 7.6 - Field Implementation Mix Design for the High-Volume Recycled Material

Cementitious Amount, lb/yd^3	517
w/cm	0.44
Amount of Fly Ash	50%
Amount of RCA Replacement	100%
Amount of Fine Aggregate	40%
Amount of Coarse Aggregate	60%
Master Builders AE-90, oz/cwt	0.67
Glenium 7500, oz/cwt	0.0

Table 7.7 - High-Volume Recycled Materials Mix Design Weights for 1 yd^3

Cement, lb	259
Fly Ash, lb	259
w/cm	0.44
River Sand, lb	602
RCA (fine), lb	431
RCA (coarse), lb	1374
Air Entrainer, mL	81
Water Reducer, mL	0

7.5. Field Implementation Construction

To begin construction of the full-scale pavement sections, the existing asphalt and subgrade was removed. A front-end loader tractor was used throughout the excavation, as shown in Figure 7.11 and Figure 7.12 . A 25 ft. wide x 300 ft. long section was excavated 16 in. deep. The existing material was an asphalt roadway approximately 5 in. thick and placed in multiple lifts. The base material was an 8 in. thick layer of very stiff clay on top of a silty sand layer. The middle clay layer appeared to be a select fill material brought to the site for placement under the existing asphalt roadway. The existing pavement was saw cut at the edge of the radii returns to create a flush edge to tie to the proposed pavement. After all the material was excavated, the aggregate base was placed, as shown in Figure 7.13 and Figure 7.14. Metro Materials delivered and placed the aggregate throughout the proposed site in approximately two 4 in. lifts. After the aggregate was placed, it was adjusted to create a 1.5% cross slope with a crown at the center. To determine the appropriate depths, a surveying level was used. Once the appropriate cross slopes were in place the aggregate was compacted using a five ton vibratory roller, as shown in Figure 7.15 and Figure 7.16. When the aggregate base was finished, the concrete panels were ready for construction. The panels were constructed one panel at a time beginning with the panel closest to the asphalt in the north bound lane, then moving to the south bound lane. The panels were constructed following the numerical system in Figure 7.2. The first steps to constructing each panel was the placement of the concrete (Figure 7.17). For this, the ready-mix concrete truck assisted with helping speed up placing the concrete. Shovels were used to appropriately spread and level the concrete to the required depth (Figure 7.18). While placing the concrete, a hand-held, vibratory compactor was used to consolidate the fresh concrete (Figure 7.19 and Figure 7.20). After the concrete was placed and consolidated, a 12 ft. wide motorized screed was used for final leveling and compaction (Figure 7.21). With the concrete in place and level, a standard 3 ft. wide bull float was used on the main area of the panel (Figure 7.22 and Figure 7.23) and hand floats were used on each of the edges (Figure 7.24). A v-notch edger was used on the center joint, while a typical rounded edge was used on the other three sides. Once a panel was floated and edged, a broom was used for the final finish (Figure 7.25) and a water sealant was applied to the surface (Figure 7.26). For each panel, plastic and blankets covered each placement for two days after construction (Figure 7.27). The plastic helped retain water in the concrete and the blankets helped retain the heat. Testing and monitoring on the panels began after the panel was finished construction.

Figure 7.11 - Excavation of the Existing Asphalt and Subgrade

Figure 7.12 - Excavation of the Existing Subgrade

Figure 7.13 - Field Implementation Aggregate Base Placement

Figure 7.14 - Field Implementation Aggregate Base Placement

Figure 7.15 - 5 Ton Vibratory Compactor

Figure 7.16 - Compaction of the Aggregate Base Using Vibratory Roller

Figure 7.17 - Concrete Placement using Ready-Mix Truck

Figure 7.18 - Spreading and Leveling the Concrete with Shovels

Figure 7.19 - Vibrating to Consolidate the Fresh Concrete

Figure 7.20 - Vibrating the Fresh Concrete

Figure 7.21 - Screeding for Final Leveling of the Concrete

Figure 7.22 - Floating the Center of Panel

Figure 7.23 - Using the Bull Float

Figure 7.24 - Hand Floating and Edging the Panel

Figure 7.25 - Broom Finish on the Concrete Panel

Figure 7.26 - Applying Sealant to the Finished Pavement

Figure 7.27 - Plastic and Blankets Covering the Finished Pavement

8. Field Implementation Testing Results

8.1. Introduction

For the field implementation tests, the fresh test results were one test while hardened results presented are the average of three tests performed. These results were compared among the similar mixes and to the base Class A mix. The three main panels investigated were 4 (Class A), 6 (Cement- Limiting), and 8 (High-Volume Recycled Materials). The other panels' results were used to provide base additional information to the curing response. Due to a miscommunication, the complete 28 day tests for Panel 4 were not conducted, but another mix following the same mix design was performed and all the 28 day tests were completed. This mix is referenced below as Class A (inside) mix. On Panel 7, the concrete arrived with #57 Rock and without any RCA replacement, but the mix did incorporate 50% fly ash replacement for cementitious materials.

8.2. ODOT Class A Field Implementation Results

8.2.1. Fresh Concrete Testing Results

During the construction of the pavement panels, multiple tests were performed on the concrete once it arrived. The four tests conducted were the slump test, unit weight, air content, and yield. The slump test ranged from 2 in. to 5 in. Each of these results were outside the allotted variance in the ODOT Specifications. The air content ranged from 4.80% to 7.6%. Three of the four tests meet the ODOT requirements. The unit weight and yield were performed just to compare to the experimental mix design. The unit weight varied in the range of

117 to 118 lb/ft^3. Table 8.1 outlines all the fresh concrete results from Panels 1-4 and the additional mix.

Table 8.1 - Class A Field Implementation Fresh Concrete Results

Panel 1			
Test	**Time**		
Slump	Time of Pour	4.25	in
Air Content	Time of Pour	4.80	%
Unit Weight	Time of Pour	117	lb/ft^3
Panel 2			
Test	**Time**		
Slump	Time of Pour	2.00	in
Air Content	Time of Pour	5.80	%
Unit Weight	Time of Pour	117	lb/ft^3
Panel 3			
Test	**Time**		
Slump	Time of Pour	5.00	in
Air Content	Time of Pour	7.60	%
Unit Weight	Time of Pour	115	lb/ft^3
Panel 4			
Test	**Time**		
Slump	Time of Pour	3.75	in
Air Content	Time of Pour	6.00	%
Unit Weight	Time of Pour	118	lb/ft^3
Inside			
Test	**Time**		
Slump	Time of Pour	5.00	in
Air Content	Time of Pour	NA	%
Unit Weight	Time of Pour	NA	lb/ft^3

8.2.2. Hardened Concrete Testing Results

Compressive strength results were taken for each of the panels constructed. For Panels 1-4, only 7, 14, and 28 day tests were conducted, and for the additional mix, 1 and 3 days were included as well. At 7 days, the strength ranged from 2,500 psi to 3,500 psi. At 14 days, some unexpected results were recorded. Two of the panels had an increasing trend at 14 days; however, two panels and the inside mix maintained the same strength or decreased. The overall range of compressive strengths at 14 days was 2,700 psi to 4,100 psi. At 28 days, all panels had increasing compressive strength compared to 14 days. The range of strengths at 28 days was between 3,250 psi and 4,500 psi. Figure 8.1 shows a plot of the compressive strength results of each of the Class A mixes at 7, 14, and 28 days .Table 8.2 shows that all of the compressive strengths for the mixes pass the 3,000 psi minimum set within the ODOT Specifications. Examining the results from the additional inside batch from 1 – 28 days, there is adequate early strength, with approximately 1,500 psi at 1 day and 2,750 psi at 3 days, which continues to increase to 28 were it produced the highest strengths of each of the mixes. Figure 8.2 shows the breakdown of the 1 day through 28 day compressive strength results for the Class A inside mix.

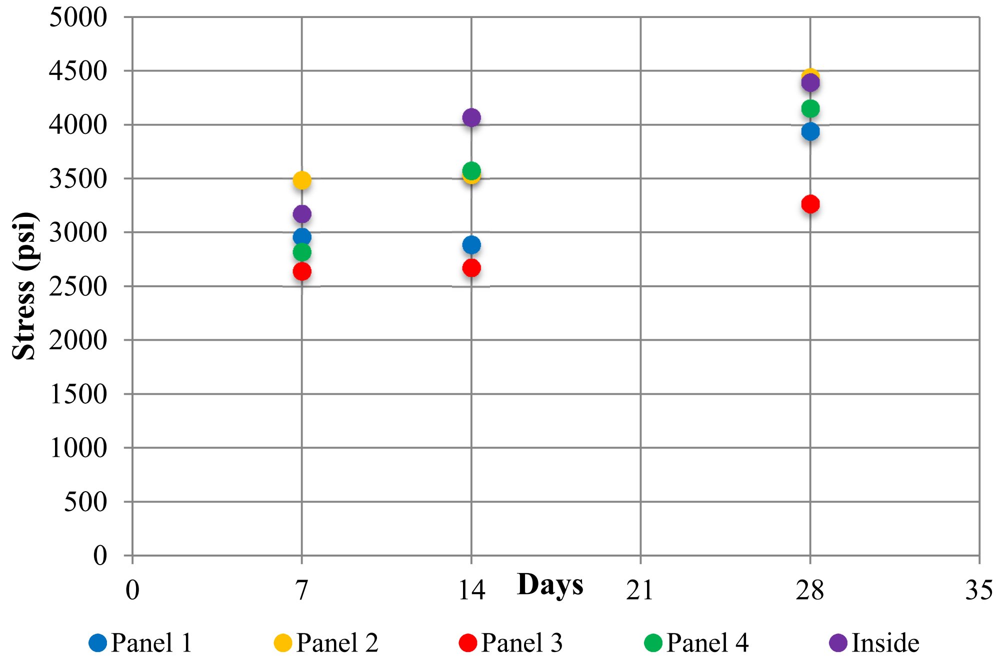


Figure 8.1 - Class A 7-28 Day Compressive Strength Results

Table 8.2 - Class A Field Implementation Compressive Strength Concrete Results

	Panel 1	
	Load (lb)	Stress (psi)
7 Day	37180	2960
14 Day	36260	2890
28 Day	49650	3940
	Panel 2	
	Load (lb)	Stress (psi)
7 Day	43830	3490
14 Day	44480	3540
28 Day	55810	4440
	Panel 3	
	Load (lb)	Stress (psi)
7 Day	33230	2640
14 Day	34000	2670
28 Day	41040	3270

Table 8.2 Cont. - Class A Field Implementation Compressive Strength Concrete Results

	Panel 4	
	Load (lb)	Stress (psi)
7 Day	35430	2820
14 Day	44910	3570
28 Day	52200	4150
	Inside	
	Load (lb)	Stress (psi)
1 Day	17840	1420
3 Day	33670	2680
7 Day	39910	3180
14 Day	51100	4070
28 Day	54910	4390

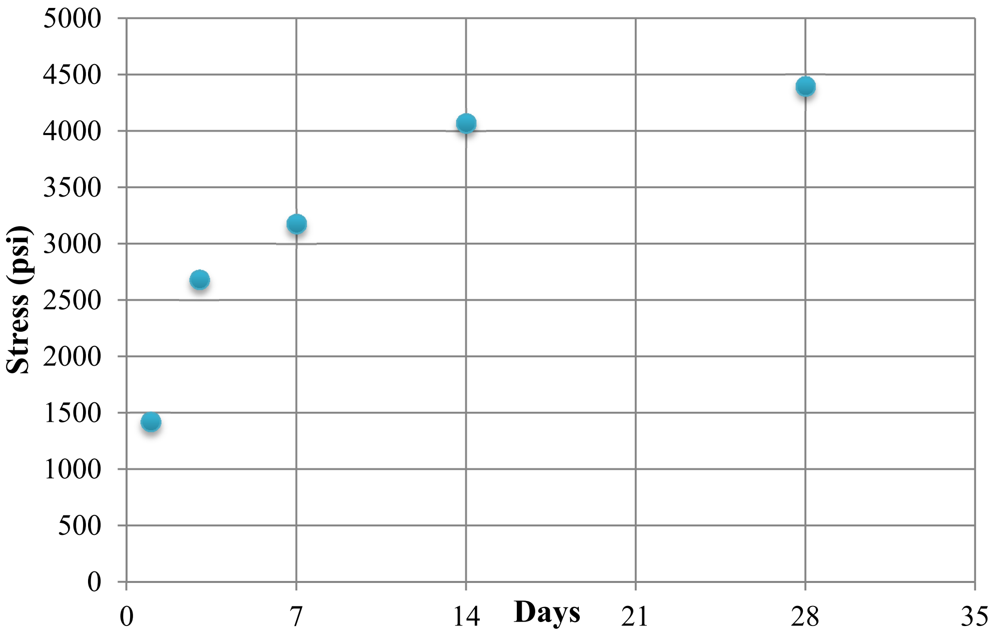


Figure 8.2 - Inside Mix Compressive Strength Results Over 1-28 Days

Multiple other tests were performed on the additional mix. The split tensile strength was 360 psi, modulus of rupture was 570 psi, and the modulus of elasticity was 3,500,000 psi. Table 8.3 shows all the data gained from the addition mix which was the base data compared to the experimental results.

Table 8.3 - Class A Additional Mix 28 Day Testing Results

Class A (Inside)			
Test	Time	Load (lb)	Stress (psi)
Compressive Strength	1 Day	17840	1420
	3 Day	33670	2680
	7 Day	39910	3180
	14 Day	51100	4070
	28 Day	54910	4390
Tensile Strength	28 Day	17930	360
Modulus of Rupture	28 Day	570 psi	
Modulus of Elasticity	28 Day	3,500,000 psi	

8.2.3. Vibrating Wire Strain Gage Results

In addition to the laboratory tests, as previously mentioned, VWSGs were placed directly in the concrete Panel 4 to determine the actual response in the field. Following the gage placement that was previously discussed, the strains were monitored every hour, over the 28 day test period to monitor the shrinkage and temperature response. One of the gages in Tree #1 did not work correctly and did not produce any data. The data was investigated in reference to their particular tree. The overall trend for all for the VWSGs was for the strains to decrease to -100 microstrain, then fluctuate between -50 and -150 microstrain due to the ambient daily temperature changes. Also, the VWSGs that are located 2 in. from the base layer have lower strain values when compared to the strains of the VWSGs placed higher in the concrete panel. For Trees #1 and #2 there is an initial concrete expansion over the first couple of hours during curing, however; this trend is not seen on Tree #2. Figure 8.3, Figure 8.4, and Figure 8.5 show the strain variations over the initial 28 day time period following casting.

With the pronounced diurnal strain changes, the weather data for the mean temperature (Weather Underground) for each day was plotted with the Tree #2 stain data. Tree #2 was selected as the base reference, due to the location in the middle of the panel and the two gage system. This shows that the overall strain data follows the temperature patterns. As the temperature decreases, the strains increase more negatively, and as the temperature increases, the strains increase positively. Figure 8.6 shows the relationship between the mean temperature and the strain effects on Tree #2

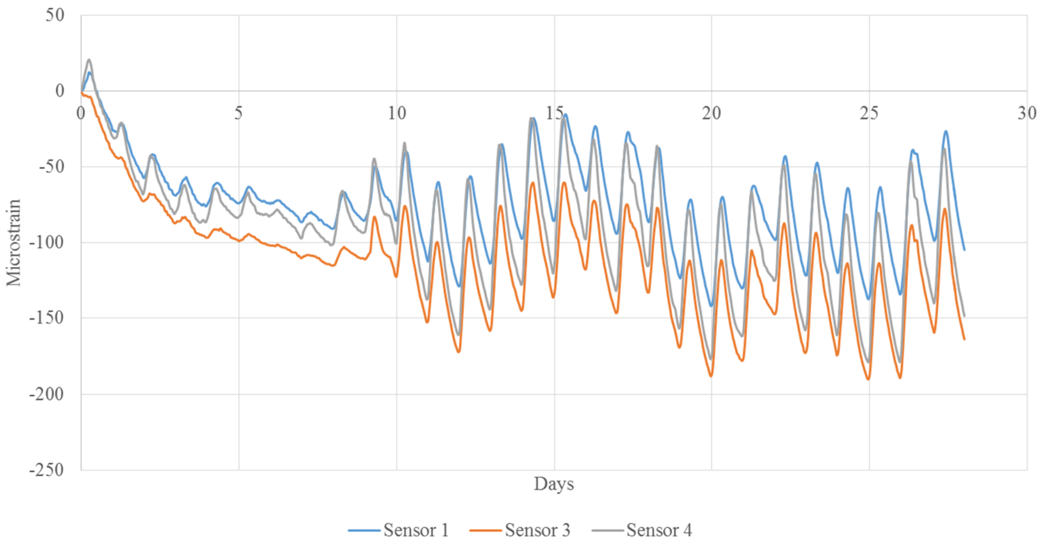


Figure 8.3 - Class A Tree #1 VWSG Data

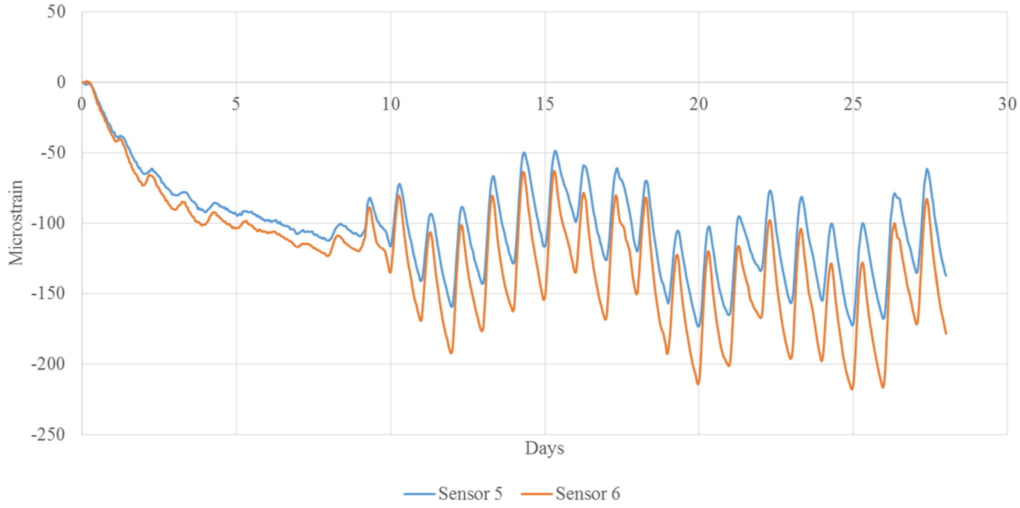


Figure 8.4 - Class A Tree #2 VWSG Data

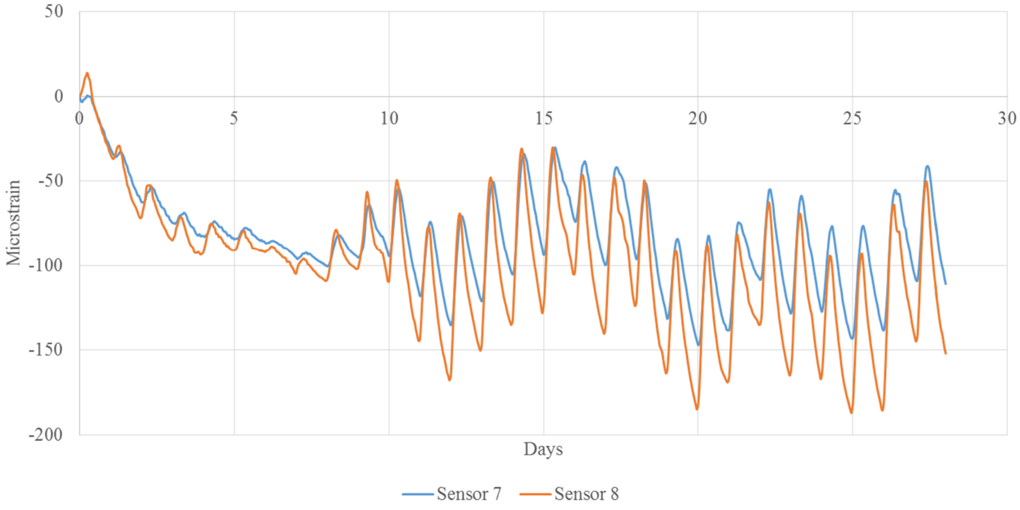


Figure 8.5 - Class A Tree #3 VWSG Data

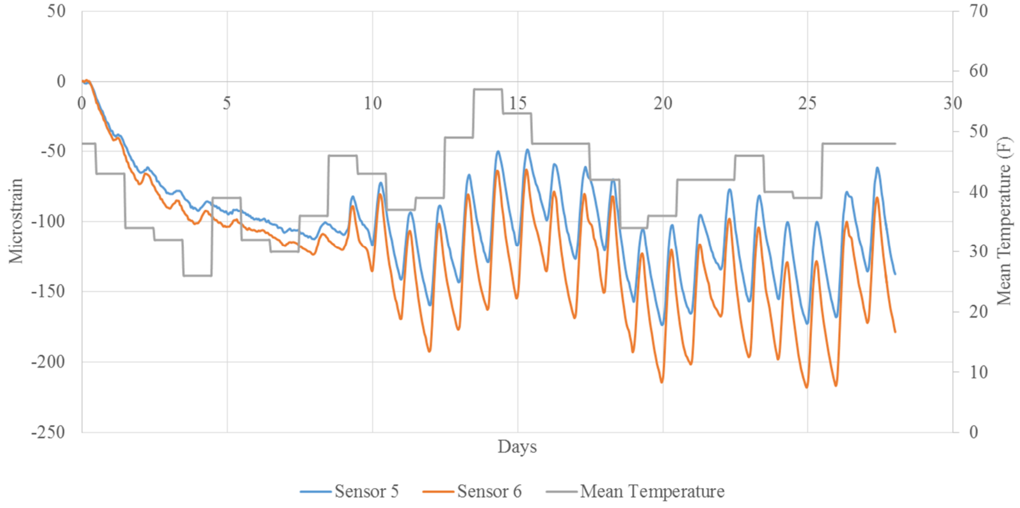


Figure 8.6 - Class A VWSG with Mean Ambient Temperatures

8.3. Cement-Limiting Field Implementation Results

8.3.1. Fresh Concrete Testing Results

Again, upon the arrival of the concrete, the fresh properties were tested and recorded. The slump was the only fresh concrete test preformed due to the limited concrete availability as a result of a less than anticipated yield for this mix design. A 2 in. and 1.75 in. slump were recorded for Panels 5 and 6, respectively. These results meet the bounds set by the ODOT Specifications. Table 8.4 shows the fresh concrete properties for the Cement-Limiting mix design.

Table 8.4 - Cement-Limiting Concrete Fresh Properties Results

	Slump (in.)
Panel 5	2.00
Panel 6	1.75

8.3.2. Hardened Concrete Testing Results

For Panel 5, compressive strengths were recorded at 7, 14, and 28 days and compared to Panel 6. For all three test dates, the results were within a 200 psi range. At 7 days, the range was 3,300 to 3,500, at 14 days the range was 4,000 to 4,100, and at 28 days the range was 4,500 to 4,700. Figure 8.3 shows these ranges of strengths over the 7, 14, and 28 day testing period. Table 8.5 shows the recorded data compressive strengths for Panels 5 and 6 over the 7, 14, and 28 day period.

Additional results were taken on Panel 6 at 1 and 3 days to compare to the Class A compressive strengths. Examining the entire range, 1-28 days, the strength results are very similar to the Class A strength results. Figure 8.4 shows the strength relationship between the Cement-Limiting and Class A concretes. The largest variation was at 7 days with an approximate 500 psi separation. At 1 day and 28 days the strengths are practically the same values.

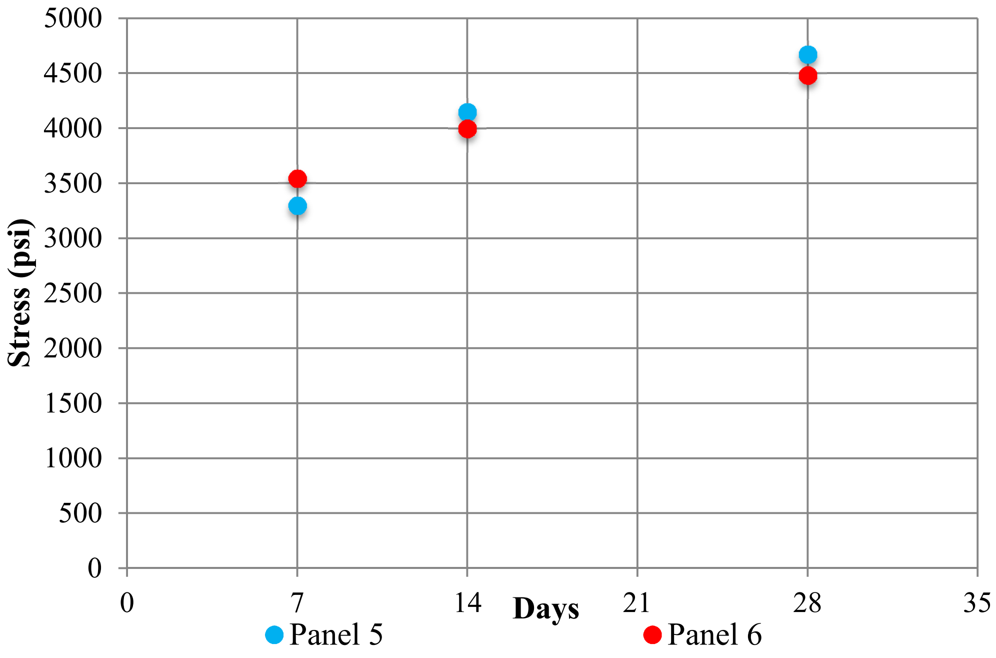


Figure 8.7 - Cement-Limiting Field Implementation Chart

Table 8.5 – Cement-Limiting Field Implementation 7-28 Day Compressive Strengths

	Panel 5	
	Load (lb)	Stress (psi)
7 Day	41410	3300
14 Day	52100	4150
28 Day	58690	4670
	Panel 6	
	Load (lb)	Stress (psi)
7 Day	44510	3540
14 Day	50220	4000
28 Day	56310	4480

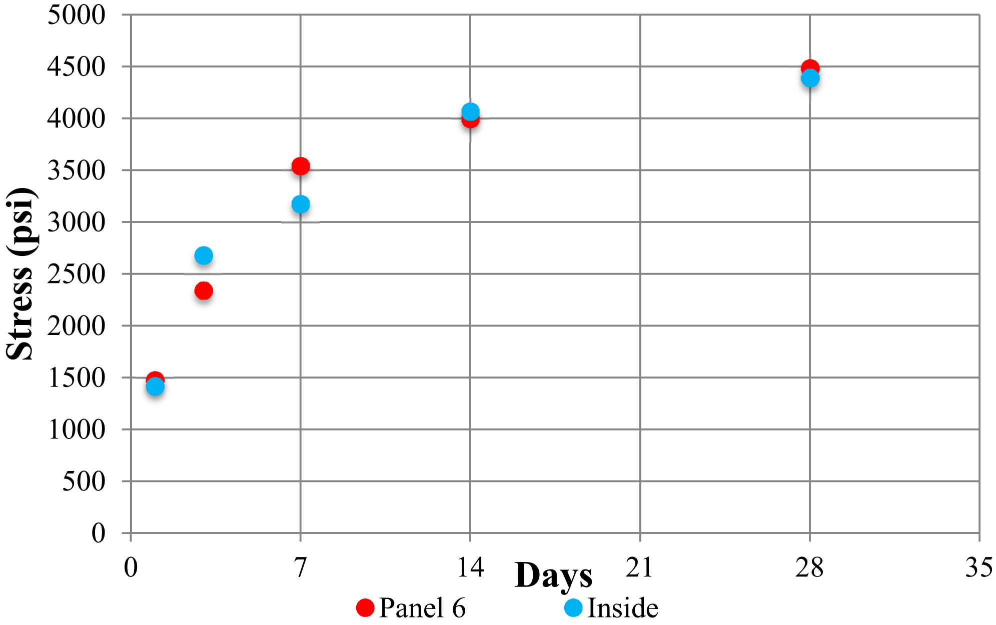


Figure 8.8 - Panel 6 vs. Inside Mix 1-28 Day Compressive Strength Results

The additional tests and compressive strength results performed on Panel 6 are recorded in Table 8.6. The split tensile strength was 390 psi, modulus of rupture was 620, and the modulus of elasticity was 4,320,000 psi. When compared to the results in Table 8.7, the Cement-Limiting mix design recorded higher results for each of the tests performed.

Table 8.6 - Cement-Limiting Additional Hardened Concrete Tests

Panel 6			
Test	Time	Load (lb)	Stress (psi)
Compressive Strength	1 Day	18560	1480
	3 Day	29420	2340
	7 Day	44510	3540
	14 Day	50220	4000
	28 Day	56310	4480
Tensile Strength	28 Day	19480	390
Modulus of Rupture	28 Day	620 psi	
Modulus of Elasticity	28 Day	4,320,000 psi	

Table 8.7 - Cement-Limiting and Class A Hardened Property Test Results

Test	**Time**	**Class A**	**Cement-Limiting**
Tensile Strength (psi)	28 Days	360	390
Modulus of Rupture (psi)	28 Days	570	620
Modulus of Elasticity (psi)	28 Days	3,500,000	4,320,000

8.3.3. Vibrating Wire Strain Gage Results

To investigate the full scale performance in the field, the VWSG imbedded in the concrete and monitored over the first 28 days to investigate the shrinkage and temperature responses throughout the panel. The overall trend of the strain data shows that an early positive strain for approximately 1 day, then a decrease to negative strains. The data does not approach a single negative trend value until 20 days, but instead fluctuate between -75 microstrain and -175 microstrain then levels off at about -100 microstrain. The data does not experience daily fluctuations until approximately day 6, which the strains vary by 50 microstrain per day. The VWSGs located 2 in. from the base layer have lower strain values when compared to the strains of the VWSGs placed higher in the concrete panel. Figure 8.9, Figure 8.10, and Figure 8.11 show the strain variations over the initial 28 day time period following casting for each of the Cement-Limiting trees.

Due to the diurnal fluctuations of the strain data, the weather data for the mean temperature (Weather Underground) for each day was plotted with the Tree #2 stain data. Tree #2 was selected as the base reference, due to the location in the middle of the panel and the two gage system. This shows that the overall strain data follows the temperature patterns. As the temperature decreases, the strains increase more negatively, and as the temperature increases, the strains increase positively. Figure 8.12 shows the relationship between the mean temperature and the strain effects on Tree #2.

Comparing the strain data in the Cement-Limiting to the Class A there are some differences, especially at the start. The initial ten days of curing shows the largest variation in strain data. The Cement-Limiting underwent expansion (positive strains) for approximately 1 day, whereas the Class A mix did not register any expansion. Over this initial 10 day period, the Class A recorded larger negative strains. The Cement-Limiting began to experience daily fluctuations earlier in the curing cycle. The data indicates that the daily fluctuations began around day 6 for the Cement-Limiting and day 10 for the Class A. The two mixes again diverge at 15 days when the Cement-Limiting drops to around -150 microstrain and the Class A increases to around -75 microstrain. Both mixes begin to approach -125 microstrain at 20 days and remain constant through the end of the 28 day monitoring period. Figure 8.13 shows the relationship of the strain results over the 28 day testing period for both the Cement-Limiting and Class A concrete mixes.

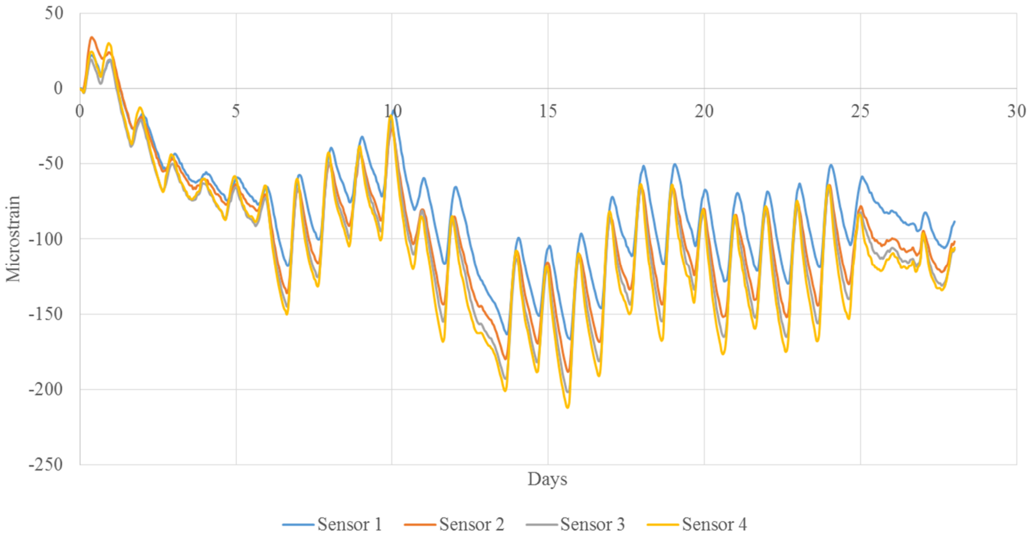


Figure 8.9 - Cement-Limiting Tree #1 Strain Data

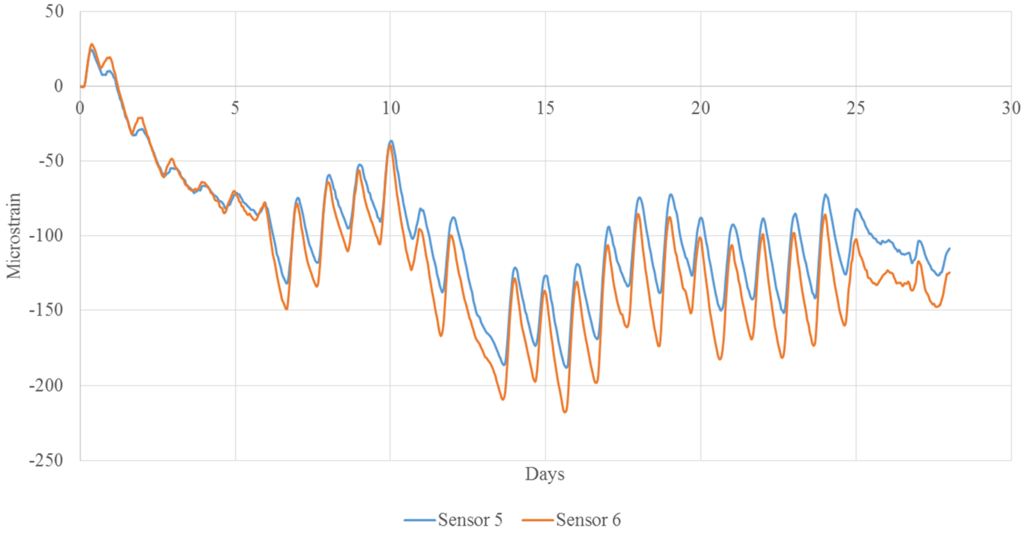


Figure 8.10 - Cement-Limiting Tree #2 Strain Data

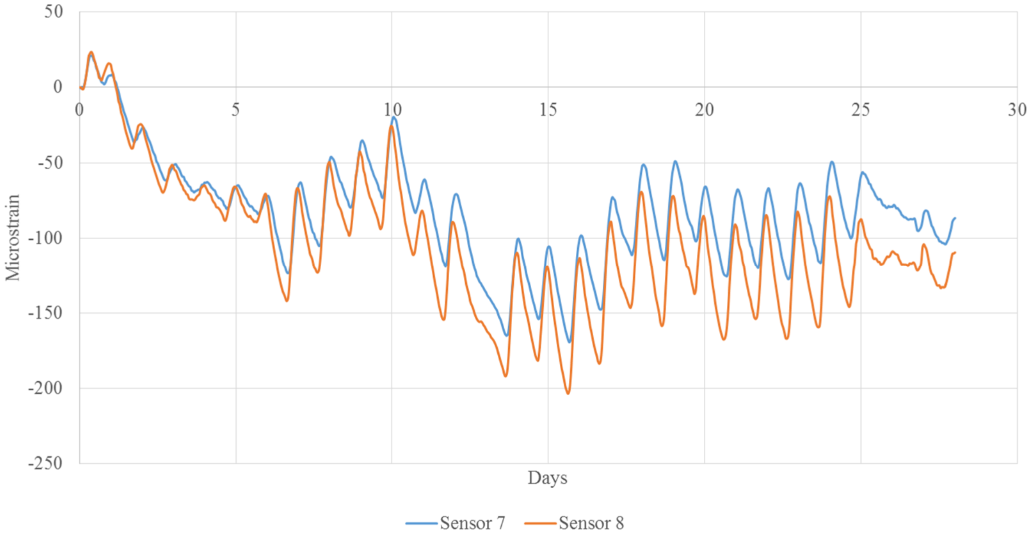


Figure 8.11 - Cement-Limiting Tree #3 Strain Data

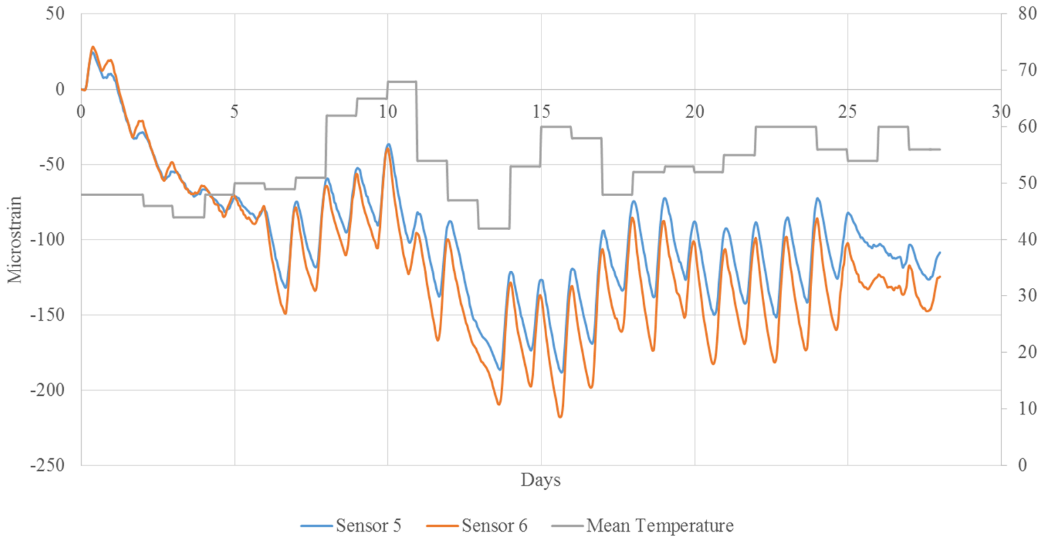


Figure 8.12 - Cement-Limiting VWSG with Mean Ambient Temperatures

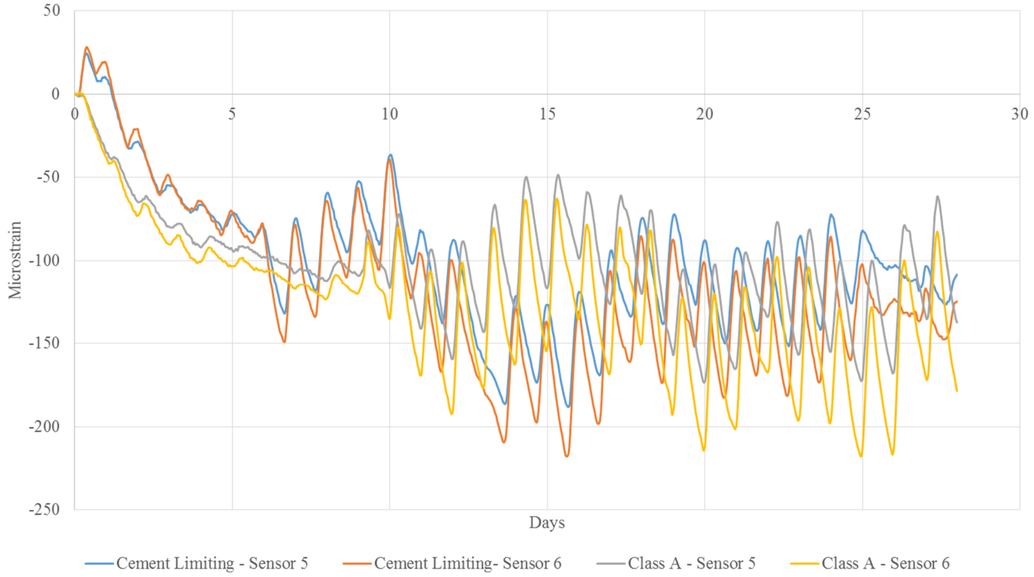


Figure 8.13 - Comparison of Cement-Limiting and Class A Strain Data

8.3.4. Field Implementation Discussion

The workability played a more important role when constructing the full scale test section, when compared to typical test specimens. Placing and finishing a 16 ft x 11 ft panel can produce problems if the mix has low workability. For both the Cement-Limiting mix on Panels 5 and 6, the slumps met the ODOT Specifications, which the Class A did not. However, the Cement-Limiting panels had significantly higher workability and cohesiveness. The w/cm was higher in the Cement-Limiting mix, but lowering the cement content and no addition of water reducer proved more applicable during these test sections. These comparisons show the limited capabilities and application of the slump test, especially when the mixes contain a variety of aggregates, w/cm, and admixtures.

Concrete's hardened properties and performance is vital to make sure the pavement withstands the demand of the traffic loading. The results of the Cement-Limiting mix proved to not only meet the requirement in the Specifications, but exceeded the required compressive strength by approximately 1,500 psi. When compared to the Class A mix, the compressive strengths performed virtually the same. Due to additional hardened tests performed, MOR, MOE, and tensile strength, the Cement-Limiting surpassed the performance of the Class A mix. However, the Cement-Limiting mix did not meet the 700 psi requirement set in the ODOT OHL192 (Oklahoma Department of Transportation 2008). Reducing the cement content could have limited the flexural strength due to the inability to completely fill the aggregate voids with cementious material. However, comparable to the acceptable ODOT Class A mix, the MOR result still out performed at 390 psi to 360 psi.

The strain data showed insight into the shrinkage and temperature response of full scale pavement sections. The Cement-Limiting recorded lower strain values around -110 microstrain compared to the -125 microstrain of the Class A mix. Also, the daily fluctuations were less of the 28 day monitoring period. The lower strain values could have been due to the increase in aggregate content that was able to absorb the forces exerted during the shrinkage and thermal process. In the Class A mix, the void space is filled with cementious material, which is generally weaker than the aggregates that fill the void space in the Cement-Limiting mix.

8.4. High-Volume Recycled Material Field Implementation Results

8.4.1. Fresh Concrete Testing Results

The fresh property tests were conducted on the concrete once it arrived at the project site. The slump was 3 in. for Panel 7 and 6.5 in. for Panel 8. Both of these results fell outside the ODOT Specification range. The air contents were 5.50% and 7.20% for Panels 7 and 8, respectively. The unit weight was higher in Panel 7 at 113 lb/ft^3 than the 109 lb/ft^3 for Panel 8. All of the fresh property test results for Panels 7 and 8 are outlined in Table 8.8.

Table 8.8 - High-Volume Recycled Field Implementation Fresh Property Results

High-Volume Recycled Material - Panel 7			
Test	Time		
Slump	Time of Pour	3.00	in
Unit Weight	Time of Pour	113	lb/ft^3
Air Content	Time of Pour	5.50	%
High-Volume Recycled Material - Panel 8			
Test	Time		
Slump	Time of Pour	6.50	in
Unit Weight	Time of Pour	109	lb/ft^3
Air Content	Time of Pour	7.20	%

8.4.2. Hardened Concrete Testing Results

The compressive strengths were recorded at 1, 3, 7, 14, and 28 days for both panels to monitor the early strength response of the high fly ash replacement. After 1 and 3 days, the strengths were similar, but at 7 days the difference in strength was approximately 1,000 psi and continued to increase to 28 days. At 28 days, the results for Panel 7 were 5,330 psi and for Panel 8 were 2,820, showing an over 2,000 psi difference in the 28 day strengths between Panels 7 and 8. Figure 8.14 shows the compressive strengths and how each varies over the tested period. Table 8.9 shows all the compressive strength results over the recorded time for each tested panel.

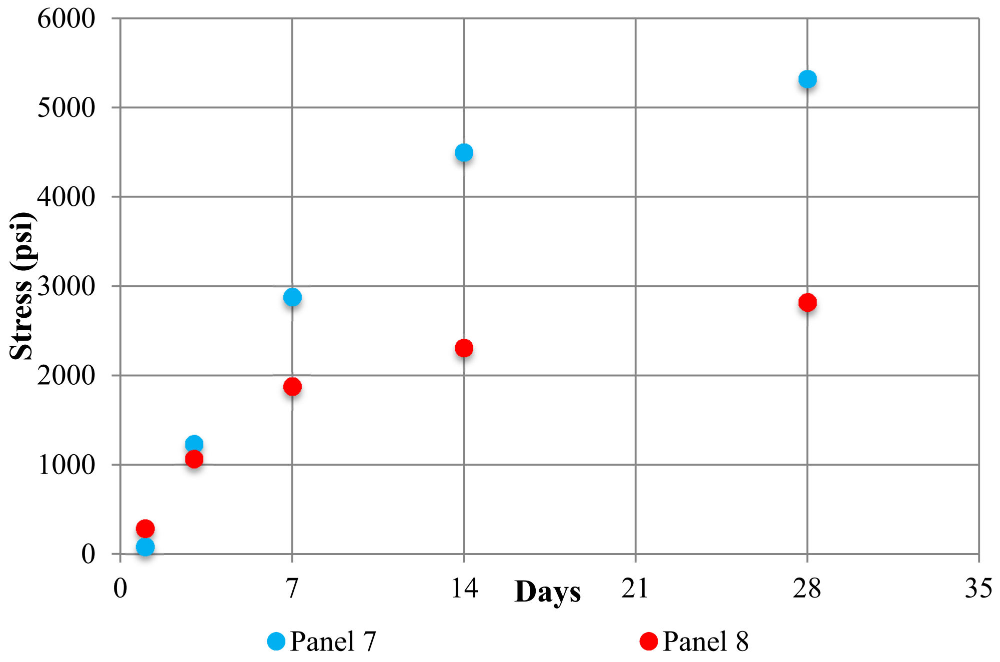


Figure 8.14 - High-Volume Recycled Material Compressive Strengths Chart

Table 8.9 - High-Volume Recycled Materials Compressive Strength Results

	High-Volume Recycled Material - Panel 7	
	Load (lb)	Stress (psi)
1 Day	2310	80
3 Day	22810	1230
7 Day	81510	2880
14 Day	127240	4500
28 Day	150570	5330
	High-Volume Recycled Material - Panel 8	
	Load (lb)	Stress (psi)
1 Day	7940	280
3 Day	30090	1070
7 Day	53110	1890
14 Day	65280	2310
28 Day	79690	2820

Using these results, the performance was compared to the Class A mix over the 28 day test period. At 1 day and 3 days, the Class A had approximately 1,000 higher strengths at 1 day and 1,500 psi at 3 days than both High-Volume Recycled Mixes. At 7 days, Panel 7 increased to match the Class A strength at 3,000 psi. Panel 8 still showed slower strength gain, only reaching 2,000 psi at 7 days. At 14 days, the Panel 7 mix surpassed the Class A strength, with both reaching values higher than 4,000 psi. At 28 days, Panel 8 only reached 2,820 psi, which does not quite reach the ODOT specified 3,000 psi at 28 days. However, due the high content of fly ash, the 56 day strength will likely easily reach the 3,000 psi minimum compressive strength. Figure 8.15 shows the variation in compressive strengths for both High-Volume Recycled Mixes and the Class A mix.

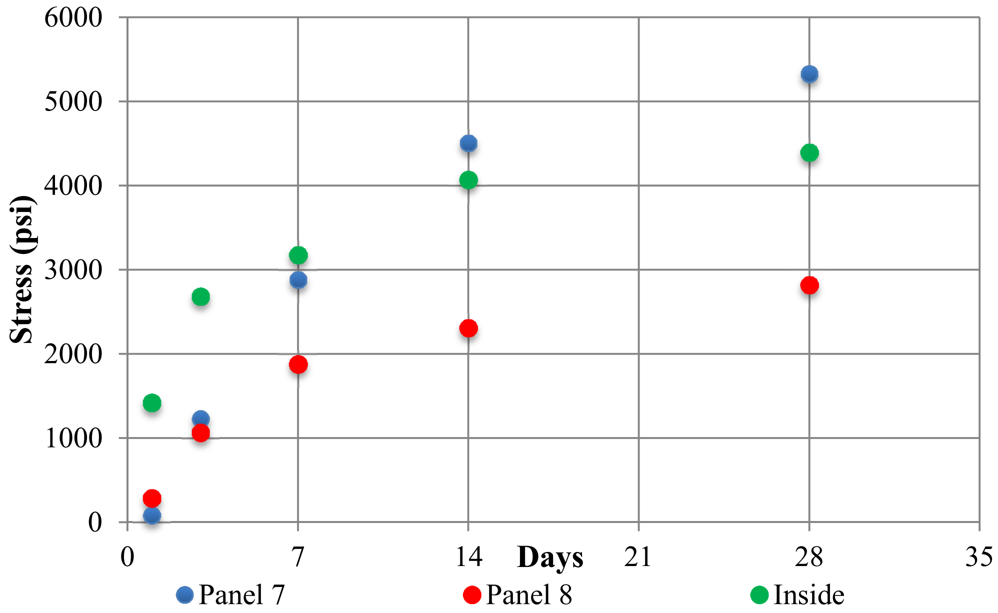


Figure 8.15 – Panels 8 and 7 vs. Inside Mix Compressive Strengths Over 28 Days

Additional hardened tests were conducted on Panel 8 and compared to the performance of the Class A mix. For Panel 8, the tensile strength was 165 psi, modulus of rupture was 280 psi, and modulus of elasticity was 1,930,000 psi. Table 8.10 outlines all the hardened testing results performed on Panel 8. These results are approximately 50% when comparing the results to the ODOT Class A mix. Table 8.11 shows the comparison of the tensile strength, modulus of rupture, and modulus of elasticity results.

Table 8.10 - Hardened Testing Results on Panel 8

High-Volume Recycled Material - Panel 8			
Test	Time	Load (lb)	Stress (psi)
Compressive Strength	1 Day	7940	280
	3 Day	30090	1070
	7 Day	53110	1880
	14 Day	65280	2310
	28 Day	79690	2820
Tensile Strength	28 Day	33140	170
Modulus of Rupture	28 Day	280 psi	
Modulus of Elasticity	28 Day	1,930,000 psi	

Table 8.11 - High-Volume Recycled Material and Class A Hardened Concrete Test Results

Test	Time	Class A	High-Volume Recycled Material
Tensile Strength (psi)	28 Days	360	170
Modulus of Rupture (psi)	28 Days	570	280
Modulus of Elasticity (psi)	28 Days	3,500,000	1,930,000

8.4.3. Vibrating Wire Strain Gage Results

In addition to the laboratory tests, as previously mentioned, VWSG were placed directly in the High-Volume Recycled Material concrete panels to determine the strain response in the field. Following the gage placement that was previously discussed, the strains were monitored every hour, over the 28 day time period following casting. The overall trend of the VWSGs show an expansion over the initial 4 days of curing, then slow increase in negative strains to where the data levels off at approximately -75 microstrains. For the High-Volume Recycled Material, there isn't a transition period from positive strains to the daily fluctuations, which began at 4 days. The daily fluctuations were at an approximate range of 50 microstrains. The VWSGs located 2 in. from the base layer have lower strain values when compared to the strains of the VWSGs placed higher in the concrete panel. Figure 8.16, Figure 8.17, and Figure 8.18 show the strain variations over the initial 28 day period following casting.

With the pronounced diurnal strain changes, the weather data for the mean temperature (Weather Underground) for each day was plotted next to the Tree #2 stain data. Tree #2 was selected as the base reference, due to the location in the middle of the panel and the two gage system. This shows that the overall strain data follows the temperature patterns. As the temperature decreases, the strains increase more negatively, and as the temperature increases, the strains increase positively. Figure 8.19 shows the relationship between the mean temperature and the strain effects on Tree #2.

Comparing the strain performance of the High-Volume Recycled Material to the Class A over the 28 day testing period, some varying results appear. The major difference is in the first 10 days. The High-Volume Recycled Material mix experienced significant expansion over the first 4 days of curing, then proceeded to increase in negative strain. The Class A mix began with negative strains and experienced no initial expansion. The daily fluctuations for the strain effects began at the 4 day mark for the High-Volume Recycled Material Mix, then followed closely with the ambient temperature changes. This was sooner than the Class A mix, which did not begin the daily fluctuations until day 10. Both mixes tend to converge around -100 microstrain at 10 days and stay fairly close the remaining 18 days, expect for a significant temperature change at 20 days. Figure 8.20 shows the relationship of the strain results over the 28 Day testing period for both the High-Volume Recycled Material and Class A concrete mixes.

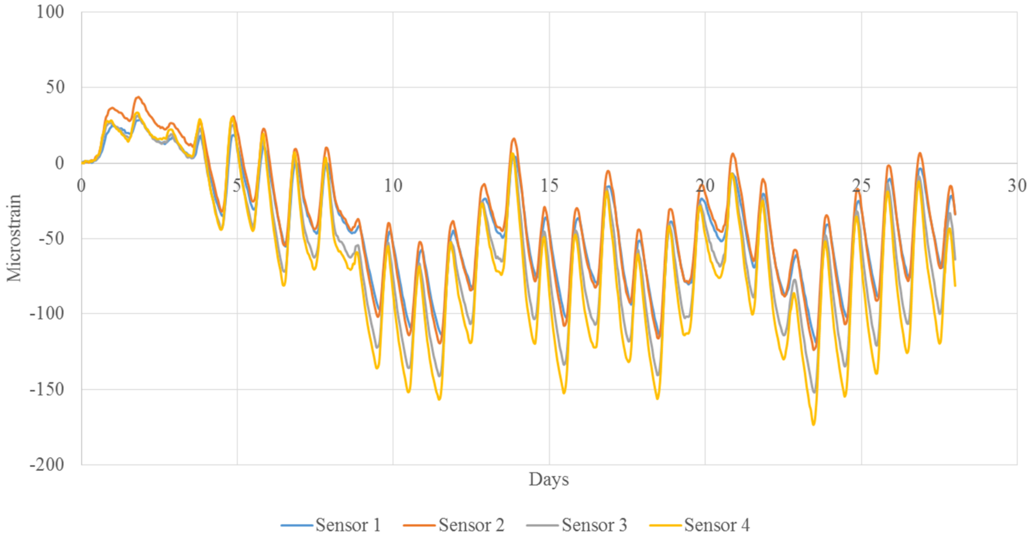


Figure 8.16 - High-Volume Recycled Materials Tree #1 Strain Data

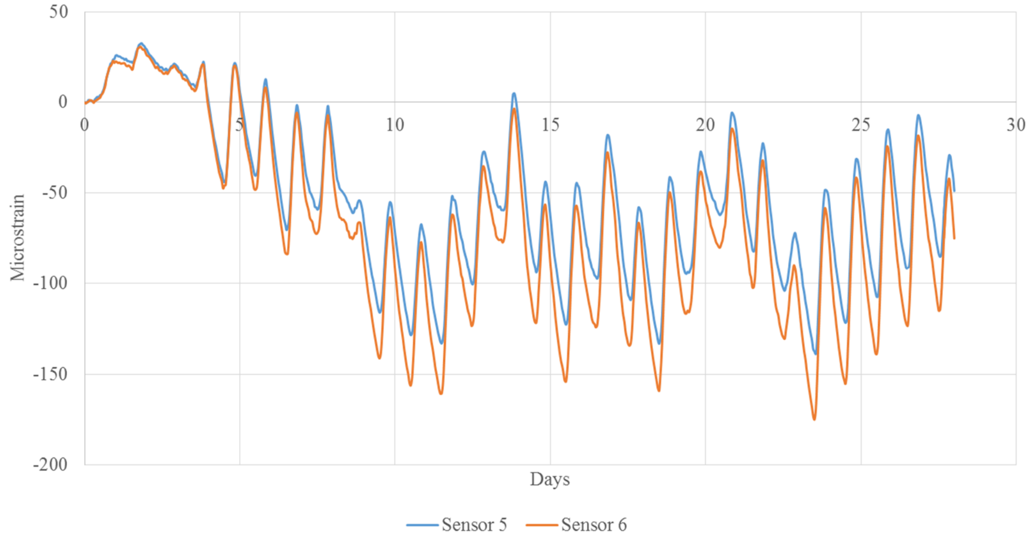


Figure 8.17 - High-Volume Recycled Material Tree #2 Strain Data

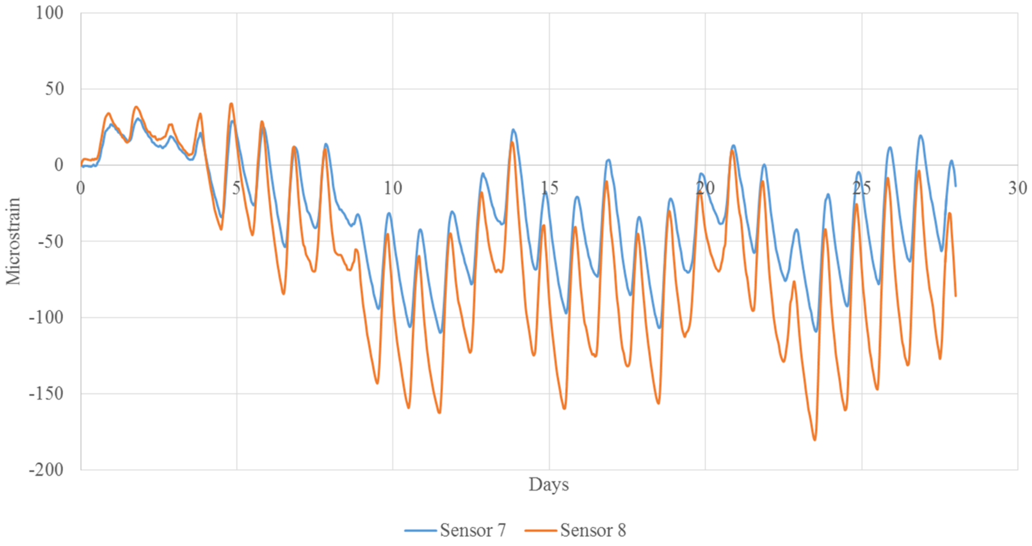


Figure 8.18 - High-Volume Recycled Material Tree #3 Strain Data

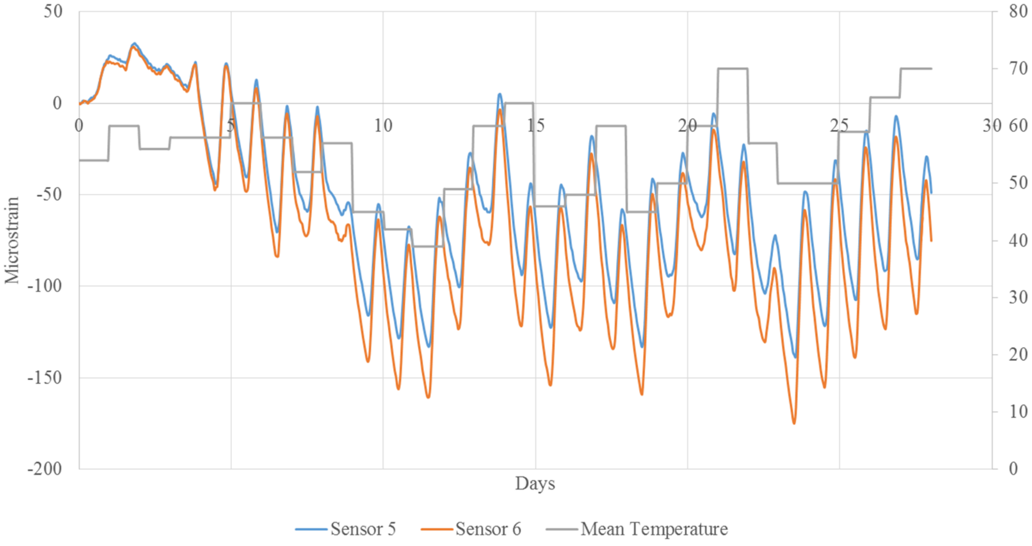


Figure 8.19 - High-Volume Recycled Material Strain Data with Mean Ambient Outside Temperature

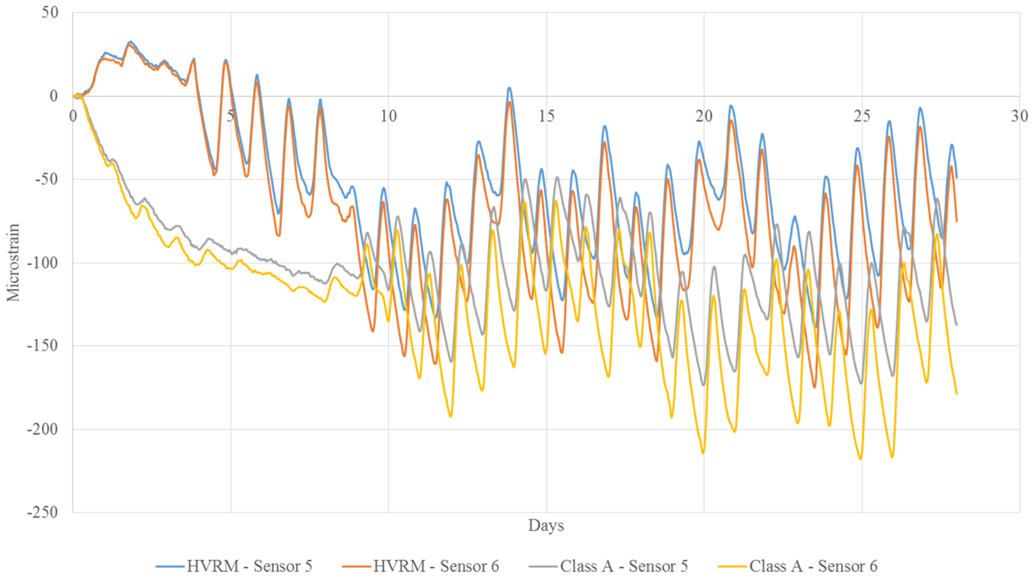


Figure 8.20 - High-Volume Recycled Materials and Class A Strain Data Comparison

8.4.4. Field Implementation Results Discussion

The High-Volume Recycled Material mix provided some interesting fresh concrete, hardened, and strain results. The fresh properties of the High-Volume Recycled mix were higher than the ODOT Specification for both slump and air content. In this case, the high slump results lead to a workable mix that proved easy to construct and finish out of all the previous panels. The unit weight was around 10 lb/ft^3 than the Class A at 109 lb/ft^3. This lower unit weight negatively affected the expected yield of the mix delivered to the site. This difference is directly related to the unit weight of the aggregates themselves. The RCA recorded a significantly less specific gravity and DRUW than the #57 Rock it replaced. This could have negatively influenced the strength results, where a typically more dense material is stronger than a less dense material due to the increased particles and ability to distribute an external load over the additional particles.

The compressive strength results indicated some highly variable results. Comparing Panel 7, which included #57 Rock, which produced a 28 day strength that was approximately 2,500 psi higher than Panel 8 and 1,000 psi higher than the Class A. The strength results from Panel 7 indicate that high levels of fly ash can produce sufficient strengths for concrete pavements. Panel 8 missed the ODOT Specified 3,000 psi by 110 psi at 28 days. However, with the 50% fly ash replacement, 3,000 psi is expected if 56 day tests were performed. The additional hardened tests, MOR, MOE, and tensile strength results tended to follow the compressive strength results where they were significantly less than that of the Class A mix. This might be related to the unit weight as previously mentioned, or are related to the specific RCA source. Unfortunately, if the RCA source is a significantly low strength concrete before recycling, that will limit the strength capacity of the RCA concrete mix. That also means the opposite is true, a high strength concrete can lead to higher strength RCA concrete mixes.

The strain results lead to interesting results as well, with the overall strain values around -75 microstrain, which was less than the Class A mix which recorded strain values around -125 microstrain. The initial expansion results lasted approximately 4 days, which was longer than the Class A mix. This expansion could be due to the higher absorption rates of the RCA limiting the available water for the reaction of the cement and fly ash, resulting in a swelling of the materials in the void space of the mix. The initial response after the expansion period to daily temperature fluctuations indicates the calcium hydroxide reactions was significantly less than the Class A, which was not affected by the daily temperatures until around day 10. This also contributes to the strong correlation between the ambient temperatures effect of the strains. Eventually the strains for the Class A mix begin to slightly follow the trend to the temperatures, but the High-Volume Recycled Materials mix strain data is highly dependent on the temperature, as well demonstrated in Figure 8.19.

9. Findings, Conclusions, and Additional Research

9.1. Introduction

This research incorporates two methods of reducing the cost and ecological footprint of concrete pavements while increasing their sustainability. The first method optimized the aggregate content in order to limit the overall cement content, and the second incorporated large volumes of recycled materials into the mix design.

The following sections describe the findings determined throughout the research process, the conclusions developed based on these findings, and additional research to expand the knowledge learned during this research.

9.2. Findings

9.2.1. Cement-Limiting Concrete Mix

Looking back at the laboratory results and field implementation results, there are some interesting findings. Due to the variety of mix designs and admixtures, the slump test can be misleading to the workability and compressive strengths. Typically, the lower the slump, the lower the workability and the higher the compressive strengths. However, in both the laboratory and field implementation mix designs, the higher slump produced higher compressive strengths. These findings indicate that due to the multiple mix design options tested (i.e. aggregate optimization and admixtures, water reducer or fly ash), the slump test less useful in characterizing the workability of fresh concrete mix.

Next, there is a substantial impact on the concrete if the percent fine aggregate is greater than 50%. For the Coarseness/Workability mix design, the percent fine aggregate was 52%, where the Percent Retained Method and Power

45 Method contained less than 50%. The strength difference for the Percent Retained Method and the Power 45 Method was negligible, but the difference for the Coarseness/Workability was approximately 2,000 psi. This difference is 2/3 the specified ODOT requirement for Class A concrete.

While monitoring the strain results over the 28 day curing period, the overall strains for the Cement-Limiting mix design were less than that of the Class A mix design. The strains monitored were a combination of shrinkage and thermal effects, some of which caused curling and warping of the panel, and the less the strains, the less curling and warping. The more curling and warping a concrete panel exhibits, the increased chance of cracking, which can significantly decrease the service life of the roadway. The strain results also indicated a strong relation with the ambient outside temperatures, indicating that warmer weather causes lower strains in the concrete. Furthermore, in general, the Cement-Limiting panel experienced less shrinkage compared to the Class A panel.

Lastly, each of the results from the hardened tests from the Cement-Limiting mix design performed higher than the commonly used Class A mix. Overall, reducing the cement content by 20% did not have a consequence on the concrete performance.

9.2.2. High-Volume Recycled Material Concrete Mix

Analyzing both the laboratory results and field implementation results show interesting findings for the High-Volume Recycled Material mix design. Mix designs containing greater than 50% fly ash proved difficult in reaching the specified ODOT strength requirements. Multiple mixes containing 50% fly ash or less closely met the ODOT Specification, while mixes significantly above 50% fly ash did not approach the required strengths.

Next, the two recycled materials, RCA and fly ash, individually and when combined met the required 3,000 psi compressive strength. Previous research indicated that implementation of RCA and fly ash caused a reduction in strength results. In this research both the laboratory and field implementation mixes that incorporated both RCA and fly ash, closely met or surpassed ODOT Specifications.

The strain results over the 28 day curing period show a strong relationship with the ambient outside temperatures, indicating that warmer weather causes overall lower strains in the concrete. This correlation could result in additionally curling and warping of the concrete panel if temperatures vary enough. Also, the overall strains monitored in the High-Volume Recycled Materials mix designs were less than that of the Class A mix design. The strains monitored were a combination of shrinkage and thermal effects, some of which caused curling and warping of the panel, and the less variation in between the top and bottom strains, the less curling and warping. The more curling and warping a concrete panel exhibits, the increased chance of cracking which can significantly decrease the service life of the roadway. Furthermore, in general, the High-Volume Recycled Materials panel experienced less overall shrinkage compared to the Class A panel.

9.3. Conclusions

With the significant releasing of carbon dioxide from the creation of cement and many roadway projects reaching the end of their design life and need replacing, conventional concrete construction will only escalate these negative environmental impacts. Two mix design methods were investigated to increase the sustainability of concrete pavement. The first method was to reduce the cement content, and the second was to incorporate recycled construction materials. Both of these methods underwent multiple laboratory tests and field implementation to evaluate all-around concrete performance. The Cement-Limiting mix design adequately performed or surpassed the Class A mix and the ODOT Specifications in the tests. The High-Volume Recycled Material mix design performed up to the ODOT Specifications, but not the Class A mix.

The goal of this research is to inform people of environmental impacts of typical concrete and develop multiple environmentally sustainable mix designs that perform above the minimum ODOT Specifications that are and more environmentally sustainable. Through this research, it shows that Cement-Limiting and High-Volume Recycled Concrete mix design methods are the start to a more environmentally sustainable, adequately performing concrete.

9.4. Recommendations for Future Research

To take these results and continue to build upon creating an environmentally sustainable concrete pavement, additional areas should be investigated. For the Cement-Limiting, additional aggregates could be looked to add to the coarse aggregates or another fine aggregate to supplement the River Sand. The additional fine aggregate could help limit the uniform gradation we typically see in using only one fine aggregate. Also, due to the performance in both fresh and hardened tests, reducing the cement content should be investigated. For the High-Volume Recycled

Materials, continue investigating RCA and fly ash combinations for more consistent results. Also, using select gradations of RCA could incorporate a self-imposed aggregate optimization. For all the outside panels, additional full scale testing should be performed to provide additional information. Tests like traffic loading response, profilograph, and falling weight deflectometer are excellent methods to measure the in-situ response. Each of these additional recommendations will better improve the knowledge of environmentally sustainable concrete pavements to work towards full scale implementation.

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