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PHASE II - REDUCING PORTLAND CEMENT CONTENT AND IMPROVING CONCRETE DURABILITY

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SI* (MODERN METRIC) CONVERSION FACTOR

APPROXIMATE CONVERSIONS TO SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

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16. Abstract

This study evaluated the effects of (1) reducing the cementitious paste volume (CPV) and (2) the use of optimized aggregate gradation (OAG) technique on Florida Class II, Class II - Bridge Deck, Class IV, and Class V concretes using portland limestone cement (PLC).

The main findings from this study are as follows. (1) PLC concrete can provide similar properties as the concrete using ordinary portland cement. (2) The pH values of fresh concrete are not affected by reducing the CPV if the concrete has sufficient CPV. (3) The temperature of hydration in concrete can be reduced by reducing the CPV. (4) The strength of the concrete is affected mainly by the ratio of water to cementitious materials (w/cm). Increasing the CPV of concrete cannot increase the strength of the concrete. (5) Average surface resistivity (SR) and rapid chloride permeability test (RCPT) results showed that the electrical resistance of the concrete is lower when the concrete had a higher CPV. This indicates that a concrete with a higher CPV could have higher permeability, which reduces the durability of the concrete. (6) The drying shrinkage of concrete is increased with higher CPV. (7) When OAG technique is applied to the concrete, the workability of the fresh concrete could be improved. (8) Using a concrete with reduced CPV could result in not only improved concrete durability, but also reduced material cost and less environmental impact in terms of reduced emission of carbon dioxide.

Based on the results of this study, the minimum CPVs of Florida Class II, Class II - Bridge Deck, Class IV, and Class V concretes were determined to be 22.5%, 25.0%, 25.0%, and 27.5%, respectively. A mix design method using minimum CPV and OAG for Florida concrete was developed and recommended for use.

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Minimized cementitious materials content, reduced paste volume, portland-limestone cement, optimized aggregate gradation, concrete mixture design, durability of concrete.	No restrictions.

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EXECUTIVE SUMMARY

Background and Objective of Study

The concrete industry in Florida and the U.S. is presently facing two major challenges, namely (1) the rising cost of cement and (2) the shortage of fly ash. The possible solutions to these challenges include (1) a more effective design of concrete mixes in which the cementitious materials content can be minimized, and (2) use of Type IL cement, which is a blended cement incorporating 5% to 15% limestone.

The database of Materials Acceptance and Certification (MAC) system has indicated that most of Florida Department of Transportation (FDOT) approved concrete mixes have an excess of cement paste. The main reason for this is due to the mistaken assumption that concrete can always be made stronger and with quicker strength gain by increasing the content of cementitious material. In reality, 10% to 20% of the cementitious materials content in many FDOT concrete mixes can be removed without any adverse effects on the plastic or hardened concrete properties (Tia et al., 2019). This reduction in cementitious materials content can be maximized by using an intermediate-size coarse aggregate along with the original aggregates. Usually, the nominal maximum size of intermediate-size coarse aggregate is around 3/8 inch. If concrete mixes have a poor gradation, intermediate-size coarse aggregate could be incorporated to optimize the aggregate gradation of the concrete (Shilstone, 1990). By adjusting the proportion of the coarse and intermediate-size aggregate blends, an optimum packing of aggregates (or optimum aggregate gradation, OAG) can be obtained such that the aggregate volume content is maximized; thus, the cement paste volume can be minimized (Shilstone and Shilstone Jr. 1997a).

At present, concrete used for a certain application in Florida is usually specified by the class of concrete, and the class of concrete is defined by its required compressive strength, ratio of water to cementitious materials (w/cm), and cementitious materials content. It is envisioned that to effectively implement the findings of this study, concrete used for a certain application could be specified by its required strength and durability properties, instead of by the class of concrete as presently defined. In this way, a concrete which may not meet the requirements for a certain class of concrete in terms of its cementitious materials content, but yet has better durability properties and lower cementitious materials content, may be approved for use. This would result in reducing cost of concrete, reducing environmental impact by conserving natural resources, lowering energy consumption, and lowering carbon dioxide emission. In order to be able to implement these research findings effectively, there is a great need (1) to collect more test data to further validate these findings and (2) to expand the scope of the testing program to cover all classes of concrete most commonly used in Florida. Phase II of this study was conducted to meet this need.

The main objectives of the study were follows:

(1) To conduct a laboratory testing program to investigate the effects of cementitious paste volume (CPV) on the workability, compressive strength, and durability of all concrete classes of Florida Department of Transportation. The effects of the application of the optimized aggregate gradation (OAG) technique and the use of Type IL cement in these concrete mixes were investigated.

(2) To recommend implementation of the findings to achieve concrete reclassification plan for Florida Department of Transportation concretes.

(3) To recommend implementation of the findings from this study to achieve reduced cost of concrete, reduced environmental impact, and improved durability of concrete.

Scope of the Study

A laboratory testing program was conducted to evaluate the effects of reducing cementitious materials content, the feasibility of using portland-limestone cement (Type IL) and the benefits of optimized aggregate gradation (OAG) technique. In this research, four reference concrete mixes which met the requirements for Florida Class II, Class II - Bridge Deck, Class IV and Class V concrete were tested. The Florida Class II concrete evaluated had a w/cm of 0.50 and designed CPV of 22.5%, 25.0%, 27.5% and 30.0%. The Florida Class II - Bridge Deck concrete evaluated had a w/cm of 0.44 and designed CPV of 22.5%, 25.0%, 27.5% and 30.0%. The Florida Class IV concrete evaluated had a w/cm of 0.38 and designed CPV of 25.0%, 27.5% 30.0% and 32.5%. The Florida Class V concrete evaluated had a w/cm of 0.32 and designed CPV of 25.0%, 27.5% 30.0% and 32.5%. The cementitious materials used consisted of 80% portland limestone cement (Type IL, portland cement contains 14% limestone) cement and 20% Class F fly ash.

Findings

The following are the main findings of this study on Florida Class II, Class II - Bridge Deck, Class IV, and Class V concretes:

- Concrete using portland limestone cement can provide similar properties as concrete using ordinary portland cement.
- The pH values of fresh concrete were not affected by reducing the CPV and w/cm when the concrete had sufficient CPV.
- The amount of heat released during hydration which is directly related to the temperature rise of the concrete, can be decreased by reducing the CPV. Moreover, the greater volume fraction of aggregate resulting from the denser packing of the OAG also inhibits the temperature rise due to the increased thermal mass. Thus, using the OAG technique in concrete can help mitigate the early cracking issue of concrete.
- Based on the average strength results (compressive strength, splitting tensile strength, and modulus of rupture), the strength of the concrete is affected mainly by the w/cm. Increasing the CPV in a concrete mix design normally does not increase the strength of the concrete.
- Average surface resistivity (SR) and rapid chloride permeability test (RCPT) results showed that the electrical resistance of the concrete was lower when the concrete had a higher CPV. This indicates that the concrete with higher CPV could have higher permeability, which reduced the durability of the concrete.
- Based on the results of drying shrinkage tests, CPV was inversely proportional to the early-age shrinkage of the concrete.
- The OAG technique did not significantly increase the strength or lower the permeability of the concrete at an early age, but the length change of the hardened concrete was reduced. Moreover, when the OAG technique was applied to the concrete, the workability was improved and the temperature of hydration of the fresh concrete could be reduced by decreasing the paste content.

Potential Reduction in Material Cost and Emission of Carbon Dioxide

The potential reduction in materials cost and emission of carbon dioxide from the use of reduced cementitious paste content in concrete is substantial.

For a typical Florida Class II, Class II (Bridge Deck), Class IV, and Class V concretes using OAGs, a 5% CPV reduction could result in a saving per cubic yard of concrete of about $3.50, $4.20, $4.60, and $4.90, respectively.

For a typical Florida Class II, Class II Deck, Class IV, and Class V concretes using OAGs, a 5% CPV reduction would result in a reduction in carbon dioxide emission per cubic yard of concrete of about 68, 70, 80, and 85 pounds, respectively.

Recommended Mix Design Method

Based on the results of this study, a mix design method to achieve concrete with minimum cement paste volume was recommended. The recommended mix design method is intended to achieve concretes with minimum paste volumes which can be used in normal Florida Classes I through V concretes. The designed concrete will be made with normal weight aggregate and will have an air content range from 0% to 6% and a slump range from 2 to 4 inches. The recommended method includes six main steps: (1) select the w/cm, (2) select the cementitious paste volume, (3) select the dosage of superplasticizer, (4) calculate the water and cementitious materials content, (5) determine the combined aggregate proportions, and (6) evaluate the trial mixture and make necessary adjustments. This method can keep the concrete mixes from overdesign with excess cementitious materials content. By minimizing the paste volume and optimizing aggregate gradation, the designed mix would provide better workability, quality, and durability of the concrete.

Recommendations

The results of the laboratory testing program and statistical analyses indicate that the minimum CPV of Class II was 22.5%, the minimum CPV of Class II-Bridge Deck, and Class IV concrete was above 25.0% and the minimum CPV of Class V concrete was above 27.5%. Moreover, portland-limestone cement is becoming widely used in Florida, and the results of this research can be used to support and improve FDOT Standard Specifications for Road and Bridge Construction-Section 346. The use of an intermediate-size aggregate and OAG in the design of concrete should be incorporated in the FDOT Standard Specifications for Road and Bridge Construction in the future.

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CHAPTER 1
INTRODUCTION

1.1 Background

The concrete industry in Florida and the U.S. is presently facing two major challenges, namely (1) the rising cost of cement and (2) the shortage of fly ash. Some of the possible solutions to these challenges include (1) a more effective design of concrete mixes in which the cementitious materials content can be minimized, and (2) use of Type IL cement, which is a blended cement incorporating 5% to 15% limestone.

According to preliminary investigation, the database of Materials Acceptance and Certification (MAC) system has indicated that most of Florida Department of Transportation (FDOT) approved concrete mixes have an excess of cement paste. The main reason for this is due to the mistaken assumption that concrete can always be made stronger and with quicker strength gain by increasing the content of cementitious materials. In reality, 10% to 20% of the cementitious materials content in most FDOT concrete mixes can be removed without any adverse effects on the plastic or hardened concrete properties. This reduction in cementitious materials content can be maximized by using an intermediate-size coarse aggregate along with the original aggregates. Usually, the nominal maximum size of intermediate-size coarse aggregate is around 3/8 inches. If concrete mixes have a poor gradation, intermediate-size coarse aggregate can be incorporated to optimize the aggregate gradation of the concrete (Shilstone, 1990). By adjusting the proportion of the coarse and intermediate size aggregate blends, an optimum packing of aggregates (or optimum aggregate gradation, OAG) can be obtained such that the aggregate volume content is maximized; thus, the cement paste volume can be minimized (Shilstone and Shilstone Jr. 1997a).

The use of OAG along with the reduction of paste (cementitious materials and water) content can improve the properties of the concrete mix, including the following:

(1) Improved workability of fresh concrete

(2) Reduced drying shrinkage

(3) Increased resistance to intrusion of chlorides and sulfates

(4) Reduced heat of hydration

(5) Increased thermal conductivity

(6) Reduced coefficient of thermal expansion.

The reduction in the use of cementitious materials would significantly reduce the cost of concrete, and reduce the environmental impact by conserving natural resources, lowering energy consumption, and lowering carbon dioxide emission.

Research is needed to find ways to reduce the cementitious materials content of current Florida concrete effectively. The study "Phase I - Reducing Portland Cement Content and Improving Concrete Durability" was funded by FDOT to meet this research need. The phase I study had three main objectives as follows:

(1) To investigate an effective method to achieve OAG in Class I pavement concrete and Class IV structural concrete.

(2) To evaluate the effects of OAG and minimizing paste content (MPC) on properties of fresh and hardened concrete, for Class I pavement concrete and Class IV structural concrete.

(3) To evaluate the effects of using Type IL cement instead of Type I/II cement on properties of fresh and hardened concrete, for Class I pavement concrete and Class IV structural concrete.

The findings from this study indicate that the cementitious materials content of Florida concrete can be reduced up to 25% by weight without loss in fresh concrete workability and strength. The concrete with lower cementitious materials content has lower permeability and shrinkage than the reference concrete. This means that concrete with a lower cementitious materials content could have better cracking resistance and durability than the reference concrete mixes. Moreover, when the concrete aggregate gradation is enhanced by the OAG technique, the amount of reduction in cementitious materials content could be further increased. The test results also show that concrete using Type IL cement has similar workability as the concrete using Type I/II cement. The permeability of concrete using Type IL cement is slightly lower than that of the concrete using Type I/II cement. The strength of concrete using Type IL cement is slightly lower than that of the concrete using Type I/II cement in Class IV structural concrete. However, the strength of concrete using Type IL cement passed the FDOT specification limits. The findings from this phase I study are as follows:

(1) Increasing the cementitious materials content without changing the w/cm cannot improve the properties of the hardened concrete in Florida Class I pavement and Class IV structural concrete.

(2) The OAG technique can be applied to the design of Class I pavement concrete and Class IV structural concrete to achieve a reduction of cementitious materials content of up to 25% without loss in fresh concrete workability and strength of hardened concrete, and resulting in improved resistance to shrinkage cracking and improved durability.

(3) Type IL cement can be used as substitute for Type I/II cement in both Class I pavement concrete and Class IV structural concrete.

1.2 Project Objectives

At present, concrete used for a certain application in Florida is usually specified by the class of concrete, and the class of concrete is defined by its required compressive strength, water to cementitious ratio (w/cm), and cementitious materials content. It is envisioned that to effectively implement the findings of this study, concrete used for a certain application could be specified by its required strength and durability properties, instead of by the class of concrete as presently defined. In this way, a concrete, which may not meet the requirements for a certain class of concrete in terms of its cementitious materials content, but yet has better durability properties and lower cementitious materials content, may be approved for use. This would result in reducing cost of concrete, reducing environmental impact by conserving natural resources, lowering energy consumption, and lowering carbon dioxide emission.

In order to be able to implement these research findings effectively, there is a great need (1) to collect more test data to further validate these findings, and (2) to expand the scope of the testing program to cover all classes of concrete most commonly used in Florida. Phase II of this study was conducted to meet this need.

The main objectives of this study are follows:

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(2) To recommend implementation of the findings to achieve concrete reclassification plan for Florida Department of Transportation concretes.

(3) To recommend implementation of the findings from this study to achieve reduced cost of concrete, reduced environmental impact, and improved durability of concrete.

CHAPTER 2
LITERATURE REVIEW

This chapter presents the different methods of reducing the cementitious materials content, the current requirements for different classes of Florida concrete, and a literature review of past research findings on concrete mixtures designs. This information was used to aid the planning and execution of this research project.

2.1 Durability of Concrete

Durability of the concrete is defined as the ability of concrete to resist weathering action, chemical attack, abrasion, or any other process of deterioration to retain its original form, quality, and serviceability when exposed to its intended service environment (Mehta and Monteiro, 2006). Most of the durability problems are caused by the materials deterioration. Although the material deteriorations typically do not have an immediate safety issue, it will progressively increase structural damage, which puts a potential danger to the structures (Tang et al., 2015). To solve the durability issues, researches have explored different materials or techniques to mitigate the material deteriorations. As a result, many assume that requiring a certain level of strength or a maximum allowable w/cm would ensure a durable concrete. This assumption can be misleading; durable concrete must have properties for resisting the extreme environment.

Major durability problems in concrete include alkali aggregate reaction, sulfate attack, steel corrosion, and freeze-thaw damage. Farny and Kerkhoff's research indicated that a high alkali content of concrete is due to a high cement content of concrete (Frany and Kerkhoff, 2007). On the other hand, Li's research stated that the alkali content of portland cement could have a significant effect on the fresh and hardened properties, because of high cementitious materials content and low w/cm ratio of these mixtures (Li et al., 2016). Sulfate attack is a complicated process and depends on many parameters, such as the cement content, w/cm, or permeability of concrete (Li, 2011.; Cullu et al., 2014). Chloride attacks steel in reinforced concrete structure and causes the concrete deterioration which reduces the durability of concrete (Yurdakul et al., 2014). At the same w/cm, increasing cement content increases the chloride penetration. Concrete with high cement content possess higher chloride binding capacity; it will increase the potential of steel corrosion in concrete (Wassermann et al., 2009.; Arachchige, 2008.).

Although Florida's concretes are not designed for freeze-thaw resistance, lower permeability could increase the freeze-thaw resistance. At the same w/cm, reducing cement content could reduce the permeability of concrete (Yurdakul et al., 2014). Therefore, one of the possible solutions to these problems is by using more effective designs of concrete mixes in which the cementitious materials contents are minimized.

Previous research has indicated that 10 to 25% of the cement content in Florida's pavement concrete and structural concrete mixes can be removed without any adverse effects on plastic and hardened concrete properties (Tia et al. 2019). This reduction in cement content can be maximized by using an intermediate-size coarse aggregate along with the original aggregate (for example, using standard #89 coarse aggregate with #57 coarse aggregate). By adjusting the gradation of the coarse aggregate blend, an optimum packing of aggregate (optimum aggregate gradation, OAG) can be obtained such that the aggregate volume is maximized. The use of OAG along with the reduction of cement paste content can improve the properties of the concrete mix, including (1) improved workability of fresh concrete, (2) reduced drying shrinkage, (3) increased resistance to intrusion of chlorides and sulfates, (4) reduced heat of hydration, and (5) reduced coefficient of thermal expansion. The reduction in the use of cementitious materials would significantly not only improve the durability of concrete but also reduce the cost of concrete. Moreover, the research has indicated that the blended cement, PLC (Type IL) can be used in Florida's pavement concrete and structural concrete mixes, instead of OPC (Type I/II). The advantages of PLC concrete are (1) lower heat of hydration, (2) lower coefficient of thermal expansion and (3) lower cost and carbon dioxide emission. These three improvements would greatly reduce the tendency for thermal cracking of the concrete. The application of PLC can reduce the environmental impact by conserving natural resources, lowering energy consumption, and lowering carbon dioxide emission, because PLC incorporates up to 15% limestone filler by weight. The substitution of 10-15% limestone filler for ordinary PC could provide multiple benefits for PCC by reducing cost, mitigating the environmental impact of PC production, and improving durability (Nadelman, 2016). The incorporation of OAG in concrete mix design could enable reductions in paste cement content and improve properties of fresh concrete. Due to better aggregate gradation, the shrinkage and durability of the designed concrete can be improved over those of ordinary PCC (Alexander,1996). Pervious research concluded that PLC and OAG methods can effectively reduce cement content in Florida's pavement and structural concrete.

2.2. Reducing Cementitious materials content in Concrete Mixtures

There are several methods to reduce the cementitious materials content in concrete. The reduction of paste volume, the use of PLC, the application of the OAG and the substitution of cement with SCMs are common methods in North America. This section presents the methodology and principle of each method. All of these methods were applied in this research.

2.2.1 Cementitious Paste Volume

The role of cement paste in concrete is to fill the voids between the aggregates, provide workability, and bind the aggregates together with cementitious hydration products. The amount of cement used in concrete influences not only the properties of the fresh concrete, such as workability, density, and temperature, but also the properties of the hardened concrete, such as strength, shrinkage, permeability, and cracking potential.

The cementitious materials content of a concrete could affect the properties of fresh concrete in some cases. Many researchers stated that slump was lower at lower CPV when the w/cm is the same (Chu., 2019., Chung et al., 2020a) Typically, a higher cement content can give concrete better workability and a quicker setting time (Marar and Eren, 2011); however, if the cement content is too high, the concrete will lose its workability and become sticky (Dhir et al., 2006). For concrete mixtures with relatively low cement content, there may be insufficient paste for adequate workability and sufficient binding of the aggregate particles, thus concrete strength would be reduced and may not reach the target strength (Yurdakul, 2014).

On the other hand, a minimum CPV of concrete mixes is a requirement for mix design in most of the concrete specifications of state highway agencies. Specifying a minimum cement content for concrete is found in 46% of the state DOT specifications reviewed (Lobo, 2019). The database of MAC system has indicated that most Florida concrete mixtures have an excess of paste (Chung et al., 2020). However, the CPV of concrete had no correlation with the compressive strength or results of other strength tests of concrete regardless of the type of aggregates (Fowler et al., 2008). Many researchers stated that reducing CPV of the concrete would not affect the compressive strength, Young’s modulus, flexural strength and splitting tensile strength of the hardened concrete (Chu., 2019.; Chung et al., 2020a).

Cementitious materials content in concrete is a factor used in ACI 209 for prediction of shrinkage and creep (ACI 209, 2008). Shrinkage of concrete can be expected to be a function of the volume of paste (non-aggregate content + water) in the concrete (Allahham et al.,2016). For a given w/cm, an increase in the cement content increases the free drying shrinkage (Yurdakul et al., 2014.; Zhang et al., 2014). This means that high cementitious material contents can cause shrinkage-cracking problems in concrete. The AASHTO T334 ring-shrinkage test can be used to assess the shrinkage tendency of concrete. Tritsch's research found that the highest paste contents of MoDOT concrete mixtures had higher cracking tendencies based on the AASHTO T334 ring-shrinkage test (Tritsch et al., 2005). The test results indicated that cementitious materials content directly affected the shrinkage of concrete and its cracking tendency.

Moreover, a cement paste reduction can sometimes help improve the durability, sustainability, and economy of concrete mixtures (Cool et al., 2017) Permeability is the primary parameter used to evaluate durability when resistance to intrusion of deleterious substances is considered. Many researches have concluded that reduction of CPV to certain point can improve the permeability of the concrete (Marar and Eren, 2011.; Yurdakul et al., 2014.; Chung et al., 2020b). It is because the capillary porosity decreased when the CPV of the concrete decreased. Based on the previous work, minimizing the CPV of the concrete which also reduces the cement content provides a sustainable way for improving the properties of concrete without increasing the costs.

2.2.2 Portland Limestone Cement

Natural limestone is crushed and then finely ground into ordinary portland cement. ASTM C150 (ASTM C150, 2018) requires the incorporated limestone in Portland cement (5% by mass) to be naturally occurring and to consist of greater than 70% by mass of calcium carbonate. Portland limestone cement (Type IL) is specified in ASTM C595 (ASTM C595, 2018) to contain a proportion of limestone in the range between 5 to 15% by mass. European standard EN 197-1 permits PLC to have two different classes, namely CEM II/A-L (6-20% limestone content, by weight) and CEM II/B-L (21-35% limestone content, by mass) (EN197-1,2011) In 2017, FDOT allowed PLC containing up to 15% limestone filler (by mass) to be used in approved concrete mixture designs.

Generally, the mean particle size of limestone filler is between 3.0 to 13.6 µm and the mean specific gravity is between 2.6 to 2.7 (Li and Kwan, 2015; Aqel and Panesar, 2016.; Panesar et al., 2017.; Turk et al., 2017.). When replacing cement, limestone filler indirectly influences the physical and chemical behavior. The physical effect is caused by (1) modification of particle size distribution, (2) dilution and (3) heterogeneous nucleation. Modification of particle size distribution and heterogeneous nucleation can improve the properties of concrete whereas dilution can have potentially adverse effects. The chemical effect of limestone filler on the cement system is mainly the suppression of tricalcium aluminate (C3A) hydration in the first 16 hours of hydration, which is caused by the chemical reaction between limestone filler with monosulfate and calcium aluminate hydrate (Hawkins et al., 1996.; Nadelman, 2016.; Panesar and Zang, 2020).
However, the use of up to 15% limestone content, by weight, in PLC typically does not significantly affect the properties of concrete substantially because the cement manufacturers tailor the properties of Type IL cements to match those of Type I/II. Since limestone is a filler material, its function is not like that of an SCM, which is to improve hardened properties by forming additional C-S-H by the aqueous pozzolanic reaction of the silica in the pozzolan with the calcium released from cement hydration. Hardened properties of PLC may be very similar to those of Type I/II in typical concrete mixes, but their use in mix designs incorporating optimized aggregate gradations and reduced paste contents needs investigation.

In 2014, Shannon et al. found that greater strength and durability of concrete can be obtained when SCMs are used in combination with PLC. According to their research, PLC produced higher strengths than OPC in essentially all mixtures with fly ash replacement, most notably at the 40% replacement level (Shannon et al., 2014). Cost et al. used Alabama coarse (Size 57) limestone aggregate, intermediate-size rounded gravel (Size 8), and natural sand to mix with PLC and OPC. The study evaluated increasing SCM replacement levels for PLC, including 0% SCM, 40% fly ash (C and F), and 30% slag cement with 20% Class C fly ash (50% total replacement). The results showed that consistent concrete strength benefits were observed with PLCs relative to OPCs with different SCM contents. Also, the PLC reduced the setting time relative to OPC (Cost et al., 2014).

The strength improves because of the chemical reaction between the $C\bar{C}$ in the limestone and alumina-containing phases in SCMs. The reaction product improves the strength and lowers the permeability as compared to PCC with SCMs (De Weerdt et al., 2011). However, some SCMs do not have sufficient alumina content to benefit from this reaction with PLC. Most of the research results showed that the performance of PLC-SCM concrete is better than PCC-SCM concrete; however, other properties of PLC-SCM concretes, for example, shrinkage and crack resistance, still need to be investigated.

2.2.3 Supplementary Cementitious Materials

In order to save energy and reduce carbon dioxide emissions, class F fly ash, ground granulated blast furnace slag (GGBFS), silica fume, and other supplementary cementing materials (SCMs) have been used in concrete construction as cement substitutes. The SCMs can replace 10 to 70 % of PC to decrease the amount of cement usage and additionally enhance the properties of concrete, but the demand for portland cement concrete is still growing annually. Recently, with energy trends resulting in a move away from coal burning power production, there has been a decrease in the amount of quality fly ash available for use in concrete(Diaz-Loya et al., 2017). According to the forecast of FA utilization from the American Road and Transportation Builders Association in 2015, the demand for FA will increase by at least 53% during the next 20 years (ARTBA, 2015). Therefore, reducing the cementitious materials content of concrete can mitigate the shortage of SCMs.

Figure 2-1 showed the relative positions of portland cement, limestone filler, fly ash, slag cement, silica fume and metakaolin on a ternary phase diagram (CaO-SiO2-Al2O3) (Panesar and Zang, 2020). Fly ash is an industrial by-product from the combustion of pulverized coal in electronic power plants. According to ASTM C618, wo general classes of fly ash can be defined: low-calcium fly ash (ASTM Class F) produced by burning anthracite or bituminous coal; and high-calcium fly ash (ASTM Class C) produced by burning lignite or sub-bituminous coal (Nochaiya et al.,2010.; ASTM C618, 2019). Because of their properties, the percentage of Class C fly ash used as a percent of total cementitious material in concrete mixes usually ranges from 20 to 35 percent by mass; however, the percentage of Class F fly ash is from 15 to 25 percent. (American Coal Ash Association, 1995.; ACI 211. 4 R-93, 1996). Generally, fly ash could improve the workability, decrease water demand and reduce heat of hydration on fresh concrete. Due to pozzolanic reaction the ultimate strength of hardened concrete and permeability will be improved at the same time. Thus, fly ash can improve the durability performance of the concretes (Uysal and Akyuncu, 2012).

Slag cement is a by-product from the iron manufacturing industry. GGBFS is ground to suitable fineness and is used to replace a portion of portland cement. The benefits of using slag cement include better workability and finishability, higher ultimate strength, lower permeability, and light color (Osborne, 1999.; Divsholi et al., 2014.; Ozbay et al, 2016).

Silica fume is a by-product from the industry of elemental silicon or alloys containing silicon. Because of its chemical and physical properties, silica fume is a very reactive pozzolanic material. Silica fume is known to produce a high strength concrete and is used in a cement replacement and an additive to improve concrete properties (Neville, 1995.; Précontrainte, 1998). Previous researchers indicated that the compressive strength of concrete is significant improved at early age when silica fume is added to concrete (Huang and Feldman, 1985.; Mazloom et al., 2004). However, some researchers stated that the slump loss of concrete increases according to the percentage of silica fume for low w/cm of 0.25 (Duval and Kadri, 1998). Metakaolin is an amorphous aluminosilicate that is produced by the calcination of kaolinitic clay minerals at temperatures between 600 ℃ and 900 ℃. It is being used very commonly as pozzolanic material in concrete and has exhibited considerable influence in enhancing the mechanical and durability properties of concrete (Ambroise et al., 1994). Previous researchers showed that metakaolin can improve the workability of the fresh concrete, increase the strength of concrete, reduce the permeability, and improve finishability (Bai et al., 2003). Moreover, metakaolin replacement of cement is effective in improving the resistance of concrete to sulfate attack (Khatiband and Wild, 1998).

Many different types of SCMs have been evaluated, such as sugarcane bagasse ash or pulverized bottom ash. Most of the SCMs can be used for improved concrete performance and properties. Thus, the use of SCMs as cement replacement is another way to reduce cement content in concrete so as to enhance the sustainability of construction materials.

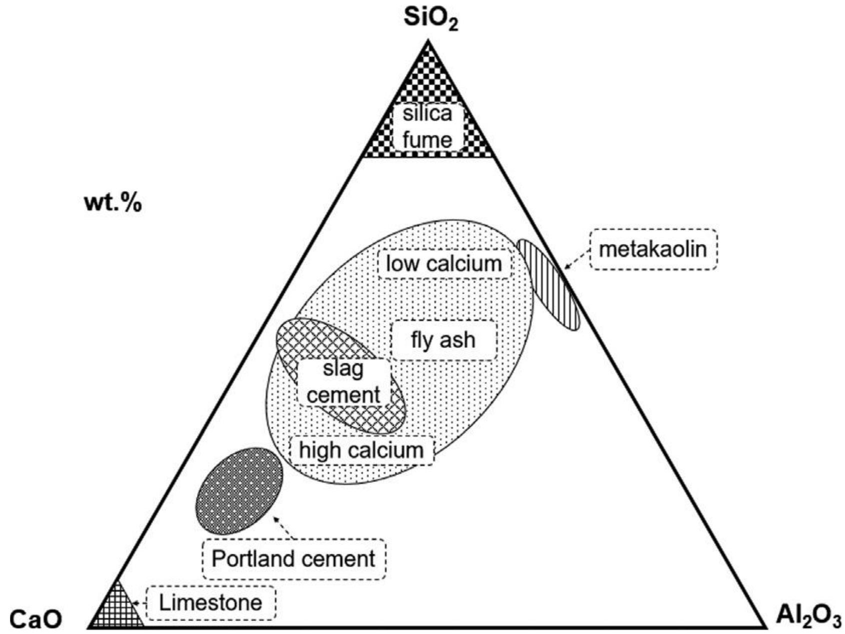

Figure 2-1 Ternary diagram of SCMs (Panesar and Zang, 2020)

2.2.4 Optimized Aggregate Gradation

In the early 19th Century, the Maximum Density Method, the 0.45 Power Chart, the Surface Area and Particle Interface Method, and Fineness Modulus were used to achieve OAG. A century later, additional concepts and methods, including the Aggregate Suspension Mixture Proportioning Method, ACI mix design procedure, classifying mixes based on Workability Factor versus Coarseness Factor (Modified Coarseness Factor Chart), and use of a Percent Retained Chart, were developed to obtain OAG in concrete mix designs.

2.2.4.1 Modified Coarseness Factor Chart

In 1970, Shilstone started work on OAG of concrete with the assumption that the aggregate gradation of concrete directly influences the properties of the concrete (Shilstone, 1990). The modified coarseness factor chart (MCFC) is based on two major factors, CF and adjusted workability factor (WFadj). The MCFC uses three parameters identified as Q, I, and W. Q is the plus 3/8 inches (9.5 mm) sieve particles. I is the sum of the minus 3/8 inches (9.5 mm) and plus No.8 (2.36 mm) sieve particles, which are the intermediate-size particles that fill major voids and aid in mix mobility. W is the minus No. 8 (2.36 mm) sieve particles, which give the mixture workability, functioning similar to that of ball bearings in machines. These three parameters are the foundation of his theory. Shilstone introduced two important factors to predict the properties of concrete. First, CF which is the proportion of plus 3/8 inches coarse particles (Q) in relation to the total coarse particles (Q+I). The following is the formula for CF (Shilstone, 1990.; Shilstone and Shilstone Jr., 1997a):

$$CF = \frac{Q}{(Q+I)} \times 100 \qquad 2\text{-}1$$

where,

Q = the amount of aggregates on 3/8 inches sieve,

I = the amount of aggregates between 3/8 inches and No. 8 sieve.

The second factor is the WF_{adj}. It is the percentage of aggregate passing the No.8 sieve. The quantity of cementitious material will influence the workability of concrete; so Shilstone adjusted WF based on the cementitious materials content. The original WF was based on a 6-bag mixture (564 lb of cement per cubic yard of concrete). WF was adjusted based on the difference between the given mixture and the 6-bag mixture, at an amount of 2.5% per sack. CF and WF_{adj} can indicate the properties of the fresh concrete. The following is the formula for WF_{adj}:

$$WF_{adj} = W + {(2.5\% \times \Delta C)}/{WT} \qquad 2\text{-}2$$

where,

WFadj = adjusted WF,

W = cumulative % passing the No.8 sieve,

ΔC = cement content difference to 564 lb/yd^3,

WT = weight of cement bag, 94 lb.

Shilstone and Shilstone Jr.'s initial research hypothesis was that workability of concrete can be controlled by aggregate gradation; concrete slump may be estimated by adjusting the combined aggregate gradation without adjusting the w/cm or affecting strength (Shilstone and Shilstone Jr., 1997b). There are three important principles established by his research: (1) For every combination of aggregates mixed with a given amount of cementitious materials and cast at a constant consistency, there is an optimum aggregate combination that can be cast at the lowest w/cm and produce the highest strength. (2) The optimum mixture has the least particle interference and responds best to a high frequency, high amplitude vibrator. (3) The optimum mixture cannot be used for all construction due to variations in placing and finishing needs (Shilstone and Shilstone Jr., 1997b).

Shilstone developed a useful chart for evaluating the workability of concrete mixes, referred to as the MCFC. MCFC can also be used as a tool to design the mixture with various cement content and aggregated gradation (Moini, 2015). Aggregate gradation information can be used to calculate CF and WF_{adj}, from which the plastic properties of the fresh concrete can be predicted. Based on the values of WF and CF, the chart (Figure 2-2) can predict and evaluate the properties of concrete by the different zone on the chart. Based on Shilstone's empirical research, Zone I predicts that the workability of the concrete is similar to the properties of the concrete with gap gradation, Zone II predicts that the workability of the concrete is as good as the one of the concrete with well-graded gradation. Zone III predicts that the workability of the concrete is close to the concrete containing small aggregates. Zone IV predicts that the concrete is sandy. Zone V predicts that the concrete is a coarse mix, such as a pervious concrete.

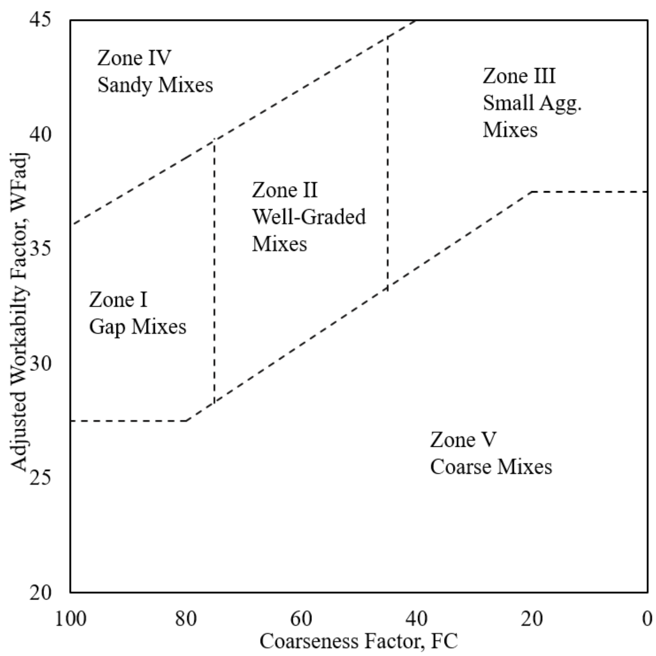

Figure 2-2 Modified coarseness factor chart

2.2.4.2 Individual Percent Retained

The Individual Percent Retained Chart (IPR) is a plot of the individual percent of the total aggregate content retained on each of the different sieves. Designed aggregate distributions can be classified by the content ranges bracketing the percentages retained for each sieve. The 8-18 distribution, also referred to as the Haystack distribution, is shown on the IPR method in Figure 2-

3. The intent is to keep the individual retained percentages between 8 and 18 percent for sieves No.30 through the sieve one size below the Nominal Maximum Aggregates Size (NMAS), and to keep all sieve sizes below 18 percent retained (Richardson, 2005). The IPR plot should not have a significant valley between the 3/8 in. and the lowest specific sieve size. Figure 2-3 shows the modified IPR: 8-18 Chart. The well-graded aggregate from an 8-18 distribution can reduce the total surface area of the aggregate, so it can reduce the water demand of the concrete. Moreover, ACI 302.1 R-96 recommends 8 to 18 percent retained on each sieve for a 1½-in. NMAS gradation, but 8 to 22 percent for ¾- and 1-in. NMAS (ACI302, 1997).

Since the development of the 8-18 chart in 1974, some research has shown that it may not always produce a mix with adequate workability. Ley's research reported that the 8-18 method was insufficient to ensure adequate workability for slip-form-paver mixtures (Ley et al., 2014). The Box Test was developed in 2012 to ensure that slip-form mixes had sufficient workability, yet would be stiff enough to hold straight-formed edges (Taylor, 2012). Cook studied the aggregate combinations used in over 400 concrete mix designs and developed specifications that are summarized in what is now referred to as the Tarantula Curve (Cook et al., 2017). This aggregate distribution varies from the 8-18 distribution in that for most fractions, the upper and lower bounds are broadened, except for those on the #8 and #16 sieves, which are reduced.

The Tarantula Curve (TC, Figure 2-4) was developed using historical concrete pavement mix designs from the Minnesota Department of Transportation (MnDOT). Contractors refined mix designs as the corresponding concrete performance was improved through trial and error. The fit to the Tarantula Curve was found to improve in relation to the refinement of the mix designs; increases in performance were mirrored by better fits to the Tarantula Curve. Similar results have been reported for mixture designs in Iowa and North Dakota. Research in Texas also verified that concrete with aggregate gradation optimized using the TC showed excellent response to vibration for concrete with low cementitious materials content (Taylor, 2015). Most research confirmed that the TC is a reliable tool for OAG, but most of the concrete mixtures used in this research were pavement mixtures.

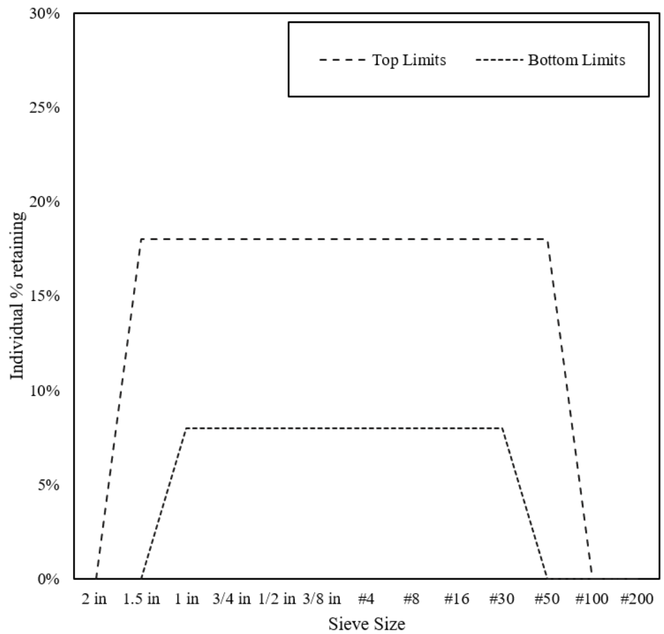

Figure 2-3 Modified individual percent retained chart: 8-18 charts

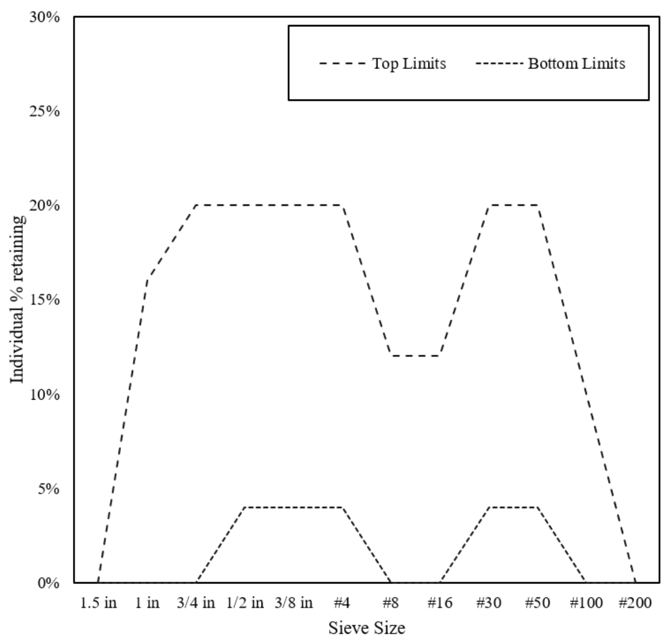

Figure 2-4 Modified individual percent retained chart: tarantula curve

2.2.4.3 Maximum Density Line (0.45 Power Chart)

The Maximum Density Line (MDL) gradation was developed by Nijboer in 1948. It can give the greatest packing of combined aggregates when the gradation is plotted as a straight line with a slope of 0.45 on a log percent passing versus log particle size (Nijboer, 1948). Kennedy et al. also showed that to be true, based on results of numerical modeling (Kennedy et al., 1994). This method is widely used for bituminous concrete to reduce the air voids and determine the amount of asphalt in mixture design. There are two different methods to draw an MDL for actual gradations. In the first method, the MDL is drawn from the percent passing the No.200 (75μm) sieve to the first sieve passing 100 percent. Another method is that MDL is the line drawn from the origin to the maximum sieve size (STP1147, 1992).

The 0.45 power chart, shown in Figure 2-5 is a chart for plotting the percent passing for each sieve on the y-axis and sieve sizes raised to the 0.45 power on the x-axis (FHWA, 2002). Sieve sizes include the following: 1 ½ in. (37.5 mm), 1 in. (25.0 mm), 3/4 in. (19.0 mm), 1/2 in. (12.5 mm), 3/8 in. (9.5 mm), No.4 (4.75 mm), No. 8 (2.36 mm), No. 16 (1.18 mm), No. 30 (600 μm), No. 50 (300 μm), No. 100 (150 μm), No. 200 (75 μm). The MDL is drawn from the origin of the chart to the Maximum Aggregate Size (Quiroga and Fowler, 2004). To verify that the MDL gradation can make optimal concrete, Panchalan and Ramakrishnan evaluated aggregate gradations using five different target powers, 0.35, 0.4, 0.45, 0.5 and 0.55 in the Fuller Distribution (FD) equation and compared the properties of these concrete mixes with those of a control concrete. The results show that concrete using an aggregate gradation following the FD with a 0.45 exponent has better workability and higher strength than concrete mixes using different aggregate gradations. Thus, the 0.45 power curve can be adopted with confidence to obtain the densest packing of aggregate, and it can be used for any type of aggregate. Quiroga and Fowler compared the different OAG methods in their research, and they found that the 0.45 power chart is a useful tool to optimize the grading of aggregate blends. The results showed that gradings close to the 0.45 power chart line had a packing density close to maximum. Mixtures with high fines content should have a gradation plot below the maximum density straight line to produce better behavior (Panchalan and Ramakrishnan, 2007).

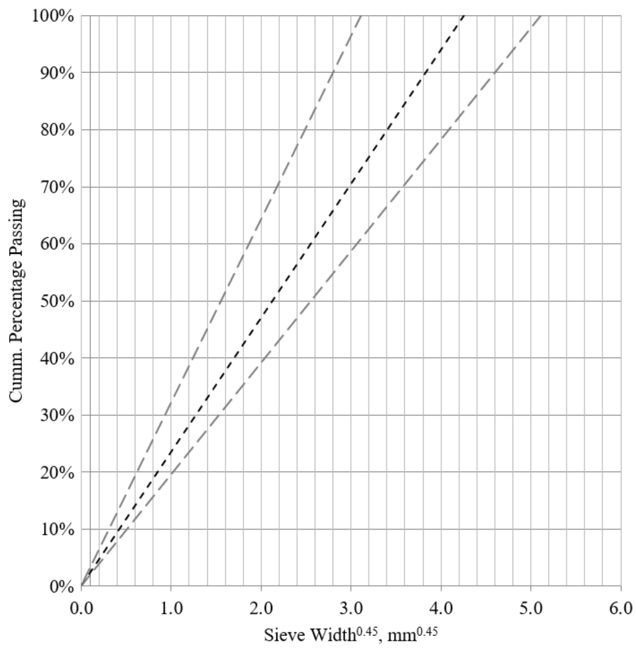

Figure 2-5 Maximum density curves for 0.45 power gradation

2.3 Previous Work

There are many parameters which can influence the properties of concrete. These parameters include w/cm, cementitious paste volume, and aggregates gradation, this section presents the effects of these parameters based on review of previous research.

2.3.1 The effect of w/cm

The water to cementitious material ratio (w/cm) is the most significant parameter in influencing the strength and durability of concrete (Kolias and Georgiou, 2005.; Piasta and Zarzycki, 2017). In 1919, Abram's principle of concrete strength was developed. Abrams' research stated that the strength of the ordinary portland cement (OPC) concrete increases as the w/cm decreases so long as the concrete mix is of workable plasticity (Abrams, 1927). His data also showed that the size and gradation of the aggregate and the quantity of cement do not significantly affect the strength, as long as the concrete mix has sufficient water to produce a workable mix. The following is the formula of Abram's principle (Abrams, 1927).

$$\sigma_c = {}^{A}\!/_{B^{(w/cm)}} \qquad \text{2-3}$$

where,

σc = concrete compressive strength,

A = empirical constant (A = 14,000 psi),

B = constant dependent on cement properties (typically B = 4),

w/cm = water/cement ratio by weight.

However, Abram's principle was based on OPC. Currently, most of the concrete mix designs use blended cement and use supplementary cementitious materials (SCM). Since Abram's formula did not consider the effect of SCM, different modified formula of Abram's principle (Figure 2-6) have been developed. Oluokun developed the modified formula for the concrete mix with fly ash in 1994 (Oluokun, 1994) Bhanja and Sengupta developed the modified formula for concrete with silica fume in 2003 (Bhanja and Sengupta, 2003). Abram's Law is not directly applicable, and modified expressions are needed to predict the compressive strength of concrete with different SCMs. Consequently, the compressive strength is still estimated by the water to cementitious materials ratio. A concrete usually has higher compressive strength when the concrete has a lower w/cm. However, blended cement, such as PLC has been widely used in the world. PLC has lower carbon dioxide emissions and costs than OPC. Many researches showed that the strength of PLC concrete is similar to that of OPC concrete.

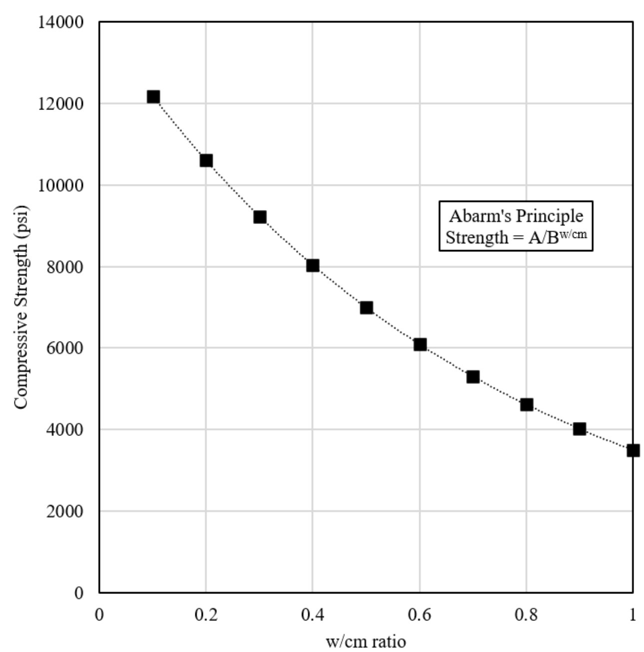

Figure 2-6 Compressive Strength versus w/cm based on Abram's Principle

According to the definition of ACI, the durability of concrete is “the ability to resist weathering action, chemical attack, abrasion, or any other process of deterioration and retain its original form, quality, and serviceability when exposed to its environment” (ACI 201. 2R-16, 2016) Permeability is the primary parameter used to evaluate durability when resistance to intrusion of deleterious substances is considered. The properties that affect the permeability of concrete include the particle size distribution, packing density of the solid components, w/cm, cementitious material content, cement fineness, and degree of hydration. Generally, the permeability of the concrete will be lower at lower w/cm (Mindess, et al., 2003)

For a constant cementitious paste volume, there was considerable variability in the relationship between w/c and shrinkage, although there was some tendency towards decreased shrinkage for concrete mixtures with higher w/c (Lindquist et al.,2008) Other researchers found that the w/cm plays a relatively minor role as compared to the effect of cementitious paste volume on drying shrinkage(Blanks et al., 1940.; Deshpande et al., 2007). Figure 2-7 shows the results from Deshpande’s research. Thus, it can be seen that the w/cm of the concrete is not the main parameter to influence the drying shrinkage (Deshpande et al., 2007).

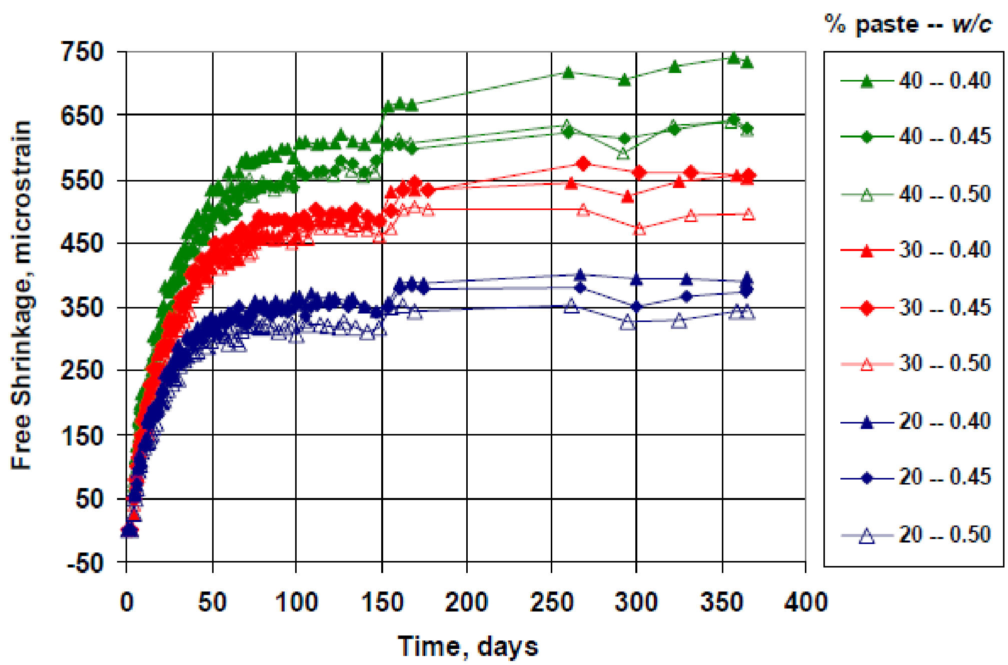

Figure 2-7 The drying shrinkage results from Deshpande’s research (Deshpande et al., 2007)

2.3.2 The effect of cementitious paste volume

Many researchers have shown that high CPV would not affect the strength of the concrete in conventional concrete (Dhir et al., 2006.; Wassermann et al., 2009.; Yurdakul et al., 2014). Previous research also showed that the strength results, such as compressive strength, modulus of rupture and split tensile strength would not be affected by CPV in Florida pavement and structural concrete (Chung et al., 2020a).

The strength of concrete only would be reduced when the concrete has insufficient CPV. On the other hand, the permeability of concrete is not just affected by w/cm. Many researchers stated that the CPV of the concrete affected the permeability of the concrete (Yurdakul et al., 2014.; Obla et al., 2017) Yurdakul applied the rapid chloride permeability test to measure the permeability of concrete with different cement contents. For a given w/cm, chloride penetration increased when the cement content increased because the capillary porosity increased, and more pores were available for chloride penetration (Yurdakul et al., 2014.) The CPV of the concrete affects the permeability of the concrete. Higher CPV could cause the high permeability of the concrete (ACI 209, 2008). It could reduce the durability and increase the cracking potential of the concrete. Thus, using an optimal CPV in concrete is still a key factor in controlling the concrete properties.

The CPV has long been recognized as a main parameter influencing shrinkage of concrete. Pickett found the relationship between the concrete shrinkage, the aggregate content, and the paste content (Pickett, 1956). In his formula, the aggregate and cement paste were assumed to be elastic, and the final formula was expressed in two different forms (Equation 2-4 and 2-5)

$$S_c = S_p(1 - V_g)^{\alpha} \tag{2-4}$$

$$\alpha = 3(1-\mu)/(1+\mu+2(1-2\mu_g)\,{}^{E}/_{E_g}) \tag{2-5}$$

Where,

S_c = Shrinkage of concrete,

S_p = Shrinkage of paste,

V_g = Aggregate volumetric fraction,

α = Material constant,

μ = Poisson's ratio of concrete,

μ_g = Poisson's ratio of aggregate,

E = Elastic modulus of concrete,

E_g = Elastic modulus of aggregate.

It should also be noted that α is assumed to be independent of the w/cm ratio and the aggregate volumetric fraction (Pickett, 1956). If material-specific information is not available, typical values suggested for α would range from 1.2 to 1.7.Deshpande's research indicated that the paste volume is the major factor to influence the shrinkage of the concrete. Figure 2-7 shows that the shrinkage of the concrete significantly increased when the CPV increased (Deshpande et al., 2007). The CPV of the concrete would affect the permeability and shrinkage of the concrete. In order to improve the durability of the concrete, optimal CPV of concrete mixes is important.

2.3.3 The effect of aggregates gradation

While the OAG technique alone cannot affect the properties of hardened concrete, there are several reasons that make OAG important for producing quality concrete. First, OAG technique could be used to design concrete containing a high volume of aggregate and a low volume of cementitious paste. Previous research concluded that OAG technique could improve the workability of fresh concrete (Shilstone and Shilstone Jr., 1997a.; Dhir et al., 2006.; Ley et al., 2014., Taylor, 2015.; Cook et al., 2017.; Chung et al., 2020b). Moreover, better workability can reduce the w/cm of the concrete mix to achieve the target slump. The strength of the concrete increased when w/cm of concrete decreased. OAG technique can not only reduce the CPV but also decrease the w/cm of the concrete. On the other hand, Crouch et al. found it necessary to optimize aggregate gradation in order to meet the goal of the high-performance concrete mix. The high-performance concrete mix was able to lower the w/cm by 8.3% with no detrimental effects on plastic properties (Crouch et al., 2000).

Besides, concrete with OAG and low CPV can reduce the permeability and shrinkage of the concrete. It would improve the properties of the hardened concrete because of lower CPV or w/cm (Crouch et al., 2000.; Rached et al., 2010.; Cook et al., 2017). In addition to reduced shrinkage with the reduction of cementitious materials content, concretes with well-graded aggregates provide less segregation, better cohesiveness, and improved workability compared to the concrete with poor-graded aggregate. According to previous research, the permeability of the concrete with OAG provides comparable results to the standard concrete in pavement and structural concrete mixes. The results of surface resistivity and rapid chloride permeability tests on standard mixes and OAG mixes showed the same level of permeability based on the AASHTO

TP 95 and ASTM C1202 (Chung et al., 2020a). Figures 2-8 and 2-9 show the cross-section of the standard concrete and OAG concrete from previous research.

Moreover, OAG technique was typically applied on the concrete mixes with recycled materials. Segregation commonly happened on the concrete with recycled materials, such as reclaimed asphalt pavement (RAP). Also, when a third aggregate of un-separated RAP was added with coarse and fine aggregate, the traditional ACI procedure is not suitable for use. However, the method of OAG takes advantage of designing concrete mix with three different aggregates by managing the proportion of each aggregate. Previous research indicated that OAG technique could be used to improve the concrete with RAP (Han et al., 2018). OAG technique can enable the use of different recycled materials or combinations of aggregates in concrete mixes.

Figure 2-8 Cross-section of standard concrete

Figure 2-9 Cross-section of concrete with OAG

2.4 Design of Florida Concrete

This section presents the requirements of concrete mixtures, including plastic properties of fresh concrete, 28-day compressive strength, and surface resistance of the hardened concrete based on FDOT standard specifications for road and bridge constructions.

2.4.1 The Requirements of Concrete Mixtures

Figure 2-10 shows the percentage of FDOT approved concrete mixes by classes based on FDOT database. It can be seen the mixes with 0.50 w/cm cover the Class II concrete (18.6%). The mixes with 0.44 w/cm cover the Class II - Bridge Deck concrete (10.6%). The mixes with 0.41 w/cm cover the most popular mixes - Class IV concrete (41.2%). In the end, the mixes with 0.37 w/cm include the Class V concrete (2.2%). Thus, the envisioned mix designs to be evaluated will encompass, Class II, II-Bridge Deck, IV, and V concrete based on w/cm and cementitious materials content.

Based on Section 346 of FDOT standard specifications for road and bridge constructions, Table 2-1 shows the minimum cementitious materials content and maximum w/cm of different classes of concrete. In the Class II concrete, the minimum cementitious materials content is 470 lb/yd^3, maximum w/cm is 0.53, and the estimated CPV is 23.6%. In the Class II - Bridge Deck concrete, the minimum cementitious materials content is 611 lb/yd^3, maximum w/cm is 0.44, and the estimated CPV is 27.5%. In the Class IV concrete, the minimum cementitious materials content is 658 lb/yd^3, maximum w/cm is 0.41, and the estimated CPV is 28.4%. In the Class V concrete, the minimum cementitious materials content is 752 lb/yd^3, maximum w/cm is 0.37, and the estimated CPV is 30.7%.

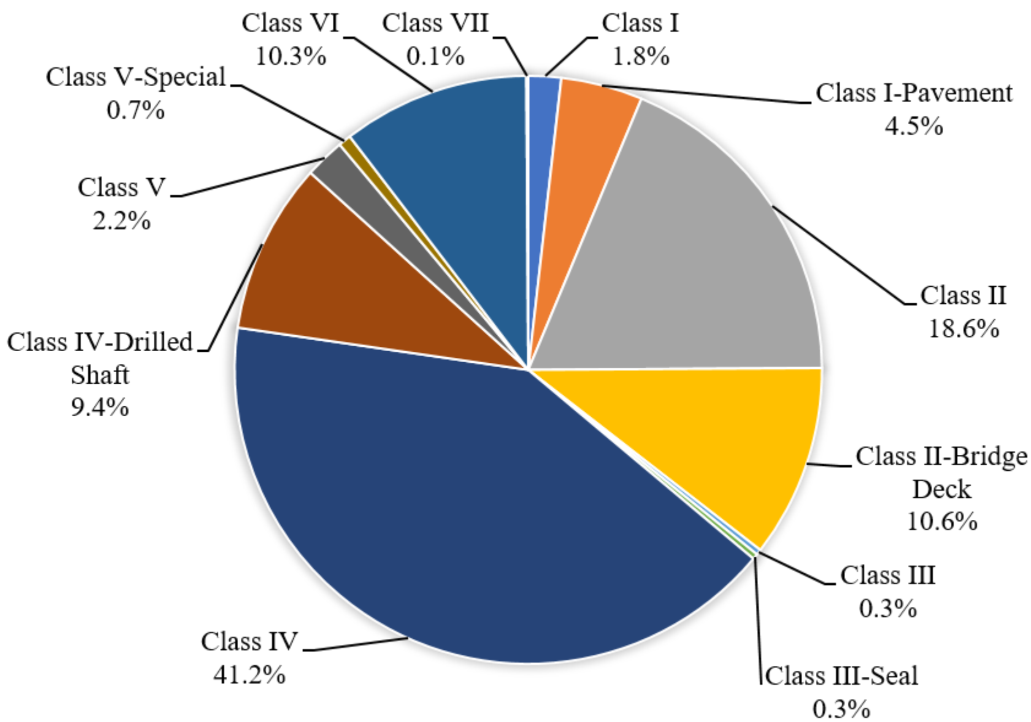

Figure 2-10 Percentage of FDOT-approved concrete mixes by classes

Table 2-1 Florida concrete master proportions

Class of Concrete	Minimum Total Cementitious Materials Content pounds per cubic yard [1]	Maximum Water to Cementitious Materials Ratio pounds per pounds[2]	[4]Estimated Cementitious Paste Volume (%)
Class I	470	0.53	23.6%
Class I-Pavement	470	0.50	22.8%
Class II	470	0.53	23.6%
Class II - Bridge Deck	611	0.44	27.5%
Class III	611	0.44	27.5%
Class III-Seal	611	0.53	30.7%
Class IV	658	0.41[3]	28.4%
Class IV-Drilled Shaft	658	0.41	28.4%
Class V-Special	752	0.37[3]	30.7%
Class V	752	0.37[3]	30.7%
Class VI	752	0.37[3]	30.7%
Class VII	752	0.37[3]	30.7%

Note:
[1] A lower total cementitious materials content may be used provided the plastic, hardened, and durability properties meet the requirements of this Section.
[2] The calculation of the water to cementitious materials ratio (w/cm) is based on the total cementitious materials content including cement and any supplementary cementitious materials that are used in the mix.
[3] When silica fume or metakaolin is used, the maximum water to cementitious materials ratio will be 0.35. When ultrafine fly ash is used, the maximum water to cementitious materials ratio will be 0.30.
[4] Estimated cementitious paste volume is computed by the minimum total cementitious materials content and maximum water to cementitious materials ratio.

2.4.2 The Requirements of Fresh Concrete

Table 2-2 presents the requirement of slump and air content on Florida concrete. The slump requirement of the Class II, II-Bridge Deck, IV and V is 3 inches. The air concrete of the fresh concrete is below 6 %, since freeze and thaw of the concrete is not typical in Florida.

Table 2-2 Requirement of slump and air content on Florida concrete

Class of Concrete	Target Slump (inches)	Air Content (%)
Class I	3.0*	<6.0%
Class I-Pavement	2.0	
Class II	3.0*	
Class II - Bridge Deck	3.0*	
Class III	3.0*	
Class III-Seal	8.0	
Class IV	3.0*	
Class IV-Drilled Shaft	8.5	
Class V-Special	3.0*	
Class V	3.0*	
Class VI	3.0*	
Class VII	3.0*	

Note: *The engineer may allow a maximum target slump of 7 inches when a type F, G, I or II admixture is used. when flowing concrete is used, the target slump is 9 inches.

2.4.3 The Strength Requirements

Table 2-23 shows the 28-day strength requirements for the different classes of concrete in Florida. In the Class II concrete, the 28-day specified minimum compressive strength is 3,400 psi, 28-day specified required minimum compressive strength is 4,600 psi. In the Class II - Bridge Deck concrete, the 28-day specified minimum compressive strength is 4,500 psi, 28-day specified required minimum compressive strength is 5,700 psi. In the Class IV concrete, the 28-day specified minimum compressive strength is 5,500 psi, 28-day specified required minimum compressive strength is 6,750 psi. In the Class V concrete, the 28-day specified minimum compressive strength is 6,500 psi, 28-day specified required minimum compressive strength is 7,850 psi. All the concrete mixes need to pass the 28-day compressive strength requirements in Florida.

Table 2-3 28-day compressive strength requirements for concrete

Class of Concrete	28-day Specified Minimum Compressive Strength (psi)	Overdesign Strength (psi)	28-day Required Minimum Compressive Strength (psi)
Class I	3,000	1,200	4,200
Class I-Pavement	3,000	1,200	4,200
Class II	3,400	1,200	4,600
Class II - Bridge Deck	4,500	1,200	5,700
Class III	5,000	1,200	6,200
Class III-Seal	3,000	1,200	4,200
Class IV	5,500	1,250	6,750
Class IV-Drilled Shaft	4,000	1,200	5,200
Class V-Special	6,000	1,300	7,300
Class V	6,500	1,350	7,850
Class VI	8,500	1,550	10,050
Class VII	10,000	1,700	11,700

2.4.4 The Durability Requirements

Table 2-4 presents the chloride content limits for concrete construction. Currently, FDOT does not specify durability property requirements for different classes of concrete. However, the requirements of permeability or another requirement could be added into an individual contract.

Table 2-4 Chloride content limits for concrete construction

Application	Maximum Allowable Chloride Content (lb/yd^3)
Non-Reinforced Concrete	No Test Needed
Reinforced Concrete[1]	0.70
Reinforced Concrete[2]	0.40
Prestressed Concrete	0.40

Note:
[1]Slightly Aggressive Environment
[2]Moderately or Extremely Aggressive Environment

CHAPTER 3
MATERIALS AND EXPERIMENTAL PROGRAM

This chapter presents the materials used for this research study and the concrete mixture designs used. It also presents the properties of the materials and the aggregate gradation of the concrete mixture designs.

3.1 Selection of Materials

All the materials selected were approved by the FDOT States Materials Office in Gainesville, Florida. In this research, portland limestone cement (Type IL) was used. Type IL cement was from Florida local cement plants approved by FDOT. A Class F fly ash was used as a SCM. All the cementitious materials used passed the ASTM requirements. The aggregates used included silica sand, intermediate-size limestone aggregates (IA, No.89) and coarse limestone aggregates (CA, No. 57). The nominal maximum size of the IA is 9.5mm (3/8 inches) and the nominal maximum size of CA is 25.4 mm (1 inch). The FM of silica sand is between 2.5 to 2.8, which is typical of silica sand used in the USA. The admixtures used included an air-entraining admixture (ASTM C260, 2016), and Type D and Type F water-reducing admixtures (ASTM C494, 2017). Dosage rates selected were based on the mix design and mixing condition.

3.2 Materials Properties

3.2.1 Cementitious Materials

Type IL cement was used for all the concrete productions in this research. The physical and chemical properties for the cement were provided by the FDOT State Materials Office. Tables 3-1 and 3-2 show that Type IL cement passed the requirements of the FDOT specification. Table 3-3 shows the physical and chemical properties of the Class F fly ash used in this research. The physical and chemical properties of the Class F fly ash passed the requirements of the ASTM C618 standard.

Table 3-1 Chemical analysis of Type IL cement

Item	Spec. Limit (AASHTO M240M/M240)	Test Result (Type IL)
Magnesium oxide (MgO) (%)	-	1.1
Sulfate reported as sulfate (SO_3) (%)	3.0 max	2.4
Sulfide sulfur (%)	-	-
Insoluble residue (%)	-	-
Loss of ignition (%)	10.0 max	5.9

Table 3-2 Physical analysis of Type IL cement

Item	Spec. Limit (AASHTO M240M/M240)	Test Result (Type IL)
Blaine Fineness (m^2/kg)	-	529
Residue on No.325 sieve (%)	-	2
Density	-	3.17
Autoclave expansion (%)	0.80 max	0.0
Autoclave contraction (%)	0.20 max	0.03
Time of setting, Vicat test		
Initial set, minutes	45 min	109
Final set, hours	7 max	3.86
Air Content of mortar (volume %)	12 max	8
Compressive Strength		
3 days (psi [MPa])	1,890 [13.0] min	4,109 [28.3]
7 days (psi [MPa])	2,900 [20.0] min	5,225 [36.0]
28 days (psi [MPa]])	3,620 [25.0] min	7,042 [48.6]

Table 3-3 Properties of fly ash used

Test Items	ASTM C618	Fly Ash (Class F)
SiO_2+Al_2O_3+Fe_2O_3, min, %	70.0	88.6
CaO, max %	18.0	3.4
SO_3, max, %	5.0	1.7
Moisture Content, max, %	3.0	0.2
Loss on ignition, max, %	6.0	3.1
Fineness, max, %	34.0	14.8
Strength at 7 days, %	75.0	97.0
Strength at 28 days, %	75.0	97.0
Water requirement, %	105.0	97.0
Autoclave expansion, max, %	0.8	−0.05
Density	-	2.44

3.2.2 Fine Aggregate

The fine aggregate is a silica sand, mined from FDOT plant number 87-090. The test results of the fine aggregates were provided by the FDOT States Materials Office and are shown in Tables 3-4 and 3-5. Table 3-4 presents the specific gravity and water absorption of the sands used. Table 3-5 shows the gradation and fineness modulus. The specific gravities of all fine aggregates had low variation from each other. From the gradation results (Table 3-5 and Figure 3-1), it can be observed that all fine aggregates were very similar. The gradations of all sands were within the range of the FDOT specification limits. The fineness modulus values were also very close to one another.

Table 3-4 Specific gravity and absorption of fine aggregates

Properties	Find Aggregate-A	Find Aggregate-B	Find Aggregate-C
Bulk Specific Gravity	2.640	2.629	2.638
Bulk Specific Gravity (SSD)	2.644	2.639	2.644
Apparent Specific Gravity	2.650	2.655	2.655
Absorption Percent (%)	0.1	0.4	0.2

Table 3-5 Gradation and fineness modulus of fine aggregates

Sieve Size	Percent Passing (%)	Fine Aggregate-A	Fine Aggregate-B	Fine Aggregate-C
#4(4.75 mm)	98-100	98.9	99.9	99.6
#8(2.36 mm)	85-100	95.9	97.3	96.6
#16(1.18 mm)	65-97	86.5	88.3	85.4
#30(0.60 mm)	25-70	60.6	66.6	61.7
#50(0.30 mm)	5-35	15.4	23.2	20.1
#100(0.15 mm)	0-7	1.2	2.1	1.8
#200(0.075 mm)	0-4	0.0	0.0	0.0
Fineness Modulus*	-	2.42	2.23	2.35

* Silica sand from any one source, having a variation in Fineness Modulus greater than 0.20 either way from the Fineness Modulus of target gradations established by the producer, may be rejected. (FDOT specification, 2020)

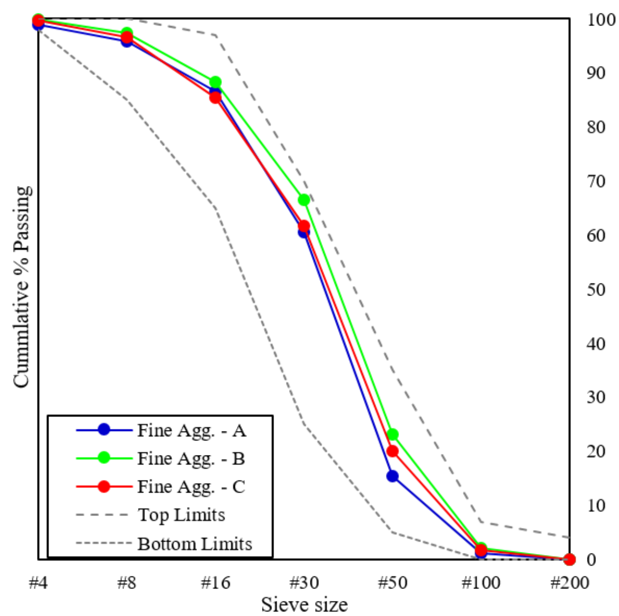

Figure 3-1 The gradation of fine aggregates

3.2.3 Intermediate-size Aggregate

The intermediate-size aggregate is a Florida local limestone, mined from FDOT plant number 87-090. The properties of the intermediate aggregate were tested by FDOT States Materials Office and are shown in Tables 3-6 and 3-7. Table 3-6 presents the specific gravity and water absorption of the intermediate limestone used. Table 3-7 shows the gradation and fineness modulus. From the gradation results (Table 3-7 and Figure 3-2), the gradation of intermediate aggregates passed the FDOT specification limits.

Table 3-6 Specific gravity and absorption of intermediate-size aggregate

Properties	Intermediate-size Aggregate-A
Bulk Specific Gravity	2.349
Bulk Specific Gravity (SSD)	2.474
Apparent Specific Gravity	2.686
Absorption Percent (%)	5.4

Table 3-7 Gradation of intermediate-size aggregate

Sieve Size	Percent Passing (%)	Int. Aggregate-A
1/2" (12.5 mm)	100	100.0
3/8" (9.5 mm)	90-100	98.6
#4(4.75 mm)	20-55	48.8
#8(2.36 mm)	5-30	10.3
#16(1.18 mm)	0-10	2.9
#50(0.30 mm)	0-5	1.2

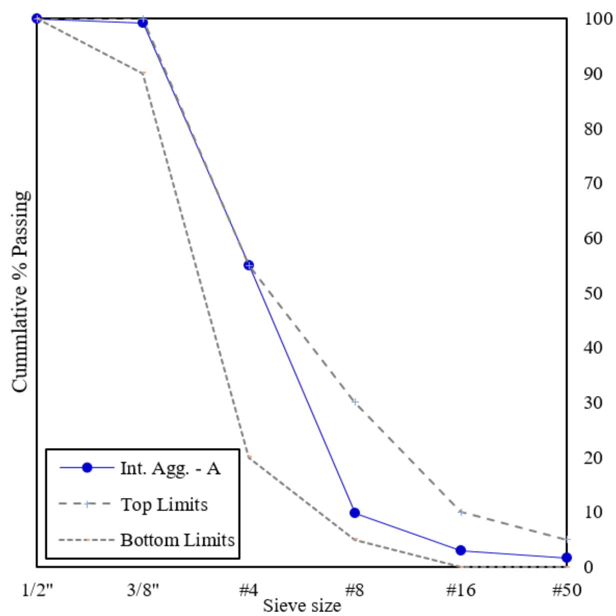

Figure 3-2 The gradation of intermediate-size aggregate

3.2.4 Coarse Aggregate

The coarse aggregate is a Florida local limestone, mined from plant number #87-090. The properties of the coarse aggregate were tested by FDOT States Materials Office and are shown in Tables 3-8 and 3-9. Table 3-8 presents the specific gravity and water absorption of the coarse limestone used. Table 3-9 shows the gradation and fineness modulus. From the gradation results (Table 3-8 and Figure 3-3), it can be observed that aggregate #A and #C are finer than #B. The gradation of these coarse aggregates all passed the FDOT specification limits.

From the gradations of the intermediate-size and coarse aggregates, it can be found that the gradation of the aggregates varied somewhat. The gradation of aggregates will influence the properties of the concrete. Thus, the OAG technique was used to adjust the combined gradation of the aggregate, so it can reduce the segregation of the concrete mix.

Table 3-8 Specific gravity and absorption of coarse aggregates

Properties	Coarse Agg.-A	Coarse Agg.-B	Coarse Agg.-C
Bulk Specific Gravity	2.381	2.352	2.398
Bulk Specific Gravity (SSD)	2.448	2.430	2.461
Apparent Specific Gravity	2.551	2.549	2.559
Absorption Percent (%)	2.8	3.3	2.6

Table 3-9 Gradation of coarse aggregates

Properties	Percent Passing (%)	Coarse Agg.-A	Coarse Agg.-B	Coarse Agg.-C
1½” (37.5 mm)	100	100.0	100.0	100.0
1” (25.0 mm)	95-100	99.8	99.2	99.6
1/2” (12.5 mm)	25-60	53.5	39.2	51.3
#4(4.75 mm)	0-10	5.5	3.4	5.2
#8(2.36 mm)	0-5	3.3	2.8	3.0

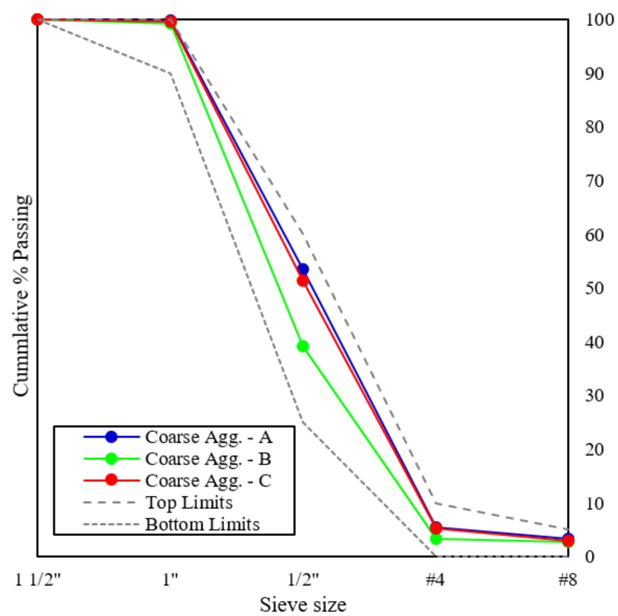

Figure 3-3 The gradation of coarse aggregates

3.3 Concrete Mix Design

In this research, the concrete mixtures used Type IL cement with different water to cementitious materials ratios. Four different concrete mixtures were developed in this research. The first concrete mixture design was a typical Class II concrete (w/cm = 0.50). The second concrete mixture was a Class II - Bridge Deck (w/cm =0.44). The third concrete mixture was a Class IV concrete (w/cm =0.38) The last concrete mixture was a Class V concrete (w/cm =0.32). The reference Class II concrete mixture used 550 lb/yd^3 cementitious materials content with 0.50 w/cm. The percentage of total fine aggregate was around 43% of the total aggregate in the mixture. The reference Class II - Bridge Deck concrete mixture used 595 lb/yd^3 cementitious materials content with 0.44 w/cm. The percentage of total fine aggregate was around 40% of the total aggregate in the mixture. The reference Class IV concrete mixture used 700 lb/yd^3 cementitious materials content with 0.38 w/cm. The percentage of total fine aggregate was around 37.5% of the total aggregate in the mixture. The reference Class V concrete mixture used 765 lb/yd^3 cementitious materials content with 0.32 w/cm. The percentage of total fine aggregate was around 35.0% of the total aggregate in the mixture. Trial batch mixtures were made before every production mixture. Admixtures were added to the concrete mixtures to get the desired air content and to obtain the target slump. A total of 32 mixtures (8 concrete mixes for each class of concrete) were produced in the laboratory and evaluated in this research study. The admixtures used included an air-entraining admixture (ASTM C260, 2016), and two water-reducing admixtures (ASTM C494, 2017). Dosage rates selected were based on the cementitious materials content of the original mix design and mixing condition.

3.3.1 Reduced Cementitious Materials Content

There are two different approaches to reduce cementitious material content in this research. The first method is to minimize the cementitious paste volume (CPV) directly. The second method is to use optimized aggregate gradation. This research applies these two methods to design and develop economical concrete in Florida.

(1) Effects of Reducing Cementitious Paste Volume

Higher cementitious materials content will cause higher shrinkage, permeability, and coefficient of thermal expansion in concrete. Reducing CPV can be a way to extend concrete service life. An advanced mixture design method, Aggregate Suspension Mixes Proportioning Method, was developed by the American Concrete Institute in 2014 (ACI 211.6T, 2014). This method aims to minimize the cementitious paste and air volume in the self-consolidating concrete (SCC). The results showed that reducing the CPV of SCC has many benefits. Thus, this study investigated how an optimum minimization of the CPV could improve the performance of Florida concretes.

(2) Effects of Optimized Aggregates Gradation

OAG can enhance the packing density of aggregates so that it can decrease CPV of the concrete. Applying Type IL cement and OAG can effectively reduce the cementitious materials content in concrete. Information in the Roadway and Bridge Specification and Standards of the different DOTs have been reviewed and summarized with regards to the OAG methods used in U.S. Nine state DOTs, namely Alabama, Colorado, Kansas, Michigan, Minnesota, Montana, New Mexico, Ohio, and Utah, are using Coarseness Factor (CF) in designing concrete mixtures. Five state DOTs, namely Alabama, Colorado, Montana, Texas, and Utah are using Individual Percent Retained as their design principle for concrete. Five state DOTs, namely Alabama, Colorado, Montana, South Dakota, and Utah are using a maximum density curve, also known as the 0.45 power chart in designing concrete. On the other hand, the individual percent retained chart

(Tarantula Curve) can only design pavement concrete mix based on this methodology. Moreover, the 0.45 power chart would reduce the workability of fresh concrete, which would reduce the performance of structural concrete. Since the major goal of this research is to develop OAG method for normal concrete mixes with PLC, the CF method was judged to be more suitable to optimize the aggregate gradation of structural concrete and was thus selected to be used in this research.

Two groups of concrete mixtures using two different reduced cementitious materials content methods were developed:

1. The FDOT standard concrete (FSC) group used Type IL cement with a minimized CPV.
2. The concrete with OAG technique group used Type IL cement with minimized CPV and OAG technique.

The following are the mixes evaluated:

FSC – Florida Standard Concrete with Type IL cement.

OAG – Concrete incorporating OAG and Type IL cement.

A number is placed at the end of the mix group to designate the CPV of the mix. Table 3-10 shows all concrete mixes used in this project.

Table 3-10 CPV and w/cm of concrete mixtures to be evaluated

OAG Technique	Design Paste Volume (%)			
	Water to Cementitious Materials Ratio			
	Mix I 0.50	Mix II 0.44	Mix III 0.38	Mix IV 0.32
No	22.5 (FSC-225)	22.5 (FSC-225)	25.0 (FSC-250)	25.0 (FSC-250)
	25.0 (FSC-250)	25.0 (FSC-250)	27.5 (FSC-275)	27.5 (FSC-275)
	27.5 (FSC-275)	27.5 (FSC-275)	30.0 (FSC-300)	30.0 (FSC-300)
	30.0 (FSC-300)	30.0 (FSC-300)	32.5 (FSC-325)	32.5 (FSC-325)
Yes	22.5 (OAG-225)	22.5 (OAG-225)	25.0 (OAG-250)	25.0 (OAG-250)
	25.0 (OAG-250)	25.0 (OAG-250)	27.5 (OAG-275)	27.5 (OAG-275)
	27.5 (OAG-275)	27.5 (OAG-275)	30.0 (OAG-300)	30.0 (OAG-300)
	30.0 (OAG-300)	30.0 (OAG-300)	32.5 (OAG-325)	32.5 (OAG-325)

3.3.2 Optimized Aggregate Gradation

In 1970, Shilstone started work on OAG of concrete with the assumption that the aggregate gradation directly influences the properties of concrete. The modified coarseness factor chart (MCFC) is based on two major factors, coarseness factor (CF) and adjusted workability factor (WF_{adj}). The MCFC is divided into three segments identified as Q, I, and W. Q is the plus 3/8 inches (9.5 mm) sieve particles. I is the sum of the minus 3/8 inch (9.5 mm) and plus No.8 (2.36 mm) sieve particles, which are the intermediate-size particles that fill major voids and aid in mix mobility. W is the minus No. 8 (2.36 mm) sieve particles, which give the mixture workability, functioning similar to that of ball bearings in machines. These three parameters are the foundation of his theory. Shilstone introduced two important factors to predict the properties of concrete. First, CF which is the proportion of plus 3/8 inches coarse particles (Q) in relation to the total coarse particles (Q+I). The following is the formula for CF:

$$\mathrm{CF} = \left(\frac{\mathrm{Q}}{\mathrm{Q+I}}\right) x100 \qquad 3\text{-}1$$

where,

Q = the amount of aggregates on 3/8 inch sieve,

I = the amount of aggregates between 3/8 inch and No. 8 sieve.

The second factor is the WF_{adj}. It is the percentage of aggregate passing the No.8 sieve. The quantity of cementitious material will influence the workability of concrete; so Shilstone adjusted WF based on the cementitious materials content. The original workability factor was based on a 6 sacks (564 lb) mixture. WF was adjusted based on the difference between the given mixture and the 6-sack mixture, at an amount of 2.5% per sack. CF and WF_{adj} can indicate the properties of the fresh concrete. The following is the formula for WF_{adj}:

$$\mathrm{WF}_{adj} = \mathrm{W} + \left(\frac{2.5\%\ \mathrm{x}\ \Delta\mathrm{C}}{\mathrm{WT}}\right) \qquad 3\text{-}2$$

where,

WF_{adj} = adjusted workability factor,

W = cumulative % passing the No.8 sieve,

ΔC = cement content difference to 564 lb/yd^3,

WT = weight of cement bag, 94 lb.

His initial research hypothesis was that workability of concrete can be controlled by aggregate gradation; concrete slump may be estimated by adjusting the combined aggregate gradation without adjusting the w/cm or affecting strength. There are three important principles established by his research: (1) For every combination of aggregates mixed with a given amount of cementitious materials and cast at a constant consistency, there is an optimum aggregate combination that can be cast at the lowest water-cement ratio and produce the highest strength. (2) The optimum mixture has the least particle interference and responds best to a high frequency, high amplitude vibrator. (3) The optimum mixture cannot be used for all construction due to variations in placing and finishing needs.

Shilstone developed a useful chart for evaluating the workability of concrete mixes, referred to as the modified coarseness factor chart (MCFC), where Coarseness Factor (CF) is plotted on the X-axis and Adjusted Workability Factor (WF_{adj}) on the Y-axis. Aggregate gradation information can be used to calculate CF and WF_{adj}, from which the plastic properties of the fresh concrete can be predicted. Based on the values of WF and CF, the chart (Figure 3-4) can predict and evaluate the properties of concrete by the different zone on the chart. Table 3-11 shows the required values for the different zone. Based on empirical Shilstone's research, Zone I represents that the workability of the concrete is similar to the properties of the concrete with gap gradation, Zone II represents that the workability of the concrete is as good as the one of the concrete with well-graded gradation. Zone III represents that the workability of the concrete is close to the concrete containing small aggregates. Zone IV represents that the concrete is sandy. Zone V represents that the concrete is a coarse mix, such as a pervious concrete.

Table 3-11 The reference CF and WF of different zones on MCFC

Zone	Coarseness Factor, CF	Adjusted Workability Factor, WFadj
Zone I (GAP)	75 - 100	27 - 36
Zone II (Optimized)	43 - 75	27 - 43
Zone III (Small Agg)	0 - 43	32 - 45
Zone IV (Sandy)	43 - 100	36 - 45
Zone V (Coarse)	0 - 100	20 - 36

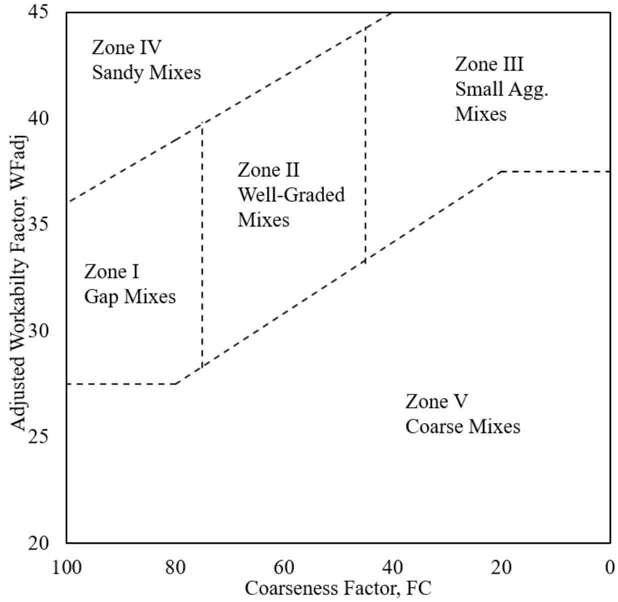

Figure 3-4 Modified coarseness factor chart

3.3.3 Class II Concrete Mix

The concrete mix to be evaluated is Florida Class II concrete with w/cm of 0.50, and total cementitious materials content (80% Type IL and 20% Class F Fly Ash) of 550 lb/yd^3. The estimated CPV of the reference mix is 27.5%. Table 3-12 shows the details of the mixture designs of Class II concrete mixture.

Figures 3-5 and 3-6 show the aggregate gradation in terms of cumulative percentage passing of each mixture. FSC mixtures are typical concrete mixes with gap-graded aggregates. Figures 3-7 and 3-8 present the individual percentage retained charts of these mixtures. The coarse aggregate and sand were combined volumetrically to produce the blended aggregates. This chart can identify any excess or lack of aggregate particles on the specific sieves. The National Concrete Pavement Technology Center developed the modified individual percentage retained chart (Tarantula Curve) to evaluate pavement concrete (Cook et al., 2015 and FHWA, 2019). When the aggregate gradations are plotted on the tarantula curve, the gradation of the OAG mix meets the limits of the tarantula curve and that of the FSC mix plots outside the limits of tarantula curve indicating a finer gradation. OAG mixture designs are based on the MCFC developed by Shilstone. Figure 3-9 shows the MCFC which was used to optimize the gradation of the OAG mixes. From the CF and WF values as presented in Figure 3-9, it can be seen that all the FSC mixes had CF and WF values which plot inside Zone IV, which indicates sandy gradation with excessive fines, while all the OAG mixes had CF and WF values which plot within Zone II, which indicates a well-graded distribution.

As specified by Standard Specifications for Road and Bridge Construction of FDOT, the slump requirement of Class II concrete is 2 to 4 inches. The required air content is to be equal to or less than 6 % in the Class II concrete.

Table 3-12 Class II concrete mixture design

Mix Design (Paste Vol.)	FSC-225 (22.5%)	FSC-250 (25.0%)	FSC-275 (27.5%)	FSC-300 (30.0%)	OAG-225 (22.5%)	OAG-250 (25.0%)	OAG-275 (27.5%)	OAG-300 (30.0%)
Mixing Date	7/23/19	9/4/19	11/5/19	7/2/19	7/23/19	9/4/19	11/5/19	7/2/19
Cement (lb/yd^3)	360	400	440	480	360	400	440	480
Fly Ash (lb/yd^3)	90	100	110	120	90	100	110	120
Fine Agg. (lb/yd^3)	1336.4	1318.2	1186.2	1066.3	1181.3	1107.8	1021.3	934.7
Intermediate Agg. (lb/yd^3)	0	0	0	0	453.4	362.3	412.5	469
Coarse Agg. (lb/yd^3)	1839.8	1747	1779.5	1777.4	1526.4	1577.9	1514.5	1458
Air Entraining Admixture (oz/cwt)	0.11	0	0	0.83	0.11	0.18	0	0.83
TypeD (oz/cwt)	4.0	4.0	4.0	0	4.0	4.0	4.0	0
TypeF (oz/cwt)	3.02	1.66	1.21	0	3.02	1.40	1.21	0

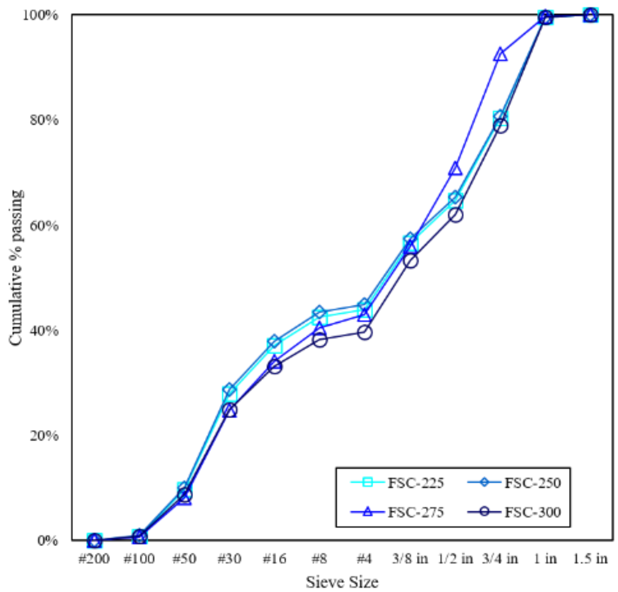

Figure 3-5 Gradation of aggregates used in the Class II concrete mixes: FSC mixes

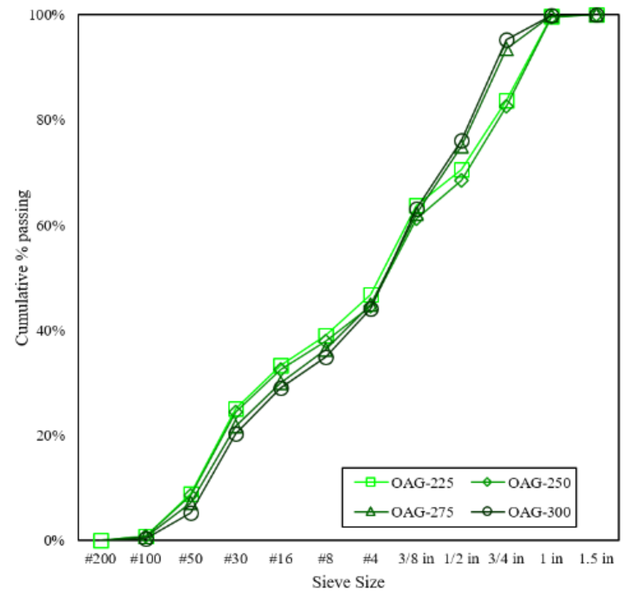

Figure 3-6 Gradation of aggregates used in the Class II concrete mixes: OAG mixes

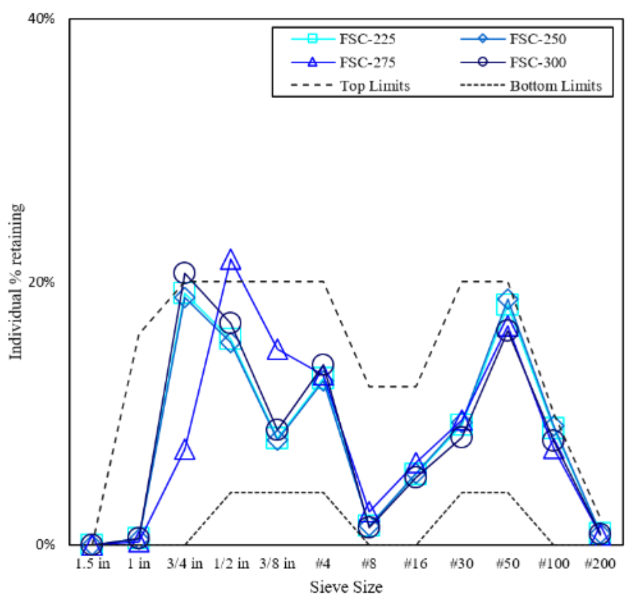

Figure 3-7 Individual percentage retained for the aggregate of the Class II concrete mixes: FSC mixes

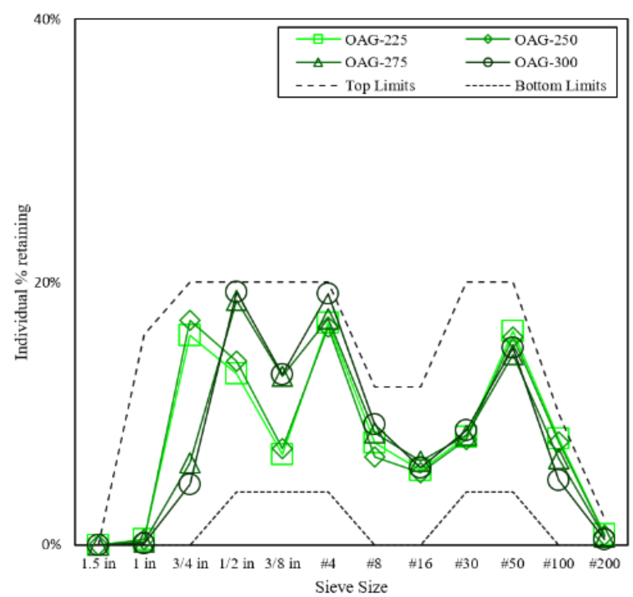

Figure 3-8 Individual percentage retained for the aggregate of the Class II concrete mixes: OAG mixes

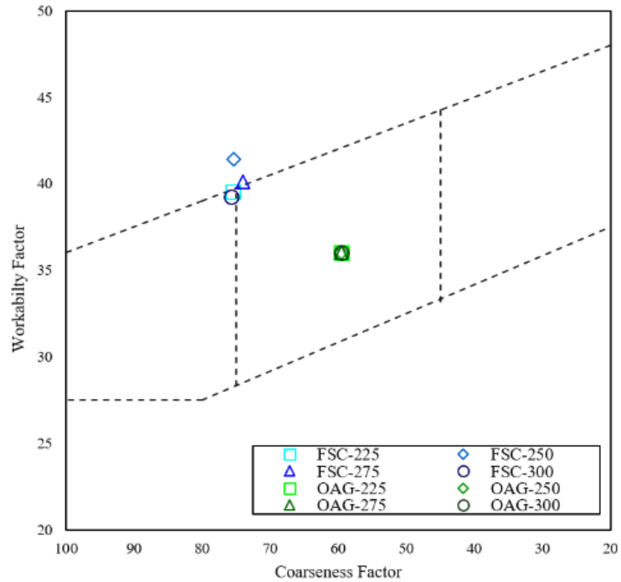

Figure 3-9 Modified coarseness factor chart for the aggregate of the Class II concrete mixes

3.3.4 Class II Bridge-Deck Concrete Mix

The concrete mix to be evaluated is a Florida Class II - Bridge Deck concrete with w/cm of 0.44, and total cementitious materials content (80% Type IL and 20% Class F Fly Ash) of 595 lb/yd^3.The estimated CPV of the reference mix is 27.5%. Table 3-13 shows the details of the mixture designs of Class II - Bridge Deck concrete mixture.

Figures 3-10 and 3-11 show the aggregate gradation in terms of the cumulative percentage passing of each mixture. The FSC mixtures are typical concrete mixes with gap-graded aggregates. Figures 3-12 and 3-13 present the individual percentage retained chart of each mixture. The gradation of both the FSC and OAG mixes plotted within the limits of the tarantula curve. Figure 3-14 shows the MCFC which was used to optimize the gradation of the OAG mixes. It can be seen that the FSC mixes with high CPV (27.5% and 30.0%) had CF and WF values which plot within Zone IV, which indicates sandy gradation with excessive fines. The FSC mixes with lower CPV (22.5% and 25.0%) had CF and WF values which plot within Zone II, which indicates a well-graded distribution. However, all the OAG mixes had CF and WF values which plot within Zone II, which indicates a well-graded distribution.

As specified by the Standard Specifications for Road and Bridge Construction of FDOT, the slump requirement of Class II - Bridge Deck concrete is 2 to 4 inches. The required air content is to be equal to or less than 6 % in the Class II - Bridge Deck concrete.

Table 3-13 Class II - Bridge Deck concrete mixture design

Mix Design (Paste Vol.)	FSC-225 (22.5%)	FSC-250 (25.0%)	FSC-275 (27.5%)	FSC-300 (30.0%)	OAG-225 (22.5%)	OAG-250 (25.0%)	OAG-275 (27.5%)	OAG-300 (30.0%)
Mixing Date	8/27/19	9/17/19	10/15/19	7/9/19	8/27/19	9/17/19	10/15/19	7/9/19
Cement (lb/yd^3)	388	432	476	516	388	432	476	516
Fly Ash (lb/yd^3)	97	108	119	129	97	108	119	129
Fine Agg. (lb/yd^3)	1255.5	1222.5	1184	1139.9	1150.7	1052.6	974.9	897
Intermediate Agg. (lb/yd^3)	0	0	0	0	373.2	458.8	425.6	448.7
Coarse Agg. (lb/yd^3)	1908.2	1834	1775.1	1710.5	1630.1	1530.9	1538.1	1484.7
Air Entraining Admixture (oz/cwt)	0.07	0	0.15	0.22	0.07	0.1	0.15	0.31
TypeD (oz/cwt)	4.0	4.0	4.0	0	4.0	4.0	4.0	0
TypeF (oz/cwt)	8/27/19	9/17/19	10/15/19	7/9/19	8/27/19	9/17/19	10/15/19	7/9/19

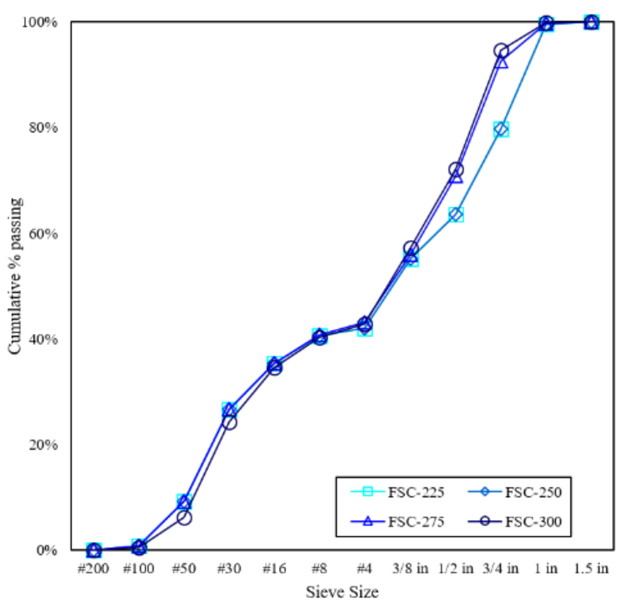

Figure 3-10 Gradation of aggregates used in the Class II - Bridge Deck mixes: FSC mixes

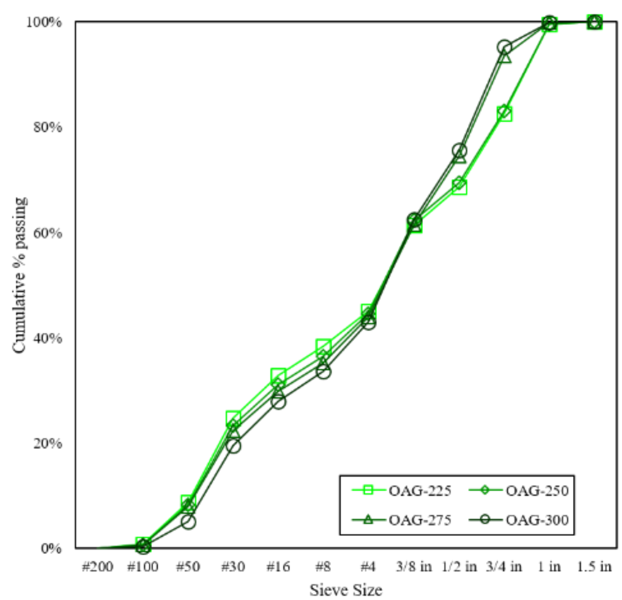

Figure 3-11 Gradation of aggregates used in the Class II - Bridge Deck mixes: OAG mixes

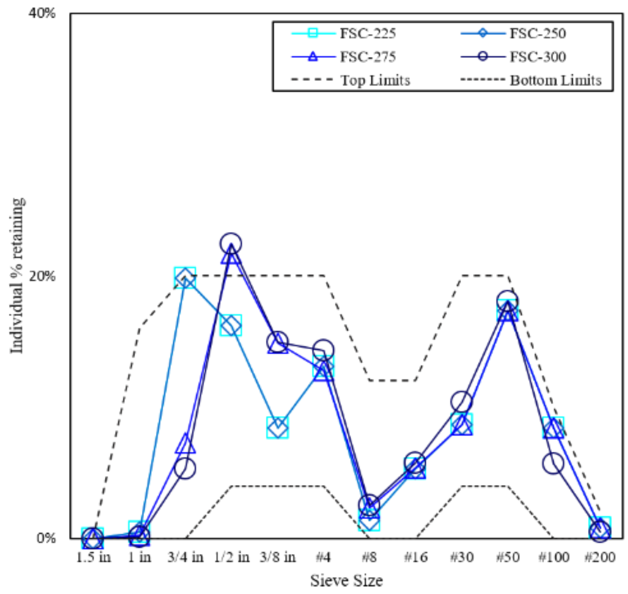

Figure 3-12 Individual percentage retained for the aggregate of the Class II - Bridge Deck mixes: FSC mixes

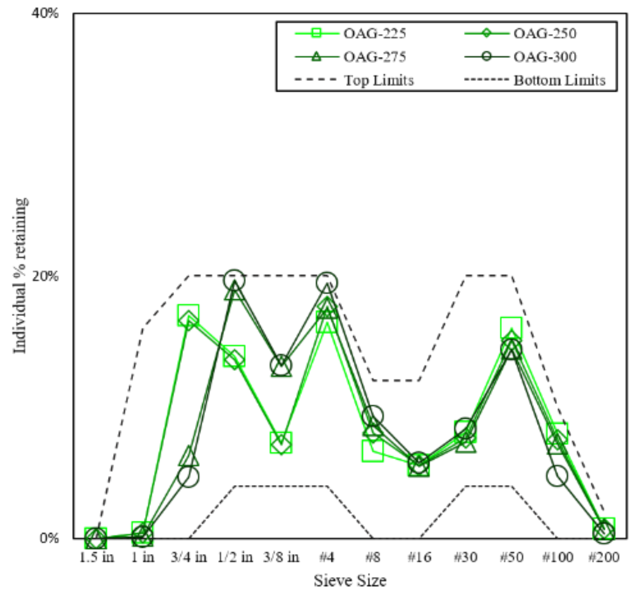

Figure 3-13 Individual percentage retained for the aggregate of the Class II - Bridge Deck mixes: OAG mixes

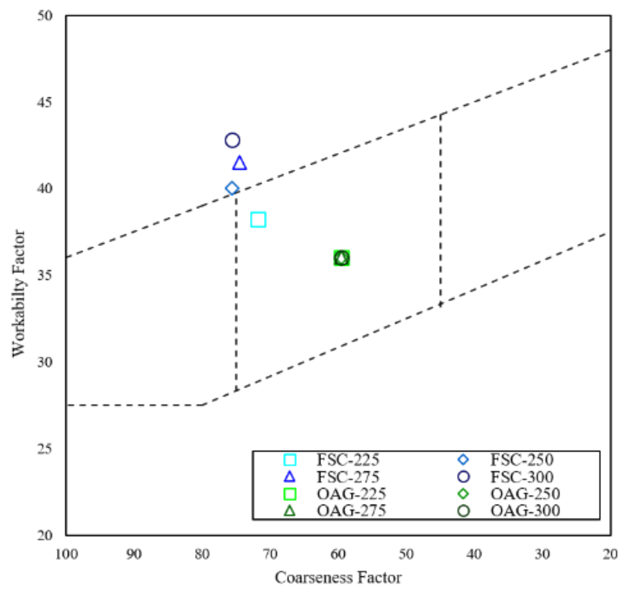

Figure 3-14 Modified coarseness factor chart for the aggregate of the Class II - Bridge Deck concrete mixes

3.3.5 Class IV Concrete Mix

The concrete mix to be evaluated is a Florida Class IV concrete with w/cm of 0.38, and total cementitious materials content (80% Type IL and 20% Class F Fly Ash) of 700 lb/yd^3.The estimated CPV of the reference mix is 30.0%. Table 3-14 shows the details of the mixture designs of Class IV concrete mixture.

Figures 3-15 and 3-16 show the aggregate gradation in terms of the cumulative percentage passing of each mixture. The FSC mixtures are typical concrete mixes with gap-graded aggregates. Figures 3-17 and 3-18 present the individual percentage retained chart of each mixture. The gradation of the OAG mixes plotted within the limits of the tarantula curve but the gradation of FSC mixes were out of the limits of the tarantula curve. Figure 3-19 shows the MCFC which was used to optimize the gradation of the OAG mixes. It can be seen that the FSC mixes with high CPV (27.5% to 32.5%) had CF and WF values which plot within Zone IV, which indicates sandy gradation with excessive fines. The FSC mixes with the lowest CPV (25.0%) had CF and WF values which plot within Zone II, which indicates a well-graded distribution. However, all the OAG mixes had CF and WF values which plot within Zone II, which indicates a well-graded distribution.

As specified by the Standard Specifications for Road and Bridge Construction of FDOT, the slump requirement of Class IV concrete is 2 to 4 inches. The required air content is to be equal to or less than 6 % in the Class IV concrete.

Table 3-14 Class IV concrete mixture design

Mix Design (Paste Vol.)	FSC-250 (25.0%)	FSC-275 (27.5%)	FSC-300 (30.0%)	FSC-325 (32.5%)	OAG-250 (25.0%)	OAG-275 (27.5%)	OAG-300 (30.0%)	OAG-325 (32.5%)
Mixing Date	11/26/19	10/8/19	11/20/19	7/30/19	11/26/19	10/8/19	11/20/19	7/30/19
Cement (lb/yd^3)	464	516	560	608	464	516	560	608
Fly Ash (lb/yd^3)	116	129	140	152	116	129	140	152
Fine Agg. (lb/yd^3)	1152.9	1103.8	1070.8	1023.9	1037.4	927.2	859.6	769.6
Intermediate Agg. (lb/yd^3)	0	0	0	0	434.4	464.1	428.6	453.7
Coarse Agg. (lb/yd^3)	1921.8	1838.6	1783.9	1708.8	1589.4	1535.8	1546.8	1488.3
Air Entraining Admixture (oz/cwt)	0.16	0.28	0.13	0.13	0.16	0.28	0.18	0.22
TypeD (oz/cwt)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
TypeF (oz/cwt)	4.31	3.09	3.12	0	4.31	3.09	2.64	0

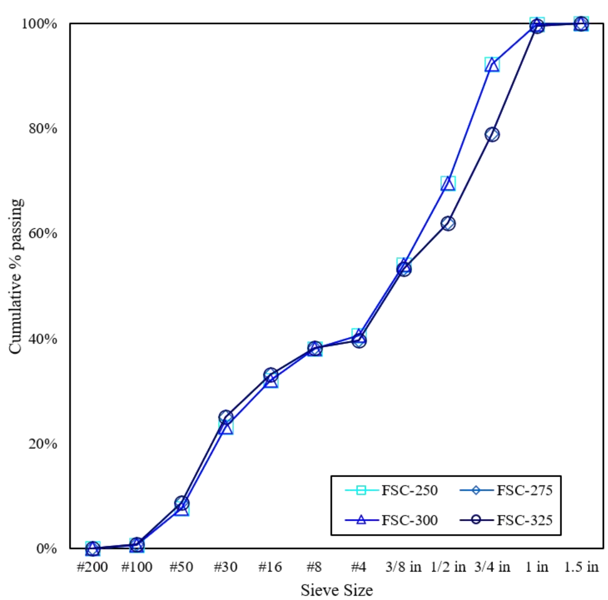

Figure 3-15 Gradation of aggregates used in the Class IV mixes: FSC mixes

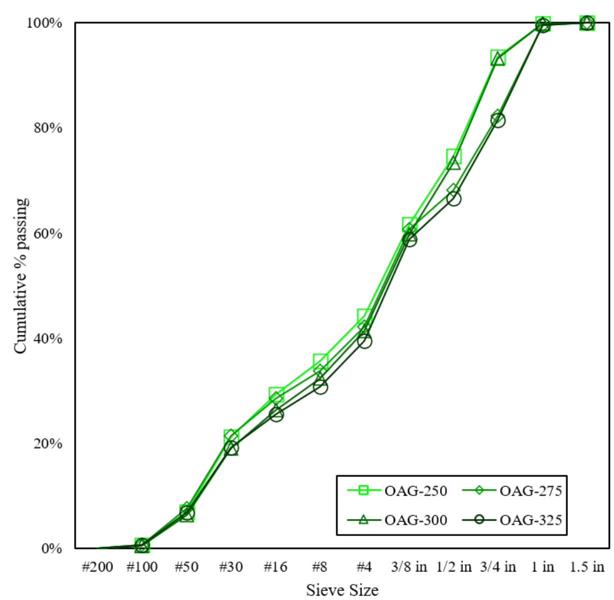

Figure 3-16 Gradation of aggregates used in the Class IV mixes: OAG mixes

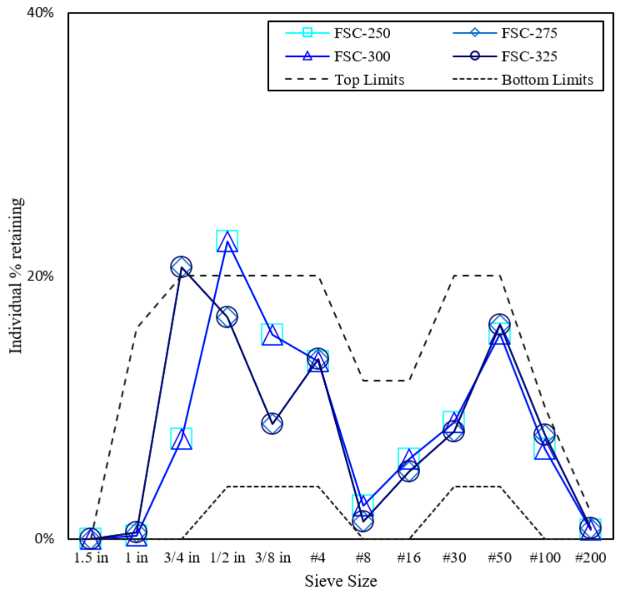

Figure 3-17 Individual percentage retained for the aggregate of the Class IV mixes: FSC mixes

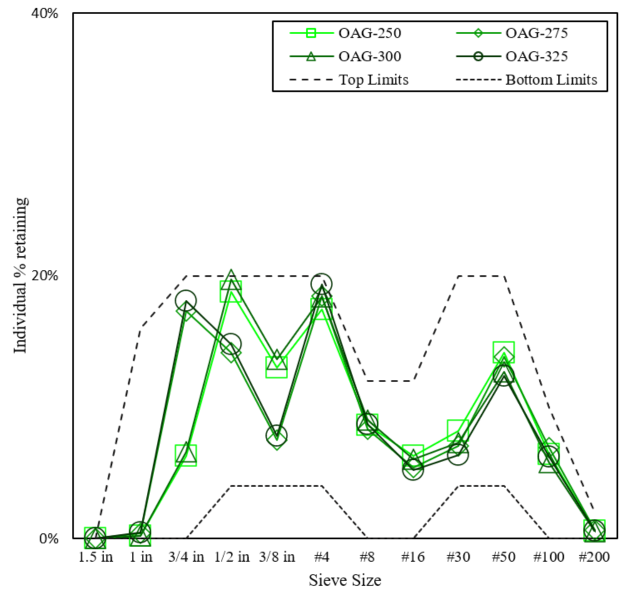

Figure 3-18 Individual percentage retained for the aggregate of the Class IV mixes: OAG mixes

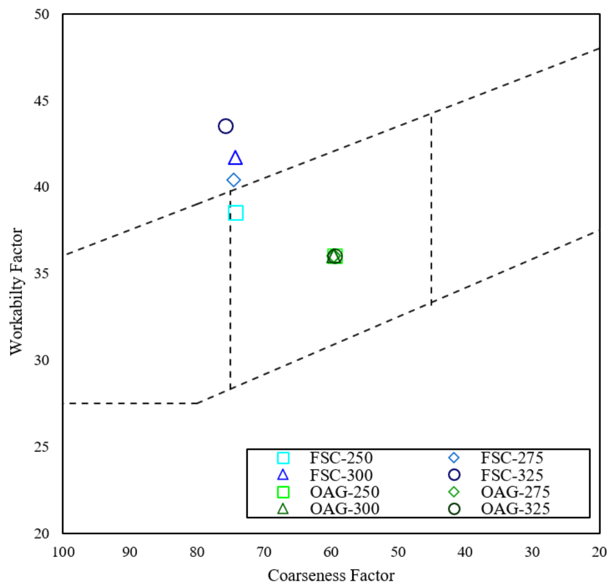

Figure 3-19 Modified coarseness factor chart for the aggregate of the Class IV concrete mixes

3.3.6 Class V Concrete Mix

The concrete mix to be evaluated is a Florida Class V concrete with w/cm of 0.32, and total cementitious materials content (80% Type IL and 20% Class F Fly Ash) of 765 lb/yd^3.The estimated CPV of the reference mix is 30.0%. Table 3-15 shows the details of the mixture designs of Class V concrete mixture.

Figures 3-20 and 3-21 show the aggregate gradation in terms of the cumulative percentage passing of each mixture. The FSC mixtures are typical concrete mixes with gap-graded aggregates. Figures 3-22 and 3-23 present the individual percentage retained chart of each mixture. The gradation of the OAG mixes plotted within the limits of the tarantula curve but the gradation of FSC mixes were out of the limits of the tarantula curve. Figure 3-24 shows the MCFC which was used to optimize the gradation of the OAG mixes. It can be seen that the FSC mixes with high CPV (30.0% and 32.5%) had CF and WF values which plot within Zone IV, which indicates sandy gradation with excessive fines. The FSC mixes with lower CPV (27.5%) had CF and WF values which plot within Zone I, which indicates a gap-graded gradation. However, the FSC mixes with the lowest CPV (25.0%) had CF and WF values which plot within Zone II, which indicates a well-graded distribution. All the OAG mixes had CF and WF values which plot within Zone II, which indicates a well-graded distribution.

As specified by the Standard Specifications for Road and Bridge Construction of FDOT, the Slump requirement of Class V concrete is 2 to 4 inches. The required air content is to be equal to or less than 6 % in the Class V concrete.

Table 3-15 Class V concrete mixture design

Mix Design (Paste Vol.)	FSC-250 (25.0%)	FSC-275 (27.5%)	FSC-300 (30.0%)	FSC-325 (32.5%)	OAG-250 (25.0%)	OAG-275 (27.5%)	OAG-300 (30.0%)	OAG-325 (32.5%)
Mixing Date	8/6/19	10/22/19	10/1/19	11/29/19	8/6/19	10/22/19	10/1/19	11/29/19
Cement (lb/yd^3)	508	560	612	664	508	560	612	664
Fly Ash (lb/yd^3)	127	140	153	166	127	140	153	166
Fine Agg. (lb/yd^3)	1367.7	1126.9	991.6	958.9	986.6	890.9	793.6	718.5
Intermediate Agg. (lb/yd^3)	0	0	0	0	390.6	444.8	474.1	438.2
Coarse Agg. (lb/yd^3)	1708.8	1837.8	1846	1781.1	1665.9	1605.5	1553.8	1561.3
Air Entraining Admixture (oz/cwt)	0.50	0.99	0.35	0.40	0.50	0.99	0.99	0.51
TypeD (oz/cwt)	5.0	5.0	4.0	4.0	5.0	5.0	4.0	4.0
TypeF (oz/cwt)	3.8	5.93	3.92	3.01	3.79	5.87	3.47	3.01

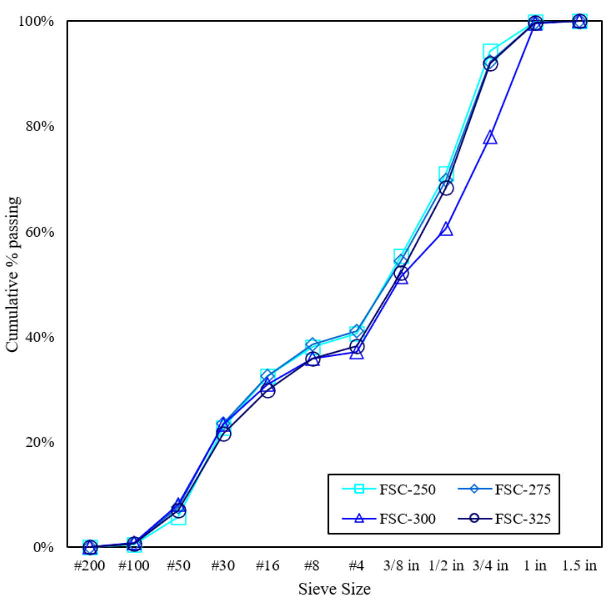

Figure 3-20 Gradation of aggregates used in the Class V mixes: FSC mixes

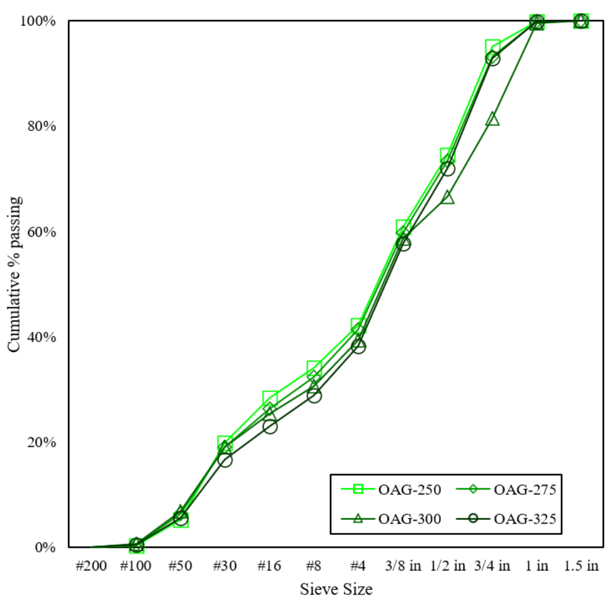

Figure 3-21 Gradation of aggregates used in the Class V mixes: OAG mixes

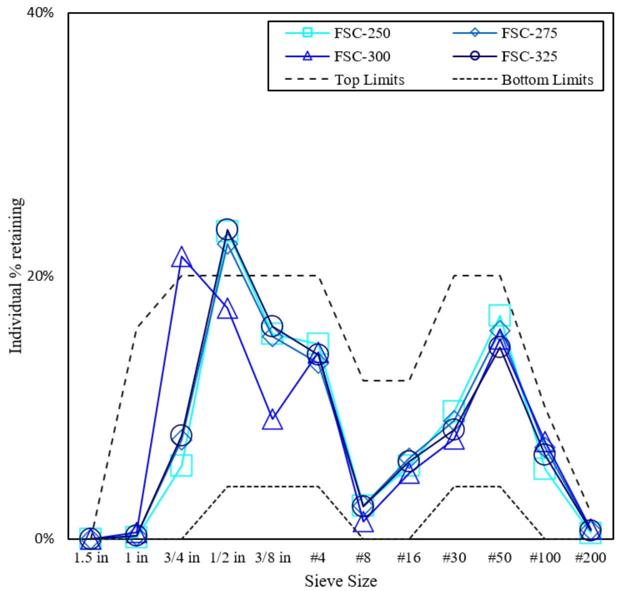

Figure 3-22 Individual percentage retained for the aggregate of the Class V: FSC mixes

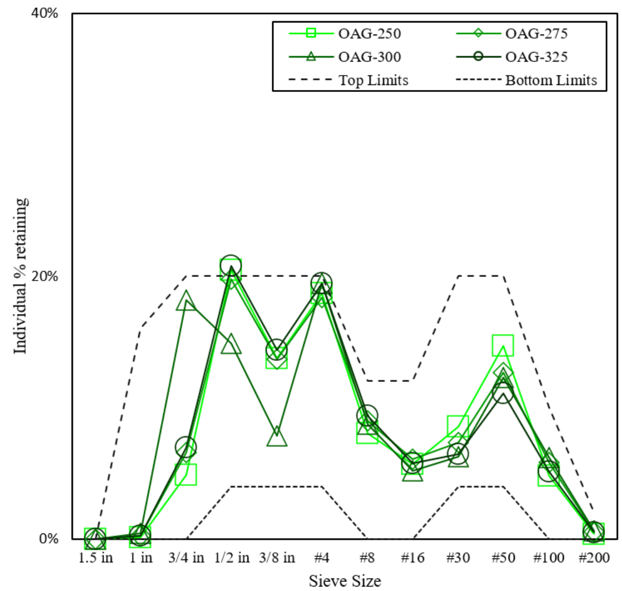

Figure 3-23 Individual percentage retained for the aggregate of the Class V: OAG mixes

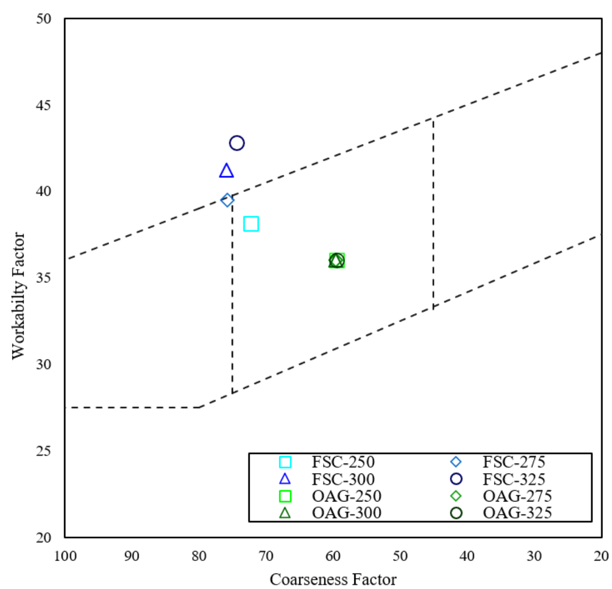

Figure 3-24 Modified coarseness factor chart for the aggregate of the Class V concrete mixes

CHAPTER 4
TEST METHODS

4.1 Fabrication and Curing of Concrete Specimens

The following steps were performed to produce concrete in the laboratory.

(1) Fill cloth bags with the coarse aggregate, intermediate aggregate, and fine aggregate required for the mixture.

(2) Submerge the coarse and intermediate aggregates in water for at least 24 hours, remove them from the water tank, and allow to drain inside the laboratory for 1.5 hours before weighing.

(3) Fill the cloth bags with the fine aggregate and dry for at least 24 hours in the oven at 230°F. Let them cool for another 24 hours inside the lab.

(4) Use the weighing scale to weigh all the materials for mixing.

(5) Collect a specimen of each aggregate and dry overnight in the oven at 230°F for moisture content determination.

(6) Weigh concrete ingredients and keep them in lidded buckets the day before mixing.

(7) Adjust the mixing water based on the moisture content of the aggregates prior to mixing.

(8) Place all the aggregates in the drum mixer and mix for 30 seconds.

(6) Add an air-entraining admixture to the mixing water.

(7) Add 1/4 of the mixing water with an air-entraining admixture into drum mixer and mix for another 30 seconds.

(8) Place the fly ash into the drum mixer, add another quarter of mixing water and mix for 30 seconds.

(9) Place the cement into the mixer and add the remaining of mixing water and mix for 30 seconds.

(10) Add the water-reducing admixture (Type D or Type F) into the mixer and mix for 3 minutes, followed by a 2-minute rest, followed by a 4-minute mixing.

(11) Perform the fresh concrete property tests to ensure workability, including air content, unit weight, temperature, and bleeding.

(12) If workability is not achieved, add more water-reducing admixture to the mix and mix it for another 3 minutes.

After the concrete was produced, some of its portions were immediately used to perform the fresh concrete property tests, including air content, unit weight, temperature, and bleeding. The remaining concrete was used to fabricate different concrete specimens as follows:

(1) Cylinders, prisms, and beams were cast.

(2) Molds were filled with concrete in two layers, and each layer was vibrated for about 45 seconds. If the concrete was stiff, it was vibrated for some additional time to ensure proper consolidation.

(3) A vibrating table was used to consolidate all specimens.

(4) The concrete specimens were covered with polyethylene sheets to prevent loss of moisture as shown in Figure 4-1.

(5) Specimens were removed from the molds after 24 hours and placed in the curing room, as shown in Figure 4-2.

Figure 4-1 Concrete specimens covered with polyethylene sheet

Figure 4-2 Curing room

4.2 Tests on Fresh Concrete

4.2.1 Slump

Slump test (Figure 4-3) was performed immediately after the concrete was produced to verify the workability of the mixtures. If the workability was not achieved, then some water-reducing admixture was added to make the concrete more workable. Because the target slump was achieved, the remaining tests on fresh concrete were performed in accordance with the ASTM

C143/C143M method. In the fresh concrete, the water content may increase or decrease without any apparent change in the slump test result. The slump of the concrete will be influenced by the properties of aggregates, the aggregate grading, concrete mix proportions, air content, temperature, and admixture. Therefore, the w/cm cannot be a determining factor to decide the slump of the fresh concrete. The slump of the fresh concrete is measured and reported to the nearest 0.25 inches.

Figure 4-3 Slump test set

4.2.2 Air Content

In this research, the air content of fresh concrete was tested following the ASTM C231/C231M method. Figure 4-4 shows the apparatus for measuring the air content of the fresh concrete. This method can determine the amount of the air voids in concrete. The air voids in concrete will provide a source of internal pressure relief within the concrete to accommodate the pressures that develop as ice crystals form in the pores and capillaries of the concrete. On the other hand, the strength of the concrete will be influence by the air content of the concrete. However, Florida concrete does not have to consider freezing and thawing conditions. Based on the FDOT Standard Specifications for Road and Bridge Construction, the air content of Florida-designed concrete is to be equal to or less than 6 %.

Figure 4-4 Type B meter of air content test

4.2.3 Density

The density test is very important to control the quality of fresh concrete. When the concrete mixes have a lower density than usual, the concrete may have higher air content, higher water content, or lighter aggregate in the concrete. Conversely, the higher density of the concrete would indicate the reverse of the above-mentioned concrete characteristics. Moreover, a change in density could affect the pumpability, placeability, finishability, and strength of all types of concrete. Based on the ASTM C138/C138M method (Figure 4-5), the density of the concrete was calculated by dividing the net mass of concrete by the volume of the measure.

$$D = \frac{M_c - M_m}{V_m}$$

where,

D = Density of Concrete, lb/ft^3,

M_c = The mass of the measure filled with concrete, lb,

M_m= The mass of measurement, lb,

V_m= The volume of measurement.

Figure 4-5 The density measurement

4.2.4 Temperature

The temperature of fresh concrete is one of the most important factors influencing the quality, time of set, strength, and other properties of the concrete. A concrete with a high initial temperature will have higher than normal early strength and lower than normal later strength. The temperature will influence the hydration of cement. Moreover, the temperature of the concrete is used to indicate the type of curing method in the field. According to ASTM C1064/C1064M, the temperature of fresh concrete of each concrete should be as identical as possible when different types of concrete are evaluated. The temperature measuring device (Figure 4-6) shall be capable of accurately measuring the temperature of the fresh concrete to ± 1°F.

Figure 4-6 Temperature measuring device of fresh concrete test

4.2.5 Bleeding Water of Concrete

ASTM C232/232M (Figure 4-7) test method was used to measure the relative quantity of mixing water that will bleed from a sample of fresh concrete. The w/cm influences bleeding water of concrete. Typically, a higher w/cm can lead to excessive bleeding. The bleed rate is significantly influenced by the types of cement and fine aggregates. Using supplementary cementitious materials can decrease the bleeding rate of concrete. If the bleeding water of the concrete is too high, it will create weak horizontal construction joints and surface of the concrete. The bleeding water was calculated as the volume of bleeding water per unit area of the concrete surface.

$$V = {V_1}/{A} \qquad 4\text{-}1$$

where,

V = bleed water per unit area of surface,

V_1 = volume of bleed water measured during the selected time interval, mL,

A = area of exposed concrete, inches^2.

Figure 4-7 Bucket for bleeding water of concrete test

4.2.6 Semi-adiabatic Temperature

The Semi-adiabatic temperature of concrete (Figure 4-8) test was performed on 4 × 8 inches concrete cylinders. Two specimens were immediately tested after casting. Each specimen was maintained under semi-adiabatic conditions up to 7 days without any external temperature control. Thus, the temperature increase in the concrete specimen was due only to the heat of hydration minus the heat lost through the thermal insulation. The apparatus will keep collating temperature data until the sample reaches a constant temperature (ambient temperature) within 7 days.

Figure 4-8 Semi-adiabatic temperature test-Calmetrix F-Cal concrete

4.2.7 pH test for concrete

The pH test of concrete was performed on fresh concrete. Two test methods were used. First, a universal indicator paper (UIP) (Figure 4-9) was used to measure the pH value of the concrete by the color. The UIP was attached to the fresh concrete for 30 seconds, and the color of the indicator paper was checked. Figure 4-10 showed the relative pH value and its corresponding color of the UIP. Secondly, the hardened concrete sample was tested in accordance with the FM 5-550 Florida method of test for pH soil and water (Figure 4-11). This method is typically used for the determination of pH in the coarse aggregate, soil, and water by using a pH/mV meter with automatic temperature compensation and a combination electrode that includes a silver chloride reference electrode, a glass bulb indicating electrode and a thermocouple. The hardened concrete sample was ground into a powder to be tested.

Figure 4-9 Universal indicator paper test on concrete

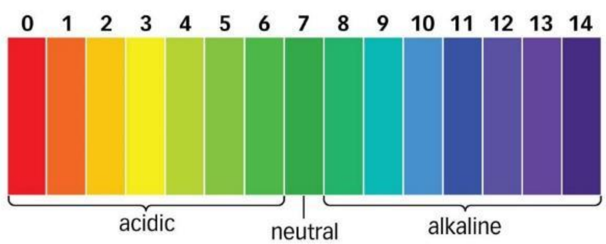

Figure 4-10 Scale of universal indicator paper

Figure 4-11 FM5-550 pH test apparatus

4.3 Tests on Hardened Concrete

4.3.1 Compressive Strength

The compressive strength test (Figure 4-12) was performed on 4 × 8 inch concrete cylinder specimens in accordance with ASTM C39. Before conducting this test, both ends of the cylinder were ground to ensure uniform load during testing. Since the both ends of the specimens were ground, no capping compound or rubber pads were used in this research. The load was continuously applied without stopping or shocking at the stress rate of 35 ± 7 psi per second. The compressive strength of the specimen was calculated by dividing the maximum load carried by the specimen during the test by the average area of the cylinder's cross section.

According to ASTM C39, there are six types of fracture patterns in crushed concrete cylinder. Figure 4-13 demonstrates 6 different types of fracture patterns. A majority of the specimens showed Type 1 and Type 4 fracture patterns in this research.

$$f = \frac{4P}{\pi D^2} \tag{4-2}$$

where,

f = Compressive strength, psi,

P= Maximum load, lbf,

D = Average measured diameter, inches.

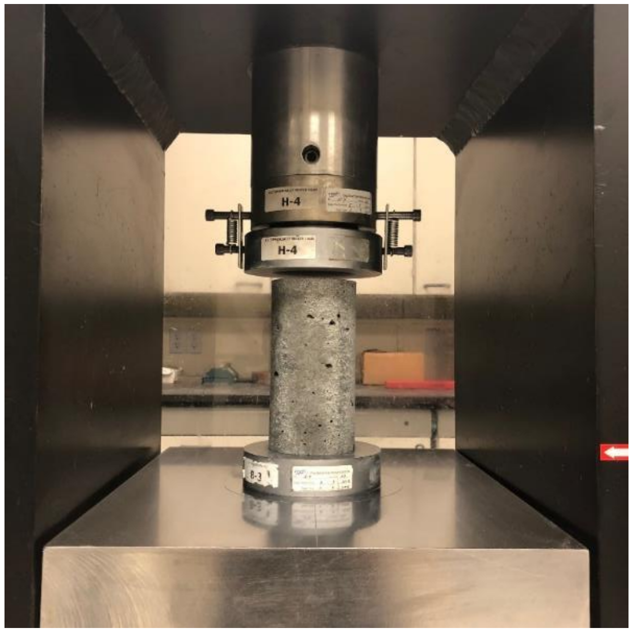

Figure 4-12 Compressive strength test apparatus

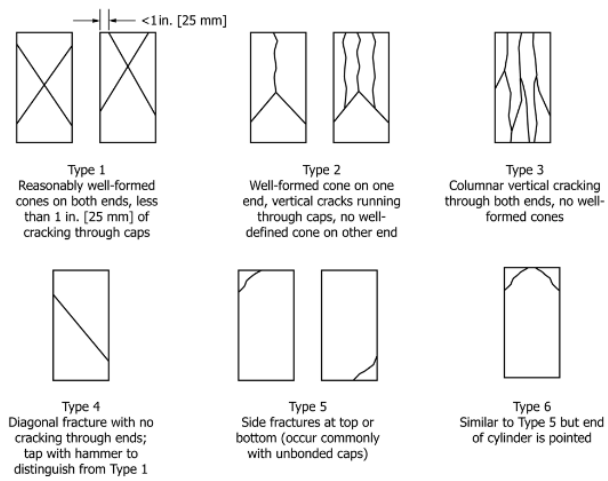

Figure 4-13 Typical fracture patterns in compressive strength test on concrete specimens

4.3.2 Modulus of Rupture

The modulus of rupture (Figure 4-14) test was performed on 4 × 4 ×14 inch concrete beam specimens in accordance with ASTM C78. Before conducting this test, the loading surface and the edges of specimens were ground by using a hand grinding stone. The grinding ensured that the applied load was uniform. The modulus of rupture was determined according to the type of fracture in the specimens.

If fracture occurred in the tension surface within the middle third of the span length, the modulus of rupture was calculated using the following equation:

$$R = \frac{PL}{bd^2} \tag{4-3}$$

where,

R = Modulus of rupture, psi,

P = Maximum load, lbf,

L = Span length, inches,

b = Average width of the specimen at fracture, inches,

d = Average depth of the specimen at fracture, inches.

If fractures occurred in the tension surface outside of the middle third of the span length by more than 5% of the span length, the modulus of rupture was calculated using the following equation:

$$R = \frac{3Pa}{bd^2} \tag{4-4}$$

where,

a = average distance between the line of fracture and the nearest support measured on the tension surface of the beam, inches.

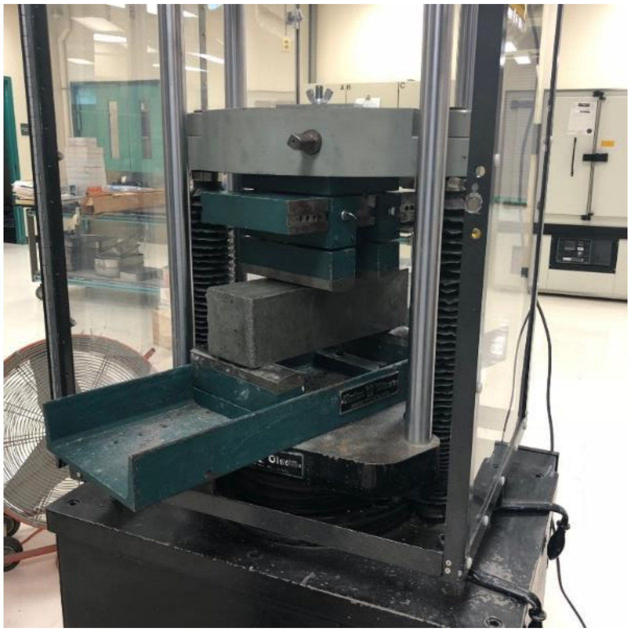
Figure 4-14 Modulus of rupture test apparatus

4.3.3 Modulus of Elasticity and Poisson's Ratio

The modulus of elasticity (Figure 4-15) and Poisson's ratio tests (Figure 4-15) were performed on 4 × 8 inch concrete cylinder specimens in accordance with ASTM C469. Before conducting this test, both ends of the cylinder were ground to ensure uniform load during testing. Since both ends of the specimens were ground, no capping compound or rubber pads were used in this research. The load was continuously applied without stopping or shocking at the stress rate 35 ± 7 psi per second until reaching 40% of the ultimate compressive strength, which was determined from the compressive strength test. The test was carried out on a compressive testing machine that had connections to the load cell and the linear variable differential transformer (LVDT). The equation used to calculate the elastic modulus is as follows.

$$E = \frac{(S_2 - S_1)}{(\varepsilon_2 - 0.000050)} \qquad 4\text{-}5$$

where,

E = Modulus of elasticity, psi,

S_2 = Stress corresponding to 40% of ultimate load, psi,

S_1 = Stress corresponding to a longitudinal strain of 50 millionths, psi,

ε_2 = Longitudinal strain produced by stress S_2.

Poisson's ratio was measured using the horizontal LVDT that measures the horizontal strain of the specimen. The Poisson's ratio was calculated using the following equation:

$$\mu = \frac{(\varepsilon_{\tau 2} - \varepsilon_{\tau 1})}{(\varepsilon_2 - 0.000050)} \qquad 4\text{-}6$$

where,

μ = Poisson's ratio,

$\varepsilon_{\tau 1}$ = Transverse strain at midheight of the specimen produced by stress S_1,

$\varepsilon_{\tau 2}$ = Transverse strain at midheight of the specimen produced by stress S_2.

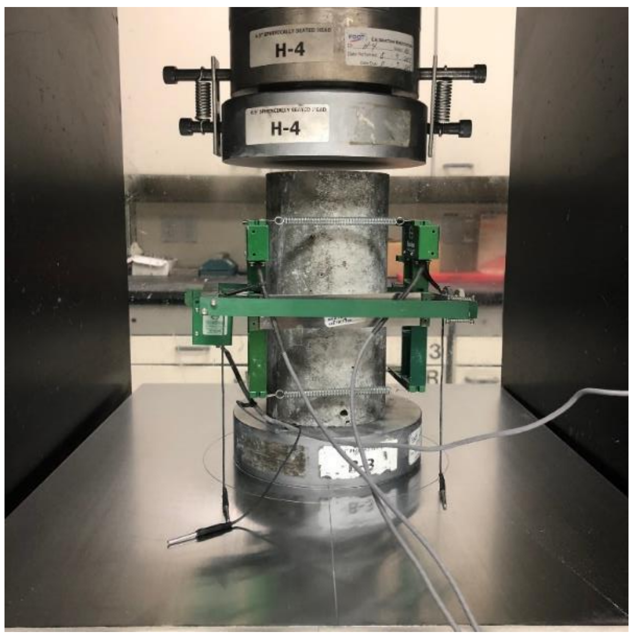

Figure 4-15 Modulus of elasticity and Poisson's ratio test apparatus

4.3.4 Drying Shrinkage

The drying shrinkage tests (Figure 4-16) were performed on 3 ×3 ×11.25 inch concrete prism specimens in accordance with ASTM C157. The specimens were removed from the molds after 24 hours of concrete mixing. An initial reading was immediately taken with a length comparator. Six specimens were placed in the moist room until 28 days. The length change of specimens at any age after the initial comparator reading was calculated as follows,

$$\Delta L_x = \frac{CRD - initialCRD}{G} \times 100 \qquad 4\text{-}7$$

where,

ΔL_x= length change of specimen at any age, %,

CRD = difference between the comparator reading of the specimen and the reference bar at any age, inches,

G = the gage length, inches.

Figure 4-16 The drying shrinkage test apparatus

4.3.5 Rapid Chloride Permeability

Permeability is a key parameter to evaluate the durability of concrete. Table 4-1 presents the correspondence between the results of RCPT (ASTM C1202, 2018). The rapid chloride penetration tests (Figure 4-17) were performed on 4 × 8 inchs concrete cylinder specimens in accordance with ASTM C1202. The test consists of monitoring the amount of electrical current passed through 50-mm (2 inches) thick slices of specimens. After the specimen preparation, a rapid-setting coating was brushed onto the side surfaces of each specimen. The specimens were placed in the vacuum desiccator after 24 hours. After 3 hours, with the vacuum pump still running, the water stopcock was opened to supply sufficient water into the container to cover the specimens. The water stopcock was closed, and the vacuum pump was operated for one additional hour. The pump was turned off and the specimens were soaked in the water for 18 ± 2 hours. The specimens were mounted in the cells and tested according to the electrical block diagram (Figure 4-18). The charge passed through each specimen was calculated as follows:

$$Q = 900(I_0 + 2I_{30} + 2I_{60} + \cdots + 2I_{300} + +2I_{360}) \quad \text{4-8}$$

where,

Q = charge passed, coulombs,

I_0 = current immediately after voltage is applied, amperes,

I_t = current at t min after voltage is applied, amperes.

Table 4-1 The permeability level of RCPT (ASTM C1202, 2018)

RCPT (Coulombs)	Permeability
>4,000	High (H)
2,000-4,000	Moderate (M)
1,000-2,000	Low (L)
100-1,000	Very Low (VL)
<100	Negligible (N)

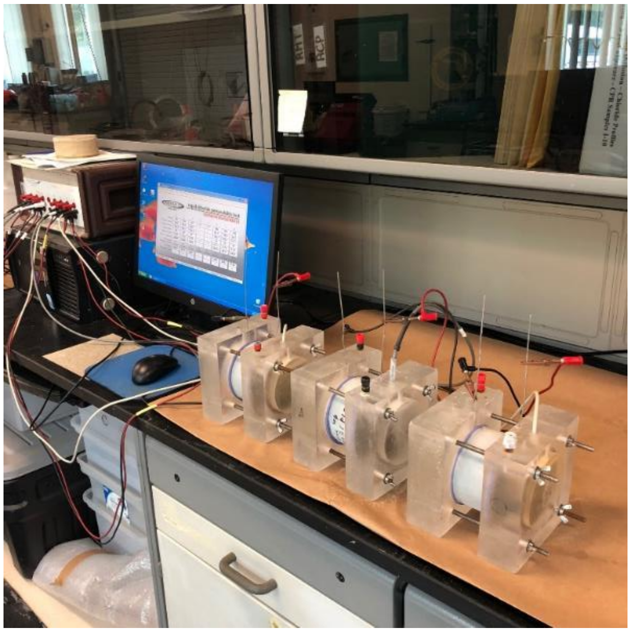

Figure 4-17 Rapid chloride penetration test apparatus

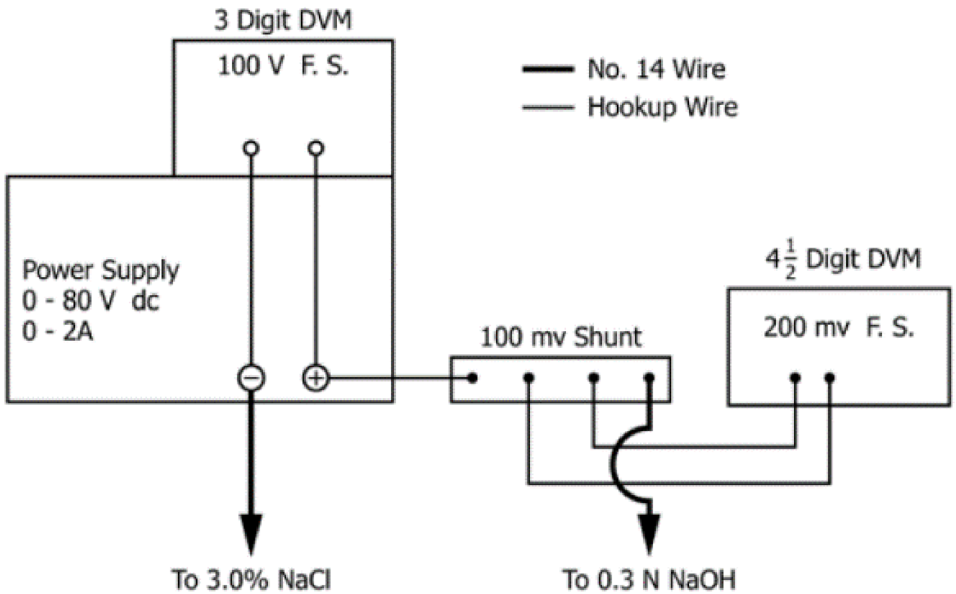

Figure 4-18 Electrical block diagram

4.3.6 Surface Resistivity

Table 4-1 presents the correspondence between the results of RCPT (AASHTO TP95, 2011).The surface resistivity measurements (Figure 4-19) were performed on 4 × 8 inches concrete cylinder specimens in accordance with AASHTO TP95. The test method consists of measuring the resistivity of specimens using a Wenner probe array. An alternating current (AC) potential difference was applied by the surface resistivity apparatus at the outer pins of the Wenner array generating current flow in the concrete. The resistivity, in kilo-ohm-centimeter (kΩ-cm).has been found to be related to the resistance of the specimen to chloride ion penetration. The average resistivity and the percent relative standard deviation (%RSD) were calculated for each sample in the set.

Table 4-2 The permeability level of SR (AASHTO TP95, 2011)

Surface Resistance	Permeability
(kΩ-cm)	High (H)
<12	Moderate (M)
21-12	Low (L)
21-37	Very Low (VL)
37-254	Negligible (N)

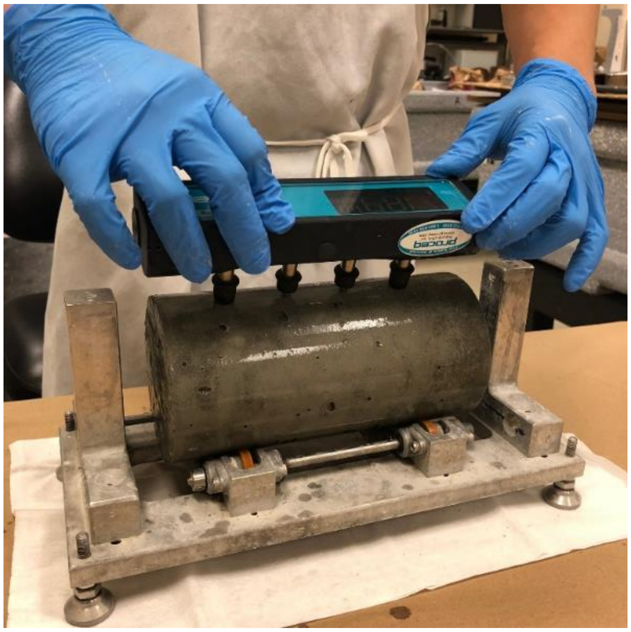
Figure 4-19 Surface resistivity test (Wenner probe array)

CHAPTER 5
TEST RESULTS

5.1 Results of Tests on Fresh Concrete

5.1.1 Class II Concrete

The results of fresh concrete tests on the Florida Class II concrete are shown in Table 5-1. The following section presents the fresh concrete properties of the Class II concrete mixtures. The slump test results showed that the concrete mixes with lower CPV have relatively lower slump. The slump of concrete was higher than the target range when the paste volume was above 30.0%. Figure 5-1 shows that the slump of the fresh concrete improves with increasing CPV of concrete. OAG mixes have higher slump than the FSC mixes. This indicates that OAG technique can be used to improve the slump of the fresh concrete.

The air content results of all mixtures were in the range of 1% to 6%. Figure 5-1 shows the air content of the FSC and OAG mixes. At the same CPV, the OAG mixes have lower air content than the FSC mixes. Because the OAG mixes have better packing density than the FSC mixes, the air content of the OAG mixes is lower. The density results were in the normal concrete range and the bleeding results of all mixtures are acceptable. The fresh concrete results showed that the minimum CPV in Class II concrete can be reduced to 22.5%.

Table 5-1 Fresh concrete properties of Class II concrete evaluated in this research study

Concrete Mix (w/cm=0.50)	Paste Volume (%)	Slump ASTM C143 (inches)	Air Content ASTM C231 (%)	Temp. ASTM C1064 (°F)	Density ASTM C138 (lb/ft^3)	Final Bleeding ASTM C232 (ml/inches^2)	pH test UIP (Color/pH)	pH test FM 5-550 (pH)
FSC225	22.5	1.50	4.5	72	139.76	0.00	/ 13	12.62
FSC250	25.0	1.50	2.7	73	142.48	0.07	/ 13	12.41
FSC275	27.5	1.50	2.3	74	141.52	0.04	/ 13	12.37
FSC300	30.0	7.00	3.3	73	139.04	0.08	/ 13	12.50
OAG225	22.5	2.00	4.0	73	141.44	0.02	/ 13	12.63
OAG250	25.0	2.50	2.2	73	142.72	0.06	/ 13	12.48
OAG275	27.5	2.25	1.9	73	141.84	0.05	/ 13	12.42
OAG300	30.0	8.00	3.6	74	138.80	0.10	/ 13	12.60

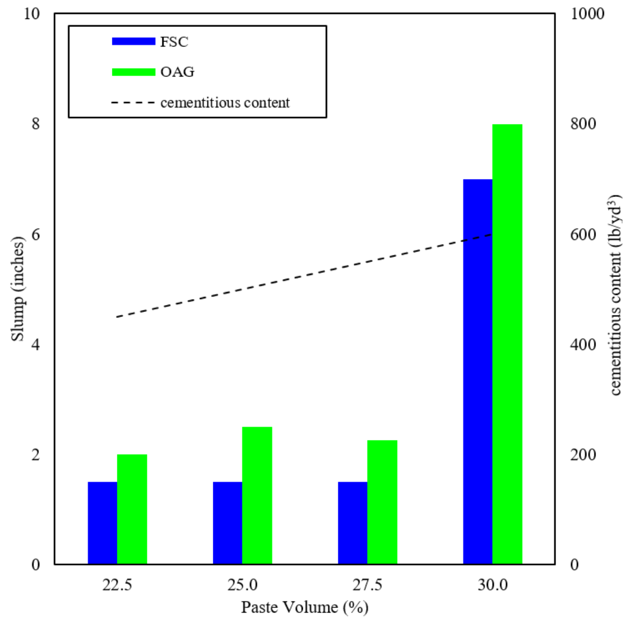

Figure 5-1 Slump of FSC and OAG mixes in Class II concrete

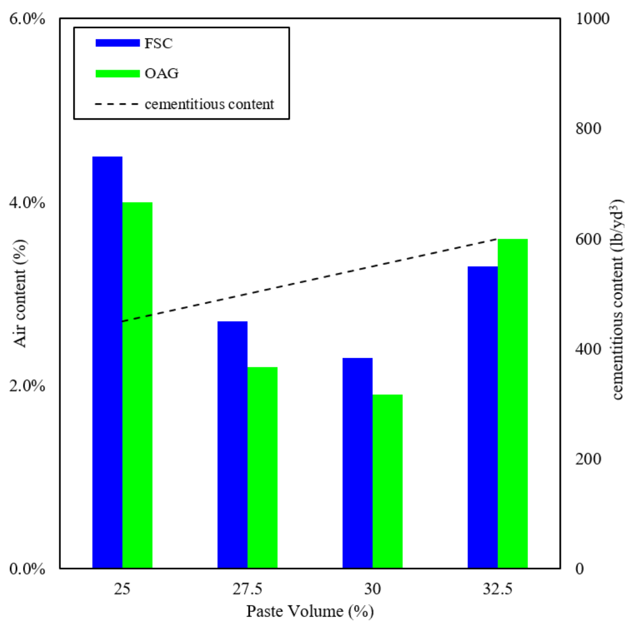

Figure 5-2 Air content of FSC and OAG mixes in Class II concrete

5.1.2 Class II Bridge Deck Concrete

The results of fresh concrete tests on Class II (Bridge Deck) mixes are shown in Table 5-2. The following section presents the fresh concrete properties of the Class II (Bridge Deck) concrete mixtures. The slump test results showed that the concrete mixes with lower CPV have lower slump. Figure 5-3 shows that the slump of the fresh concrete improves with increasing CPV of concrete. The OAG mixes have higher slump than typical FSC mixes. OAG technique can be used to improve the slump of the fresh concrete.

The air content of all mixtures was in the range of 1 to 6%. For the concretes with a higher CPV, more bleeding was observed. Figure 5-4 shows the air content of the FSC and OAG mixes. At the same CPV, the OAG mixes have lower air content than the FSC mixes. The density results were in the normal concrete range and the bleeding results of all mixtures are acceptable. The fresh concrete results showed that the minimum CPV in Class II- Bridge Deck concrete can be reduced to 22.5%.

Table 5-2 Fresh concrete properties of Class II - Bridge Deck concrete evaluated in this research study

Concrete Mix (w/cm=0.44)	Paste Volume (%)	Slump ASTM C143 (inches)	Air Content ASTM C231 (%)	Temp. ASTM C1064 (°F)	Density ASTM C138 (lb/ft^3)	Final Bleeding ASTM C232 ($ml/inches^2$)	pH test UIP (Color/pH)	pH test FM 5-550 (pH)
FSC225	22.5	2.25	3.5	73	141.04	0.09	/ 13	12.67
FSC250	25.0	4.25	3.2	74	142.16	0.00	/ 13	12.52
FSC275	27.5	4.00	3.5	74	140.96	0.00	/ 13	12.42
FSC300	30.0	4.25	1.3	74	142.96	0.55	/ 13	12.49
OAG225	22.5	4.00	4.2	73	140.80	0.10	/ 13	12.69
OAG250	25.0	4.00	3.0	74	142.24	0.02	/ 13	12.54
OAG275	27.5	4.50	5.0	73	137.76	0.00	/ 13	12.49
OAG300	30.0	5.00	1.0	74	143.60	0.57	/ 13	12.61

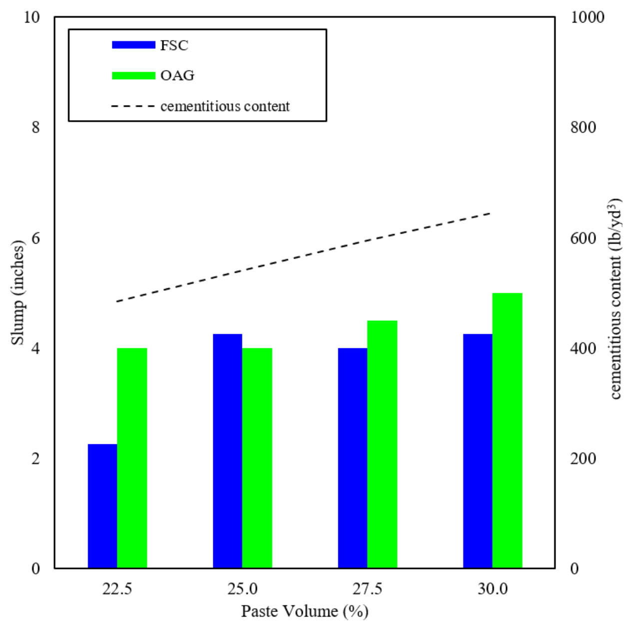

Figure 5-3 Slump of FSC and OAG mixes in Class II - Bridge Deck concrete

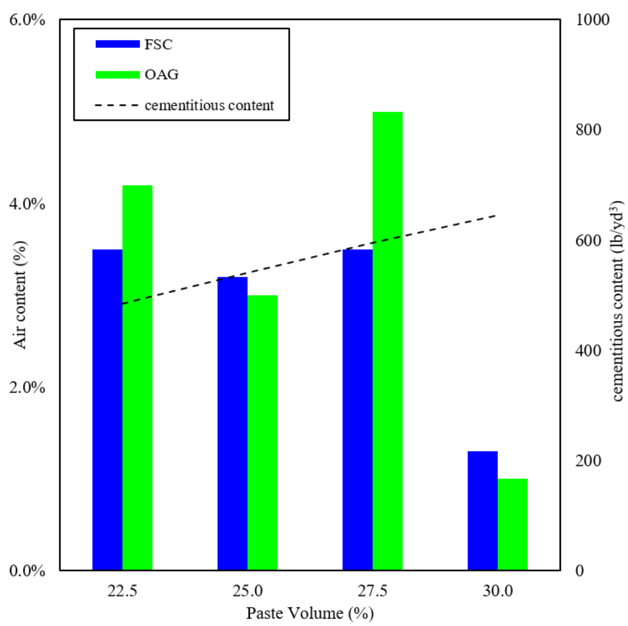

Figure 5-4 Air content of FSC and OAG mixes in Class II - Bridge Deck concrete

5.1.3 Class IV Concrete

The results of fresh concrete tests on the Class IV mixes are shown in Table 5-3. The following section presents the fresh concrete properties of the Class IV concrete mixtures. The slump test results showed that the concrete mixes with a lower CPV have lower slump. However, the OAG mixes show better slump for the mixes with low CPV. Figure 5-5 shows that the slump of the fresh concrete improves with increasing CPV of concrete. The OAG mixes have higher slump than the typical FSC mixes. OAG technique can be used to improve the slump of the fresh concrete.

The air content of all mixtures was in the range of 1 to 6%. For the concretes with a higher CPV, more bleeding was observed. Figure 5-6 shows the air content of the FSC and OAG mixes. At the same CPV, the OAG mixes have lower air content than the FSC mixes. The density results were in the normal concrete range and the bleeding results of all mixtures are acceptable. The fresh concrete results showed that the minimum CPV in Class IV concrete can be reduced to 25.0%.

Table 5-3 Fresh concrete properties of Class IV concretes evaluated in this research study

Concrete Mix (w/cm=0.38)	Paste Volume (%)	Slump ASTM C143 (inches)	Air Content ASTM C231 (%)	Temp. ASTM C1064 (°F)	Density ASTM C138 (lb/ft^3)	Final Bleeding ASTM C232 ($ml/inches^2$)	pH test UIP (Color/pH)	pH test FM 5-550 (pH)
FSC250	25.0	2.25	3.4	71	142.00	0.00	/ 13	12.47
FSC275	27.5	3.25	2.9	73	142.90	0.00	/ 13	12.45
FSC300	30.0	3.00	1.9	71	143.44	0.03	/ 13	12.38
FSC325	32.5	3.00	1.7	75	142.64	0.04	/ 13	12.70
OAG250	25.0	2.50	3.1	71	142.64	0.00	/ 13	12.49
OAG275	27.5	4.50	3.1	74	141.84	0.00	/ 13	12.52
OAG300	30.0	3.00	1.7	72	143.36	0.04	/ 13	12.40
OAG325	32.5	5.00	1.2	75	141.44	0.06	/ 13	12.63

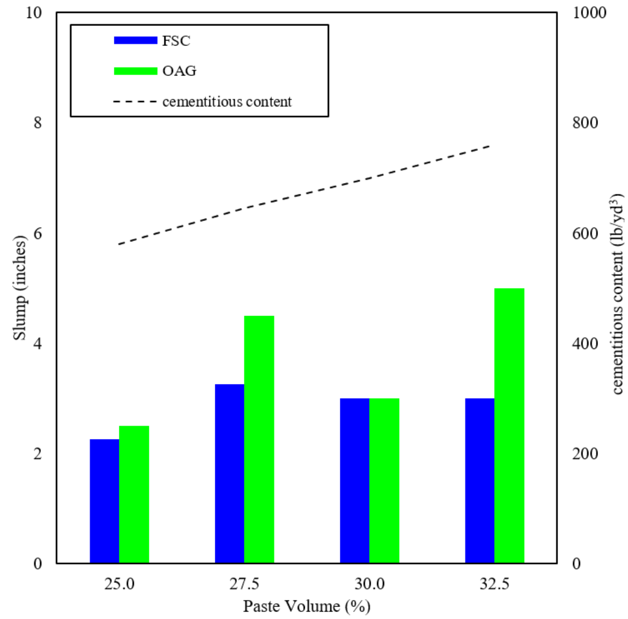

Figure 5-5 Slump of FSC and OAG mixes in Class IV concrete

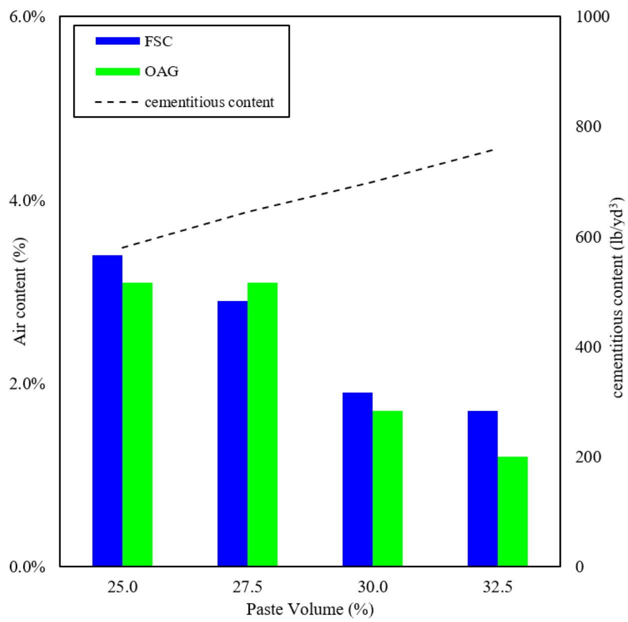

Figure 5-6 Air content of FSC and OAG mixes in Class IV concrete

5.1.4 Class V Concrete

The results of fresh concrete tests on the Class V mixes are shown in Table 5-4. The following section presents the fresh concrete properties of the Class V concrete mixtures. The slump test results showed that the concrete mixes with a lower CPV have lower slump. Figure 5-7 shows that the slump of the fresh concrete generally improves with increased CPV of concrete. Thus, the paste volume can affect the slump of the fresh concrete. The OAG mixes have similar slump as the FSC mixes. OAG technique cannot improve the workability when the concrete has a low w/cm.

The air content of all mixtures was in the range of 1 to 6%. For the concretes with a higher CPV, more bleeding was observed. Figure 5-8 shows the air content of the FSC and OAG mixes. At the same CPV, the OAG mixes have a lower air content than the FSC mixes. The density results were in the normal concrete range and the bleeding results of all mixtures are acceptable. The fresh concrete results showed that the minimum CPV in Class V concrete can be reduced to 25.0%.

Table 5-4 Fresh concrete properties of Class V concretes evaluated in this research study

Concrete Mix (w/cm=0.32)	Paste Volume (%)	Slump ASTM C143 (inches)	Air Content ASTM C231 (%)	Temp. ASTM C1064 (°F)	Density ASTM C138 (lb/ft^3)	Final Bleeding ASTM C232 (ml/inches^2)	pH test UIP (Color/pH)	pH test FM 5-550 (pH)
FSC250	25.0	2.00	3.0	72	144.32	0.00	/ 13	12.62
FSC275	27.5	2.00	3.8	73	143.12	0.00	/ 13	12.38
FSC300	30.0	4.00	2.4	75	143.52	0.00	/ 13	12.41
FSC325	32.5	3.50	2.0	74	142.32	0.00	/ 13	12.34
OAG250	25.0	2.00	3.3	73	143.84	0.00	/ 13	12.66
OAG275	27.5	2.00	2.3	73	145.20	0.00	/ 13	12.36
OAG300	30.0	3.25	2.7	75	142.96	0.00	/ 13	12.46
OAG325	32.5	3.50	1.8	72	142.80	0.00	/ 13	12.47

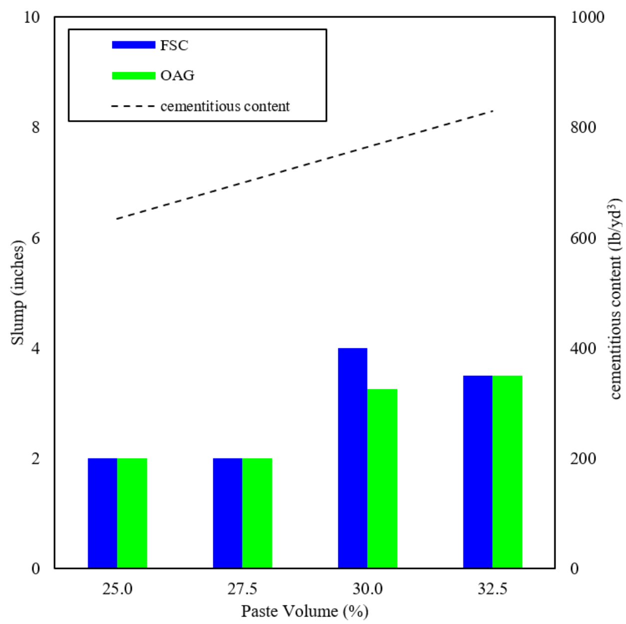

Figure 5-7 Slump of FSC and OAG mixes in Class V concrete

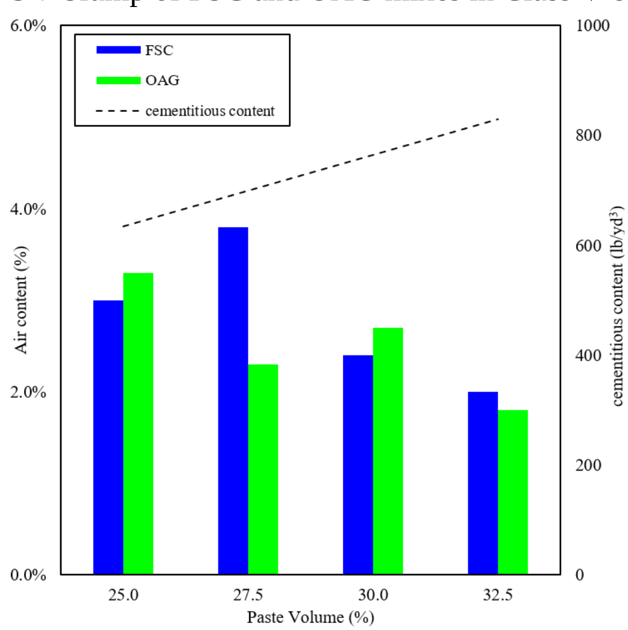

Figure 5-8 Air content of FSC and OAG mixes in Class V concrete

5.1.5 Summary of Fresh Concrete Properties

Table 5-5 shows the slump of the FSC and OAG mixes with different CPVs for different classes of concrete. When the w/cm is 0.50, 0.44, and 0.38, the slump of the concrete with the incorporation of OAG technique is greater than the slump of the corresponding conventional concrete. However, when the w/cm is 0.32, the concrete using OAG technique does not substantially improve the slump of the fresh concrete. The use of OAG technique can significantly improve the workability of the concrete only when the w/cm is higher than 0.32.

Table 5-6 shows the air content of the FSC and OAG mixes with different CPVs for different classes of concrete. The air content of the concrete with the incorporation of OAG technique is lower than the air content of the corresponding conventional concrete. The use of OAG technique can reduce the air content of the concrete.

Superplasticizer (Type F) admixture was applied on all the concrete mixes to obtain the desired workability of the fresh concrete. Figure 5-9 presents the required dosage of superplasticizer for different CPVs and w/cms. When the CPV was lower than 25.0%, the concrete required more than 6.0 fluid ounces/centum weight to achieve the target slump. It indicates that the CPV influenced the workability of the concrete. Besides, at a higher w/cm of concrete, the concrete required more Type F admixture to reach the target slump at the same CPV. Thus, the slump of the concrete was affected by its w/cm and CPV.

Table 5-5 Slump change of the concrete mixes

Mixes (Inches)	Paste Volume	Water to Cementitious Materials Ratio			
		0.50	0.44	0.38	0.32
FSC	22.5%	1.50	2.25	-	-
	25.0%	1.50	4.25	2.25	2.00
	27.5%	1.50	4.00	3.25	2.00
	30.0%	7.00	4.25	3.00	4.00
	32.5%	-	-	3.00	3.50
OAG	22.5%	2.00(▲)	4.00(▲)	-	--
	25.0%	2.50(▲)	4.00(▼)	2.50(▲)	2.00(▬)
	27.5%	2.25(▲)	4.50(▲)	4.50(▲)	2.00(▬)
	30.0%	8.00(▲)	5.00(▲)	3.00(▬)	3.25(▼)
	32.5%	-	-	5.00(▲)	3.50(▬)

Note: (▲) The slump of the OAG mix increased, (▼) the slump of the OAG mix decreased, (▬) The slump of the OAG mix did not change.

Table 5-6 Air content change of the concrete mixes

Mixes (%)	Paste Volume	Water to Cementitious Materials Ratio			
		0.50	0.44	0.38	0.32
FSC	22.5%	4.5	3.5	-	-
	25.0%	2.7	3.2	3.4	3.0
	27.5%	2.3	3.5	2.9	3.8
	30.0%	3.3	1.3	1.9	2.4
	32.5%	-	-	1.7	2.0
OAG	22.5%	4.0(▼)	4.2(▲)	-	--
	25.0%	2.2(▼)	3.0(▼)	3.1(▼)	3.3(▼)
	27.5%	1.9(▼)	5.0(▲)	3.1(▲)	2.3(▼)
	30.0%	3.6(▲)	1.0(▼)	1.7(▼)	2.7(▲)
	32.5%	-	-	1.2(▼)	1.8(▼)

Note: (▲) The air of the OAG mix increased, (▼) the air of the OAG mix decreased, (▬) The air of the OAG mix did not change.

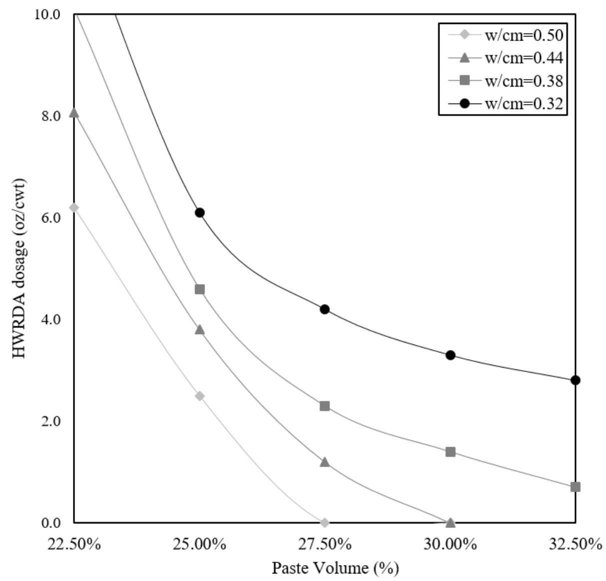

Figure 5-9 Required dosage of superplasticizer for different paste volume and w/cm

5.2 Results of Tests on Hardened Concrete

5.2.1 Compressive Strength

5.2.1.1 Class II concrete

According to FDOT Standard Specifications for Road and Bridge Construction, Section 346, the specified minimum strength of Class II concrete is 3,400 psi at 28 days. The required minimum strength of Class II concrete is 4,600 psi at 28 days. The average compressive strength results at 7, 28, 56, 91, and 182 days are shown in Table 5-7. Figure 5-10 shows the compressive strength results at 28 days. Although most of the Class II concrete mixes passed the specified and required minimum strength, except for the concrete mix with the highest CPV (30%). The concrete mixes with 30% CPV did not passed the required minimum strength (4,600 psi) of Class II. Therefore, the concrete with excessive CPV cannot improve the compressive strength in the Class II concrete mix. Figures 5-11 and 5-12 present the plots of average compressive strength of FSC and OAG mixes at 7, 28, 56, 91, and 182 days. For the FSC and OAG mixes, the compressive strengths of both groups were similar at all CPV levels. The amount of CPV would not significantly improve the strength at any age. The results indicated that the compressive strength of concrete could not be improved by increasing the paste volume. Based on the compressive strength results, and FDOT's required minimum strength, the recommended range of designed CPV is between 22.5 and 27.5 % in the Class II concrete When the OAG technique is used to optimize the aggregate gradation, the strength of concrete will not be affected.

Table 5-7 Average compressive strengths of tested Class II concrete mixtures

w/cm ratio = 0.50	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Compressive Strength (psi)	22.5% (450)		25.0% (500)		27.5% (550)		30.0% (600)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
7 days	3,400	3,480	3,560	3,690	3,630	3,570	3,200	3,140
28 days	4,650	4,810	4,930	5,090	5,250	5,160	4,350	4,140
56 days	4,950	5,140	5,520	5,680	5,700	5,750	4,650	4,740
91 days	5,430	5,650	5,650	5,560	5,960	5,800	5,400	5,240
182 days	5,600	5,890	6,510	6,180	6,090	6,310	5,430	5,330

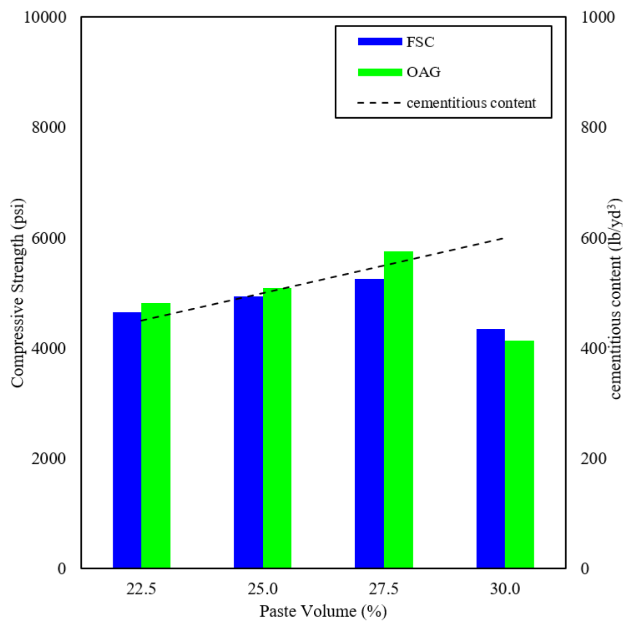

Figure 5-10 Compressive strength of Class II concretes at 28 days

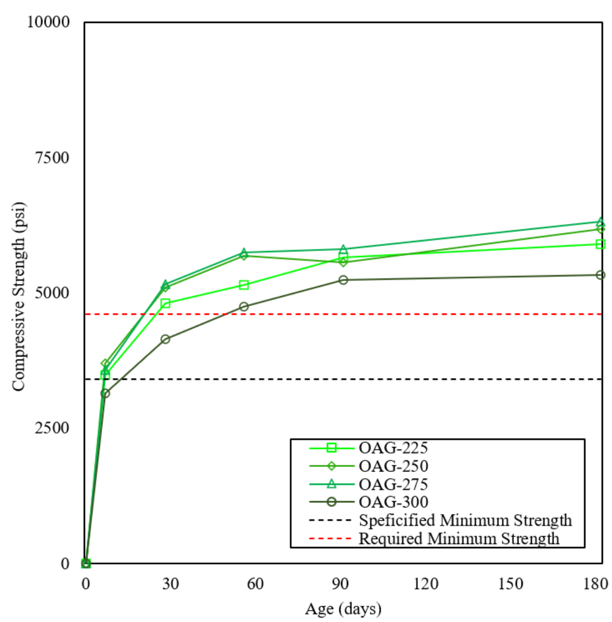

Figure 5-11 Compressive strength development of Class II concrete in FSC group

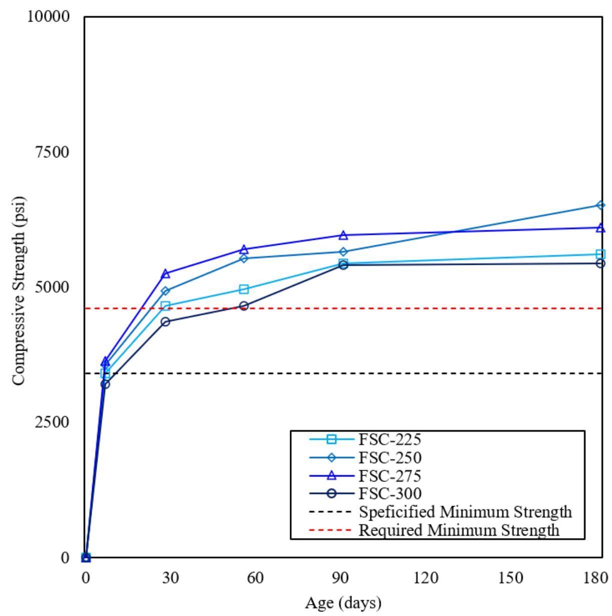

Figure 5-12 Compressive strength development of Class II concrete in OAG group

5.2.1.2 Class II - Bridge Deck concrete

According to FDOT Standard Specifications for Road and Bridge Construction, Section 346, the specified minimum strength of Class II - Bridge Deck concrete is 4,500 psi at 28 days. The required minimum strength of Class II concrete is 5,700 psi at 28 days. The average compressive strength results at 7, 28, 56, 91, and 182 days are shown in Table 5-8. Figure 5-13 shows the compressive strength results at 28 days. For the FSC and OAG mixes, compressive strengths were about the same at different CPVs at different ages. Most of the strength passed the specified minimum strength (5000 psi), except the FSC mixes with 22.5% CPV. The concrete with insufficient CPV cannot improve the compressive strength in Class II - Bridge Deck concrete mix. Figures 5-14 and 5-15 present the plots of average compressive strength of FSC and OAG mixes at 7, 28, 56, 91, and 182 days. For the FSC and OAG mixes, the compressive strengths of both groups were similar at all CPV levels. The results indicated that the compressive strength of concrete could not be improved by increasing the paste volume. Based on the compressive strength results and FDOT's required minimum strength, the recommended range of designed CPV is between 25.0 and 30.0 % in Class II - Bridge Deck concrete. When the OAG technique is used to optimize the aggregate gradation, the strength of concrete will not be affected.

Table 5-8 Average compressive strengths (psi) of tested Class II - Bridge Deck concrete mixture

w/cm ratio = 0.44	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Compressive Strength (psi)	22.5% (485)		25.0% (450)		27.5% (595)		30.0% (645)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
7 days	3,440	3,950	5,060	5,170	5,060	4,560	4,740	4,660
28 days	4,560	5,190	6,170	6,060	6,400	5,790	5,780	5,740
56 days	5,250	5,750	6,740	6,920	7,030	6,460	6,800	6,610
91 days	5,430	5,990	6,970	7,050	7,110	6,840	6,890	7,020
182 days	6,450	6,950	7,450	7,470	7,200	7,170	7,590	7,200

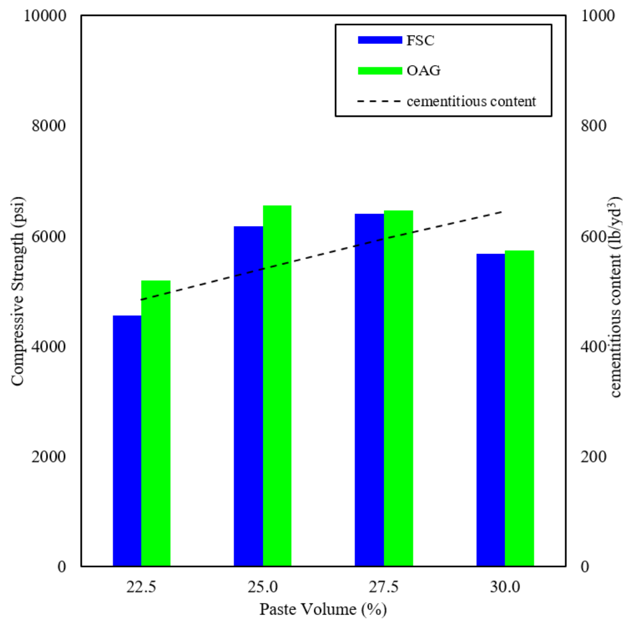

Figure 5-13 Compressive strength of Class II - Bridge Deck concretes at 28 days

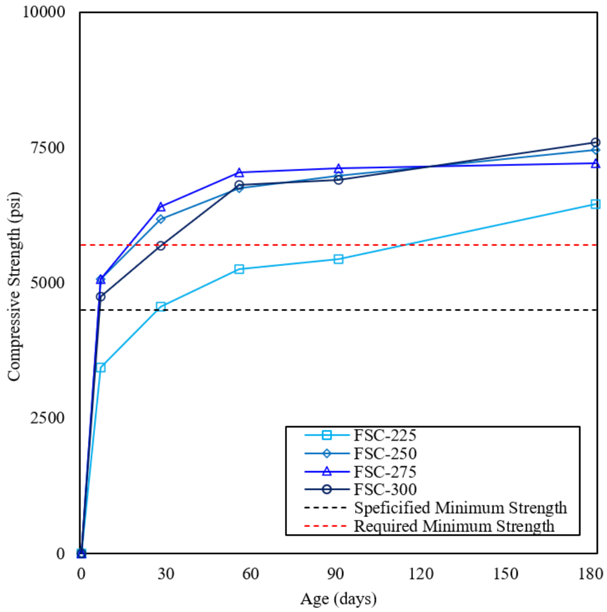

Figure 5-14 Compressive strength development of Class II - Bridge Deck concrete in FSC group

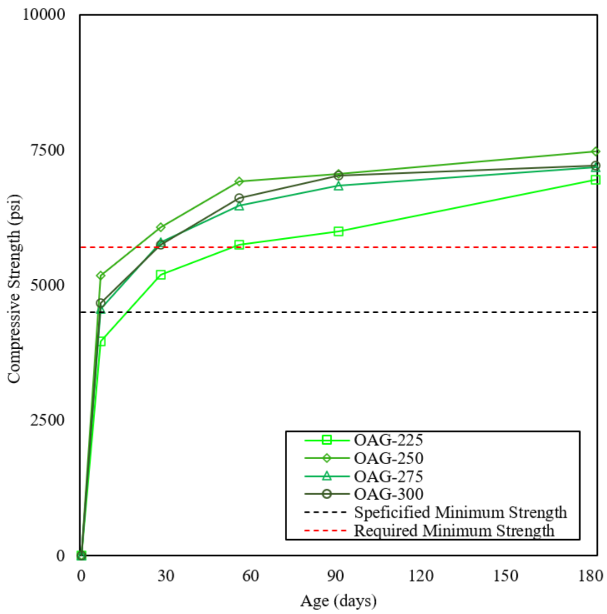

Figure 5-15 Compressive strength development of Class II - Bridge Deck concrete in OAG group

5.2.1.3 Class IV concrete

According to FDOT Standard Specifications for Road and Bridge Construction, Section 346, the specified minimum strength of Class IV concrete is 5,500 psi at 28 days. The required minimum strength of Class IV concrete is 6,750 psi at 28 days. The average compressive strength results at 7, 28, 56, 91, and 182 days are shown in Table 5-9. Figure 5-16 shows the compressive strength results at 28 days. All of the Class IV concrete mixes passed the specified and required minimum strength. However, the compressive strength of the concrete with the highest CPV (32.5%) showed the lowest compressive strength. The excessive CPV in the concrete did not improve the compressive strength in Class IV concrete. Figures 5-17 and 5-18 present the plots of average compressive strength of FSC and OAG mixes at 7, 28, 56, 91, and 182 days. For the FSC and OAG mixes, the compressive strengths of both groups were similar at all CPV levels. The amount of CPV did not significantly improve the strength of any age. The results indicated that the compressive strength of concrete could not be improved by increasing the paste volume. Based on the compressive strength results, and FDOT's required minimum strength, the recommended range of designed CPV is between 25.0 and 30.0 % in the Class IV concrete When the OAG technique is used to optimize the aggregate gradations, the strength of concrete will not be affected.

Table 5-9 Average compressive strengths (psi) of tested Class IV concrete mixtures

w/cm ratio = 0.38	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Compressive Strength (psi)	25.0% (580)		27.5% (645)		30.0% (700)		32.5% (760)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
7 days	6,460	6,160	5,610	6,180	6,600	6,550	5,930	5,610
28 days	7,490	7,450	7,630	7,450	7,830	7,940	6,850	7,120
56 days	7,980	8,110	7,740	8,110	8,600	8,350	7,930	7,560
91 days	8,670	8,410	8,280	8,270	8,890	8,270	8,770	8,560
182 days	9,440	9,160	9,280	8,600	9,090	8,790	8,520	8,650

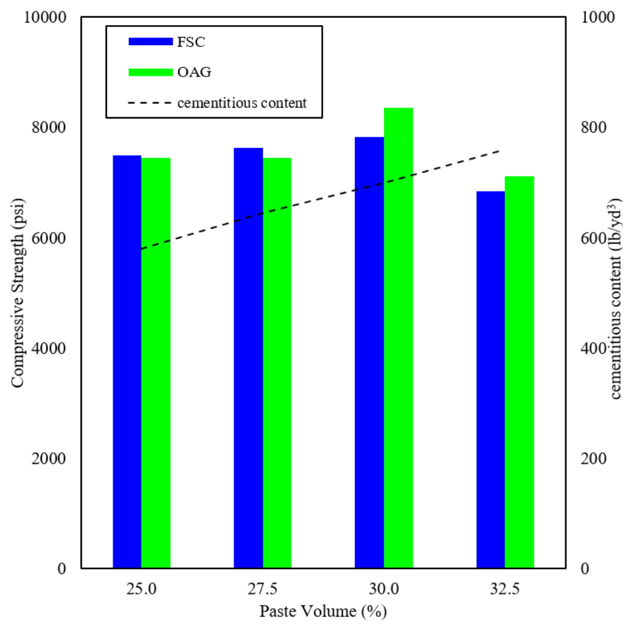

Figure 5-16 Compressive strength of Class IV concretes at 28 days

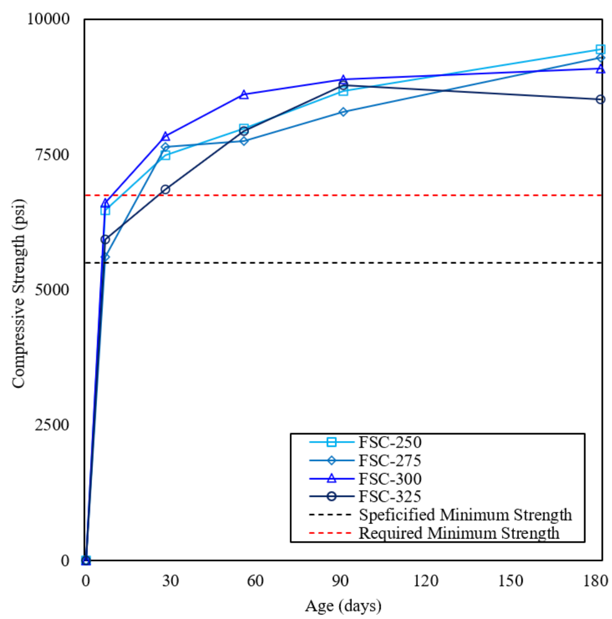

Figure 5-17 Compressive strength development of Class IV concretes in FSC group

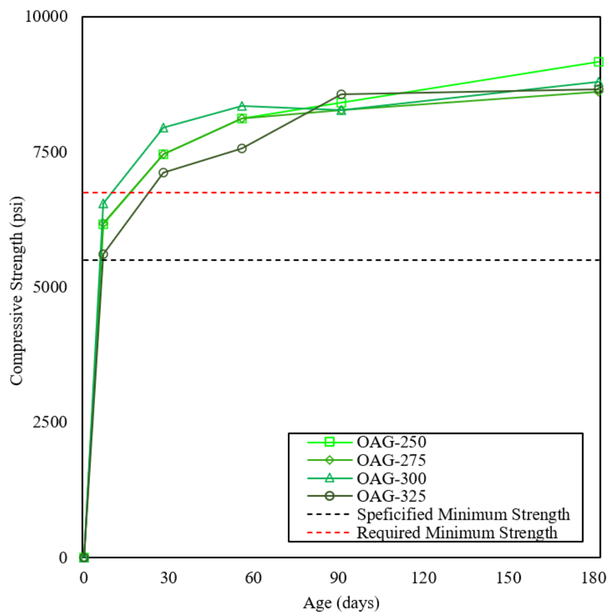

Figure 5-18. Compressive strength development of Class IV concretes in OAG group

5.2.1.4 Class V concrete

According to FDOT Standard Specifications for Road and Bridge Construction, Section 346, the specified minimum strength of Class V concrete is 6,500 psi at 28 days. The required minimum strength of Class V concrete is 7,850 psi at 28 days. The average compressive strength results at 7, 28, 56, 91, and 182 days are shown in Table 5-10. Figure 5-19 shows the compressive strength results at 28 days. For the FSC and OAG mixes, compressive strengths were about the same at different CPVs at different ages. Most of the strength passed the specified minimum strength (5000 psi), except for the FSC mixes with 25.0% CPV. The concrete with the insufficient CPV cannot improve the compressive strength in Class V concrete mix. Figures 5-20 and 5-21 present the plots of average compressive strength of FSC and OAG mixes at 7, 28, 56, 91, and 182 days. For the FSC and OAG mixes, the compressive strengths of both groups were similar at all CPV levels. The results indicated that the compressive strength of concrete could not be improved by increasing the paste volume. Based on the compressive strength results and FDOT's required minimum strength, the recommended range of designed CPV is between 25.0 and 32.5 % in Class V concrete. When the OAG technique is used to optimize the aggregate gradations, the strength of concrete will not be affected.

Table 5-10 Average compressive strengths (psi) of tested Class V concrete mixtures

w/cm ratio = 0.32	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Compressive Strength (psi)	25.0% (635)		27.5% (700)		30.0% (765)		32.5% (830)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
7 days	6,250	6,230	7,730	7,500	6,800	6,760	6,150	7,070
28 days	7,040	6,840	8,500	8,160	8,470	7,950	8,400	8,410
56 days	7,490	7,580	8,900	8,860	8,500	8,040	8,640	9,030
91 days	8,280	8,610	9,300	8,290	8,790	8,920	9,380	9,210
182 days	8,920	8,610	9,530	9,580	9,590	9,480	9,440	9,440

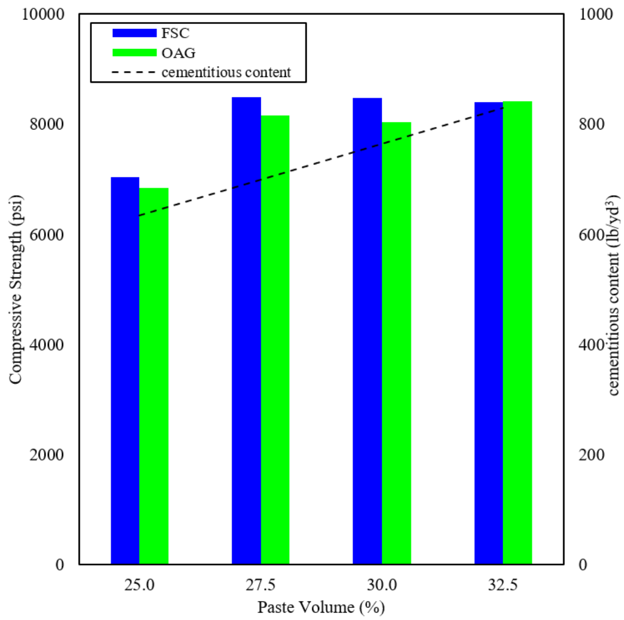

Figure 5-19 Compressive strength of Class V concretes at 28 days

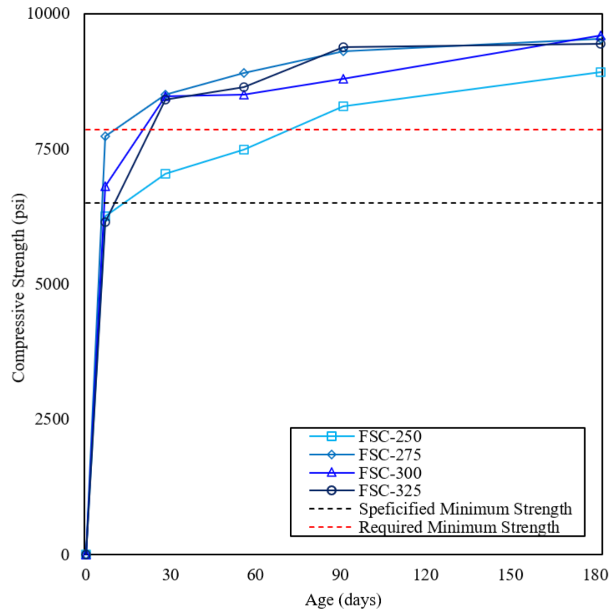

Figure 5-20 Compressive strength development of Class V concretes in FSC group

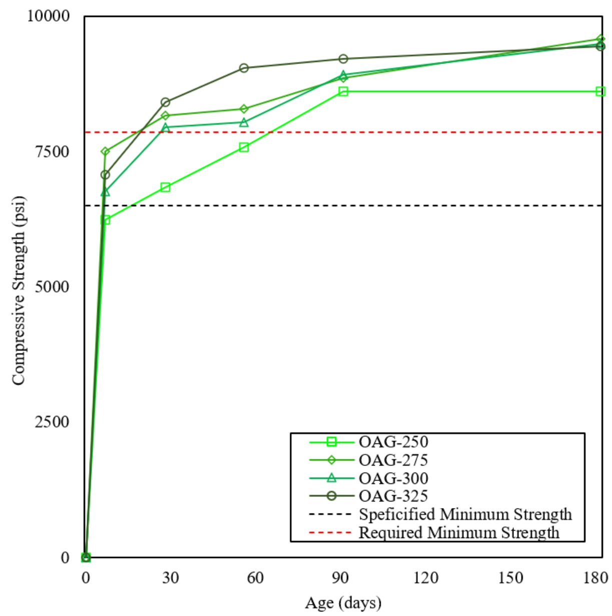

Figure 5-21 Compressive strength development of Class V concretes in OAG group

5.2.2 Modulus of Rupture (MOR)

5.2.2.1 Class II concrete

The average MOR of the Class II concretes are shown in Table 5-11. Figure 5-22 shows the MOR of the Class II concretes at 28 days. Figures 5-23 and 5-24 present the plots of MOR of the FSC and OAG mixes at 28, 56, 91 and 182 days. Similar trends as for the compressive strength plots can be seen here. For the FSC and OAG mixes, the MORs were about the same at paste volumes of 30.0, 27.5, 25.0, and 22.5%. The MOR results also indicated that the MOR of concrete could not be improved by increasing the CPV. Therefore, the minimum CPV of Class II concrete is 22.5% based on the 28-day MOR. On the other hand, the MORs of the OAG mixes were slightly higher than FSC mixes at different ages. OAG technique could slightly improve the MORs of the concrete.

Table 5-11 MORs (psi) of tested Class II concrete mixtures

w/cm ratio = 0.50	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
MOR (psi)	22.5% (450)		25.0% (500)		27.5% (550)		30.0% (600)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
28 days	585	605	610	650	660	675	655	645
56 days	565	620	655	715	675	715	665	635
91 days	645	700	690	760	695	730	675	685
182 days	685	690	760	795	790	775	695	705

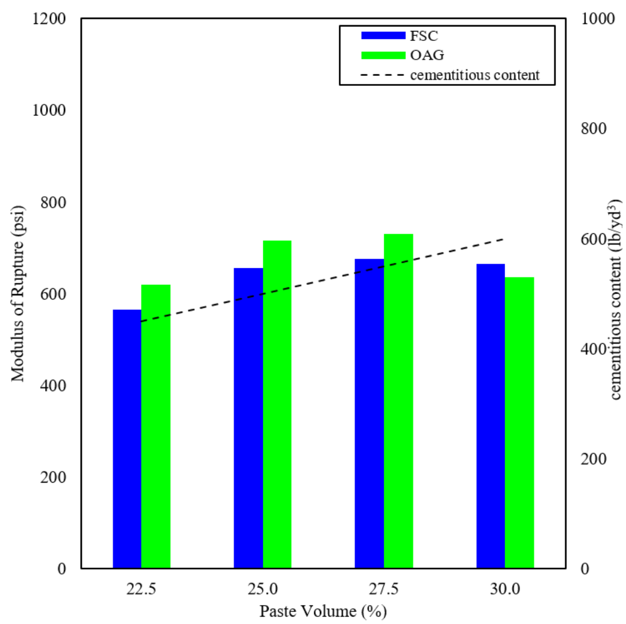

Figure 5-22 MOR of Class II concretes at 28 days

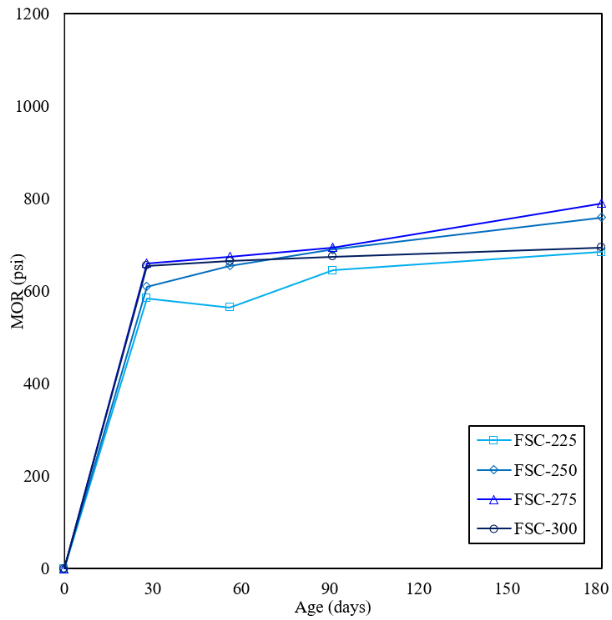

Figure 5-23 MOR development of Class II concretes in FSC group

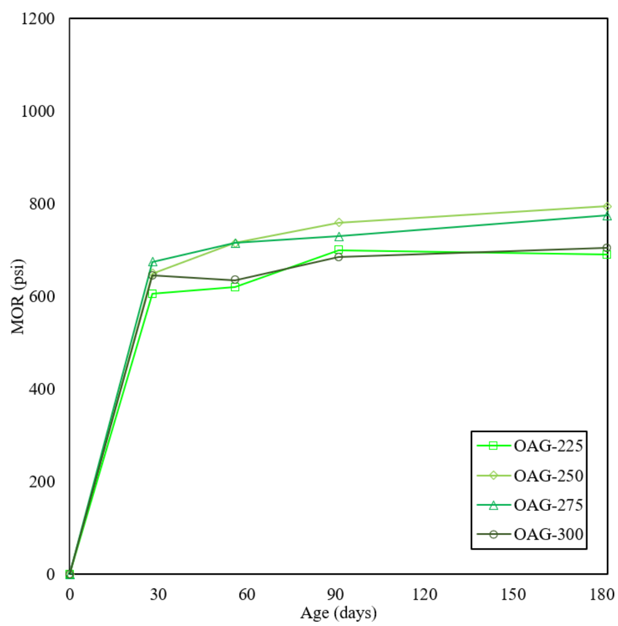

Figure 5-24 MOR development of Class II concretes in OAG group

5.2.2.2 Class II - Bridge Deck concrete

The average MOR of the Class II - Bridge Deck concretes are shown in Table 5-12. Figure 5-25 shows the MOR of the Class II- Bridge Deck concretes at 28 days. Figures 5-26 and 5-27 present the plots of MOR of the FSC and OAG mixes at 28, 56, 91, and 182 days. Similar trends as for the compressive strength plots can be seen here. For both of these mixes, the MORs were about the same at paste volumes of 30.0, 27.5, and 25.0. The concrete with the lowest CPV (22.5%)

show lower MOR than the MOR of the concrete with higher CPV. Therefore, the minimum CPV of Class II - Bridge Deck concrete is 25.0% based on the 28-day MOR. The MOR results indicated that the strength of concrete could not be improved by increasing the CPV. On the other hand, the MORs of the OAG mixes were slightly higher than the FSC mixes at different ages. OAG technique could slightly improve the MORs of the concrete.

Table 5-12 MORs (psi) of tested Class II - Bridge Deck concrete mixtures

w/cm ratio = 0.44	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
MOR (psi)	22.5% (485)		25.0% (450)		27.5% (595)		30.0% (645)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
28 days	555	645	710	740	690	705	730	775
56 days	625	695	730	815	765	785	780	790
91 days	660	730	815	830	805	820	830	815
182 days	715	820	835	810	885	825	815	855

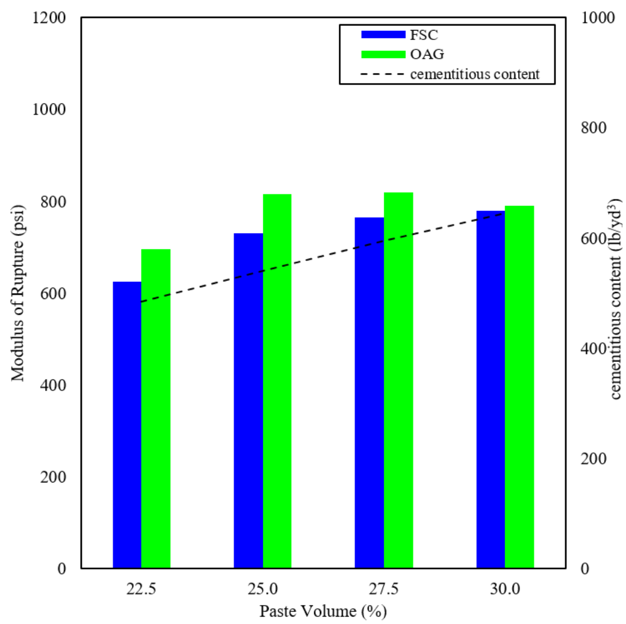

Figure 5-25 MOR of Class II - Bridge Deck concretes at 28 days

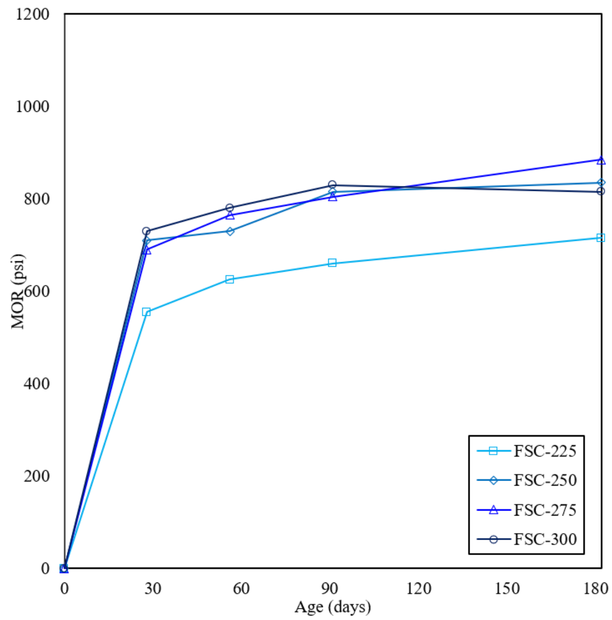

Figure 5-26 MOR development of Class II - Bridge Deck concretes in FSC group

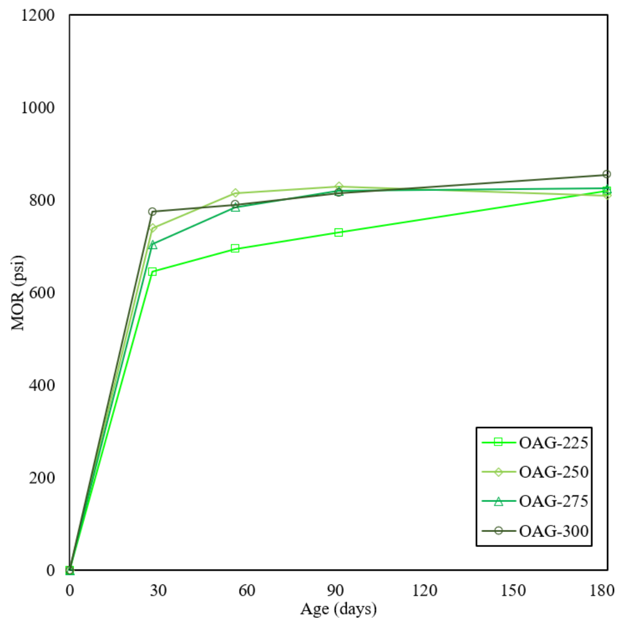

Figure 5-27 MOR development of Class II - Bridge Deck concretes in OAG group

5.2.2.3 Class IV concrete

The average MOR of the Class IV concretes are shown in Table 5-13. Figure 5-28 shows the MOR of the Class IV concretes at 28 days. Figures 5-29 and 5-30 present the plots of MOR of the FSC and OAG mixes at 28, 56, 91, and 182 days. Similar trends as for the compressive strength plots can be seen here. For the FSC and OAG mixes, the MORs were about the same at paste volumes of 32.5, 30.0, 27.5, and 25.0%. The MOR results also indicated that the strength of concrete could not be improved by increasing the CPV. Therefore, the minimum CPV of Class IV concrete is 25.0% based on the 28-day MOR. On the other hand, the MORs of the OAG mixes were slightly higher than those of the FSC mixes at different ages. OAG technique could slightly improve the MORs of the concrete.

Table 5-13 MORs (psi) of tested Class IV concrete mixtures

w/cm ratio = 0.38	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
MOR (psi)	25.0% (580)		27.5% (645)		30.0% (700)		32.5% (760)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
28 days	815	825	780	795	810	845	770	800
56 days	860	875	800	840	855	900	810	815
91 days	860	865	820	845	935	940	820	810
182 days	960	930	965	950	980	985	875	945

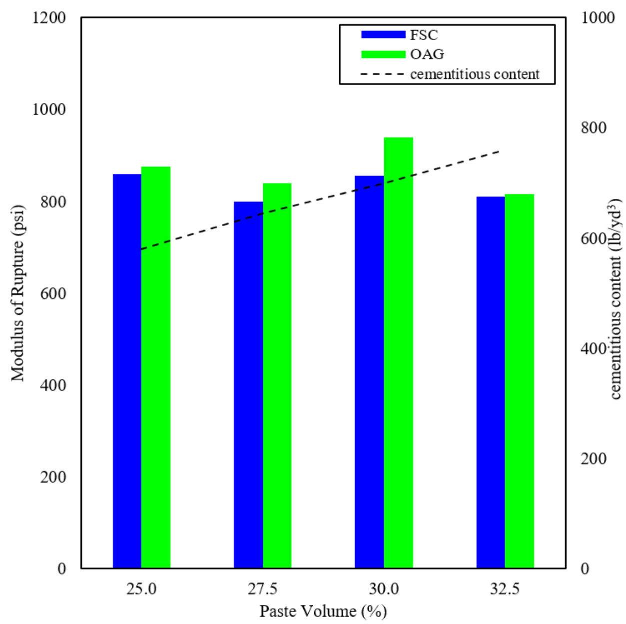

Figure 5-28 MOR of Class IV concretes at 28 days

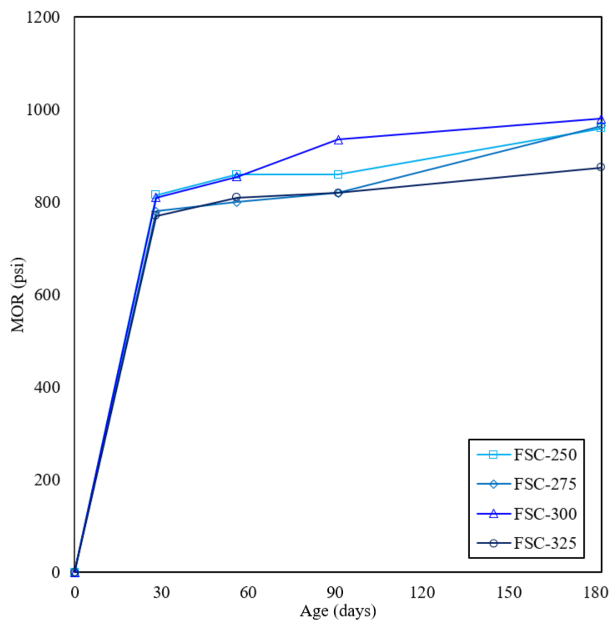

Figure 5-29 MOR development of Class IV concretes in FSC group

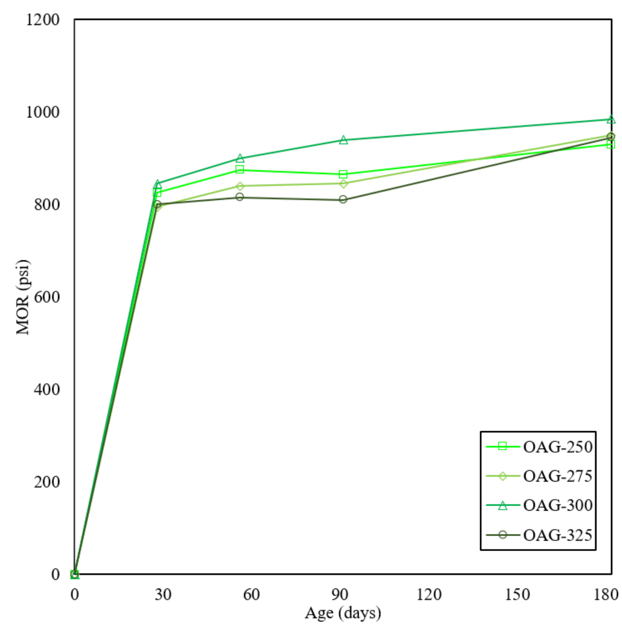

Figure 5-30 MOR development of Class IV concretes in OAG group

5.2.2.4 Class V concrete

The average MOR of the Class V concretes are shown in Table 5-14. Figure 5-31 shows the MOR of the Class V concretes at 28 days. Figures 5-32 and 5-33 present the plots of MOR of the FSC and OAG mixes at 28, 56, 91, and 182 days. Similar trends as for the compressive strength plots can be seen here. For both of these mixes, the MORs were about the same at paste volumes of 30.0, 27.5, and 25.0%. The concrete with the lowest CPV (25.0%) show lower MOR than the MOR of the concrete with higher CPV. Therefore, the minimum CPV of Class V concrete is 27.5% based on the 28-day MOR. The MOR results indicated that the strength of concrete could not be improved by increasing the CPV. On the other hand, the MORs of the OAG mixes were slightly higher than the FSC mixes at different ages. OAG technique could slightly improve the MORs of the concrete.

Table 5-14 MORs (psi) of tested Class V concrete mixtures

w/cm ratio = 0.32	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
MOR (psi)	25.0% (635)		27.5% (700)		30.0% (765)		32.5% (830)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
28 days	755	805	945	965	810	890	920	905
56 days	865	920	1000	1005	925	930	945	975
91 days	860	900	1005	1010	915	935	1005	1015
182 days	910	935	920	1045	935	960	975	965

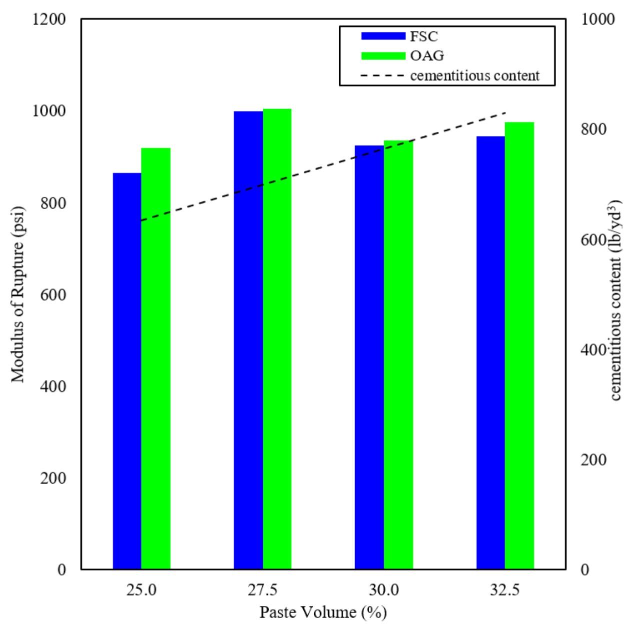

Figure 5-31 MOR of Class V concretes at 28 days

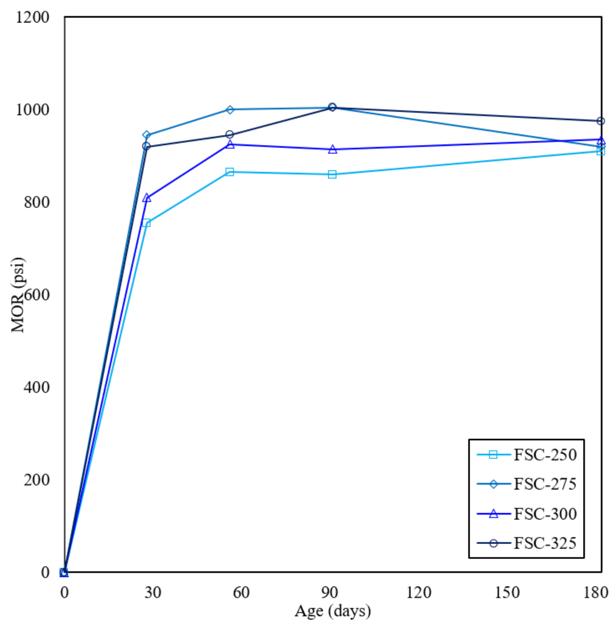

Figure 5-32 MOR development of Class V concrete in FSC group

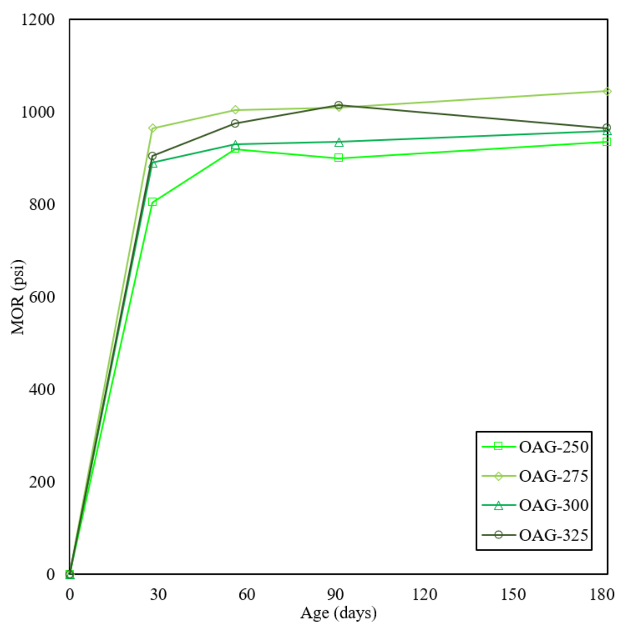

Figure 5-33 MOR development of Class V concrete in FSC group

5.2.3 Splitting Tensile Strength

5.2.3.1 Class II concrete

The average splitting tensile strength results are shown in Table 5-15. Figure 5-34 shows the splitting tensile strength of the Class II concrete at 28 days. Figures 5-35 to 5-36 present the plots of splitting tensile strength of the FSC and OAG mixes at 28, 56, 91, 182 days. For both of these mixes, the splitting tensile strengths were about the same at paste volumes of 30.0, 27.5, 25.0, and 22.5%. Therefore, the CPV of concrete would not affect splitting tensile strength. For the FSC and OAG mixes, the splitting tensile strengths were about the same for the different paste volumes. The concrete with better packing could not improve the compressive strength of Class II concrete.

Table 5-15 Average splitting tensile strength (psi) of the Class II concrete mixtures tested

w/cm ratio = 0.50	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Splitting Tensile Strength (psi)	22.5% (450)		25.0% (500)		27.5% (550)		30.0% (600)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
28 days	370	360	380	405	425	405	390	400
56 days	425	430	405	385	425	435	400	410
91 days	430	415	470	420	460	415	410	420
182 days	430	440	540	560	535	510	455	430

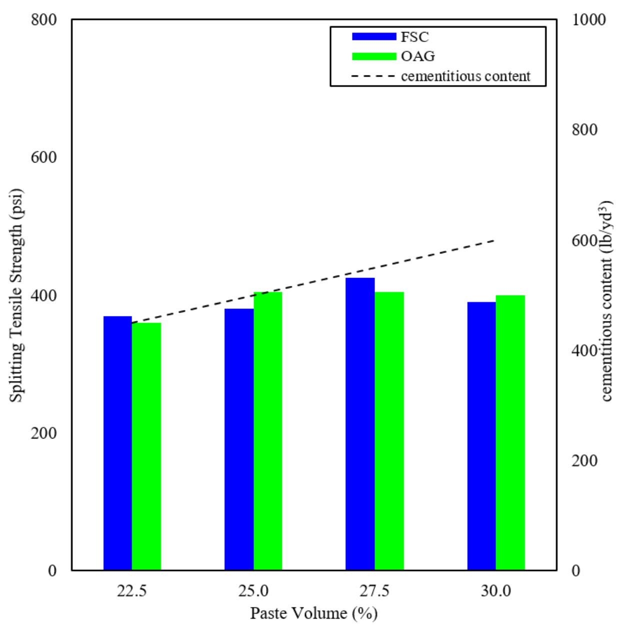

Figure 5-34 Splitting tensile strength of Class II concretes at 28 days

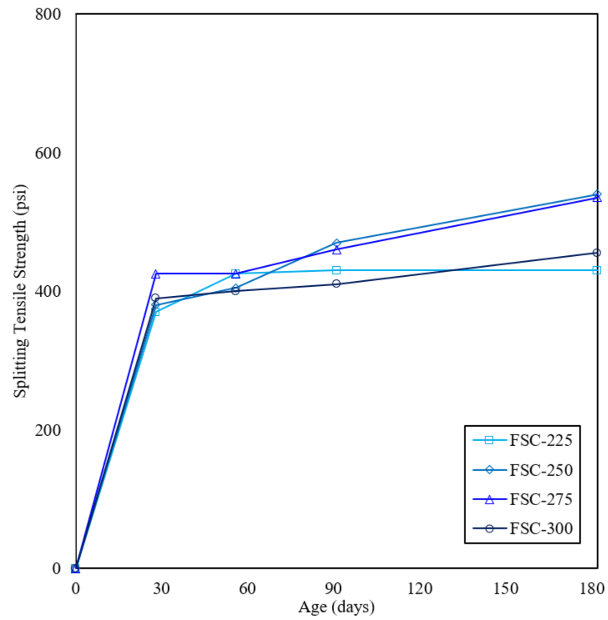

Figure 5-35 Splitting tensile strength development of of Class II concretes in FSC group

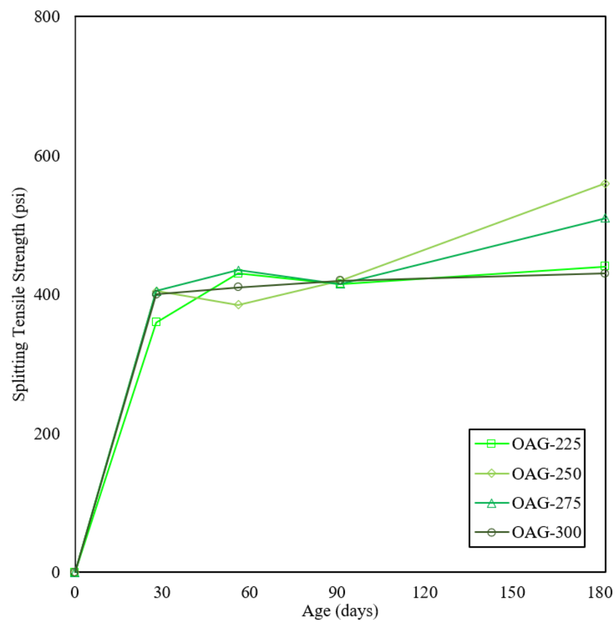

Figure 5-36 Splitting tensile strength development of of Class II concretes in OAG group

5.2.3.2 Class II - Bridge Deck concrete

The average splitting tensile strength results are shown in Table 5-16. Figure 5-37 shows the splitting tensile strength of the Class II - Bridge Deck concretes at 28 days. Figures 5-38 and 5-39 present the plots of splitting tensile strength of the FSC and OAG mixes at 28, 56, 91, and 182 days. For both of these mixes, the MORs were about the same at paste volumes of 30.0, 27.5, 25.0, 22.5%. Therefore, the CPV of concrete did not affect splitting tensile strength. The splitting tensile strengths of the OAG mixes were slightly higher than those of the FSC mixes. The concrete with better packing had slightly improved splitting tensile strength for Class II - Bridge Deck concrete.

Table 5-16 Average splitting tensile strength (psi) of the Class II - Bridge Deck concrete mixtures tested

w/cm ratio = 0.44	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Splitting Tensile Strength (psi)	22.5% (485)		25.0% (450)		27.5% (595)		30.0% (645)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
28 days	375	390	410	390	415	375	430	475
56 days	425	440	425	470	455	445	455	480
91 days	490	505	510	515	535	545	550	535
182 days	475	490	595	595	515	530	565	545

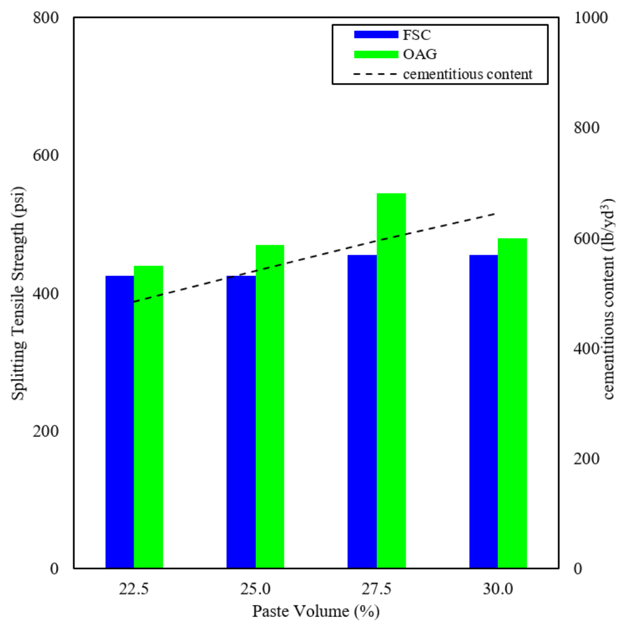

Figure 5-37 Splitting tensile strength of Class II - Bridge Deck concretes at 28 days

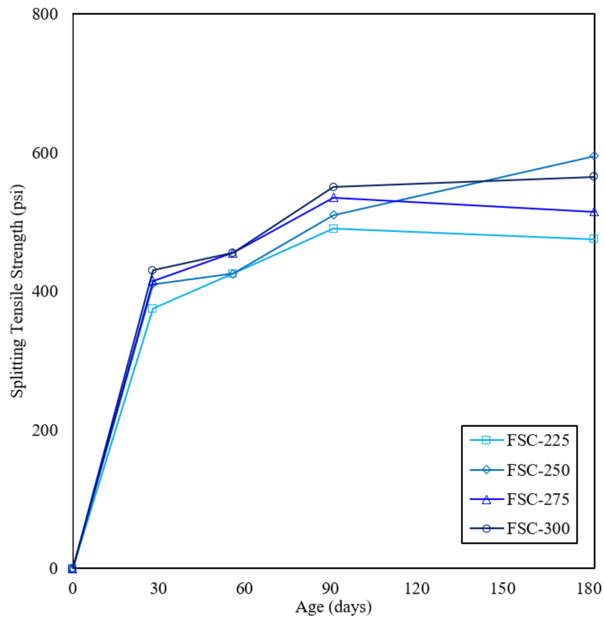

Figure 5-38 Splitting tensile strength development of of Class II - Bridge Deck concretes in FSC group

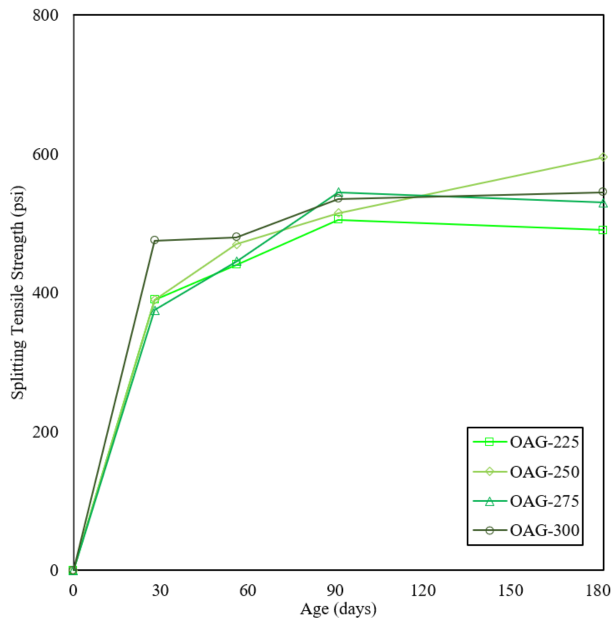

Figure 5-39 Splitting tensile strength development of of Class II - Bridge Deck concretes in OAG group

5.2.3.3 Class IV concrete

The average splitting tensile strength results are shown in Table 5-17. Figure 5-40 shows the splitting tensile strength of Class IV concretes at 28 days. Figures 5-41 and 5-42 present the plots of splitting tensile strength of the FSC and OAG mixes at 28, 56, 91, and 182 days. For both of these mixes, the splitting tensile strengths were about the same at paste volumes of 30.0, 27.5, 25.0, 22.5%. Therefore, the CPV of concrete did not affect splitting tensile strength. The splitting tensile strengths of the OAG mixes were slightly higher than those of the FSC mixes. The concrete with better packing had slightly improved compressive strength for the Class IV concretes.

Table 5-17 Average splitting tensile strength of the Class IV concrete mixtures tested

w/cm ratio = 0.38	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Splitting Tensile Strength (psi)	25.0% (580)		27.5% (645)		30.0% (700)		32.5% (760)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
28 days	440	460	430	445	510	440	470	480
56 days	475	515	440	455	485	490	520	550
91 days	505	495	545	520	485	530	560	555
182 days	560	515	580	555	535	550	600	575

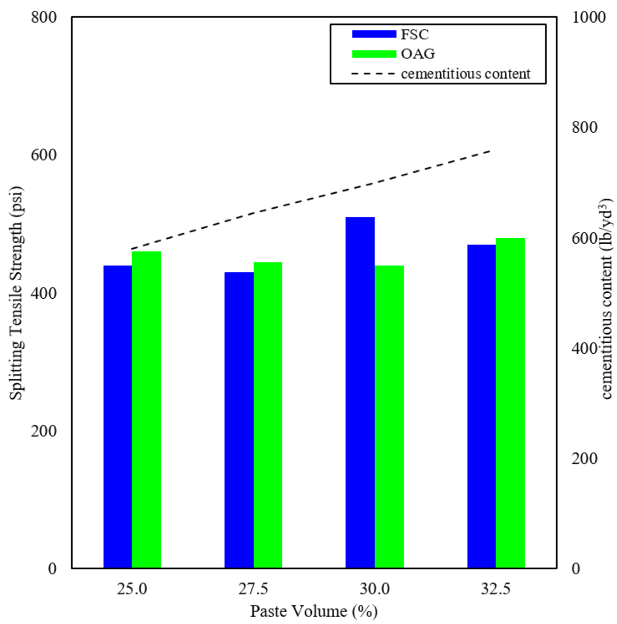

Figure 5-40 Splitting tensile strength of Class IV concretes at 28 days

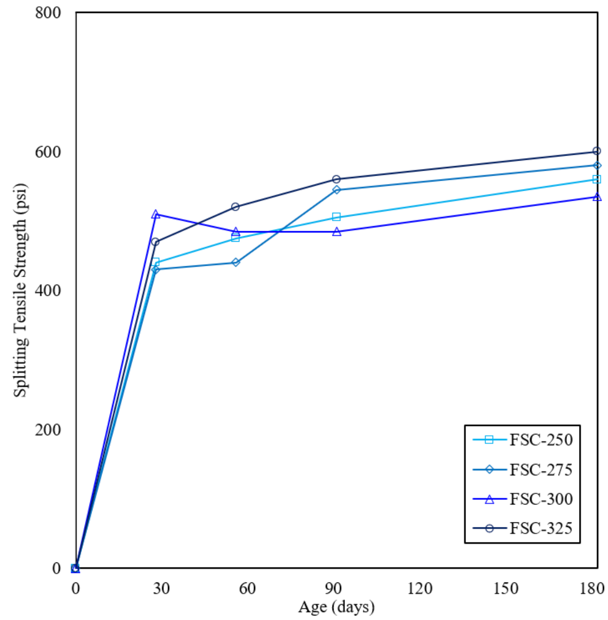

Figure 5-41 Splitting tensile strength development of of Class IV concretes in FSC group

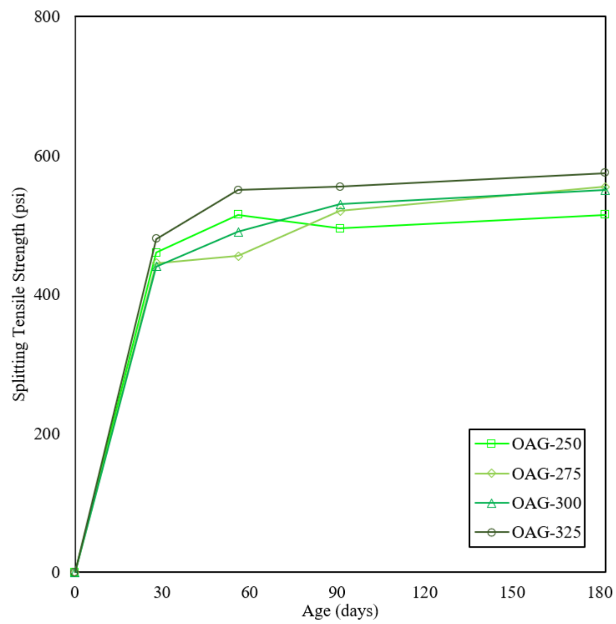

Figure 5-42 Splitting tensile strength development of of Class IV concretes in OAG group

5.2.3.4 Class V concrete

The average splitting tensile strength results are shown in Table 5-18. Figure 5-43 shows the splitting tensile strength of the Class V concretes at 28 days. Figures 5-44 and 5-45 present the plots of splitting tensile strength of FSC, and OAG mixes at 28, 56, 91, and 182 days. For both mixes, the splitting tensile strengths were about the same at paste volumes of 30.0, 27.5, 25.0, 22.5%. Therefore, the CPV of concrete would not affect splitting tensile strength. The splitting tensile strengths of OAG mixes were slightly higher than those of the FSC mixes. The concrete with better packing had slightly improved splitting tensile strength for the Class V concretes.

Table 5-18 Average splitting tensile strength (psi) of the Class V concrete mixtures tested

w/cm ratio = 0.32	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Splitting Tensile Strength (psi)	25.0% (635)		27.5% (700)		30.0% (765)		32.5% (830)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
28 days	515	505	540	505	515	485	495	480
56 days	560	495	510	545	530	505	475	500
91 days	540	545	520	545	560	550	525	500
182 days	550	610	570	575	595	660	575	585

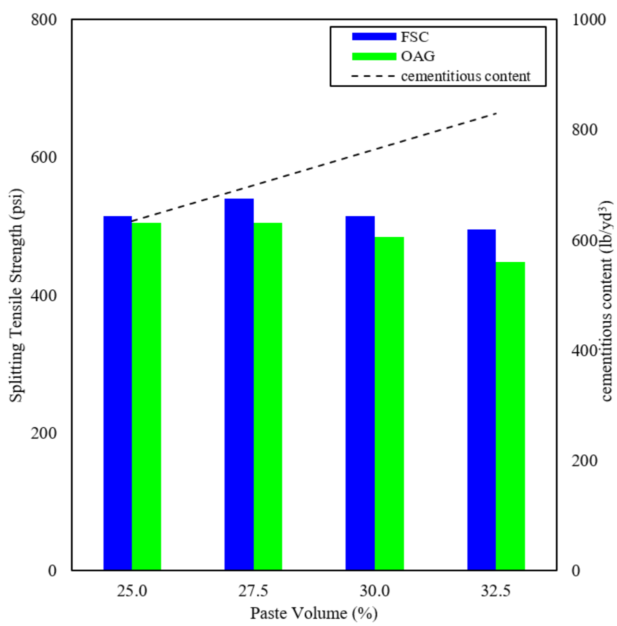

Figure 5-43 Splitting tensile strength of Class V concretes at 28 days

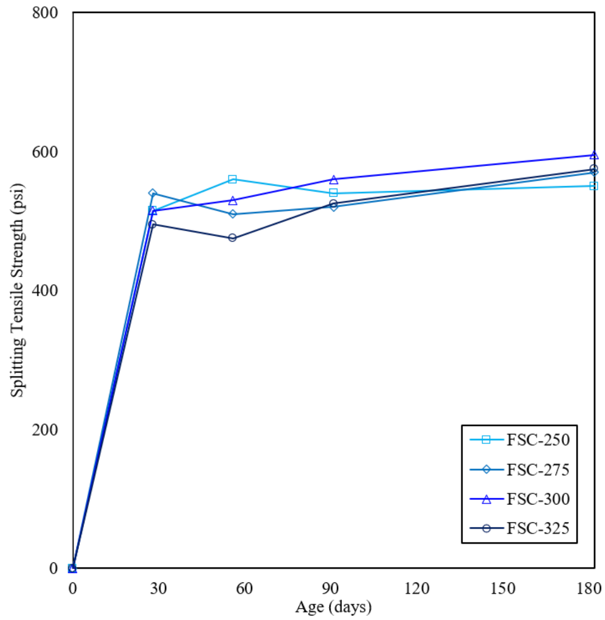

Figure 5-44 Splitting tensile strength development of of Class V concretes in FSC group

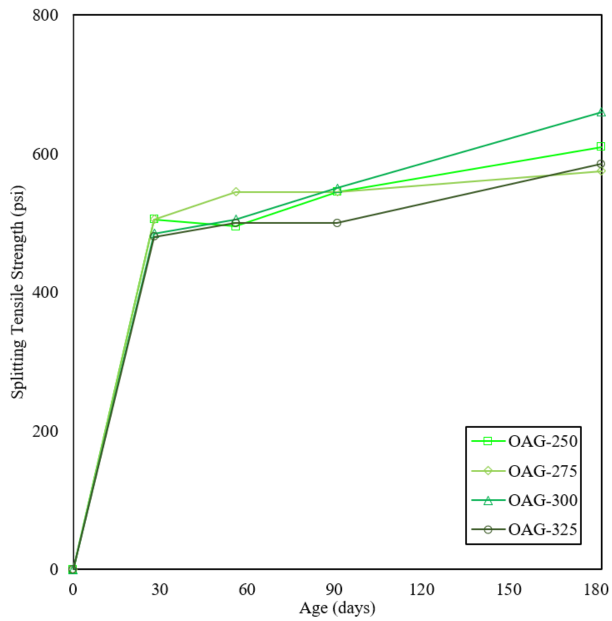

Figure 5-45 Splitting tensile strength development of of Class V concretes in OAG group

5.2.4 Modulus of Elasticity

5.2.4.1 Class II concrete

The average MOE results for the Class II concretes are shown in Table 5-19. Figure 5-46 shows the MOE of the Class II concrete at 28 days. Figures 5-47 and 5-48 present the plots of the MOE of the FSC and OAG mixes at 28, 56, 91, and 182 days. For both mixes, the MOE appeared to be unaffected by changes in CPV. The MOE of OAG mixes were lower than those of the FSC mix. The use of OAG technique could reduce the MOE of the Class II concrete.

Table 5-19 Average MOE (Mpsi) of the Class II concrete mixtures tested

w/cm ratio = 0.50	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
MOE (Mpsi)	22.5% (450)		25.0% (500)		27.5% (550)		30.0% (600)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
28 days	4.75	4.45	4.95	4.70	5.00	4.80	4.20	4.30
56 days	4.75	4.80	5.00	4.90	5.15	4.60	4.40	4.30
91 days	4.90	4.95	5.25	5.15	5.35	4.65	4.65	4.45
182 days	5.25	5.15	5.60	5.75	5.30	5.30	4.90	4.75

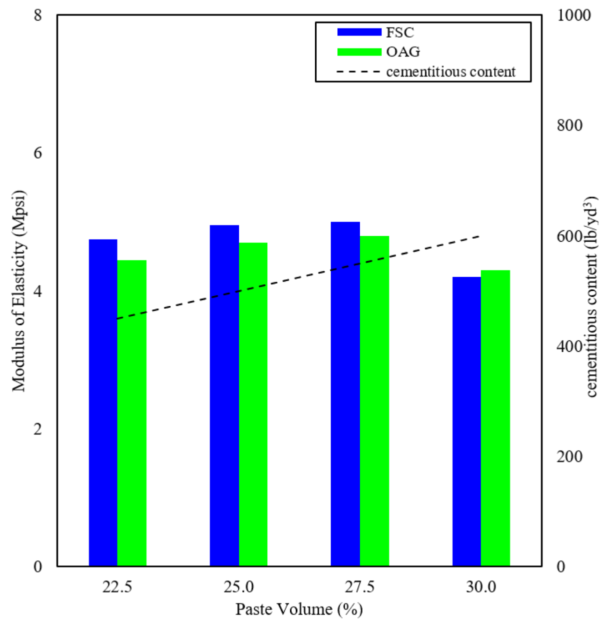

Figure 5-46 MOE of Class II concretes at 28 days

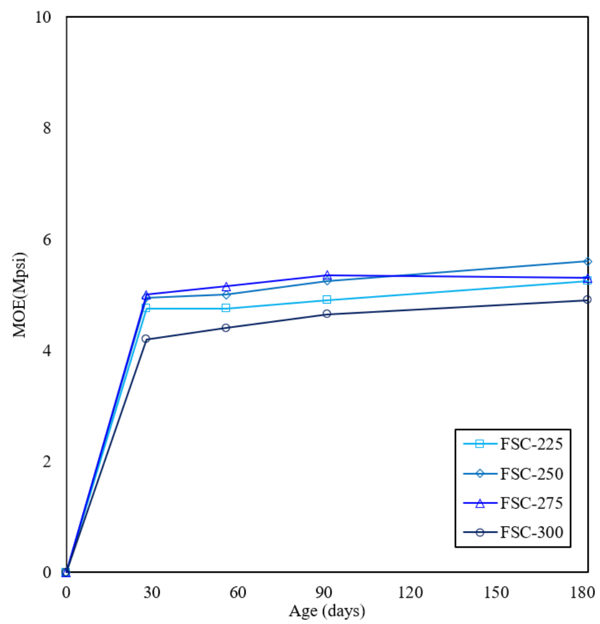

Figure 5-47 MOE development of of Class II concretes in FSC group

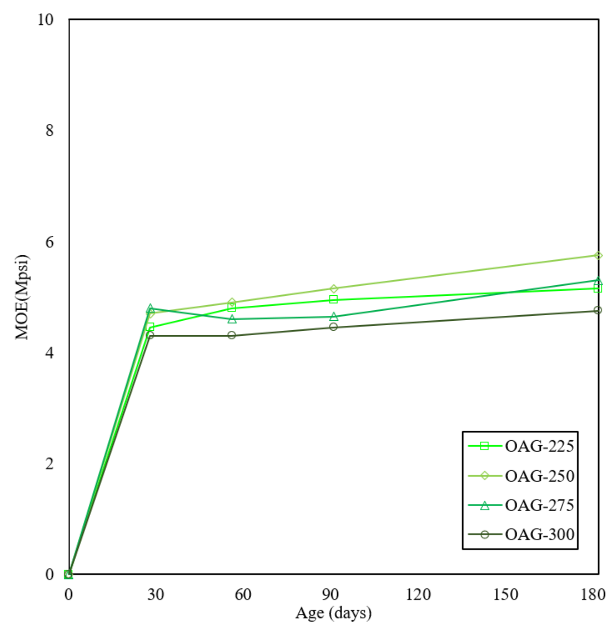

Figure 5-48 MOE development of Class II concretes in OAG group

5.2.4.2 Class II - Bridge Deck concrete

The average MOE results for the Class II - Bridge Deck concretes are shown in Table 5-20. Figure 5-49 shows the MOE of the structural concrete at 28 days. Figures 5-50 and 5-51 present the plots of MOE of the FSC and OAG mixes at 28, 56, 91, and 182 days. For both mixes, the

MOE appeared to be unaffected by changes in CPV. The MOE of the OAG mixes were lower than those of the FSC mix. The use of OAG technique could reduce the MOE of the Class II (Bridge Deck) concrete.

Table 5-20 Average MOE (Mpsi) of the Class II - Bridge Deck concrete mixtures tested

w/cm ratio = 0.44	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
MOE (Mpsi)	22.5% (485)		25.0% (450)		27.5% (595)		30.0% (645)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
28 days	4.80	4.90	5.05	4.75	4.85	4.50	4.60	4.70
56 days	5.10	5.00	5.45	5.30	5.05	4.95	5.10	5.05
91 days	5.20	5.20	5.50	5.40	5.40	4.95	5.20	5.15
182 days	5.40	5.35	5.60	5.50	5.45	5.00	5.25	5.40

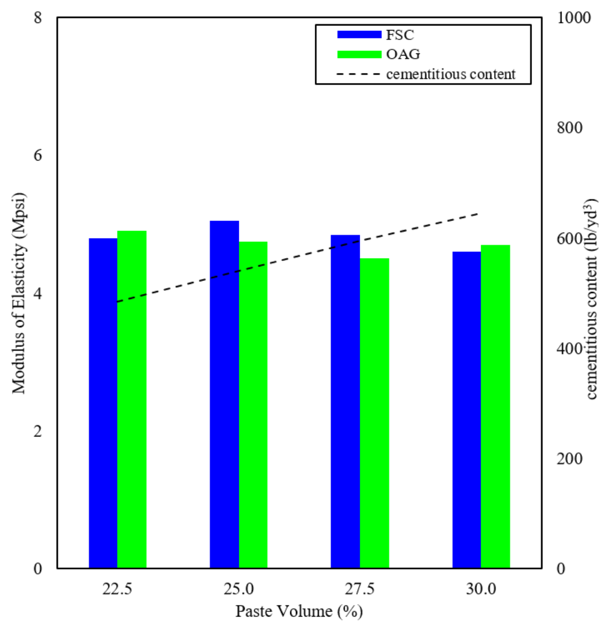

Figure 5-49 MOE of Class II - Bridge Deck concretes at 28 days

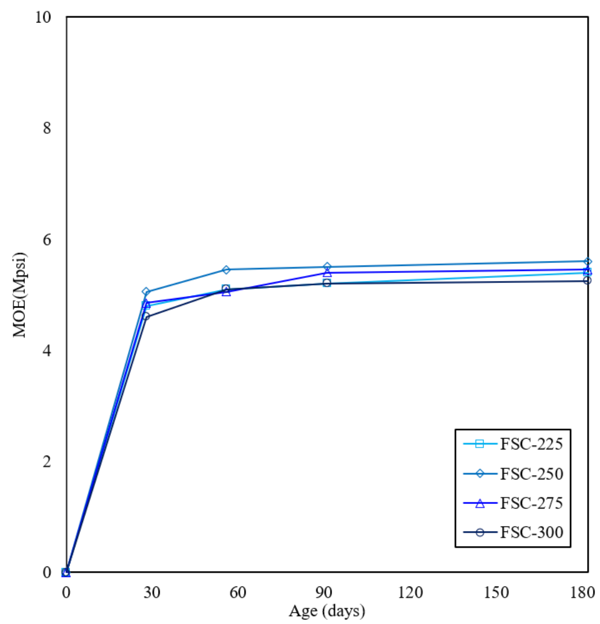

Figure 5-50 MOE development of Class II - Bridge Deck concretes in FSC group

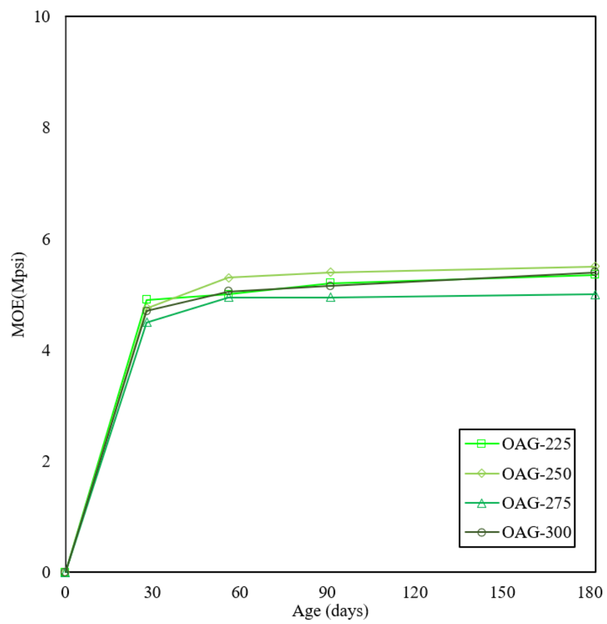

Figure 5-51 MOE development of Class II - Bridge Deck concretes in OAG group

5.2.4.3 Class IV concrete

The average MOE results for the Class IV concretes are shown in Table 5-21. Figure 5-52 shows the MOE of the structural concrete at 28 days. Figures 5-53 and 5-54 present the plots of MOE of the FSC and OAG mixes at 28, 56, 91 and 182 days. For both mixes, the MOE appeared to be unaffected by changes in CPV. The MOE results of the OAG mixes were lower than those of the FSC mix. The use of OAG technique could reduce the MOE of the Class IV concrete.

Table 5-21 Average MOE (Mpsi) of the Class IV concrete mixtures tested

w/cm ratio = 0.38	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
MOE (Mpsi)	25.0% (580)		27.5% (645)		30.0% (700)		32.5% (760)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
28 days	5.40	5.30	5.30	5.20	5.45	5.25	5.25	5.0
56 days	5.95	5.55	5.65	5.65	5.55	5.30	5.45	5.35
91 days	5.75	5.80	5.65	5.65	5.80	5.65	5.65	5.35
182 days	5.85	5.90	5.70	5.75	5.90	5.70	5.85	5.60

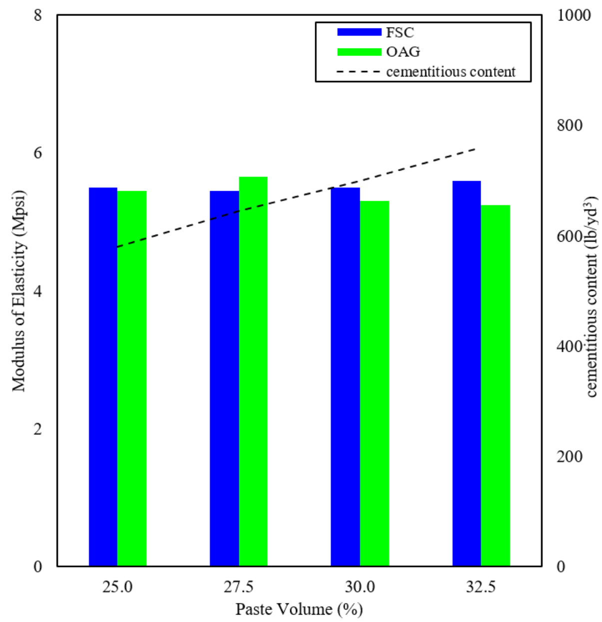

Figure 5-52 MOE of Class IV concretes at 28 days

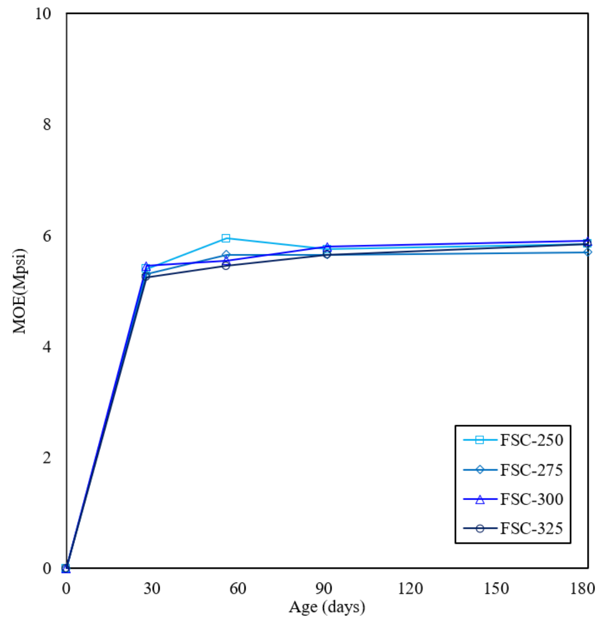

Figure 5-53 MOE development of Class IV concretes in FSC group

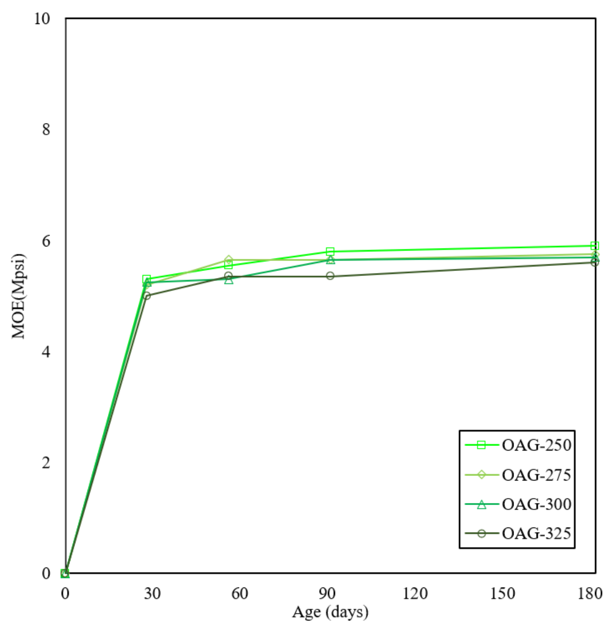

Figure 5-54 MOE development of Class IV concretes in OAG group

5.2.4.4 Class V concrete

The average MOE results for the Class V concretes are shown in Table 5-22. Figure 5-55 shows the MOE of the structural concrete at 28 days. Figures 5-56 and 5-57 present the plots of MOE of the FSC and OAG mixes at 28, 56, 91, and 182 days. For both mixes, the MOE appeared to be unaffected by changes in CPV. The MOE results of the OAG mixes were lower than those of the FSC mix. The use of OAG technique could reduce the MOE of the Class V concrete.

Table 5-22 Average MOE (Mpsi) of the Class V concrete mixtures tested

w/cm ratio = 0.32	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
MOR (Mpsi)	25.0% (635)		27.5% (700)		30.0% (765)		32.5% (830)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
28 days	5.50	5.45	5.45	5.65	5.50	5.30	5.60	5.25
56 days	5.80	5.65	5.85	5.80	5.65	5.60	5.70	5.50
91 days	6.00	5.75	5.95	5.85	5.90	5.70	5.95	5.75
182 days	6.05	5.95	6.00	5.95	6.35	6.20	6.05	5.95

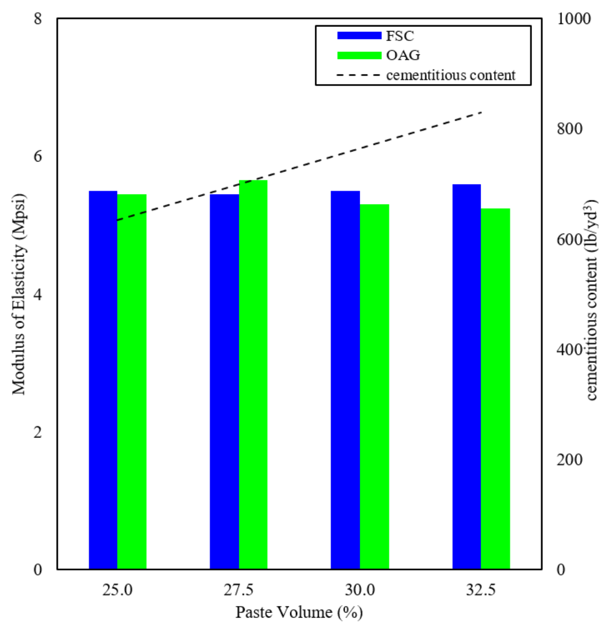

Figure 5-55 MOE of Class V concretes at 28 days

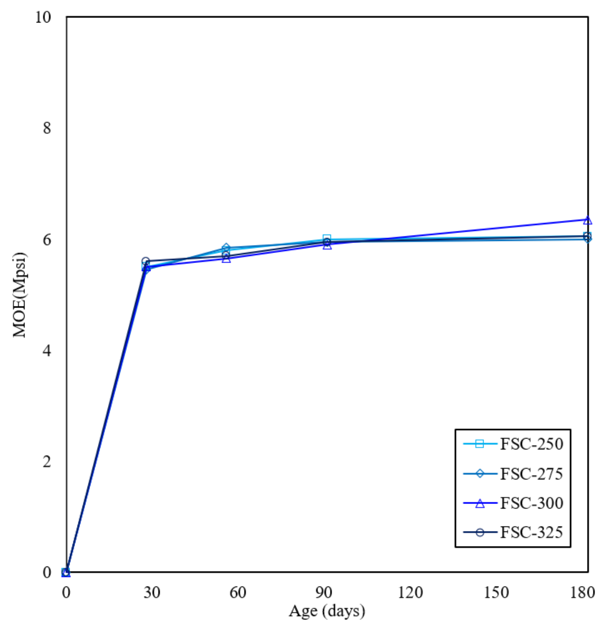

Figure 5-56 MOE development of Class V concretes in FSC group

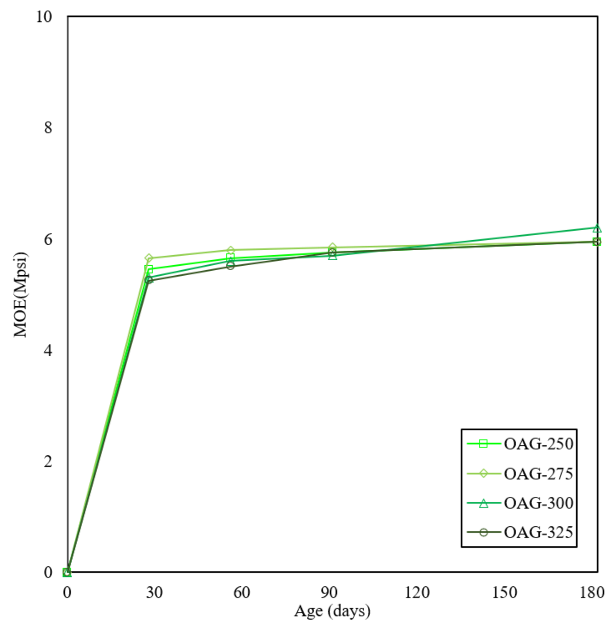

Figure 5-57 MOE development of Class V concretes in OAG group

5.2.5 Poisson's Ratio

5.2.5.1 Class II concrete

Table 5-23 summarizes the average Poisson's ratio of the Class II concrete mixtures evaluated in this research study. Figure 5-58 shows the Poisson's ratio of the Class II concrete at 28 days. Figure 5-59 and 5-60 present plots of Poisson's ratio of the FSC and OAG mixes at 28, 56, 91, and 182 days. The Poisson's ratio of all the mixtures evaluated were in the normal range (0.20-0.25) at all ages.

Table 5-23 Average Poisson's ratio of the Class II concrete mixtures tested

w/cm ratio = 0.50	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Poisson's Ratio	22.5% (450)		25.0% (500)		27.5% (550)		30.0% (600)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
28 days	0.22	0.21	0.21	0.23	0.22	0.23	0.22	0.24
56 days	0.22	0.21	0.22	0.22	0.21	0.22	0.22	0.21
91 days	0.21	0.21	0.22	0.22	0.22	0.23	0.22	0.22
182 days	0.22	0.22	0.20	0.22	0.22	0.22	0.22	0.23

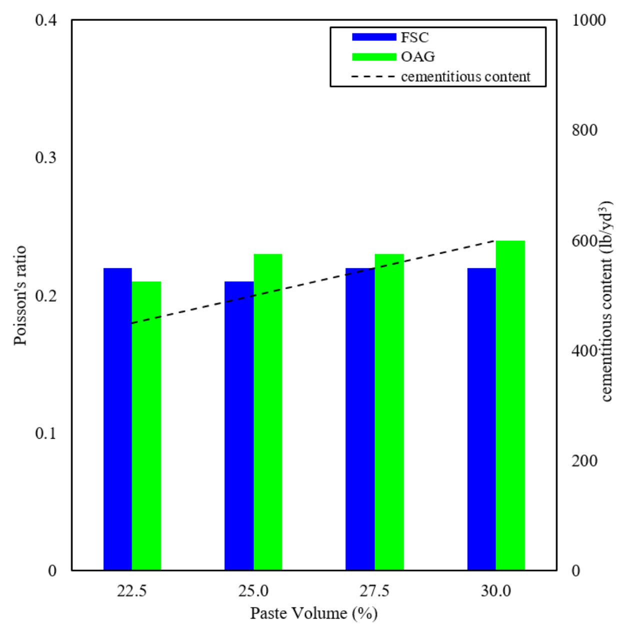

Figure 5-58 Poisson's ratio of Class II concretes at 28 days

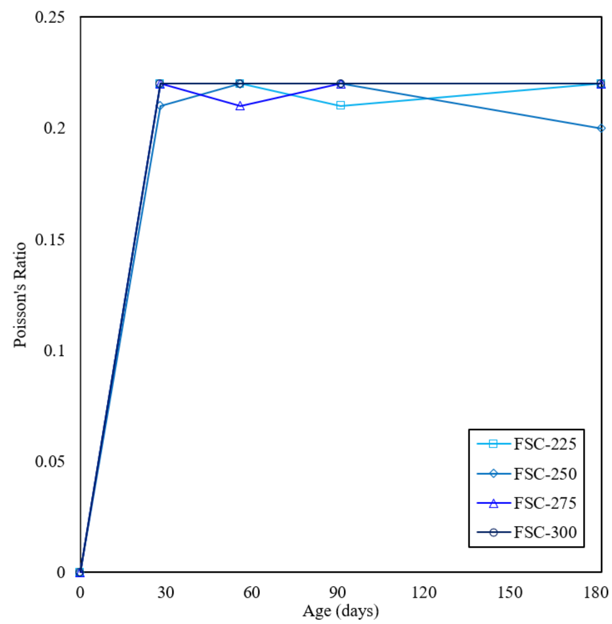

Figure 5-59 Poisson’s ratio development of Class II concretes in FSC group

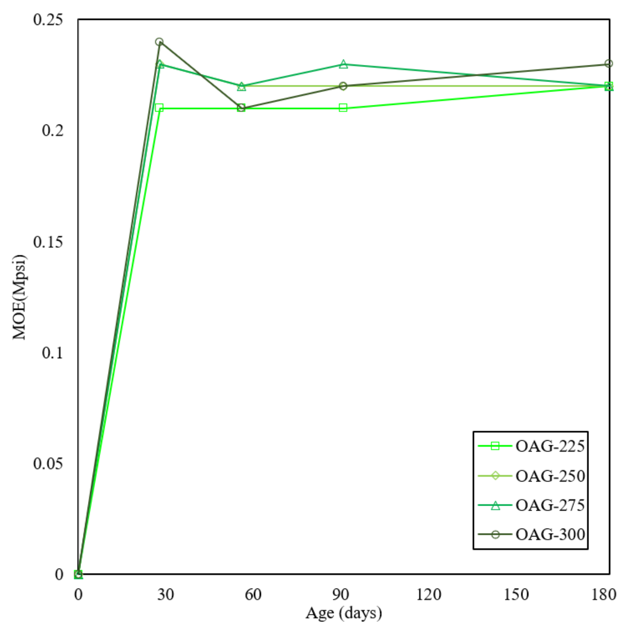

Figure 5-60 Poisson’s ratio development of Class II concretes in OAG group

5.2.5.2 Class II - Bridge Deck concrete

Table 5-24 summarizes the average Poisson’s ratio of the Class II - Bridge Deck concrete mixtures evaluated in this research study. Figure 5-61 shows the Poisson’s ratio of the Class II - Bridge Deck concrete at 28 days. Figure 5-62 and 5-63 present plots of Poisson’s ratio of the FSC and OAG mixes at 28, 56, 91, and 182 days. The Poisson's ratio of all the mixtures evaluated were in the normal range (0.20-0.25) at all ages.

Table 5-24 Average Poisson's ratio of the Class II - Bridge Deck concrete mixture tested

w/cm ratio = 0.44	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Poisson's Ratio	22.5% (485)		25.0% (450)		27.5% (595)		30.0% (645)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
28 days	0.22	0.21	0.21	0.23	0.22	0.23	0.22	0.24
56 days	0.22	0.21	0.22	0.22	0.21	0.22	0.22	0.21
91 days	0.21	0.21	0.22	0.22	0.22	0.23	0.22	0.22
182 days	0.22	0.21	0.22	0.22	0.22	0.22	0.22	0.22

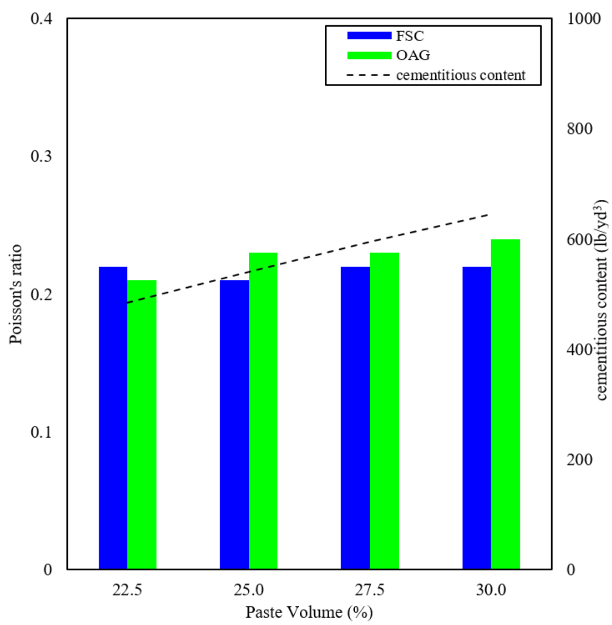

Figure 5-61 Poisson's ratio of Class II - Bridge Deck concretes at 28 days

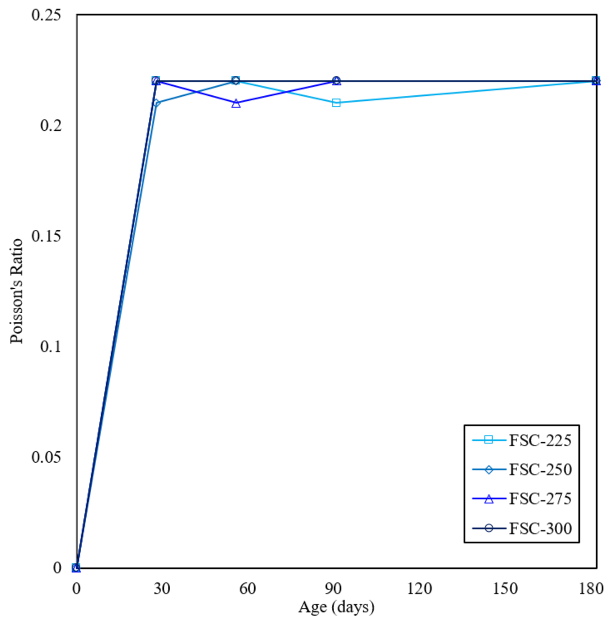

Figure 5-62 Poisson's ratio development of Class II - Bridge Deck concretes in FSC group

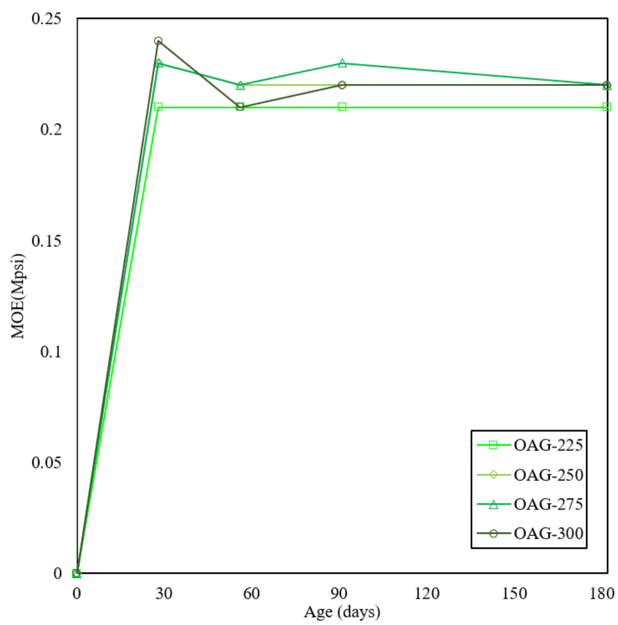

Figure 5-63 Poisson's ratio development of Class II - Bridge Deck concretes in OAG group

5.2.5.3 Class IV concrete

Table 5-25 summarizes the average Poisson's ratio of all the Class IV concrete mixtures evaluated in this research study. Figure 5-64 shows the Poisson's ratio of the Class IV concrete at 28 days. Figure 5-65 and 5-66 present plots of Poisson's ratio of the FSC and OAG mixes at 28, 56, 91, and 182 days. The Poisson's ratio of all the mixtures evaluated were in the normal range (0.20-0.25) at all ages.

Table 5-25 Average Poisson's ratio of the Class IV concrete mixtures tested

w/cm ratio = 0.38	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Poisson's ratio	25.0% (580)		27.5% (645)		30.0% (700)		32.5% (760)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
28 days	0.22	0.21	0.22	0.23	0.22	0.22	0.23	0.23
56 days	0.22	0.21	0.22	0.23	0.22	0.23	0.22	0.23
91 days	0.21	0.23	0.21	0.23	0.22	0.24	0.22	0.24
182 days	0.22	0.23	0.22	0.23	0.23	0.22	0.23	0.24

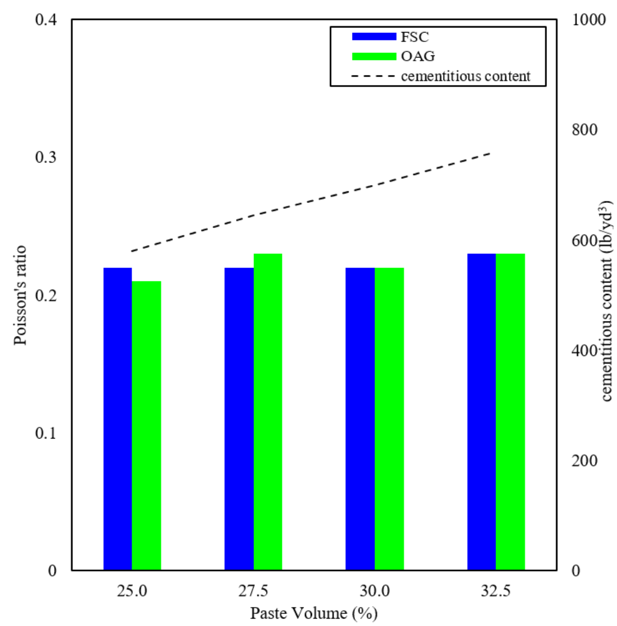

Figure 5-64 Poisson's ratio of Class IV concretes at 28 days

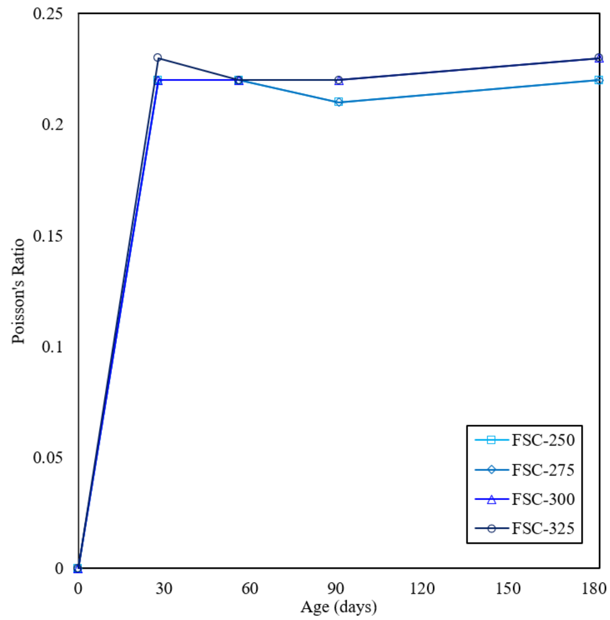

Figure 5-65 Poisson’s ratio development of Class IV concretes in FSC group

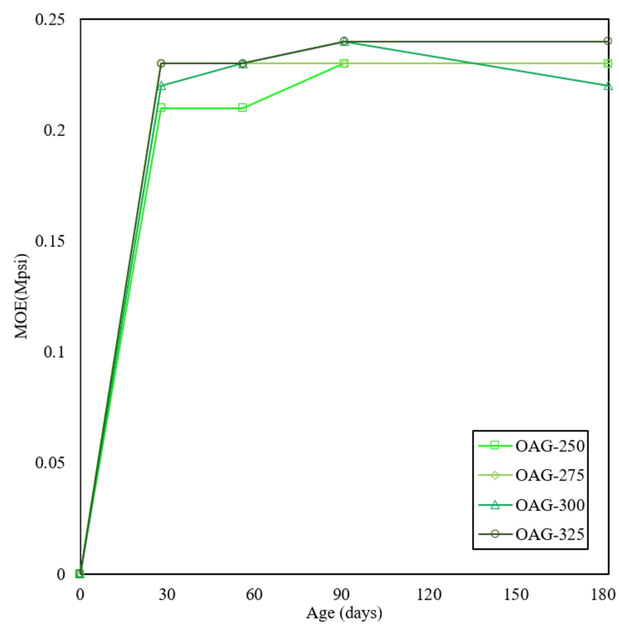

Figure 5-66 Poisson’s ratio development of Class IV concretes in OAG group

5.2.5.4 Class V concrete

Table 5-26 summarizes the average Poisson’s ratio of the Class V concrete mixtures evaluated in this research study. Figure 5-67 shows the Poisson’s ratio of the Class V concrete at 28 days. Figure 5-68 and 5-69 present plots of Poisson’s ratio of the FSC and OAG mixes at 28, 56, 91, and 182 days. The Poisson's ratio of all the mixtures evaluated were in the normal range (0.20-0.25) at all ages.

Table 5-26 Average Poisson's ratio of the Class V concrete mixtures tested

w/cm ratio = 0.32	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Poisson's ratio	25.0% (635)		27.5% (700)		30.0% (765)		32.5% (830)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
28 days	0.22	0.22	0.22	0.23	0.23	0.23	0.22	0.23
56 days	0.24	0.23	0.21	0.22	0.23	0.23	0.22	0.24
91 days	0.24	0.23	0.22	0.24	0.22	0.23	0.23	0.24
182 days	0.22	0.23	0.22	0.24	0.21	0.23	0.23	0.23

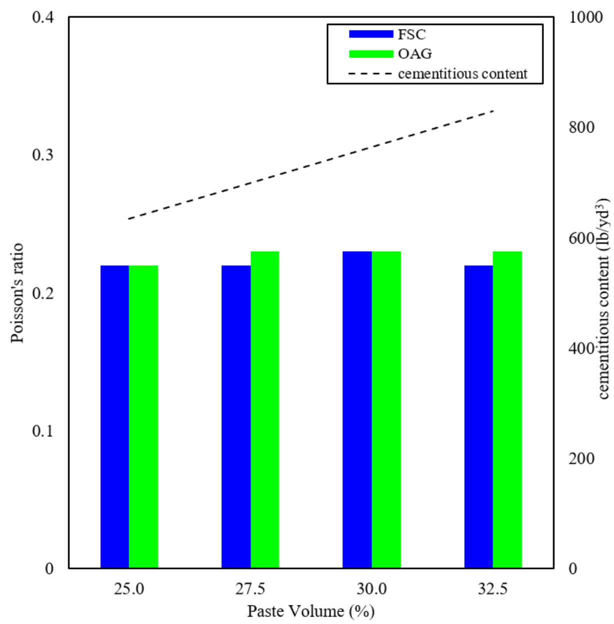

Figure 5-67 Poisson's ratio of Class V concretes at 28 days

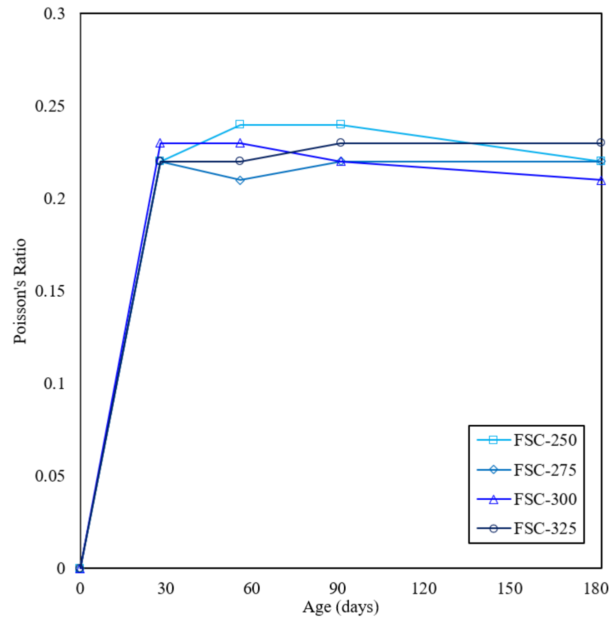

Figure 5-68 Poisson’s ratio development of Class V concretes in FSC group

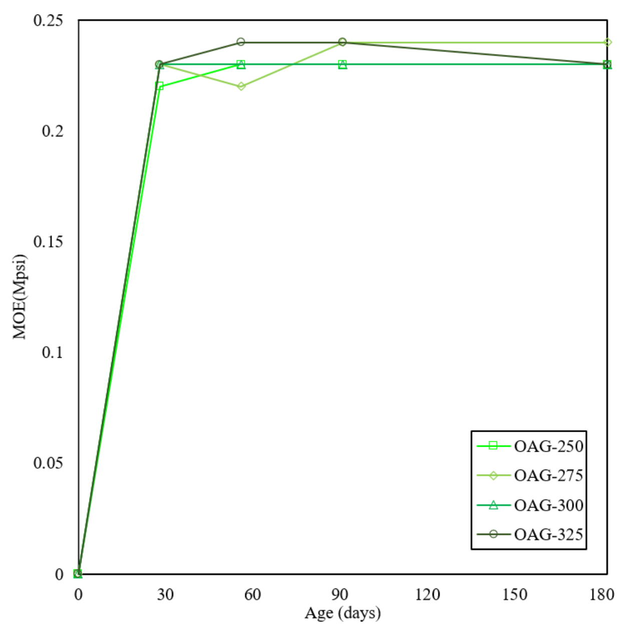

Figure 5-69 Poisson’s ratio development of Class V concretes in OAG group

5.2.6 Drying Shrinkage

5.2.6.1 Class II concrete

The average drying shrinkage results, measured up to 365 days, are shown in Table 5-27. Figure 5-70 shows the drying shrinkage of the Class II concrete at 365 days. The average drying shrinkage results, measured up to 365 days, are shown in Figures 5-71 and 5-72. The drying shrinkage specimens were stored in the curing room for 28 days before they were left to dry, according to standard ASTM C157. The results showed that concrete specimens expanded up to 28 days before they were left to dry, and the expansion increased when the paste volume was increased. It seems that the OAG mixes have a lower expansion than the FSC mixes. The drying shrinkage decreased when the paste volume was decreased. For all mixes, drying shrinkage decreased with reduced paste volume, and the OAG mixes appeared to have lower drying shrinkage than the FSC mixes. When the OAG technique is used in conjunction with Type IL cement, drying shrinkage of the concrete can be reduced substantially.

Table 5-27 Average drying shrinkage (με) of the Class II concrete mixtures

w/cm ratio = 0.50	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Drying Shrinkage (με)	22.5% (450)		25.0% (500)		27.5% (550)		30.0% (600)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
7 days	83	70	62	72	88	63	112	112
28 days	110	97	103	102	115	100	150	150
56 days	-77	-77	-193	-177	-152	-173	-143	-143
91 days	-228	-183	-230	-218	-255	-222	-260	-260
182 days	-233	-200	-258	-240	-277	-255	-272	-272
365 days	-257	-242	-282	-262	-290	-270	-298	-298

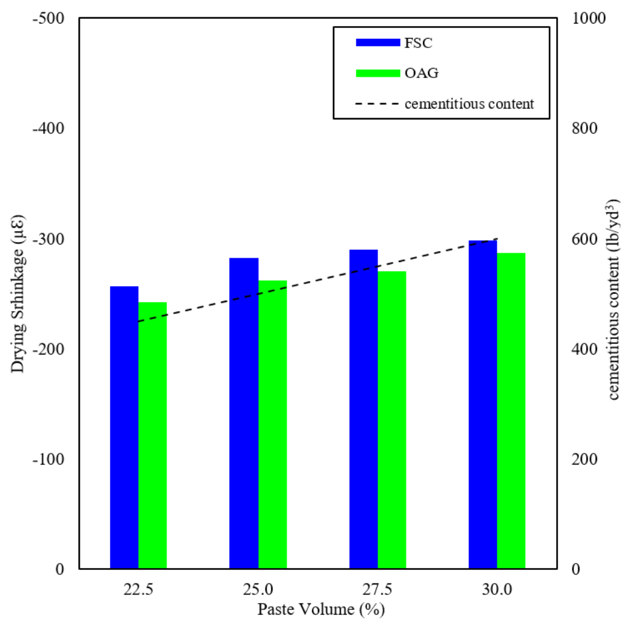

Figure 5-70 Drying shrinkage of Class II concretes at 365 days

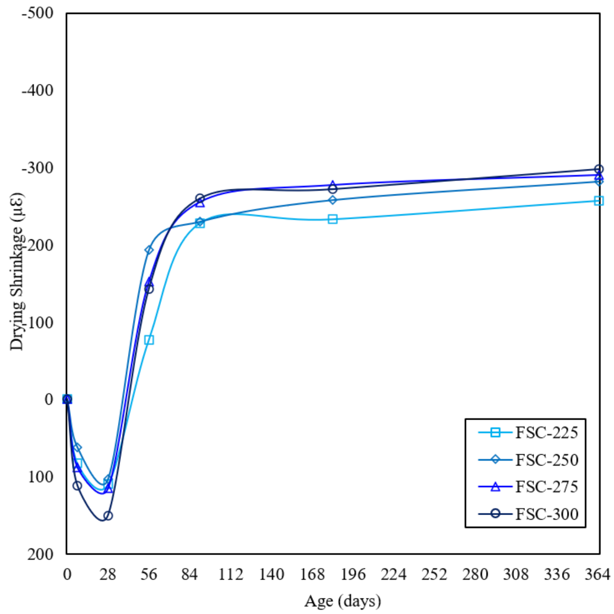

Figure 5-71 Drying shrinkage development of Class II concretes in FSC group

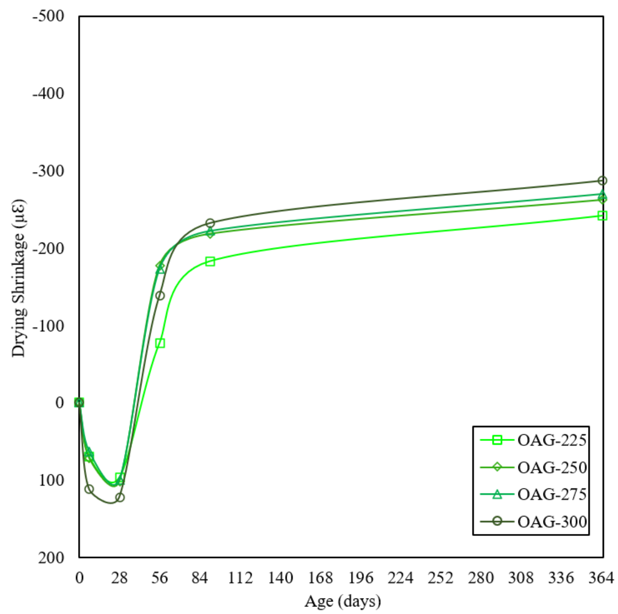

Figure 5-72 Drying shrinkage development of Class II concretes in OAG group

5.2.6.2 Class II - Bridge Deck concrete

The average drying shrinkage results for the Class II - Bridge Deck concrete mixes, measured up to 365 days, are shown in Table 5-28. Figure 5-73 shows the drying shrinkage of the Class II - Bridge Deck concrete at 365 days. The average drying shrinkage results, measured up to 365 days, are shown in Figures 5-74 and 5-75. According to the results, the higher paste volume could cause the concrete to expand more during curing. The drying shrinkage decreased when the paste volume was decreased. For all mixes, drying shrinkage decreased with reduced paste volume, and the OAG mixes appeared to have lower drying shrinkage than the FSC mixes. It seems that the OAG mixes have lower expansion during curing than the FSC mixes. OAG technique could reduce the drying shrinkage of concrete. When the OAG technique is used in conjunction with Type IL cement, drying shrinkage of the concrete can be reduced substantially.

Table 5-28 Average drying shrinkage (με) of the Class II - Bridge Deck concrete mixtures

w/cm ratio = 0.44	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Drying Shrinkage (με)	22.5% (485)		25.0% (450)		27.5% (595)		30.0% (645)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
7 days	38	37	37	42	67	53	55	85
28 days	85	87	60	62	117	107	117	122
56 days	-105	-117	-167	-182	-218	-197	-222	-180
91 days	-188	-182	-207	-203	-257	-227	-250	-218
182 days	-227	-223	-210	-233	-237	-247	-262	-245
365 days	-268	-240	-258	-255	-283	-285	-330	-303

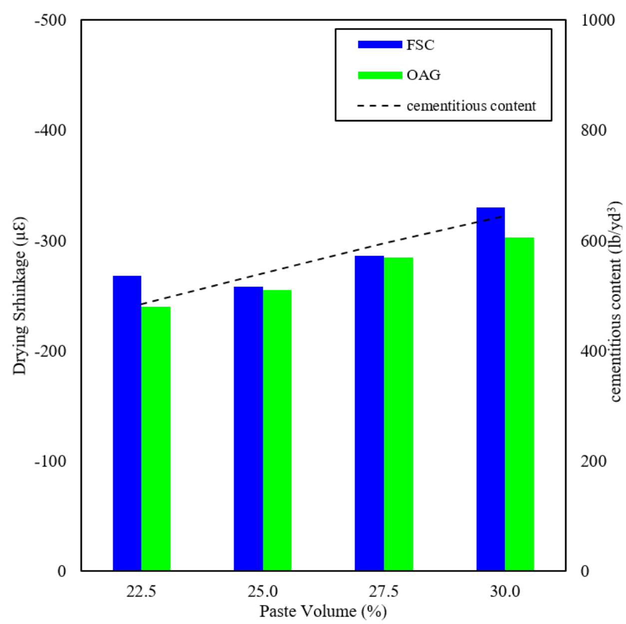

Figure 5-73 Drying shrinkage of Class II - Bridge Deck concretes at 91 days

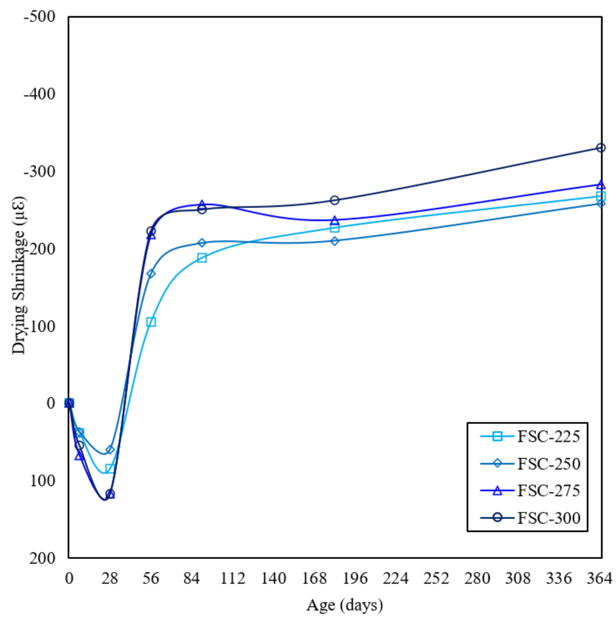

Figure 5-74 Drying shrinkage development of Class II - Bridge Deck concretes in FSC group

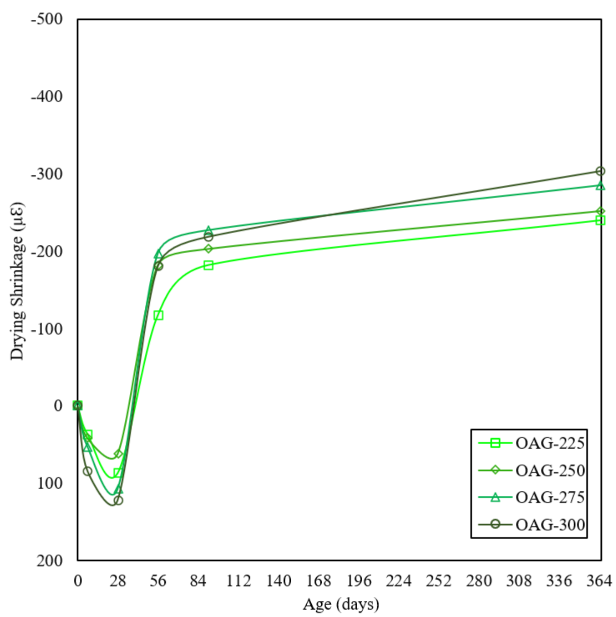

Figure 5-75 Drying shrinkage development of Class II - Bridge Deck concretes in OAG group

5.2.6.3 Class IV concrete

The average drying shrinkage results for the Class IV concrete mixes, measured up to 365 days, are shown in Table 5-29. Figure 5-76 shows the drying shrinkage of the Class IV concrete at 365 days. The average drying shrinkage results, measured up to 365 days, are shown in Figures 5-77 and 5-78. According to the results, the higher paste volume could cause the concrete to expand more during curing in the first 28 days. The drying shrinkage decreased when the paste volume was decreased. For all mixes, drying shrinkage decreased with reduced paste volume, and the OAG mixes appeared to have lower drying shrinkage than the FSC mixes. It seems that the OAG mixes has a lower expansion during curing than the FSC mixes. OAG technique could reduce the drying shrinkage of concrete. When the OAG technique is used in conjunction with Type IL cement, drying shrinkage of the concrete can be reduced substantially.

Table 5-29 Average drying shrinkage (με) of the Class IV concrete mixtures

w/cm ratio = 0.38	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Drying Shrinkage (με)	25.0% (580)		27.5% (645)		30.0% (700)		32.5% (760)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
7 days	-7	22	7	12	15	2	48	27
28 days	75	80	40	25	53	93	52	43
56 days	-97	-100	-180	-153	-153	-138	-187	-152
91 days	-200	-187	-252	-222	-257	-263	-292	-272
182 days	-283	-265	-313	-247	-347	-327	-357	-360
365 days	-300	-277	-345	-290	-388	-378	-388	-375

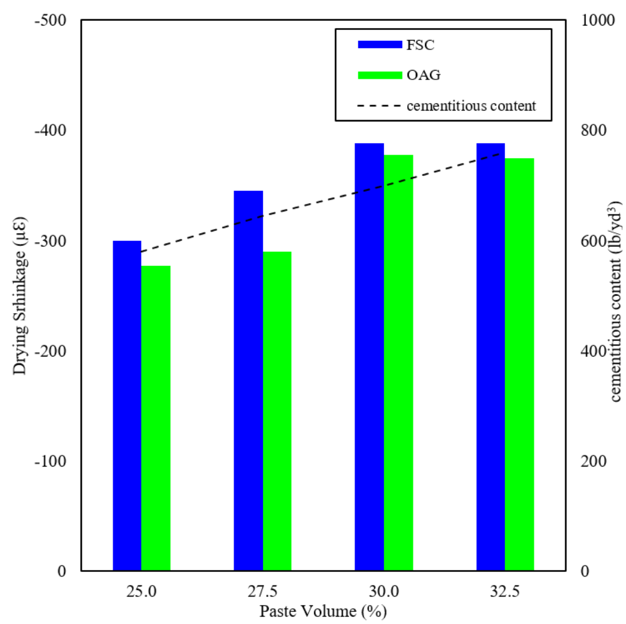

Figure 5-76 Drying shrinkage of Class IV concretes at 365 days

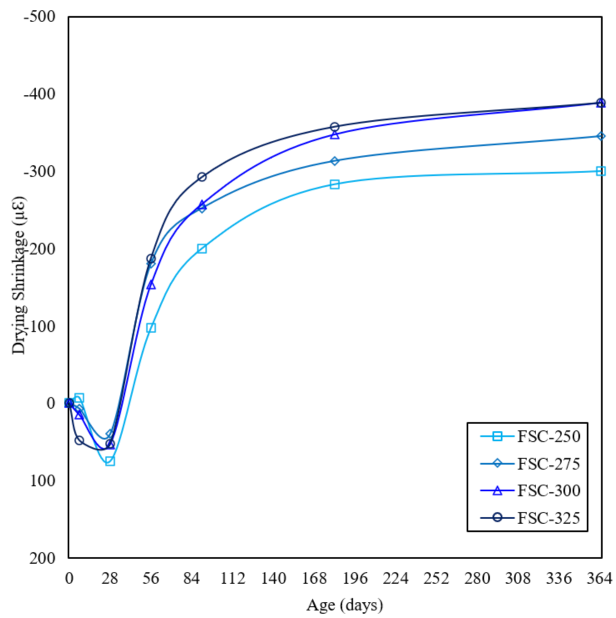

Figure 5-77 Drying shrinkage development of Class IV concretes in FSC group

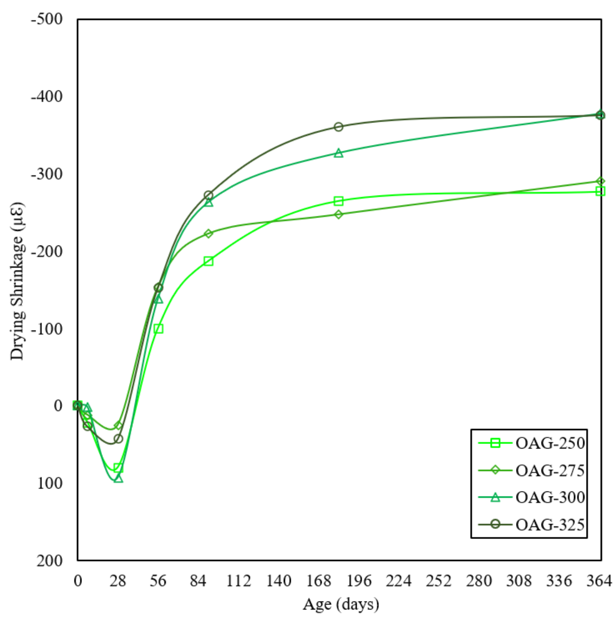

Figure 5-78 Drying shrinkage development of Class IV concretes in OAG group

5.2.6.4 Class V concrete

The average drying shrinkage results for the Class V concrete mixes, measured up to 365 days, are shown in Table 5-30. Figure 5-79 shows the drying shrinkage of the Class V concrete at 365 days. The average drying shrinkage results, measured up to 365 days, are shown in Figures 5-80 and 5-81. According to the results, the higher paste volume could cause the concrete to expand more during curing. The drying shrinkage decreased when the paste volume was decreased. For all mixes, drying shrinkage decreased with reduced paste volume, and the OAG mixes appeared to have lower drying shrinkage than the FSC mixes. It seems that the OAG mixes has a lower expansion during curing than the FSC mixes. OAG technique could reduce the drying shrinkage of concrete. When the OAG technique is used in conjunction with Type IL cement, drying shrinkage of the concrete can be reduced substantially.

Table 5-30 Average drying shrinkage (με) of the Class V concrete mixtures

w/cm ratio = 0.32	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Drying Shrinkage (με)	25.0% (635)		27.5% (700)		30.0% (765)		32.5% (830)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
7 days	-5	-2	20	27	-5	23	30	15
28 days	23	3	13	88	33	80	82	43
56 days	-162	-187	-180	-120	-247	-220	-243	-227
91 days	-217	-233	-268	-242	-280	-267	-305	-277
182 days	-273	-270	-293	-287	-392	-312	-417	-383
365 days	-317	-357	-358	-368	-415	-410	-445	-425

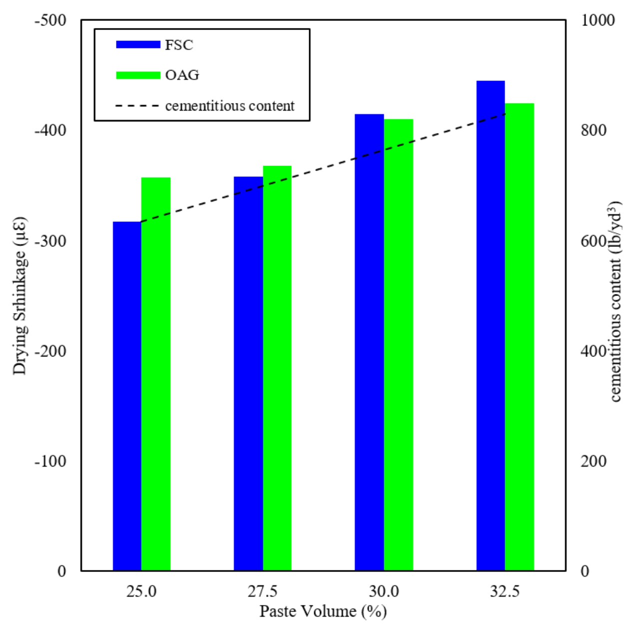

Figure 5-79 Drying shrinkage of Class V concretes at 365 days

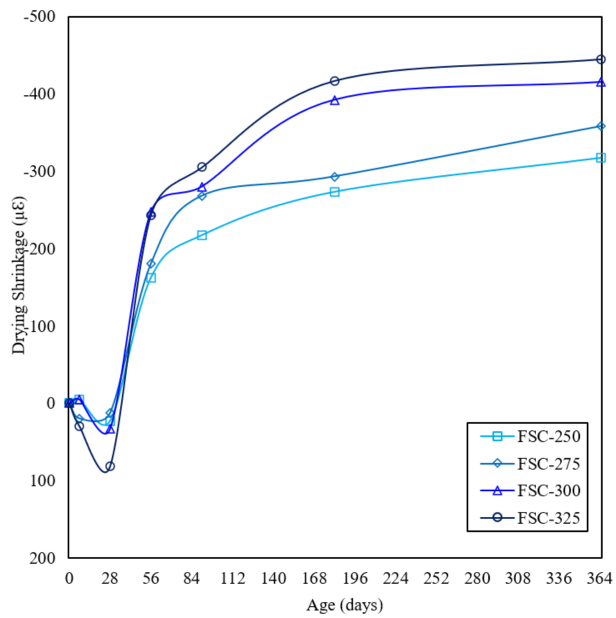

Figure 5-80 Drying shrinkage development of Class V concretes in FSC group

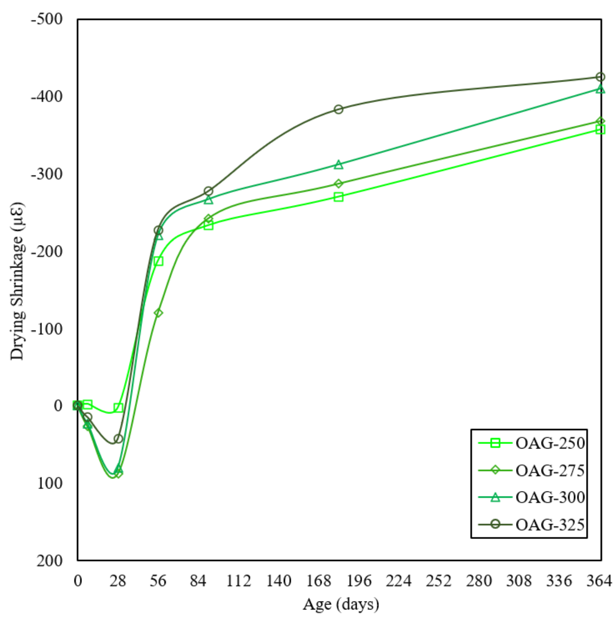

Figure 5-81 Drying shrinkage development of Class V concretes in OAG group

5.2.7 Results of Electrical Resistance Tests

5.2.7.1 Class II concrete

Table 5-31 summarizes the Rapid Chloride Permeability Test (RCPT) results for all the Class II concrete mixes evaluated in this research study. The RCPT results for the FSC and OAG concrete mixes are similar to one another and showed no strong trends with CPV according to this set of test results. All the concrete mixes have similar level of electrical resistance. The RCPT values of the concrete mixes decreased as the CPV decreased. This indicates that reducing the CPV reduces the permeability of the concrete.

The average results of SR tests for the Class II concrete mixes are shown in Table 5-32. The SR results of the FSC and OAG mixes also were similar to one another. The SR results mostly increased when the CPV decreased. The SR results for the FSC concretes were inversely related to the RCPT results, in which the highest resistivity occurred at 22.5% paste volume and the lowest resistivity (highest conductivity) occurred at 30.0% paste volume.

Figure 5-82 shows the plot of the results of RCPT versus the results of SR. The results of these two permeability tests have a strong inverse correlation with each other. The permeability of this mix at 28 days ranged from moderate to high in value. When the paste volume was under 22.5%, the permeability level was moderate. When the paste volume was above 25.0%, the permeability level of the concrete was high. Thus, the permeability of the concrete at an early age was lower when the concrete paste volume was lower. It can indicate that reducing paste volume might mitigate the early cracking of concrete.

Table 5-31 RCPT (coulombs) results of tested Class II concrete mixtures

w/cm ratio = 0.50	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
RCPT (coulombs)	22.5% (450)		25.0% (500)		27.5% (550)		30.0% (600)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
28 days	3,507	3,909	3,644	3,875	4,257	4,328	4,546	4,712
Level of Permeability	M	M	M	M	H	H	H	H
365 days	704	863	810	841	884	959	992	999
Level of Permeability	VL	VL	VL	VL	VL	VL	VL	VL

Note: Table 4-1 and 4-2 presents the values of RCPT and SR tests in terms of concrete permeability level. H=High, M=Moderate, L=Low, VL=Very low, and N=Negligible

Table 5-32 Average surface resistivity (kΩ-cm) of tested Class II concrete mixtures

w/cm ratio = 0.50	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Surface Resistivity (kΩ-cm)	22.5% (450)		25.0% (500)		27.5% (550)		30.0% (600)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
7 days	6.62	5.44	5.25	5.22	5.53	5.32	5.08	4.93
28 days	11.91	12.02	10.75	9.45	8.47	8.05	8.79	8.81
56 days	16.09	17.08	15.37	14.41	12.81	13.28	12.20	11.41
91 days	19.70	19.50	18.49	18.35	18.37	18.50	14.88	14.30
182 days	36.38	38.55	33.71	31.05	29.11	26.18	28.86	28.11
365 days	48.03	46.62	43.13	44.55	43.41	42.63	30.10	26.53

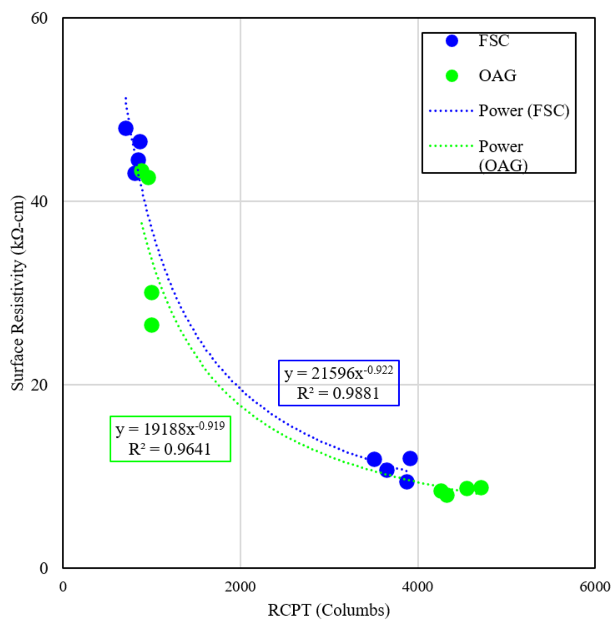

Figure 5-82 RCPT versus surface resistivity in Class II concretes at 28 and 365 days

5.2.7.2 Class II - Bridge Deck concrete

Table 5-33 summarizes the RCPT results for all the Class II - Bridge Deck concrete mixes evaluated in this research study. The RCPT results for the FSC and OAG concrete mixes are similar to one another and showed no strong trends with CPV according to this set of test results. All the concrete mixes have similar level of electrical resistance. But the RCPT values of the concrete mix decreased when the concrete had lower CPV. This indicates that reducing the CPV reduces the permeability of the concrete.

The average results of SR tests for the Class II - Bridge Deck concrete mixes are shown in Table 5-34. The SR results of the FSC and OAG mixes also were similar to one another. The SR results mostly increased when the CPV decreased. The SR results for the FSC concretes were inversely proportional to the RCPT results, in which the highest resistivity occurred at 22.5% paste volume and the lowest resistivity (highest conductivity) occurred at 30.0% paste volume.

Figure 5-83 shows the plot of the results of RCPT versus the results of SR. The results of these two permeability tests have a strong correlation with each other. The permeability of this mix was between moderate to high in value. When the paste volume was under 22.5%, the permeability level was moderate. When the paste volume was above 27.5%, the permeability level of the concrete was high. Thus, the permeability of the concrete at an early age decreased when the concrete decreased.

Table 5-33 RCPT (coulombs) results of tested Class II - Bridge Deck concrete mixtures

w/cm ratio = 0.44	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
RCPT (coulombs)	22.5% (485)		25.0% (450)		27.5% (595)		30.0% (645)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
28 days	2,707	2,956	2,462	2,423	3,049	3,459	4,108	4,285
Level of Permeability	M	M	M	M	M	M	H	H
365 days	583	682	621	695	771	810	920	961
Level of Permeability	VL	VL	VL	VL	VL	VL	VL	VL

Note: Table 4-1 and 4-2 presents the values of RCPT and SR tests in terms of concrete permeability level. H=High, M=Moderate, L=Low, VL=Very low, and N=Negligible

Table 5-34 Average surface resistivity (kΩ-cm) of tested Class II - Bridge Deck concrete mixtures

w/cm ratio = 0.44	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
surface resistivity (kΩ-cm)	22.5% (485)		25.0% (450)		27.5% (595)		30.0% (645)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
7 days	6.39	6.73	6.29	5.98	6.12	5.75	4.92	4.93
28 days	10.00	9.90	9.79	9.71	9.68	9.23	8.96	9.30
56 days	16.24	15.06	15.85	14.96	15.44	15.18	13.47	12.96
91 days	23.04	22.57	22.22	22.49	21.71	22.38	18.95	17.58
182 days	34.10	33.65	34.47	33.01	32.40	31.53	24.57	23.14
365 days	53.95	50.17	51.62	47.27	51.35	48.28	42.28	40.28

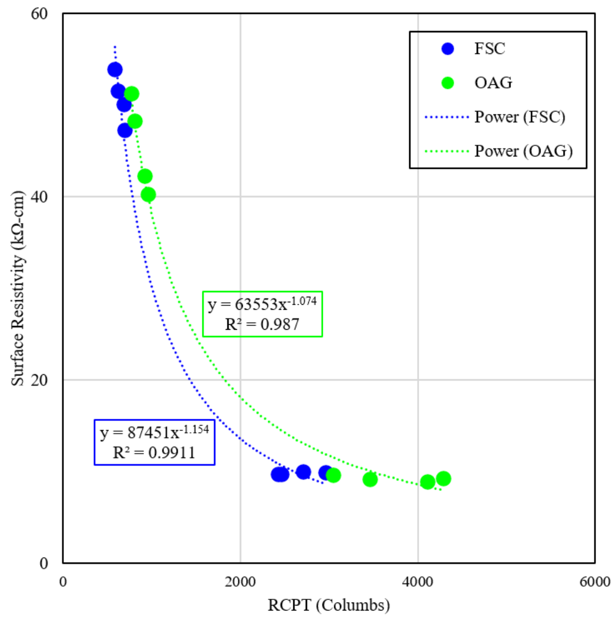

Figure 5-83 RCPT versus surface resistivity of Class II - Bridge Deck concretes at 28 and 365 days

5.2.7.3 Class IV concrete

Table 5-35 summarizes the RCPT results for all the Class IV concrete mixes evaluated in this research study. The RCPT results for the FSC and OAG concrete mixes are similar to one another and showed no strong trends with CPV according to this set of test results. All the concrete mixes had similar levels of electrical resistance. But RCPT values of the concrete mix decreased as the concrete CPV decreased. This indicates that reducing the CPV reduces the permeability of the concrete.

The average results of SR tests for the Class IV concrete mixes are shown in Table 5-36. The SR results of the FSC and OAG mixes also were similar to one another. The SR results mostly increased when the CPV decreased. The SR results for the FSC concretes appear to correlate to the RCPT results, in which the highest resistivity occurred at 25.0% paste volume and the lowest resistivity (highest conductivity) occurred at 32.5% paste volume.

Figure 5-84 shows the plot of the results of RCPT versus the results of SR for the Class IV concrete, along with the AASHTO T277 and ASTM C1202 limits for the various categories of durability. The results of these two permeability tests have a strong inverse relationship. The permeability of this mix at 28 days was between moderate to high in value. When the paste volume was under 25.0%, the permeability level was moderate. When the paste volume was above 30.0%, the permeability level of the concrete was high. Thus, the permeability of the concrete at an early age was reduced when the concrete paste volume was reduced.

Table 5-35 RCPT (coulombs) results of tested Class IV concrete mixtures

w/cm ratio = 0.38	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
RCPT (coulombs)	25.0% (580)		27.5% (645)		30.0% (700)		32.5% (760)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
28 days	1,940	2,218	2,554	3,021	2,433	2,853	2,965	3,810
Level of Permeability	M	M	M	M	M	M	H	H
365 days	465	527	486	533	502	432	580	658
Level of Permeability	VL	VL	VL	VL	VL	VL	VL	VL

Note: Table 4-1 and 4-2 presents the values of RCPT and SR tests in terms of concrete permeability level. H=High, M=Moderate, L=Low, VL=Very low, and N=Negligible

Table 5-36 Average surface resistivity (kΩ-cm) of tested Class IV concrete mixtures

w/cm ratio = 0.38	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Surface Resistivity (kΩ-cm)	25.0% (580)		27.5% (645)		30.0% (700)		32.5% (760)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
7 days	7.38	7.57	6.91	6.72	6.97	6.02	6.40	5.67
28 days	12.94	12.48	12.21	12.70	11.26	10.81	9.87	8.41
56 days	20.17	19.14	19.62	18.95	17.35	16.11	16.71	15.15
91 days	28.81	28.00	24.90	23.68	24.04	22.72	23.73	22.63
182 days	32.13	33.85	31.44	29.99	30.96	30.66	29.87	28.51
365 days	58.23	53.38	55.91	51.59	55.31	51.25	45.44	43.48

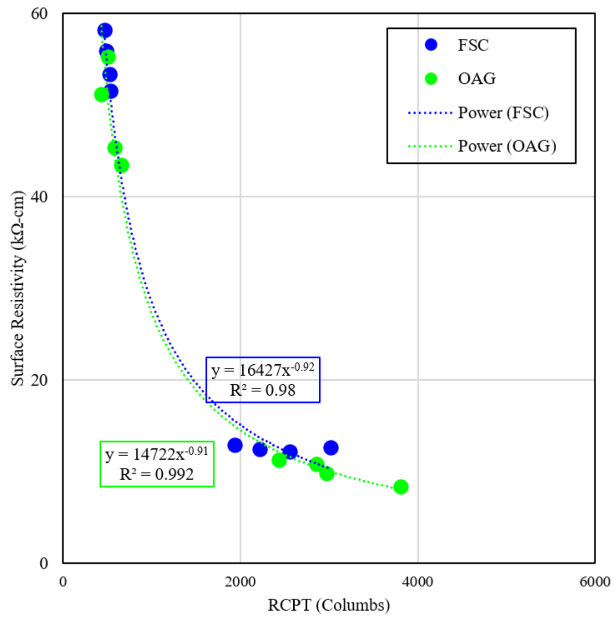

Figure 5-84 RCPT versus surface resistivity of Class IV concretes at 28 and 365 days

5.2.7.4 Class V concrete

Table 5-37 summarizes the RCPT results for all the Class V concrete mixes evaluated in this research study. The RCPT results for the FSC and OAG concrete mixes are similar to one another and show no strong trends with CPV. All the concrete mixes have similar electrical resistance values. RCPT values of the concrete mix decreased when the concrete CPV decreased. This indicates that reducing the CPV reduces the permeability of the concrete.

The average results of SR tests for the Class V concretes are shown in Table 6-38. The SR results of the FSC and OAG mixes were similar to one another. The SR results mostly increased when the CPV decreased. The SR results for the FSC concretes appear to correlate to the RCPT results, in which the highest resistivity occurred at 25.0% paste volume and the lowest resistivity (highest conductivity) occurred at 32.5% paste volume.

Figure 5-85 shows the plot of the results of RCPT versus the results of SR for the Class V concrete mixes, along with the AASHTO T277 and ASTM C1202 categories for the various durability levels. Although the lack of intermediate SR-RCPT values limits the ability to evaluate any correlation between the results of the two tests for the Class V concrete mixes, they are expected to have a high degree of correlation. The permeability of the Class V concrete mixes were between low to moderate in value. When the paste volume was under 25.0%, the permeability level was low. When the paste volume was above 32.5%, the permeability level of the concrete was moderate. Thus, the permeability of the concrete at an early age was reduced when the concrete paste volume decreased.

Table 5-37 RCPT (coulombs) of tested Class V concrete mixtures

w/cm ratio = 0.32	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
RCPT (coulombs)	25.0% (635)		27.5% (700)		30.0% (765)		32.5% (830)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
28 days	1,719	1,647	1,915	2,048	1,915	1,985	2,342	2,422
Level of Permeability	L	L	L	M	L	L	M	M
365 days	306	369	407	446	452	490	542	532
Level of Permeability	VL	VL	VL	VL	VL	VL	VL	VL

Note: Table 4-1 and 4-2 presents the values of RCPT and SR tests in terms of concrete permeability level. H=High, M=Moderate, L=Low, VL=Very low, and N=Negligible

Table 5-38 Average surface resistivity (kΩ-cm) of tested Class V concrete mixtures

w/cm ratio = 0.32	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Surface Resistivity (kΩ-cm)	25.0% (635)		27.5% (700)		30.0% (765)		32.5% (830)	
Ages	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
7 days	9.38	8.84	8.55	7.93	7.48	7.39	7.11	6.90
28 days	14.36	13.19	12.02	11.43	12.74	11.51	10.92	9.92
56 days	22.89	21.18	19.11	18.90	19.58	19.80	16.73	16.16
91 days	27.91	27.86	26.72	26.28	26.70	25.77	24.65	23.32
182 days	39.58	37.80	34.58	34.23	31.25	30.74	31.39	28.32
365 days	59.97	58.02	59.90	58.78	59.03	55.25	54.73	51.53

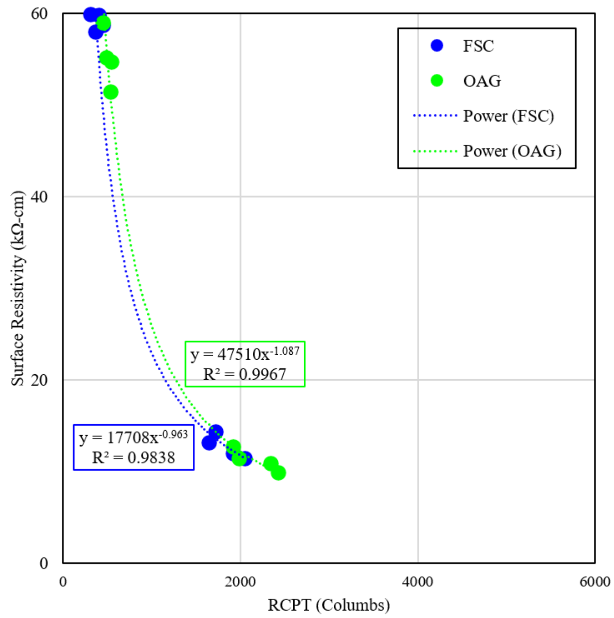

Figure 5-85 RCPT versus surface resistivity in Class V concretes at 28 and 365 days

5.2.8 Semi-adiabatic Temperature

5.2.8.1 Class II concrete

Table 5-39 show the summary of the semi-adiabatic temperature of the Class II concrete. Figures 5-86 to 5-87 present the semi-adiabatic temperature versus time for the Class II concrete investigated in this study. The results showed that the temperature of hydration of the concrete with high paste volume is higher than that of the concrete with lower paste volume in different mixes, which was expected due to the higher cementitious material contents. Concrete with a higher paste volume has a higher potential for thermal cracking at an early age. The OAG groups showed lower peak temperatures and slower rates of temperate rise than the FSC group. Type IL cement can be expected to reduce the temperature of hydration based on these results. The use of OAG technique might reduce the temperature of hydration in Class II concrete.

Table 5-39 The results of semi-adiabatic temperature of Class II concrete

w/cm ratio = 0.50	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Temperature (°F)	22.5% (450)		25.0% (500)		27.5% (550)		30.0% (600)	
	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
Initial	95.22	95.14	103.32	100.82	104.71	99.52	104.86	102.43
Maximum	70.10	72.98	72.25	74.14	71.98	70.49	73.24	72.61
Magnitude	25.12	22.17	31.07	26.69	32.73	29.03	31.63	29.83
Age of Max	12	12	12	12	12	12	12	12

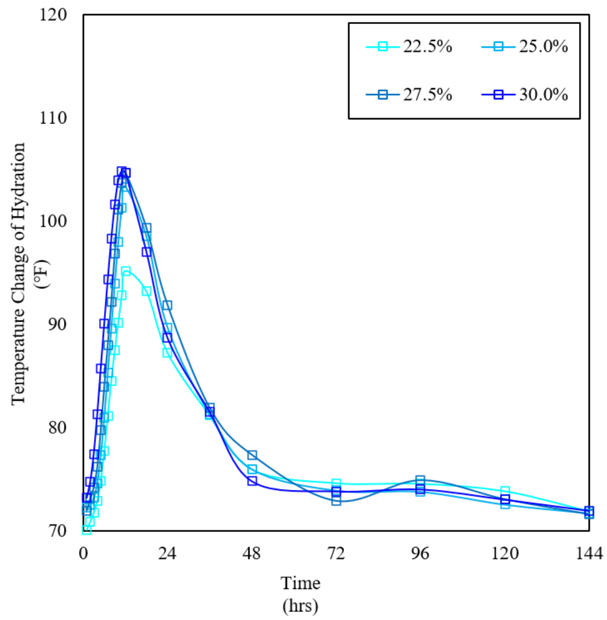

Figure 5-86 Semi-adiabatic temperature versus time of Class II concretes in FSC group

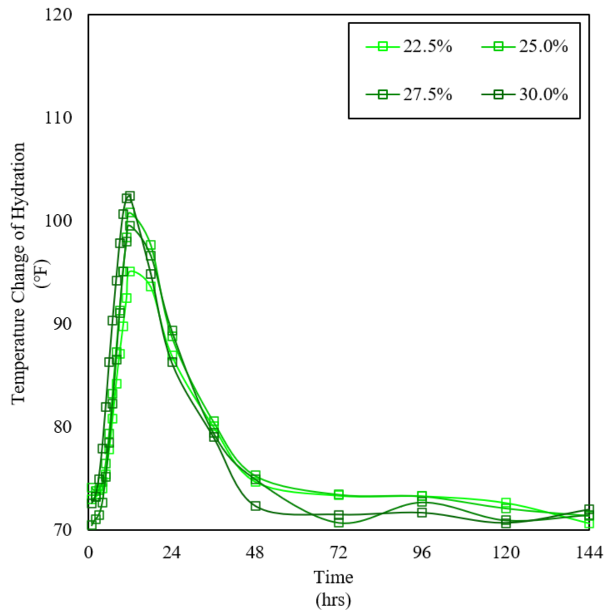

Figure 5-87 Semi-adiabatic temperature versus time of Class II concretes in OAG group

5.2.8.2 Class II - Bridge Deck concrete

Table 5-40 show the summary of the semi-adiabatic temperature of the Class II – Bridge Deck concrete. Figures 5-88 to 5-89 present the semi-adiabatic temperature versus time for the Class II- Bridge Deck concrete investigated in the present study. The results showed that the temperature of hydration of the concrete with high paste volume is higher than that of the concrete with lower paste volume in different mixes, which was expected due to the higher cementitious material contents. Concrete with a higher paste volume has a higher potential for thermal cracking at an early age. The OAG group showed lower peak temperatures and the slower rates of temperate rise than the FSC group. The use of OAG technique might reduce the temperature of hydration in Class II - Bridge Deck concrete.

Table 5-40 The results of semi-adiabatic temperature of Class II-Bridge Deck concrete

w/cm ratio = 0.44	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Temperature (°F)	22.5% (485)		25.0% (450)		27.5% (595)		30.0% (645)	
	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
Initial	97.68	97.09	100.94	98.08	108.00	104.11	111.39	111.39
Maximum	71.82	71.38	73.47	74.65	73.47	73.51	72.92	72.58
Magnitude	25.87	25.71	27.47	23.43	34.52	30.60	38.47	38.81
Age of Max	12	12	12	12	12	12	11	10

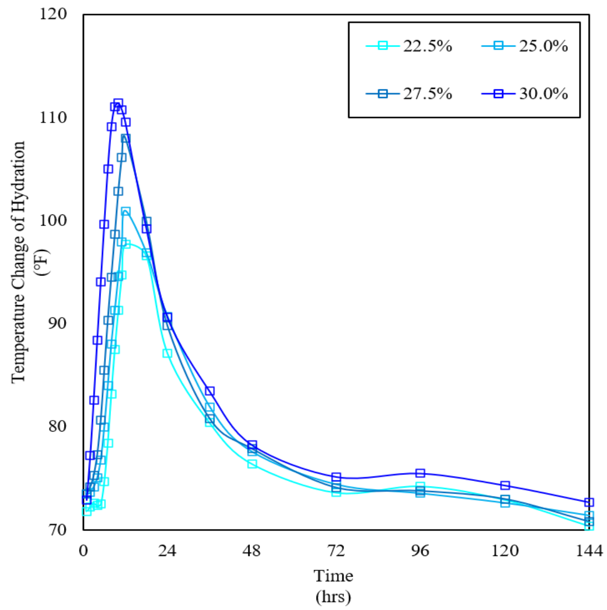

Figure 5-88 Semi-adiabatic temperature versus time of Class II - Bridge Deck concretes in FSC group

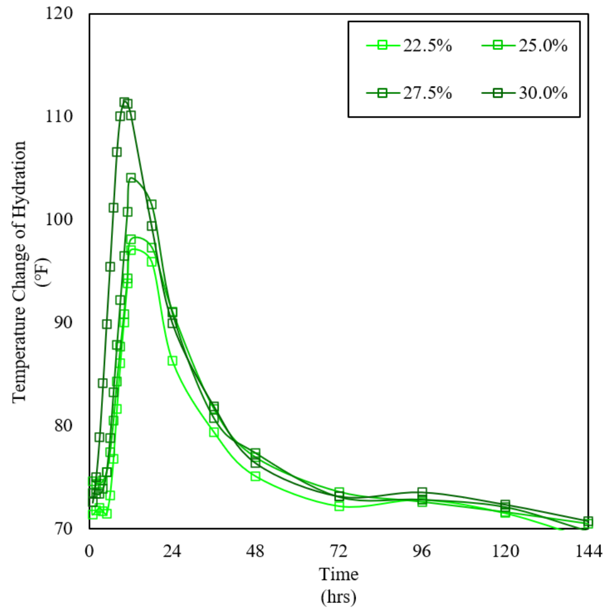

Figure 5-89 Semi-adiabatic temperature versus time of Class II - Bridge Deck concrete in OAG group

5.2.8.3 Class IV concrete

Table 5-41 show the summary of the semi-adiabatic temperature of the Class IV concrete. Figures 5-90 to 5-91 present the semi-adiabatic temperature versus time for the Class IV concrete investigated in the present study. The results showed that the temperature of hydration of the concrete with high paste volume is higher than that of the concrete with lower paste volume in different mixes, which was expected due to the higher cementitious material contents. The OAG groups showed lower peak temperatures and the slower rates of temperate rise than the FSC group. The use of OAG technique might reduce the temperature of hydration in Class IV concrete.

Table 5-41 The results of semi-adiabatic temperature of Class IV concrete

w/cm ratio = 0.38	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Temperature (°F)	25.0% (580)		27.5% (645)		30.0% (700)		32.5% (760)	
	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
Initial	106.12	102.41	111.80	108.63	119.55	114.70	118.36	112.74
Maximum	67.30	67.05	71.11	69.76	69.98	68.40	73.43	74.16
Magnitude	38.82	35.36	40.70	38.87	49.57	46.31	44.94	38.58
Age of Max	12	12	12	12	12	12	12	12

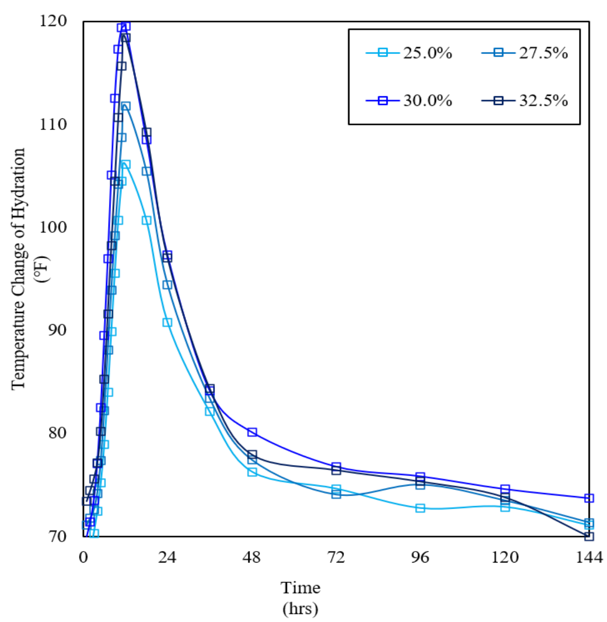

Figure 5-90 Semi-adiabatic temperature versus time of Class IV concretes in FSC group

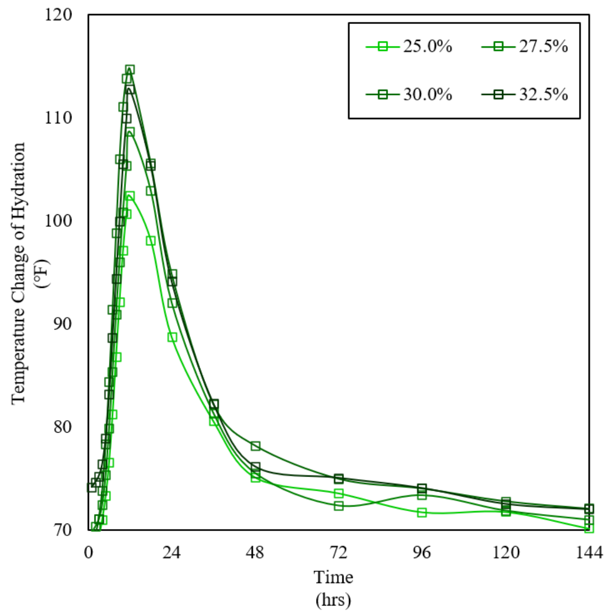

Figure 5-91 Semi-adiabatic temperature versus time of Class IV concretes in OAG group

5.2.8.4 Class V concrete

Table 5-42 show the summary of the semi-adiabatic temperature of the Class V concrete. Figures 5-92 to 5-93 present the semi-adiabatic temperature versus time for the Class V concrete investigated in the present study. The results showed that the temperature of hydration of the concrete with high paste volume is higher than that of the concrete with lower paste volume in different mixes, which was expected due to the higher cementitious material contents. Concrete with a higher paste volume has a higher potential for thermal cracking at an early age. The OAG groups showed lower peak temperatures and the slower rates of temperate rise than the FSC group. The use of OAG technique might reduce the temperature of hydration in Class V concrete.

Table 5-42 The results of semi-adiabatic temperature of Class V concrete

w/cm ratio = 0.38	Paste Volume (%) (Cementitious Materials Content, lb/yd^3)							
Temperature (°F)	25.0% (635)		27.5% (700)		30.0% (765)		32.5% (830)	
	FSC	OAG	FSC	OAG	FSC	OAG	FSC	OAG
Initial	107.01	105.22	112.06	104.46	119.27	115.52	119.14	118.15
Maximum	69.03	69.03	72.18	69.92	68.47	69.94	73.96	71.16
Magnitude	37.98	36.19	39.88	34.54	50.81	45.59	45.18	46.99
Age of Max	12	12	12	12	12	12	12	12

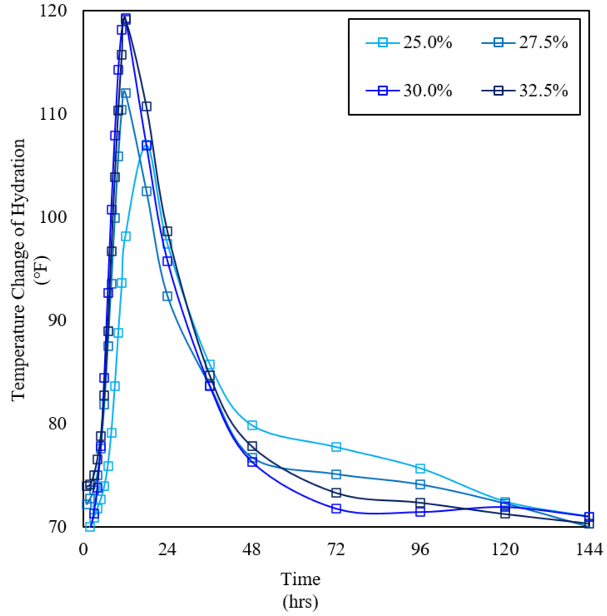

Figure 5-92 Semi-adiabatic temperature versus time of Class V concretes in FSC group

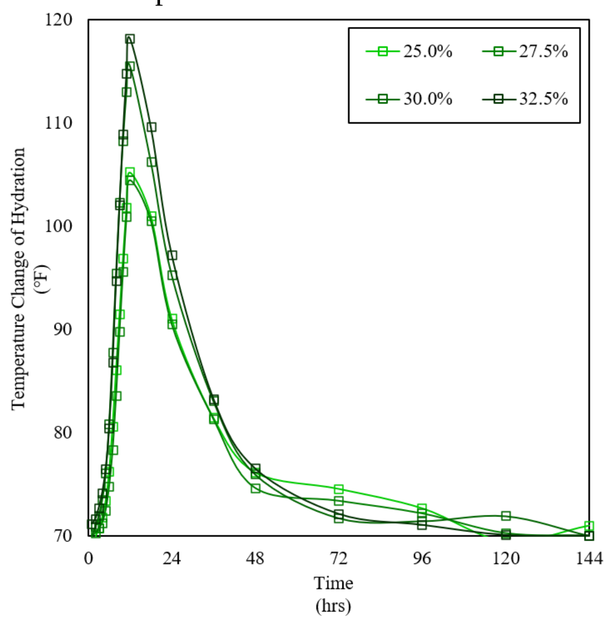

Figure 5-93 Semi-adiabatic temperature versus time of Class V concretes in OAG group

5.3 Statistical Analysis of Test Results

5.3.1 Analysis of Compressive Strength

5.3.1.1 Class II concrete

Tables 5-43 and 5-44 show the results of t-test to compare the compressive strength of the FSC and OAG concrete mixes at 28 and 91 days for Class II concrete mixes. The results indicate that there was no statistically significant difference between the compressive strength of the OAG mixes and that of the FSC mixes at α level (probability of error) of 5%.

Table 5-43 The results of t-test on the comparison of compressive strength of Class II concrete between FSC and OAG mixes at 28 days

Paste Volume	FSC		OAG		df	t	P-Value
	S1	Mean	S2	Mean			
22.5%	100.2	4645.3	177.3	4805.3	4	-1.3609	0.2625
25.0%	57.6	4932.7	63.5	5093.3	4	-3.2458	0.3019
27.5%	317.3	5250.7	382.1	5163.3	4	0.3046	0.7764
30.0%	99.4	4354.3	264.8	4142.7	4	1.2961	0.2997

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

Table 5-44 The results of t-test on the comparison of compressive strength of Class II concrete between FSC and OAG mixes at 91 days

Paste Volume	FSC		OAG		df	t	P-Value
	S1	Mean	S2	Mean			
22.5%	102.3	5427.0	250.6	5649.7	4	-1.4248	0.2607
25.0%	345.9	5650.7	501.7	5557.0	4	0.2662	0.8048
27.5%	165.8	5956.7	257.5	5796.7	4	0.9049	0.4248
30.0%	300.1	5396.3	401.0	5238.0	4	0.5476	0.6153

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

5.3.1.2 Class II - Bridge Deck concrete

Tables 5-45 and 5-46 show the results of t-test to compare the compressive strength of the FSC and OAG concrete mixes at 28 and 91 days for Class II - Bridge Deck concrete mixes. There was no statistical difference in compressive strength between FSC and OAG concrete mixes at 91 days. The results of t-test on the 28-day data were somewhat inconclusive. At 28 days, the t-test results showed that there was statistically significant difference at paste volumes of 22.5% and 27.5%, but no statistically significant difference at paste volumes of 25% and 30%.

Table 5-45 The results of t-test on the comparison of compressive strength of Class II - Bridge Deck concrete between FSC and OAG mixes at 28 days

Paste Volume	FSC		OAG		df	t	P-Value
	S1	Mean	S2	Mean			
22.5%	115.7	4557.0	266.0	5189.0	4	-3.7739	0.0383*
25.0%	331.7	6165.0	316.6	6056.7	4	0.4092	0.7034
27.5%	103.5	6401.3	172.1	5788.7	4	5.2846	0.0105*
30.0%	365.0	5678.3	667.4	5736.0	4	-0.1313	0.9036

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

Table 5-46 The results of t-test on the comparison of compressive strength of Class II - Bridge Deck concrete between FSC and OAG mixes at 91 days

Paste Volume	FSC		OAG		df	t	P-Value
	S1	Mean	S2	Mean			
22.5%	151.1	5434.0	640.8	5993.7	4	-1.4725	0.2668
25.0%	489.0	6974.7	222.2	7054.0	4	-0.2558	0.8157
27.5%	223.2	7108.0	291.4	6838.0	4	1.2741	0.2759
30.0%	757.6	6889.7	552.6	7019.7	4	-0.2401	0.8231

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

5.3.1.3 Class IV concrete

Tables 5-47 and 5-48 show the results of t-test to compare the compressive strength of the FSC and OAG concrete mixes at 28 and 91 days for Class IV concrete mixes. There was no statistically significant difference in compressive strength between FSC and OAG concrete mixes at 28 and 91 days.

Table 5-47 The results of t-test on the comparison of compressive strength of Class IV concrete between FSC and OAG mixes at 28 days

Paste Volume	FSC		OAG		df	t	P-Value
	S1	Mean	S2	Mean			
25.0%	284.1	7488.0	319.4	7451.0	4	0.1499	0.8882
27.5%	456.5	7630.3	428.5	7447.3	4	0.5062	0.6394
30.0%	397.0	7829.7	129.7	7942.7	4	-0.4686	0.6782
32.5%	282.6	6852.7	70.0	7124.3	4	-1.6164	0.2338

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

Table 5-48 The results of t-test on the comparison of compressive strength of Class IV concrete between FSC and OAG at 91 days

Paste Volume	FSC		OAG		df	t	P-Value
	S1	Mean	S2	Mean			
25.0%	327.6	8666.7	42.7	8410.7	4	1.3421	0.3079
27.5%	303.8	8281.7	687.6	8268.0	4	0.0315	0.9770
30.0%	837.4	8890.0	840.7	8267.0	4	0.9094	0.4146
32.5%	682.0	8770.3	417.2	8562.3	4	0.4506	0.6801

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

5.3.1.4 Class V concrete

Tables 5-49 and 5-50 show the results of the t-test to compare the compressive strength of the FSC and OAG concrete mixes at 28 and 91 days for Class V concrete mixes. There was no statistically significant difference in compressive strength between FSC and OAG concrete mixes at 28 and 91 days.

Table 5-49 The results of t-test on the comparison of compressive strength of Class V concrete between FSC and OAG mixes at 28 days

Paste	FSC		OAG		df	t	P-Value
Volume	S1	Mean	S2	Mean			
25.0%	85.0	7035.0	791.4	6844.7	4	0.4142	0.7181
27.5%	572.0	8504.0	258.6	8161.3	4	0.9455	0.4190
30.0%	63.9	8471.7	452.8	7948.3	4	1.9823	0.1810
32.5%	227.1	8403.0	306.9	8407.3	4	-0.0197	0.9853

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

Table 5-50 The results of t-test on the comparison of compressive strength of Class V concrete between FSC and OAG mixes at 91 days

Paste	FSC		OAG		df	t	P-Value
Volume	S1	Mean	S2	Mean			
25.0%	219.8	8277.0	438.2	8609.0	4	-1.173	0.3269
27.5%	515.4	9299.7	436.0	8290.7	4	2.5889	0.0624
30.0%	229.8	8789.7	341.7	8922.0	4	-0.5567	0.6114
32.5%	590.7	9382.0	347.7	9205.0	4	0.4473	0.6829

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

5.3.2 Analysis of Modulus of Rupture

5.3.2.1 Class II concrete

Tables 5-51 and 5-52 show the results of the t-test to compare the MOR of the FSC and OAG mixes at 28 and 91 days for Class II concrete mixes. There was no statistically significant difference in MOR between FSC and OAG concrete mixes at 28 days. At 91 days, the MOR of the OAG mixes were significantly higher than those of the FSC mixes at paste volume of 22.5 and 25%, but the difference was not significant at paste volume of 27.5 and 30%.

Table 5-51 The results of t-test on the comparison of MOR of Class II concrete between FSC and OAG at 28 days

Paste	FSC		OAG		df	t	P-Value
Volume	S1	Mean	S2	Mean			
22.5%	10.0	585.0	13.2	605.0	4	-2.0889	0.1101
25.0%	29.3	608.3	5.8	648.3	4	-2.3202	0.1370
27.5%	29.3	661.7	18.9	673.3	4	-0.5793	0.5983
30.0%	28.4	653.3	5.8	643.3	4	0.5970	0.6070

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

Table 5-52 The results of t-test on the comparison of MOR of Class II concrete between FSC and OAG at 91 days

Paste Volume	FSC S1	FSC Mean	OAG S2	OAG Mean	df	t	P-Value
22.5%	10.4	646.7	20.2	701.7	4	-4.1910	0.0249*
25.0%	21.8	690.0	13.2	760.0	4	-4.7556	0.0142*
27.5%	13.2	695.0	22.9	730.0	4	-2.2913	0.1003
30.0%	12.6	676.7	32.8	685.0	4	-0.4110	0.7128

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

5.3.2.2 Class II - Bridge Deck concrete

Tables 5-53 and 5-54 show the results of the t-test to compare the MOR of the FSC and OAG mixes at 28 and 91 days for Class II - Bridge Deck concrete mixes. There was no significant difference in MOR between FSC and OAG concrete mixes at 28 and 91 days.

Table 5-53 The results of t-test on the comparison of MOR of Class II - Bridge Deck concrete between FSC and OAG mixes at 28 days

Paste Volume	FSC S1	FSC Mean	OAG S2	OAG Mean	df	t	P-Value
22.5%	18.9	646.7	12.6	556.7	4	6.858	0.0039*
25.0%	31.8	708.3	32.1	741.7	4	-1.2778	0.2705
27.5%	27.5	688.3	62.9	703.3	4	-0.3783	0.7326
30.0%	40.4	728.3	33.3	783.3	4	-1.8193	0.1456

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

Table 5-54 The results of t-test on the comparison of MOR of Class II - Bridge Deck concrete between FSC and OAG mixes at 91 days

Paste Volume	FSC S1	FSC Mean	OAG S2	OAG Mean	df	t	P-Value
22.5%	18.9	728.3	15.0	660.0	4	4.9004	0.0092*
25.0%	20.2	816.7	41.6	831.7	4	-0.5614	0.6151
27.5%	7.6	803.3	12.6	821.7	4	-2.1573	0.1117
30.0%	7.6	831.7	25.7	816.7	4	0.9705	0.4208

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

5.3.2.3 Class IV concrete

Tables 5-55 and 5-56 show the results of the t-test to compare the MOR of the FSC and OAG mixes at 28 and 91 days for Class IV concrete mixes. There was no significant difference in MOR between FSC and OAG concrete mixes at 28 days and 91 days.

Table 5-55 The results of t-test on the comparison of MOR of Class IV concrete between FSC and OAG mixes at 28 days

Paste	FSC		OAG		df	t	P-Value
Volume	S1	Mean	S2	Mean			
25.0%	32.1	813.3	20.0	825.0	4	-0.5337	0.6270
27.5%	26.5	780.0	17.3	795.0	4	-0.8216	0.4643
30.0%	17.6	811.7	28.4	843.3	4	-1.6414	0.1901
32.5%	7.6	768.3	27.5	796.7	4	-1.7173	0.2112

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

Table 5-56 The results of t-test on the comparison of MOR of Class IV concrete between FSC and OAG mixes at 91 days

Paste	FSC		OAG		df	t	P-Value
Volume	S1	Mean	S2	Mean			
25.0%	23.1	858.3	36.2	863.3	4	-0.2018	0.8516
27.5%	75.9	821.7	13.2	845.0	4	-0.5247	0.6495
30.0%	53.9	933.3	18.9	938.3	4	-0.1515	0.8910
32.5%	12.6	821.7	11.5	811.7	4	1.0142	0.3682

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

5.3.2.4 Class V concrete

Tables 5-57 and 5-58 show the results of the t-test to compare the MOR of the FSC and OAG mixes at 28 and 91 days for Class V concrete mixes. There was no significant difference in MOR between FSC and OAG concrete mixes at 28 days and 91 days.

Table 5-57 The results of t-test on the comparison of MOR of Class V concrete between FSC and OAG mixes at 28 days

Paste	FSC		OAG		df	t	P-Value
Volume	S1	Mean	S2	Mean			
25.0%	44.8	756.7	32.1	803.3	4	-1.4656	0.2237
27.5%	55.3	946.7	12.6	966.7	4	-0.6108	0.5983
30.0%	74.9	808.3	20.0	890.0	4	-1.8249	0.1936
32.5%	22.9	920.0	24.7	903.3	4	0.8575	0.4398

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

Table 5-58 The results of t-test on the comparison of MOR of Class V concrete between FSC and OAG mixes at 91 days

Paste	FSC		OAG		df	t	P-Value
Volume	S1	Mean	S2	Mean			
25.0%	18.0	860.0	27.5	901.7	4	-2.1926	0.1043
27.5%	45.4	1006.7	27.8	1010.0	4	-0.1085	0.9199
30.0%	25.2	916.7	26.5	935.0	4	-0.8696	0.4337
32.5%	56.3	1005.0	27.8	1015.0	4	-0.2756	0.8012

*When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

5.3.3 Analysis of Splitting Tensile Strength

5.3.3.1 Class II concrete

Tables 5-59 and 5-60 show the results of the t-test to compare the splitting tensile strength of the FSC and OAG mixes for Class II concrete mixes. There was no significant difference in splitting tensile strength between FSC and OAG concrete mixes at 28 days and 91 days.

Table 5-59 The results of t-test on the comparison of splitting tensile strength of Class II concrete between FSC and OAG mixes at 28 days

Paste Volume	FSC		OAG		df	t	P-Value
	S1	Mean	S2	Mean			
22.5%	14.4	368.3	36.1	360.0	4	0.3716	0.7381
25.0%	30.4	380.0	46.5	403.3	4	-0.7278	0.5130
27.5%	5.8	423.3	20.2	406.7	4	1.3736	0.2870
30.0%	5.8	388.3	15.0	400.0	4	-1.2572	0.3105

*When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

Table 5-60 The results of t-test on the comparison of splitting tensile strength of Class II concrete between FSC and OAG mixes at 91 days

Paste Volume	FSC		OAG		df	t	P-Value
	S1	Mean	S2	Mean			
22.5%	80.5	430.0	41.6	413.3	4	0.3186	0.7709
25.0%	21.8	470.0	14.4	418.3	4	3.4234	0.3033
27.5%	26.5	460.0	7.6	413.3	4	2.9352	0.0826
30.0%	32.5	411.7	55.8	421.7	4	-0.2683	0.8047

*When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

5.3.3.2 Class II - Bridge Deck concrete

Tables 5-61 and 5-62 show the results of the t-test to compare the splitting tensile strength of the FSC and OAG mixes for Class II - Bridge Deck concrete mixes. There was no significant difference in splitting tensile strength between FSC and OAG concrete mixes at 28 days and 91 days.

Table 5-61 The results of t-test on the comparison of splitting tensile strength of Class II - Bridge Deck concrete between FSC and OAG mixes at 28 days

Paste	FSC		OAG		df	t	P-Value
Volume	S1	Mean	S2	Mean			
22.5%	13.2	375.0	27.8	390.0	4	-0.8429	0.4639
25.0%	18.9	411.7	23.1	391.7	4	1.1601	0.3128
27.5%	30.1	416.7	39.7	375.0	4	1.4482	0.2260
30.0%	27.5	428.3	80.5	475.0	4	-0.9504	0.4256

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

Table 5-62 The results of t-test on the comparison of splitting tensile strength of Class II - Bridge Deck concrete between FSC and OAG mixes at 91 days

Paste	FSC		OAG		df	t	P-Value
Volume	S1	Mean	S2	Mean			
22.5%	25.7	491.7	45.8	505.0	4	-0.4397	0.6886
25.0%	15.3	511.7	15.3	516.7	4	-0.4009	0.7090
27.5%	25.0	535.0	45.4	543.3	4	-0.2786	0.7980
30.0%	43.6	550.0	56.2	518.3	4	0.7712	0.4861

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

5.3.3.3 Class IV concrete

Tables 5-63 and 5-64 show the results of the t-test to compare the splitting tensile strength of the FSC and OAG mixes for Class IV concrete mixes. There was no significant difference in splitting tensile strength between FSC and OAG concrete mixes at 28 days and 91 days.

Table 5-63 The results of t-test on the comparison of splitting tensile strength of Class IV concrete between FSC and OAG mixes at 28 days

Paste	FSC		OAG		df	t	P-Value
Volume	S1	Mean	S2	Mean			
25.0%	25.7	438.3	46.2	461.7	4	-0.7649	0.4979
27.5%	75.2	431.7	27.5	446.7	4	-0.3243	0.7706
30.0%	48.6	511.7	45.4	441.7	4	1.8244	0.1425
32.5%	43.1	471.7	52.9	480.0	4	-0.2115	0.8433

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

Table 5-64 The results of t-test on the comparison of splitting tensile strength of Class IV concrete between FSC and OAG mixes at 91 days

Paste	FSC		OAG		df	t	P-Value
Volume	S1	Mean	S2	Mean			
25.0%	58.6	503.3	10.4	493.3	4	0.2910	0.7970
27.5%	22.5	543.3	20.0	520.0	4	1.3410	0.2520
30.0%	22.9	485.0	86.7	530.0	4	-0.8687	0.4668
32.5%	7.6	561.7	59.2	553.3	4	0.2417	0.8309

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

5.3.3.4 Class V concrete

Tables 5-65 and 5-66 show the results of the t-test to compare the splitting tensile strength of the FSC and OAG mixes for Class V concrete mixes. There was no significant difference in splitting tensile strength between FSC and OAG concrete mixes at 28 days and 91 days.

Table 5-65 The results of t-test on the comparison of splitting tensile strength of Class V concrete between FSC and OAG mixes at 28 days

Paste Volume	FSC		OAG		df	t	P-Value
	S1	Mean	S2	Mean			
25.0%	57.7	590.0	20.0	595.0	4	-0.1419	0.8979
27.5%	22.9	540.0	15.3	553.3	4	-0.8386	0.4553
30.0%	49.2	515.0	92.5	483.3	4	0.5234	0.6364
32.5%	35.1	493.3	62.1	448.3	4	1.0923	0.3509

***When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

Table 5-66 The results of t-test on the comparison of splitting tensile strength of Class V concrete between FSC and OAG mixes at 91 days

Paste Volume	FSC		OAG		df	t	P-Value
	S1	Mean	S2	Mean			
25.0%	38.8	538.3	32.5	546.7	4	-0.2849	0.7903
27.5%	27.5	521.7	55.3	543.3	4	-0.6075	0.5873
30.0%	63.5	561.7	59.2	548.3	4	0.2659	0.8035
32.5%	28.9	526.7	42.5	501.7	4	0.8425	0.4529

***When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

5.3.4 Analysis of Modulus of Elasticity

5.3.4.1 Class II concrete

Tables 5-67 and 5-68 show the results of the t-test to compare the MOE of the FSC and OAG mixes for Class II concrete mixes. There was no significant difference in MOE between FSC and OAG concrete mixes at 28 days and 91 days.

Table 5-67 The results of t-test on the comparison of MOE of Class II concrete between FSC and OAG mixes at 28 days

Paste Volume	FSC		OAG		df	t	P-Value
	S1	Mean	S2	Mean			
22.5%	0.2	4.8	0.3	4.5	4	1.7491	0.1651
25.0%	0.3	4.7	0.2	5.0	4	-1.2532	0.2952
27.5%	0.1	5.0	0.1	4.8	4	1.6219	0.1807
30.0%	0.2	4.2	0.2	4.3	4	-0.5071	0.6388

***When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

Table 5-68 The results of t-test on the comparison of MOE of Class II concrete between FSC and OAG mixes at 91 days

Paste Volume	FSC		OAG		df	t	P-Value
	S1	Mean	S2	Mean			
22.5%	0.1	4.9	0.1	5.0	4	-1.0426	0.3643
25.0%	0.2	5.2	0.1	5.2	4	-0.6489	0.5647
27.5%	0.1	5.4	0.3	5.2	4	1.1468	0.3447
30.0%	0.2	4.6	0.4	4.5	4	0.6238	0.5785

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

5.3.4.2 Class II - Bridge Deck concrete

Tables 5-69 and 5-70 show the results of the t-test to compare the MOE of the FSC and OAG mixes for Class II - Bridge Deck concrete mixes. There was no significant difference in MOE between FSC and OAG concrete mixes at 28 days and 91 days.

Table 5-69 The results of t-test on the comparison of MOE of Class II - Bridge Deck concrete between FSC and OAG mixes at 28 days

Paste Volume	FSC		OAG		df	t	P-Value
	S1	Mean	S2	Mean			
22.5%	0.0	4.8	0.1	4.9	4	-1.066	0.3897
25.0%	0.5	5.0	0.2	4.8	4	0.93	0.4263
27.5%	0.0	4.9	0.1	4.5	4	4.6904	0.0356*
30.0%	0.2	4.6	0.1	4.7	4	-0.5976	0.5985

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

Table 5-70 The results of t-test on the comparison of MOE of Class II - Bridge Deck concrete between FSC and OAG mixes at 91 days

Paste Volume	FSC		OAG		df	t	P-Value
	S1	Mean	S2	Mean			
22.5%	0.2	5.2	0.0	5.2	4	0.2540	0.8226
25.0%	0.2	5.5	0.2	5.4	4	0.6547	0.5527
27.5%	0.3	5.4	0.1	5.0	4	2.8712	0.0946
30.0%	0.1	5.2	0.1	5.1	4	0.9045	0.4204

*****When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

5.3.4.3 Class IV concrete

Tables 5-71 and 5-72 show the results of the t-test to compare the MOE of the FSC and OAG mixes for Class IV concrete mixes. There was no significant difference in MOE between FSC and OAG concrete mixes at 28 days and 91 days.

Table 5-71 The results of t-test on the comparison of MOE of Class IV concrete between FSC and OAG mixes at 28 days

Paste	FSC		OAG		df	t	P-Value
Volume	S1	Mean	S2	Mean			
25.0%	0.1	5.4	0.2	5.3	4	0.5835	0.5920
27.5%	0.2	5.3	0.1	5.2	4	0.6030	0.5929
30.0%	0.1	5.5	0.1	5.2	4	4.2212	0.0157*
32.5%	0.2	5.2	0.2	5.0	4	1.7500	0.1561

*When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

Table 5-72 The results of t-test on the comparison of MOE of Class IV concrete between FSC and OAG mixes at 91 days

Paste	FSC		OAG		df	t	P-Value
Volume	S1	Mean	S2	Mean			
25.0%	0.2	5.8	0.2	5.8	4	-0.2132	0.8425
27.5%	0.6	5.6	0.2	5.7	4	-0.0432	0.9690
30.0%	0.0	5.8	0.1	5.7	4	5.0000	0.0132
32.5%	0.2	5.7	0.1	5.4	4	2.2156	0.1265

*When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

5.3.4.4Class V concrete

Tables 5-73 and 5-74 show the results of the t-test to compare the MOE of the FSC and OAG mixes for Class V concrete mixes. There was no significant difference in MOE between FSC and OAG concrete mixes at 28 days and 91 days.

Table 5-73 The results of t-test on the comparison of MOE of Class V concrete between FSC and OAG mixes at 28 days

Paste	FSC		OAG		df	t	P-Value
Volume	S1	Mean	S2	Mean			
25.0%	0.1	5.5	0.1	5.4	4	0.8000	0.4873
27.5%	0.2	5.5	0.1	5.7	4	-1.5254	0.2534
30.0%	0.1	5.5	0.2	5.3	4	2.1213	0.1170
32.5%	0.0	5.6	0.2	5.3	4	3.1623	0.0817

*When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

Table 5-74 The results of t-test on the comparison of MOE of Class V concrete between FSC and OAG mixes at 91 days

Paste	FSC		OAG		df	t	P-Value
Volume	S1	Mean	S2	Mean			
25.0%	0.3	6.0	0.1	5.8	4	1.4018	0.2784
27.5%	0.1	6.0	0.2	5.9	4	1.0321	0.3620
30.0%	0.1	5.9	0.3	5.7	4	1.1000	0.3803
32.5%	0.2	5.9	0.1	5.8	4	1.2127	0.3038

*When the P-value is less than 0.05, the difference is considered significant at a probability of error of 5%.

5.3.5 The Results of Analysis of Variance

To assess the influence of CPV, w/cm and OAG technique on the properties of concrete, a statistical method, ANOVA (Analysis of Variance) was used. The basic ANOVA model used is shown below:

Test Property = Average Property + CPV + w/cm + OAG + (CPV)(w/cm) + (CPV)(OAG) + (w/cm)(OAG) + (CPV)(w/cm)(OAG) + error

Where,

Test property = concrete properties such as slump, compressive strength, SR, etc.

CVP = effects of CVP

w/cm = effects of w/cm

OAG = effects of OAG

All the interaction terms are assumed to be negligible. The model becomes the following:

Test Property = Average Property + CPV + w/cm + OAG + error

It is also assumed that the data meet the requirement of normality and homogeneity of variances. ANOVA determines whether or not a certain factor may have statistically significant effect on the test property evaluated. A Type I error of 5% was used in determining whether or not a certain factor was significant. Currently, most of the requirements of the concrete strength are based on 28-day compressive strength. The 28-day data were analyzed in this research.

Table 5-75 shows the results of ANOVA on the 28-day data The results of ANOVA indicate that CPV and w/cm have significant effect on the all the tests (compressive strength, MOR, Splitting tensile strength, and MOE) while OAG does not have a significant effect on the hardened concrete properties. Therefore, CPV and w/cm are two critical factors of PLC concrete design.

Table 5-75 The results of ANOVA on 28-day strength and modulus of elasticity data

Properties of Concrete	Coefficients of CPV	Coefficients of w/cm	P-value of CPV	P-value of w/cm	P-value of aggregates gradation
Compressive strength	59.10	-17,839.23	0.0001^{s}	0.0001^{s}	0.6483^{ns}
MOR	8.85	-1,232.90	0.0001^{s}	0.0001^{s}	0.0914^{ns}
Splitting tensile strength	0.33	-762.15	0.0002^{s}	0.0001^{s}	0.6622^{ns}
MOE	-0.02	-5.24	0.0077^{s}	0.0001^{s}	0.0328^{ns}

Note: S: significant difference, NS: no significant difference

5.4 The Influence of Cementitious Paste Volume in Concrete System

In a concrete system, cementitious paste and aggregate are mixed in appropriate proportions. The proportions of these materials can determine the physical properties of the fresh and hardened concrete. According to this research, cementitious paste volume is a critical parameter to affect the properties of the PLC concrete. CPV is the volume of cementitious materials and water in the concrete system. Figure 5-94 demonstrates the component diagram of concrete system. Since a higher CPV of the concrete could result in a reduction of durability, using an optimized CPV in a concrete system is recommended.

Optimum CPV is required so that a durable concrete may be achieved. Excessive CPV may cause high shrinkage, and permeability of the concrete. Increasing the density of the concrete by optimizing aggregate gradation of the concrete can result in lower CPV with lower heat of hydration, drying shrinkage and permeability.

When a sample of concrete is prepared, the concrete is analyzed to determine the probable performance in a structure. The analysis focuses on four characteristics of the concrete and the influence of the CPV on those characteristics. The four characteristics are workability, strength, drying shrinkage, and permeability.

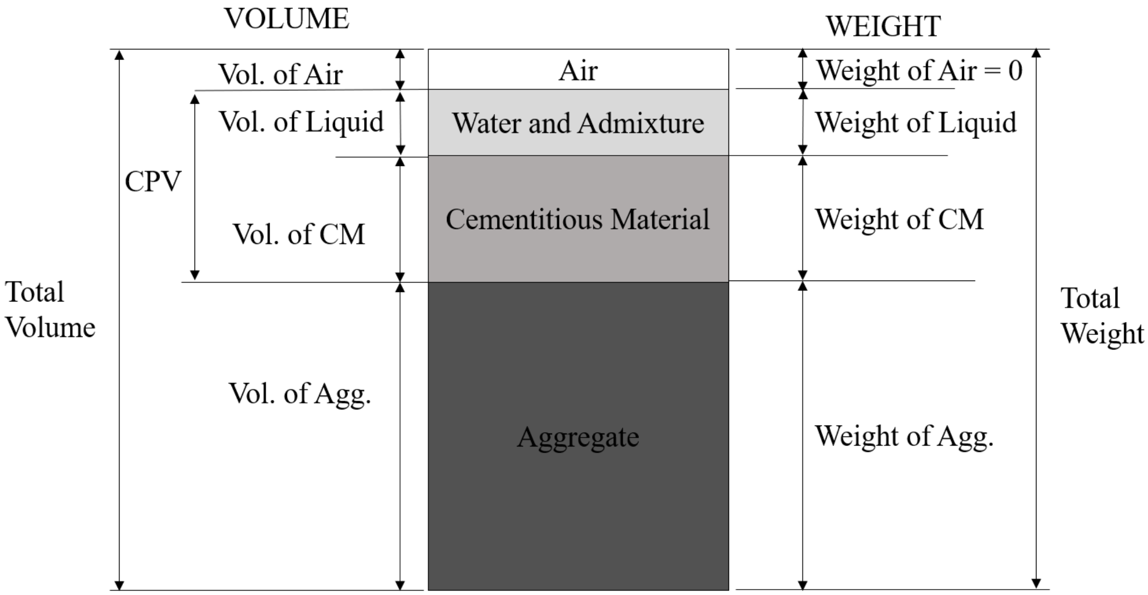

Figure 5-94 Component diagram of concrete system

5.4.1 The Effects of Workability

Workability describes the ease with which concrete may be placed, compacted, and finished. The workability of the concrete is affected by the water content, particle size distribution and quantity of cementitious materials, and sand, air content, and aggregate proportion. To enable flow of the concrete, there must be enough cementitious paste to fill the inter-particle voids and coat the particle surfaces. The paste acts as an intra-particle lubricant to minimize resistance to flow due to friction and interlocking between aggregate particles. Also, increasing the CPV of the concrete reduces the volume of the aggregate since the extra CPV resides in the space that would otherwise be occupied by the aggregate. It would also reduce the chance for aggregate interlock. Thus, a concrete with high CPV usually provides better workability. In order to increase the CPV at a fixed w/cm, the cementitious materials content needs to increase. The air content of the concrete would also improve the workability of the concrete. The internal air bubbles can help to relieve the pressures, provided the bubbles are small and closely spaced through the paste. These bubbles can increase the workability of the concrete.

The results of the experimental program show that the CPV affects the workability of the concrete. In each concrete mix, the concrete with higher CPV showed higher slump results. Figure 5-95 shows the workability versus the different CPV in each concrete mix. The results show that the concrete with high CPV provides better workability of the concrete.

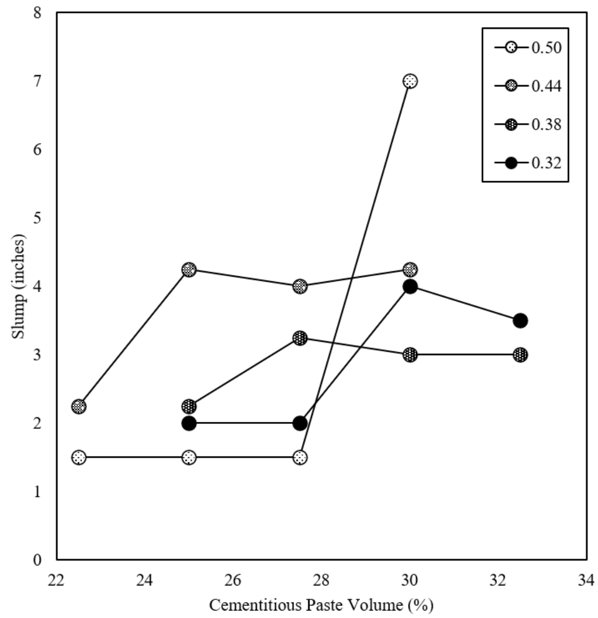

Figure 5-95 Slump of fresh concrete versus cementitious paste volume

5.4.2 The Effects of Strength

The strength of concrete is its ability to sustain applied stress without failure. Abram's principle indicates that the w/cm of the concrete is the critical parameter to determine the strength of the concrete. Increasing the CPV does not improve the strength of the concrete. Moreover, previous research observed that an increase in the CPV could reduce the compressive strength of concrete and increase its water absorption (Tracz and Sliwinski, 2012). The CPV contains volume of the water, and cementitious materials. Water within the paste is consumed by hydration leaving behind air voids that reduce the strength of concrete. There is about a 5% reduction in strength for each 1% increase in the volume of air voids (ACI 212, 2016). An increase in paste volume at a fixed w/cm increases the cementitious material content that can increase strength. However, the respective increase in water content results in an increase in porosity as the cementitious material hydrates, which tends to reduce strength. Initially, the strength increases due to increasing cementitious material content; however, the detrimental effect of increasing porosity eventually becomes dominant. These opposing effects result, as the cementitious paste content is increased, in an increase in strength of the concrete followed by a leveling off and eventual decrease in strength (Figure 5-96). This could lead to the concrete failing to reach the required strength.

The concrete CPV has to be above a minimum to provide the aggregate with insufficient coating to provide the needed workability. Insufficient CPV could result in the formation of weak interfacial transition zones and subsequently lower strengths. In general, a sufficient CPV of the concrete is needed for adequate workability, with the strength which is determined by the w/cm. Unexpectedly low strengths can occur when the concrete has insufficient CPV.

Figure 5-96 shows the 28-day compressive strength of the concrete versus CPV for concrete with different w/cm. The compressive strength of the was adversely affected only when the CPV of the concrete was insufficient. The individual aggregate particles cannot be coated adequately when the CPV of the concrete is insufficient. The concrete with insufficient CPV would not be able to provide enough strength to the concrete. The results of this research also show that the compressive strength of the concrete is affected by the w/cm.

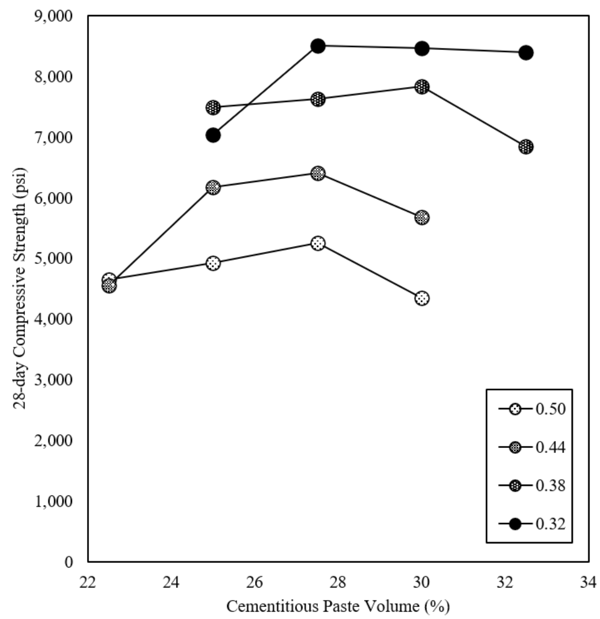

Figure 5-96 Compressive strength of the concrete at 28 days versus cementitious paste volume

5.4.3 The Effects of Shrinkage

Drying shrinkage happens when the moisture of concrete is lost. The shrinkage of the concrete is from the hydrated cement paste. The loss of moisture from concrete after it hardens is inevitable. Drying shrinkage occurs when the concrete starts losing the moisture. And the amount is proportional to the CPV. Most aggregate used for concrete has negligible drying shrinkage. The relationship between the CPV and the drying shrinkage behavior is positive.

Figure 5-97 showed the 91-day drying shrinkage of the concrete versus CPV for concrete with different w/cm. When the concrete has higher CPV, the drying shrinkage of the concrete is higher. It is because the shrinkage of the concrete is caused by the loss of moisture in cement paste.

Typically, the high CPV has relatively high water. Therefore, the drying shrinkage of the concrete would increase when the concrete has high CPV.

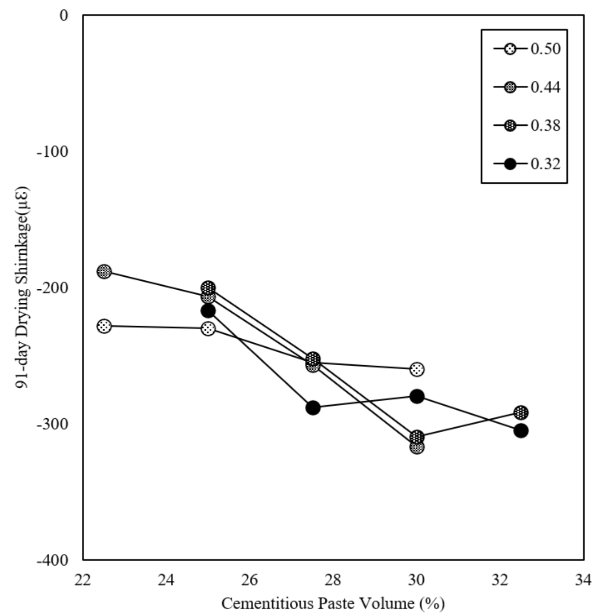

Figure 5-97 The 91-day drying shrinkage of the concrete versus cementitious paste volume

5.4.4 The Effects of Electrical Resistance

According to the definition of ACI, the durability of concrete is "the ability to resist weathering action, chemical attack, abrasion, or any other process of deterioration and retain its original form, quality, and serviceability when exposed to its environment"(ACI 201.2R-16, 2016). Permeability is the primary parameter used to evaluate durability when resistance to intrusion of deleterious substances is considered. The properties that affect the permeability of concrete include the particle size distribution, packing density of the solid components, w/cm, cementitious material content, cement fineness, and degree of hydration. Generally, the permeability of the concrete is lower at a lower w/cm. However, permeability is not just affected by w/cm. The CPV of the concrete affects the permeability of the concrete. The electrical resistance test is a popular way to evaluate the permeability of the concrete. A higher electrical resistance of the concrete indicates a lower permeability of the concrete. Since the CPV contains the volume of the water, the water content can affect the electrical resistance of the concrete. When the concrete has higher CPV, the electrical resistance of concrete decreases. Therefore, a higher CPV of the concrete can reduce the durability of the concrete.

However, insufficient CPV would also lead to high permeability of the concrete because of the additional porosity due to insufficient paste volume to fill all the voids between aggregate particles. Figure 5-98 presented the cross-section of a concrete with insufficient CPV. It can be seen that the concrete has additional voids between the aggregate and non-hydrated area. The porosity of the concrete increases the permeability of the concrete and reduces the durability of the concrete. To fill the voids and bond the aggregate cohesively in the concrete, an adequate volume of paste needs to be present. Therefore, a concrete with proper CPV could improve the durability of the concrete.

In this research, the results also show that the concrete with higher CPV reduces electrical resistance of the concrete, which indicated higher permeability of the concrete. Figure 5-99 shows the 91-day surface resistivity of the concrete versus CPV for concrete mixes for different w/cm. It can be seen that the concrete with high CPV has lower electrical resistance of the concrete. Besides, the results also show that the concrete with lower w/cm has higher electrical resistance. The water content of the concrete is the key factor to influence the electrical resistance. Mixes with higher CPV contained higher water contents and mixes with higher w/cm also contained higher water contents. Therefore, the electrical resistance of the concrete is influenced by the w/cm and CPV.

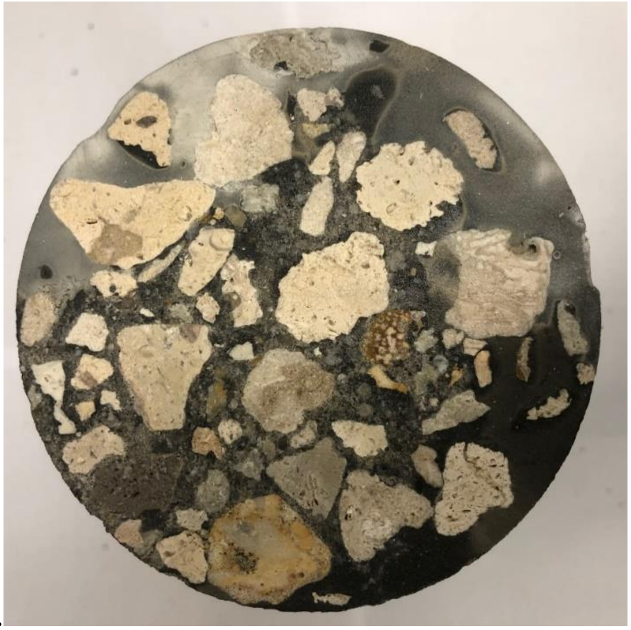
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Figure 5-98 The cross-section of the concrete with insufficient CPV

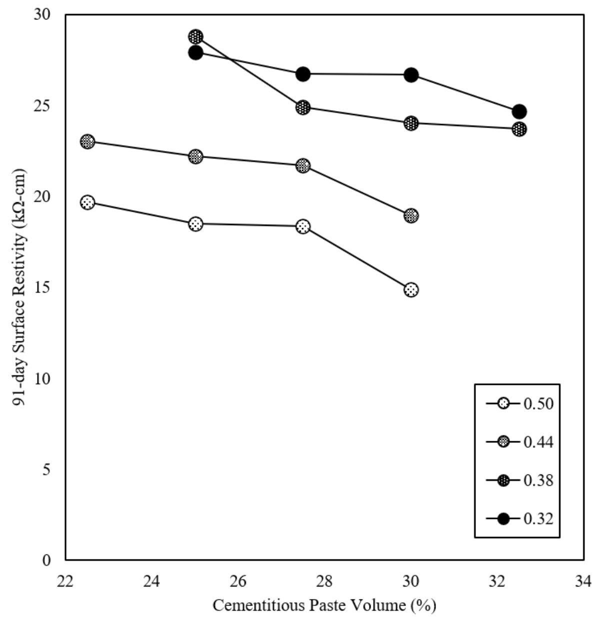


Figure 5-99 The 91-day surface resistivity of the concrete versus cementitious paste volume

5.5 Summary of Findings

The following are the main findings concerning the use of portland limestone cement (PLC) in Florida concrete mixes (Class II, Class II - Bridge Deck, Class IV, and Class V):

1. Concrete using PLC can provide similar properties as the concrete using ordinary portland cement.
2. PLC concrete mixes met the current required compressive strength based on the FDOT's standard specifications for road and bridge construction.
3. Based on the average strength results (compressive strength, splitting tensile strength, and modulus of rupture), the strength of the concrete is affected by the w/cm. Increasing the paste volume of concrete above the optimum amount cannot increase the strength of the concrete.
4. PLC concrete mixes could be used in all classes of concrete in Florida.

The following are the main findings concerning the effects of cementitious paste volume (CPV) on Florida concrete mixes:

1. Concrete with higher CPV could easily meet the slump without additional water-reducing admixture.
2. The strength of all classes of concrete did not increase when the CPV of the concrete was increased above the optimum content.
3. The strength of the concrete increased as the w/cm decreased regardless of the CPV as long as the CPV was above a certain threshold value.
4. Concrete mixes with insufficient or excessive CPV did not provide sufficient strength.
5. PLC concrete with a lower CPV had lower drying shrinkage
6. PLC concrete with a lower CPV had lower permeability as shown from the RCPT and SR test results.
7. The minimum CPV of Class II concrete was 22.5%, the minimum CPV of Class II - Bridge Deck and IV concrete was 25.0%, and the minimum CPV of Class V concrete was 27.5%.

The following are the main findings concerning the effects of optimized aggregate gradation (OAG) on Florida concrete:

1. PLC concrete using OAG had improved workability of the fresh concrete.
2. PLC concrete using OAG had similar compressive strength and mechanical properties (compressive strength, MOR, splitting tensile strength, and Poisson's ratio) as those of the concrete without using OAG.
3. PLC concrete using OAG had slightly lower MOE than those of the concrete without using OAG.
4. PLC concrete using OAG had lower drying shrinkage than that of the concrete without using OAG.
5. PLC concrete using OAG had similar permeability level as conventional concrete.

CHAPTER 6
SUSTAINABILITY EVALUATION FOR PORTLAND LIMESTONE CEMENT CONCRETE

6.1 Material Cost Analysis

An evaluation was made to determine the potential decrease in material cost due to the reduction of cementitious paste content in concrete. The calculation of the costs of cementitious materials and admixtures was based on typical U.S. market prices. The prices used were $120 per ton for portland limestone cement, $50 per ton for fly ash, $4 per gallon for air-entraining admixture, $5 per gallon for water-reducing admixture, and $15 per gallon for high range-water-reducing admixture. The aggregate costs were based on prices in the Florida market. The prices used were $20 per ton for concrete sand, and $18 per ton for coarse and intermediate aggregates. Using the different concrete mix designs (Class II, II-Bridge Deck, IV and V) in this research, the prices of the average materials were calculated and are shown in Tables 6-1 to 6-4.

Since the FSC and OAG mixes used a similar amount of admixture in the concrete mix, the materials cost between FSC and OAG mixes are very close. However, the properties of the OAG mixes are better than the FSC mixes. On the other hand, the cost results show that the paste volume plays a key factor in influencing the cost. In Class II concrete, the difference of the cost between the highest (30.0%) and lowest (22.5%) is around $4 per yd^3. In Class II - Bridge Deck concrete, the difference of the cost between the highest (30.0%) and lowest (22.5%) is around $5 per yd^3. In Class IV concrete, the difference of the cost between the highest (32.5%) and lowest (25.0%) is around $6 per yd^3. In Class V, the difference of the cost between the highest (32.5%) and lowest (25.0%) is around 7 dollars per yd^3. Based on the research results, the properties between different paste volumes are very similar. For the construction of a 48 ft. × 100 ft. long bridge deck (9 inches. thick, not including girders) utilizing a Class IV concrete with 27.5% paste volume, the concrete could result in a reduction of roughly $8,300 in materials cost. If the concrete uses the OAG technique, the potential cost of admixture could be further reduced due to the better workability of fresh concrete. Moreover, the shrinkage of the structural concrete can be mitigated by OAG technique. The durability of the concrete can be improved by OAG technique. Therefore, it is important to choose the proper paste volume in the concrete mix to save cost.

Table 6-1 Concrete materials price evaluation for Class II concrete

USD/yd^3	Paste Volume (Relative Cementitious Materials Content)			
	30.0%	27.5%	25.0%	22.5%
FSC	58.5	57.5	55.9	54.3
OAG	58.6	57.2	55.6	54.0

Table 6-2 Concrete materials price evaluation for Class II Bridge-Deck concrete

USD/yd^3	Paste Volume (Relative Cementitious Materials Content)			
	30.0%	27.5%	25.0%	22.5%
FSC	61.0	59.9	57.8	55.9
OAG	60.6	59.5	57.6	55.7

Table 6-3 Concrete materials price evaluation for Class IV concrete

USD/yd^3	Paste Volume (Relative Cementitious Materials Content)			
	32.5%	30.0%	27.5%	25.0%
FSC	66.4	64.4	62.3	60.1
OAG	66.0	64.0	62.0	59.8

Table 6-4 Concrete materials price evaluation for Class V concrete

USD/yd^3	Paste Volume (Relative Cementitious Materials Content)			
	32.5%	30.0%	27.5%	25.0%
FSC	70.1	67.6	65.6	63.4
OAG	69.7	67.3	65.2	62.7

6.2 Carbon Dioxide Emission Evaluation

An evaluation of potential reduction in carbon dioxide emissions due to reduction of cementitious paste content in concrete was made. The data for global warning potential (GWP) are taken from various sources, including the published literature and life cycle inventory (LCI) database (Santero, et al., 2013). Table 6-5 summarizes the key values used in this research. Using the different concrete mix designs (Class II, II-Bridge Deck, IV and V) in this research, the estimated carbon dioxide emissions were calculated and are shown in Tables 6-6 to 6-9.

Since the FSC and OAG mixes used a similar amount of admixture in the concrete mix, the carbon dioxide emission between the FSC and OAG mixes are very close. However, the properties of the OAG mixes are better than the FSC mixes. The estimated carbon dioxide emission results show that the paste volume plays a key factor in influencing carbon dioxide emission. In Class II concrete, the difference of the cost between the highest (30.0%) and lowest (22.5%) is around 100 lb CO_2e/yd^3. In Class II - Bridge Deck concrete, the difference of the cost between the highest (30.0%) and lowest (22.5%) is around 105 lb CO_2e/yd^3. In Class IV concrete, the difference of the cost between the highest (32.5%) and lowest (25.0%) is around 110 lb CO_2e/yd^3. In Class V concrete, the balance of the cost between the highest (32.5%) and lowest (25.0%) is around 130 lb CO_2e/yd^3. Based on the research results, the properties between different paste volumes are very similar. For the construction of a 48 ft. × 100 ft. long bridge deck (9 inches. thick, not including girders) utilizing a Class IV concrete with 27.5% paste volume, a concrete mix with could result in a total reduction of roughly 3,900 lb CO_2e/yd^3 in carbon dioxide emission. If the concrete uses the OAG technique, the shrinkage of the structural concrete can be mitigated by OAG technique. The durability of the concrete can be improved by OAG technique. Therefore, it is important to choose the proper paste volume in the concrete mix to improve the sustainable development.

Table 6-5 Inventory data for GWP of concrete materials

Material	GWP emissions factor (lb CO_2e/yd^3)[1]	Source
Type IL Cement[2]	0.835	Hasegawa (2011)
Fly ash	0.01	PE International (2011)
Coarse Aggregates	0.0032	Zapata and Gambatese (2005)
Fine Aggregates	0.0007	Ma et al. (2016)
Water	0.005	PE International (2011)
Superplasticizer	0.00069	Ma et al. (2016)

Note:[1]U.S. units for masses are proportional, i.e. 1 kg CO_2e/kg material equals 1 pound CO_2e/pound materials.
[2]Type IL cement by replacing up to15% of portland cement clinker with limestone powder.

Table 6-6 Concrete materials price evaluation for Class II concrete

lb CO_2e/yd^3	Paste Volume (Relative Cementitious Materials Content)			
	30.0%	27.5%	25.0%	22.5%
FSC	409.8	376.2	342.6	309.3
OAG	410.1	376.6	343.1	309.7

Table 6-7 Concrete materials price evaluation for Class II Bridge-Deck concrete

lb CO_2e/yd^3	Paste Volume (Relative Cementitious Materials Content)			
	30.0%	27.5%	25.0%	22.5%
FSC	439.8	406.5	369.7	333.0
OAG	440.4	406.9	370.1	333.2

Table 6-8 Concrete materials price evaluation for Class IV concrete

lb CO_2e/yd^3	Paste Volume (Relative Cementitious Materials Content)			
	32.5%	30.0%	27.5%	25.0%
FSC	517.1	477.0	440.2	396.8
OAG	517.6	477.5	440.6	397.1

Table 6-9 Concrete materials price evaluation for Class V concrete

lb CO_2e/yd^3	Paste Volume (Relative Cementitious Materials Content)			
	32.5%	30.0%	27.5%	25.0%
FSC	564.3	520.8	477.2	433.3
OAG	564.8	521.3	477.7	434.1

CHAPTER 7
RECOMMEDED MIX DESIGN METHOD

7.1 Recommended Mix Design Method for Concrete with Minimum Paste Volume

Based on the results of this study, a mix design method to achieve concrete with minimum cement paste volume was recommended. This mix method is applicable for concrete mixes with maximum size of aggregates of 1 inch. It is described in this section.

7.1.1 Application for the Recommended Mix Design method

The recommended mix design method is intended to achieve concretes with minimum paste volume that can be used in normal Florida concrete Classes I through V concretes. The designed concrete will be made with normal weight aggregate and will have an air content range from 0 to 6% and a slump range from 2 to 4 inches. This method may not suitable for mass concrete application.

7.1.2 Recommended Mix Design Procedure

The recommended method includes six main steps: (1) Select the w/cm; (2) select the cementitious paste volume; (3) select the dosage of superplasticizer; (4) calculate the water and cementitious materials content; (5) determine combined aggregate proportion; (6) evaluate the trial mixture and make necessary adjustments.

The proportions of the aggregates are based on its saturated surface dry (SSD) condition. The user should make corrections for aggregate moisture content when conducting trial or production batches.

The six main steps are described below:

1. Select the w/cm

    Select the maximum w/cm to achieve the desired compressive strength at 28 days. Table 7-1, which was developed from the results from this study, can be used to select the w/cm to be used. Much more research work still needs to be conducted to verify and refine this table.

Table 7-1 Recommend w/cm for Florida Classes I to V concretes

Classes of concrete in Florida	w/cm	Estimated Compressive Strength at 28 days (psi)1	Specified Minimum Strength (psi)	Required Minimum Strength (psi)2
Class I	Above 0.50	Lower than 4,200	3,000	4,200
Class II	0.44 to 0.50	5,500 to 4,200	3,400	4,600
Class III	0.38 to 0.44	6,750 to 5,500	5,000	6,200
Class IV	0.32 to 0.38	7,850 to 6,750	5,500	6,750
Class V	Below 0.32	Higher than 7,850	6,500	7,850

Note:

[1] Estimated compressive strength is based on this research.

[2] Required minimum strength is according to FDOT Materials Manual Chapter 9 in 2020.

2. Select the cementitious paste volume

Select the CPV to be used from Table 7-2, which was developed from the results from this research. Similarly, much more research work still needs to be conducted to verify and refine this table.

Table 7-2 Cementitious paste volume for Florida Classes I to V concretes

Classes of concrete in Florida	w/cm	Recommend paste volume range (%)
Class I	Above 0.50	22.0 to 24.0
Class II	0.44 to 0.50	24.0 to 26.0
Class III	0.38 to 0.44	26.0 to 28.0
Class IV	0.32 to 0.38	28.0 to 30.0
Class V	Below 0.32	30.0 to 32.0

3. Select the dosage of superplasticizer

Figure 7-1 presents the required dosage of superplasticizer as functions of w/cm and CPV. Figure 7-1 was created from the results from this research study; it is for producing concrete with slump range between 2 to 4 inches, and with air content below 6%. In order to improve the accuracy of the correlation, more tests are needed. However, Figure 7-1 will not be suitable for use with other admixtures.

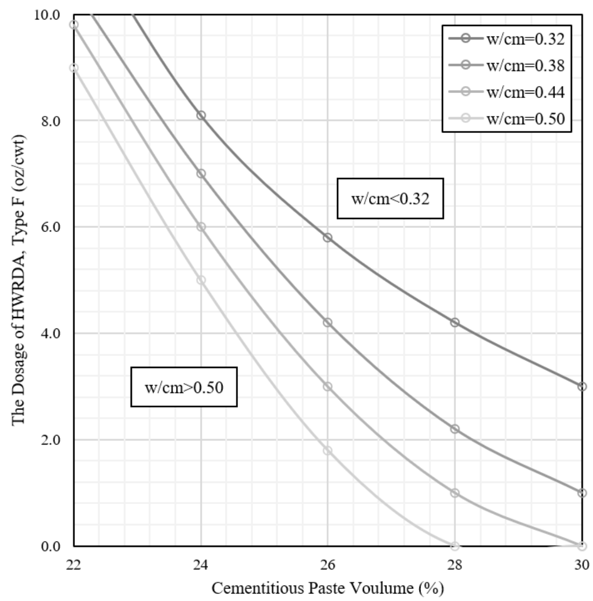


Figure 7-1 The required dosage of superplasticizer as functions of w/cm and paste volume

4. Calculate the water and cementitious materials contents

The water and cementitious materials contents of concrete can be calculated from the selected w/cm, CPV and the dosage of superplasticizer. CPV is related to the volumes of cementitious materials, water, and admixture by equation 7-1.

$$CPV(\%) = \frac{(CM_{vol.}+W_{vol.})+A_{vol.}}{27} \tag{7-1}$$

Where,

CPV=paste volume, %,

$CM_{vol.}$=volume of cementitious materials, ft^3/yd^3,

$W_{vol.}$= volume of water, ft^3/yd^3,

$A_{vol.}$= volume of admixture, ft^3/yd^3.

The CPV was determined from Table 7-2. The volume of admixture, $A_{vol.}$ was determined from Figure 7-1. Therefore, the sum of $CM_{vol.}$ and $W_{vol.}$ can be calculated from equation 7-1. In addition, the selected w/cm from Table 7-1 provides a constant ratio of cementitious materials to water. Thus, the $CM_{vol.}$ and $W_{vol.}$ can be calculated through equation 7-1 and w/cm. By knowing the $CM_{vol.}$, $W_{vol.}$, and $A_{vol.}$, the weight of materials in the cement paste can be calculated by equation 7-2.

$$Weight\ of\ materials\ = \frac{M_{vol.}}{SG_m} \tag{7-2}$$

Where,

$M_{vol.}$= Volume of material in cement paste, ft^3/yd^3,

SG_m= Specific gravity of material in cement paste,

Note that the amount of superplasticizer would be considered as a part of mixing water. The final amount of water should be equal to the sum of free water and superplasticizer.

5. Determine combined aggregate proportion

The combination of aggregates would be decided by the modified coarseness factor chart (MCFC) developed by Shilstone. The MCFC is shown in Figure 7-2. MCFC is a plot of coarseness factor (CF) versus adjusted workability factor (WF_{adj}). The designed point is the optimum point located at the center of the well-graded zone. The optimum point gives CF equal to 60 and WF_{adj} equal to 36. The calculation procedure of CF and WF_{adj} has been present in Chapter 3. In addition, the size and gradation of aggregates needs to meet the AASHTO M43. If the CF and WF_{adj} of concrete mixes are unable to reach the designed point, then an intermediate-size aggregate should be used so that the combined aggregate blend can meet the designed point.

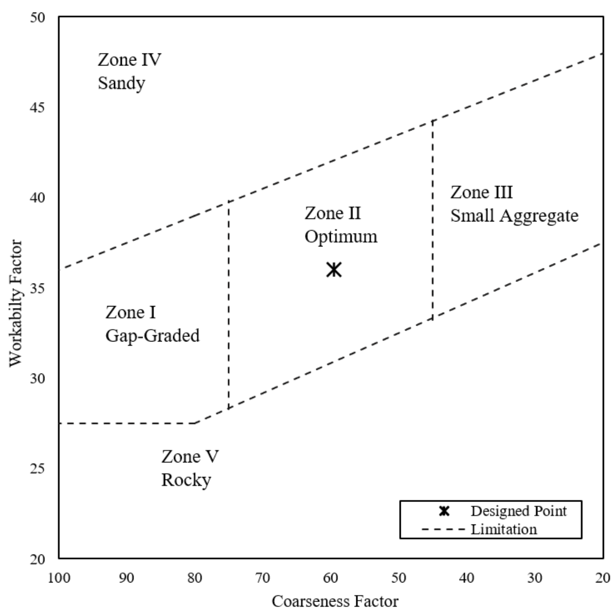


Figure 7-2 The modified coarseness factor chart for mix designs

6. Evaluate trial mixture and make necessary adjustments

Prepare trial batches for one or more mixtures developed in step 1 to 5. The actual dosage of the AEA or superplasticizer can be adjusted during trial batching or final production.

7.2 Case Study

Design a concrete mixture to meet the following requirements:

(1) Florida Class III concrete, compressive strength of 6000 psi at 28 days;

(2) Target slump of 3 inches;

(3) Coarse aggregate, maximum aggregates size is 1 inch, a crushed limestone with specific gravity (SG) of 2.45;

(4) Intermediate aggregates, maximum aggregates size is 3/8 inches, a crushed limestone with SG of 2.50;

(5) Fine aggregate, a silica sand with SG of 2.65;

(6) Type IL cement, SG of 3.15;

(7) Not exposed to freezing and thawing exposure conditions.

Step 1 Select w/cm

Based on Table 7-1, the selected maximum w/cm is 0.42.

Step 2 Select paste volume

The minimum paste volume is 26.0% according to selected w/cm (0.42).

Step 3 Select the dosage of superplasticizer for workability

The target slump is in the applicable range. According to Figure 7-1, the selected dosage of superplasticizer is 3.6 oz/cwt.

Step 4 Calculate the cementitious materials content

Based on the SG of the different cementitious materials and Equations 7-1 and 7-2, and the selected w/cm and paste volume, the calculated the cement content is 595 lb/yd^3.

Step 5 Calculate the aggregates proportions

A combination of 53.6% coarse aggregate, 14.1% of intermediate aggregate and 32.3% sand by volume is selected based on the optimized point (CF=60, WF_{adj}=36) of the MCFC. Table 7-3 is the final proportions of ingredients for this mix.

Table 7-3 Final proportions of ingredients for the designed mix

Designed Concrete w/cm=0.42 CPV=26.0%	Designed Weight of Materials per yd^3
Cement(lb/yd^3)	595.0
Water(lb/yd^3)	249.1
Air	0.0
Coarse Aggregate(lb/yd^3)	1541.8
Intermediate Aggregate(lb/yd^3)	459.6
Fine Aggregate(lb/yd^3)	1142.8
Superplasticizer(oz/yd^3)	21.3

Step 6 Trial batches

Trial batches are made. The slump was 2.5 inches, which was lower than the target of 4 inches. The air content was 1.5% Thus, the dosage of superplasticizer can be increased, and air entraining admixture can be also applied in this concrete mix. To ensure all the properties of the fresh concrete were adequate, the bleeding resistance was also measured with ASTM C232. The strength of the concrete met the 28-day compressive strength target value of 6000 psi.

CHAPTER 8
FINDINGS AND RECOMMENDATIONS

8.1 Summary of findings

The following are the main findings from this study on Florida Class II, Class II - Bridge Deck, Class IV, and Class V concretes:

1. Concrete using portland-limestone cement can provide similar properties as the concrete using ordinary portland cement.
2. The pH values of fresh concrete were not affected by reducing the CPV and w/cm when the concrete had sufficient CPV.
3. The amount of heat released during hydration, which is directly related to the temperature rise of the concrete can be decreased by reducing the CPV. Moreover, the greater volume fraction of aggregate resulting from the denser packing density of the OAG also inhibits the temperature rise due to the increased thermal mass. Thus, using the OAG technique in concrete can help mitigate the early cracking issue of concrete.
4. Based on the average strength results (compressive strength, splitting tensile strength, and modulus of rupture), the strength of the concrete is affected mainly by the w/cm. Increasing the CPV in a concrete mix design normally does not increase the strength of the concrete.
5. Average SR and RCPT results showed that the electrical resistance of the concrete was lower when the concrete had a higher CPV. This indicates that the concrete with higher CPV had a higher permeability, which reduced the durability of the concrete.
6. Based on the results of drying shrinkage results, CPV was inversely proportional to the early-age shrinkage of the concrete.
7. The OAG technique did nor significantly increase the strength or lower the permeability of the concrete at an early age, but the length change of the hardened concrete was reduced. Moreover, when the OAG technique was applied to the concrete, the workability was improved and the temperature of hydration of the fresh concrete could be reduced by decreasing the paste volume.

8.2 Recommendations

1. Require the minimum CPV of Class II concrete to be is 22.5%, and the recommended CPV to be 22.5 to 25.0%.
2. Require the minimum CPV of the Class II (Bridge Deck), and Class IV concrete need to be above 25.0%, and the recommended CPV to be 25.0 to 27.5%.
3. Require the minimum CPV of the Class V concrete needs to be above 27.5%, and the recommended CPV to be between 27.5 to 30.0%.
4. Use the results of this research to support and improve FDOT Standard Specifications for Road and Bridge Construction-Section 346.
5. Currently, Sections 901, Coarse Aggregate and 902, Fine Aggregate of FDOT Standard Specifications for Road and Bridge Construction limit the type of aggregates to be used in concrete. However, the application of the OAG technique would require the use of an intermediate-size aggregate in the mixes. The use of an intermediate-size aggregate and OAG technique in the design of concrete should be incorporated in the FDOT Standard Specifications for Road and Bridge Construction in the future.

REFERENCES

AASHTO M240M/M240, *Standard Specification for Blended Hydraulic Cement*, American Association of State Highway and Transportation Officials, Washington D.C., 2015.

AASHTO T277, *Standard Method of Test for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration*, American Association of State Highway and Transportation Officials, Washington D.C., 2015.

AASHTO TP95-11, *Standard Method of Test for Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration*, American Association of State Highway and Transportation Officials, Washington D.C., 2011.

Abrams, D.A.,Water-cement ratio as a basis of concrete quality, ACI Journal, Proceedings, vol. 23, p.452-457, Michigan, 1927.

ACI 201.2R-16, Guide to durable concrete, American Concrete Institute, Michigan, 2016.

ACI 209R-92, Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures, American Concrete Institute, Farmington Hills, Michigan, 2008.

ACI 212.3R-16, Report on chemical admixture for concrete, American Concrete Institute, Michigan, 2016.

ACI 211.4R-93. "Guide for Selecting Properties for High-Strength Concrete with Portland Cement and Fly Ash," ACI Manual of Concrete Practice, Part 1. American Concrete Institute, Detroit, Michigan, 1996.

ACI 211.6T, Aggregate suspension mixture proportioning method, TechNote of American Concrete Institute, Michigan, 2014.

ACI 302.1R-96, *Guide for Concrete Floor and Slab Construction*. American Concrete Institute, Michigan, 1997.

Alexander, M.G., "Aggregate and the deformation properties of concrete," *ACI Materials Journals*, Vol. 93(6), p. 1-9, 1996.

Allahham, J., Bordelon, A., Li, L., and Rayaprolu, S., Review and specification for shrinkage cracks of bridge decks, Utah Department of Transportation, Research Division, Utah, 2016.

Ambroise, J., Maximilien, S. and Pera, J., Properties of metakaolin blended cements, *Advanced Cement Based Materials*, Vol. 1, p.161-168, 1994.

American Coal Ash Association. Fly Ash Facts for Highway Engineers. Report of Federal Highway Administration, No. FHWA-SA-94-081, Washington, D.C., 1995.

Anson-Cartwright, M., Optimization of aggregate gradation combinations to improve concrete sustainability, Master of Science Thesis, Department of Civil Engineering, University of Toronto, Toronto, Ontario, Canada, 2011.

Aqel, M. and Panesar, D.K., Hydration kinetics and compressive strength of steam-cured cement pastes and mortars containing limestone filler, *Construction and Building Materials*, vol.113, p.359-368, 2016.

Arachchige, M., Influence of cement content on corrosion resistance. *Construction Materials*, Vol. 161, p. 31–39, 2008.

ARTBA, *Production and Use of Coal Combustion Products in the U.S.*, American Road and Transportation Builders Association, Washington, D.C., 2015.

ASTM C138/C138M, *Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete*, American Society for Testing and Materials, West Conshohocken, West Conshohocken, Philadelphia, 2017.

ASTM C143/C143M, *Standard Test Method for Slump of Hydraulic Cement Concrete*, American Society for Testing and Materials, West Conshohocken, Philadelphia, 2015.

ASTM C150, Standard Specification for Portland Cement, American Society for Testing and Materials, Pennsylvania, 2018.

ASTM C157/C157M, *Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete*, American Society for Testing and Materials, West Conshohocken, Philadelphia, 2017.

ASTM C1064/C1064M, *Standard Test Method for Temperature of Freshly Mixed Hydraulic-Cement Concrete*, American Society for Testing and Materials, West Conshohocken, Philadelphia, 2017.

ASTM C1202, *Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration*, American Association of State Highway and Transportation Officials, Washington D.C., 2018

ASTM C231/C231M, *Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method*, American Society for Testing and Materials, West Conshohocken, Philadelphia, 2017.

ASTM C232/C232M, *Standard Test Method for Bleeding of Concrete*, American Society for Testing and Materials, West Conshohocken, Philadelphia, 2014.

ASTM C260/C260M, *Standard Specification for Air-Entraining Admixtures for Concrete*, American Society for Testing and Materials, West Conshohocken, Philadelphia, 2016.

ASTM C39/C39M, *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*, American Society for Testing and Materials, West Conshohocken, Philadelphia, 2018.

ASTM C469/C469M, *Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression*, American Society for Testing and Materials, West Conshohocken, Philadelphia, 2014.

ASTM C494/C494M, *Standard Specification for Chemical Admixtures for Concrete*, American Society for Testing and Materials, West Conshohocken, Philadelphia, 2017.

ASTM C595, Standard Specification for Blended Hydraulic Cements, American Society for Testing and Materials, Pennsylvania, 2018.

ASTM C618, Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete, American Society for Testing and Materials, Pennsylvania, 2019.

ASTM C78/C78M, *Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)*, American Society for Testing and Materials, West Conshohocken, Philadelphia, 2018.

Bai, J., Wild, S., Ware,J.A. and Sabir, B.B., Using neural networks to predict workability of concrete incorporating metakaolin and fly ash. *Advances in Engineering Software*, vol. 34, p. 663-669, 2003.

Bhanja, S. and Sengupta, B., Modified water-cement ratio law for silica fume concretes, Cement and Concrete Research, vol.33, p.447-450, 2003. https://doi.org/10.1016/s0008-8846(02)00977-8.

Blanks, R. F., Vidal, E. N., Price, W. H., and Russell, F. M., The properties of concrete mixes, ACI Journal, *Proceedings*, Vol. 36, p.44, 1940.

Chu, S.H., Effect of paste volume on fresh and hardened properties of concrete, *Construction and Building Materials*, vol. 218, p. 284-294, 2019.

Chung, H.W., Subgranon, T., Wang, Y., Deford, H., and Tia, M, Evaluation of pavement concrete with low paste volume using Portland limestone cement, *ACI Material Journal*, vol. 117(2), p. 181-192, 2020a.

Chung, H.W., Subgranon, T., Tia, M., DeFord, H., and Armenteros, J., The effects of reduced paste volume in portland limestone cement concrete, *Magazine of Concrete Research*, 2020b.

Cook, M.D., Seader, J.N., Ley M.T., and Russel, B.W., Investigation of optimized graded concrete for Oklahoma – phase 2, the research report of Oklahoma Department of Transpiration, Oklahoma, 2015.

Cook, M.D., Ghaeezadah, A., and Ley, M.T., Impacts of coarse-aggregate gradation on the workability of slip-formed concrete, *Journal of Materials in Civil Engineering*, vol.30 issue 28, 2017.

Cost, V.T., Matschei, T., Shannon, J. and Howard, I.L., “Extending the use of fly ash and slag cement in concrete through the use of portland-limestone cement,” *Proceeding of 2014 International Concrete Sustainability Conference*, Boston, 2014.

Crouch, L.K., Sauter, H., and Williams, J.A., 92-MPa air-entrained HPC, *Transportation Research Record*, Vol. 1698, p.24-29, 2000.

Cullu, M. and Arslan, M., The effects of chemical attacks on physical and mechanical properties of concrete produced under cold weather conditions, *Construction and Building Materials*, Vol. 57, p.53-56, 2014.

De Weerdt, K., Ben Haha, M., Le Saout, G., Kjellsen, K. O., Justnes, H. and Lothenbach B., "Hydration mechanisms of ternary portland cement containing limestone powder and fly ash," *Cement and Concrete Research*, vol.4, pp.279-291, 2011.

Deshpande, S., Darwin, D., Browning, J., Evaluating free shrinkage of concrete for control of cracking in bridge decks, Kansas Department of Transportation Research Report, Kansas Department of Transportation, Kansas, 2007.

Dhir, R. K., McCarthy, M. J., Tittle, P. A. J., and Zhou, S., "Discussion: role of cement content in specifications for concrete durability: aggregate type influences," *Structures and Buildings*, vol.159(6), p.361–363, 2006.

Diaz-Loya, I., Juenger, M., Seraj, S., and Minkara, R., "Extending supplementary cementitious material resources: Reclaimed and remediated fly ash and natural pozzolans," *Cement and Concrete Composites*, In Press, 2017.

Divsholi, B.S., Lim, T.Y.D. and Teng, S., Durability properties and microstructure of ground granulated blast furnace slag cement concrete, *International Journal of Concrete Structures and Materials*, vol.8, p.157-164, 2014.

Duval R. and Kadri, E.H., Influence of silica fume on the workability and the compressive strength of high-performance concretes, *Cement and Concrete Research*, vol. 28, p.533-547, 1998.

European Standard EN 197-1, *Cement - Part 1: Composition, specifications, and conformity criteria for common cements*, European Committee for Standardization, 2011.

FDOT, *Standard Specifications for Road and Bridge Construction*, Florida Department of Transportation, Florida, January 2020.

FDOT, *Materials Manual*, Chapter 9, Vol. 2 Section 9.2, Florida, 2020.

FHWA, *Superpave Asphalt Mixture Design*, U.S. Department of Transportation Federal Highway Administration Workshop-workbook, Version 8, Washington, D.C., 2002.

FHWA, Blended aggregates for concrete mixture optimization- best practices for jointed concrete pavements, Tech Brief of FHWA, 2019.

FM 5-550, pH of Soil and Water, Florida Department of Transportation, Florida, 2016.

Fowler, D.W., Hahn, M.D.M., Rached M., Choi, D. and Choi, J., A study on the minimum paste volume in the design of concrete mixture, *International Journal of Concrete Structure and Materials*, vol.2(2) p.161-167, 2008.

Frany, J.A. and Kerkhoff B., *Concrete technology: diagnosis and control of Alkali-aggregate reactions in concrete*, Portland Cement Association, 2007.

Han, S., Chung, H.W., Subgranon, T. and Tia, M., Development of mix designs for minimally refined reclaimed asphalt pavement (RAP) concrete for Florida concrete test road, Sustainability, 10(8),2598, 2018.

Hasegawa, L.I.I., *Carbonation curing and performance of pervious concrete using portland limestone cement*, Graduate Theses and Dissertations, Department of Civil Engineering and Applied Mechanics, McGill University, Québec, Canada, 2011.

Hawkins, P., Tennis, P.D. and Detwiler, R.J., *The Use of Limestone in Portland*

Cement: A State-of-the-art review, Portland Cement Association, 1996.

Huang, C.Y and Feldman, R.F. Influence of silica fume on the micro-structural development in cement mortars, *Cement and Concrete Research*, vol.15, p. 285–294, 1985.

Kennedy, T., Huber, G., Harrigan, E., Cominsky, R., Hughes, C., Quintus H. and Moulthrop, J., *Superior Performing Asphalt Pavements (Superpave): The Product of SHRP Asphalt Research Program*, National Research Council, Washington, D.C., 1994.

Khatib, J.M. and Wild, S., Sulfate resistance of metakaolin mortar. Cement and Concrete Research 28, vol. 28, p.83–92, 1998.

Kolias, S. and Georgiou, C., The effect of paste volume and of water content on the strength and water absorption of concrete, Cement and Concrete Composites, vol.27 issue 2, pp.211-216, 2005.

Ley, T., Cook, D., Seader, N., Ghaeezadeh, A., and Russell, B., ”Aggregate proportioning and gradation for slip formed pavements, “*Proceeding of TTCC-National Concrete Consortium Fall 2014 Meeting Presentation*, Iowa, 2014.

Li, Z. *Advanced Concrete Technology*, John Wiley &Sons, Hoboken, 2011.

Li, L.G., and Kwan, A.K., Adding limestone fines as cementitious paste replacement to improve tensile strength, stiffness and durability of concrete, *Cement and Concrete Composite*, vol. 60, p.17-24, 2015.

Li, Z., Afshinnia, K. and Rangaraju, P.R., Effect of alkali content of cement on properties of high performance cementitious mortar, *Construction and Building Materials*, Vol. 102, Part 1, p.631-639, 2016.

Lindquist, W., Darwin, D. and Browning, J., Development and construction of low-cracking high-performance concrete (lc-hpc) bridge decks:free shrinkage, mixture optimization, and concrete production, Kansas Department of Transportation Research Report, Kansas Department of Transportation, Kansas, 2008.

Lobo, C.L., Specifying requirements for concrete mixtures, Website of Structure Magazine, 2019. (accessed 19 Feb. 2020).

Ma, F., Sha, A., Yang, P. and Huang, Y.,” The greenhouse gas emission from portland cement concrete pavement construction in China,” *International Journal of Environmental Research and Public Health*, Vol. 13(3), p. 351, 2016.

Marar, K., and Eren, Ö, “Effect of cement content and water/cement ratio on fresh concrete properties without admixtures,” *International Journal of the Physical Sciences*, vol. 6(24), p. 5752-5765, 2011.

Mazloom, M., Ramezanianpour, A.A., and Brooks, J.J., Effect of silica fume on mechanical properties of high-strength concrete, *Cement and Concrete Composites*, vol.26, p.347-357, 2004.

Mehta, P.K. and Monteiro, P.J.M., *Concrete: Microstructure, Properties and Materials* (3rd ed.), McGraw-Hill, New York, 2006.

Mindess, S., Young, J.F., and Darwin, D. Concrete, 2nd Edition, Pearson Education, Inc., Upper Saddle River, NJ, 2003.

Moini, M., *The optimization of concrete mixtures for use in highway applications*, Master of Science Thesis, Department of Civil Engineering, University of Wisconsin Milwaukee, 2015.

Nadelman, E.I., *Hydration and microstructural development of portland limestone cement-based materials*, Ph.D. Dissertation, School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, Georgia, 2016.

Neville, A.M., *Properties of concrete* (4th ed.), Longman, Essex, UK, 1995.

Nijboer, L.W., *Plasticity as a Factor in the Design of Dense Bituminous Road Carpets*, Elsevier Publishing, New York,1948.

Nochaiya, T., Wongkeo, W., and Chaipanich A., Utilization of fly ash with silica fume and properties of Portland cement–fly ash–silica fume concrete, *Fuel*, vol.89(3), p.768-744, 2010.

Obla, K.h., Hong, R., Lobo, C.L., and Kim, H., Should minimum cementitious materials contents for concrete be specified? *Journal of the Transportation Research Board*, No., 2629, p.1-8, 2017.

Oluokun, F.A., Fly ash concrete mix design and the water-cement ratio law, ACI Materials Journal, vol.91(4), 1994.

Osborne, G.J., Durability of portland blast-furnace slag cement concrete, *Cement and Concrete Composites*, vol. 21, p.11- 21, 1999.

Ozbay,E., Erdemir, M., and Durmus, H.I., Utilization and efficiency of ground granulated blast furnace slag on concrete properties – A review, *Construction and Building Materials*, vol.105, p.423-434, 2016.

Panchalan, R.K. and Ramakrishnan V., "Validity of 0.45 power chart in obtaining the optimized aggregate gradation for improving the strength aspects of high-performance concrete," *American Concrete Institute Special Publication*, Vol. 243, p. 99-108, 2007.

Panesar, D.K., Aqel, Rhead, D., and Schell, H., Effect of cement type and limestone particle size on the durability of steam cured self-consolidating concrete, *Cement and Concrete Composite*, vol. 80, p.175-189, 2017.

Panesar, D.K. and Zang, R., Performance comparison of cement replacing materials in concrete: Limestone fillers and supplementary cementing materials – A review, *Construction and Building Materials*, vol. 251, 2020.

PE International, GaBi Software, http://www.gabi-software.com/america/index/, 2011.

Piasta W., and Zarzycki, B, The effect of cement paste volume and w/c ratio on shrinkage strain, water absorption and compressive strength of high-performance concrete, Construction and Building Materials, vol.140, p.395-402, 2017.

Pickett, G., Effect of aggregate on shrinkage of concrete and a hypothesis concerning shrinkage, ACI Journal, *Proceedings*, Vol. 52, No. 1, p. 581-590, 1956.

Précontrainte, F., *Condensed silica fume in concrete,* Thomas Telford Ltd., London, 1998.

Quiroga, P.N. and Fowler, D. W., *The effects of aggregates characteristics on the performance of portland cement concrete*, International Center for Aggregates Research, Research Report ICAR 104-1F, 2004.

Rached, M., Fowler D. and Koehler, E., Use of aggregates to reduce cement content in concrete, *Proceedings of Second International Conference on Sustainable Construction Materials and Technologies*. Universita Politecnica delle Marche, Ancona, Italy, 2010.

Richardson, D.N., *Aggregate gradation optimization-literature search*, MoDOT, Research Report, Missouri, 2005.

Santero, N., Loijos, A., and Ochsendorf, J., "Greenhouse gas emissions reduction opportunities for concrete pavements*," Journal of Industrial Ecology*, Vol. 17, 2013.

Shannon, J., Howard, I.L., Cost, V.T. and Wilson, W.M., "Benefits of portland-limestone cement for concrete with rounded gravel aggregates and higher fly ash replacement rates," *Proceeding of 94th Annual Meeting of the Transportation Research Board*, Washington, D.C., 2014.

Shilstone, J. M., "Concrete mixture optimization," *Concrete International*, Vol.10, p. 33–39, 1990.

Shilstone, J.M. and Shilstone, J.M. Jr., "Concrete mix analysis-interpreting the coarseness factor chart," Newsletter: 1, 1997a.

Shilstone, J.M. and Shilstone, J.M. Jr., “Concrete performance is predictable – a comparison of two mixtures,” Newsletter: 1, 1997b.

STP 1147, *Effects of Aggregates and Mineral Fillers on Asphalt Mixture Performance*, ASTM Special Technical Publication, West Conshohocken, Pennsylvania, 1992.

Tang, S.W., Yao, Y., Andrade, C., and Li, Z.Y., Recent durability studies on concrete structure, *Cement and Concrete Research*, Vol. 78, p. 143-154, 2015.

Taylor, P., *Concrete pavement mixture design and analysis (mda): effect of aggregate systems on concrete properties*, Technical Report, National Concrete Pavement Technology Center, Iowa, 2012.

Taylor, P. Blended aggregates for concrete mixture optimization, Federal Highway Administration, Washington, D.C., 2015.

Tia., M., Chung, H.W., Subgranon, T., Han, S., “Reducing Portland Cement Content and Improving Concrete Durability,: Research Report, University of Florida, May 2019, 187 pages.

Tracz T. and Sliwinski, J., Effect of cement paste content and w/cm on concrete water absorption, Cement Lime Concrete, vol. 3, p. 131-137, 2012.

Tritsch, N., Darwin, D., and Browning, J., Evaluating shrinkage and cracking behavior of concrete using restrained ring and free shrinkage tests, Research Report, The University of Kansas Center for Research, Kansas, 2005.

Turk, K., Kina, C., and Bagdiken, M., Use of binary and ternary cementitious blends of F-Class fly-ash and limestone powder to mitigate alkali-silica reaction risk, *Construction and Building Materials*, vol. 151, p. 422-427, 2017.

Uysal M. and Akyuncu V., Durability performance of concrete incorporating Class F and Class C fly ashes, *Construction and Building Materials*, vol. 34, p.170-178, 2012.

Wassermann, R., Katz, A. and Bentur, A., Minimum cement content requirements: a must or a myth?. *Materials and Structures*, No. 42, p. 973–982, 2009.

Yurdakul, E., *Optimizing concrete mixtures with minimum cement content for performance and sustainability*, Graduate Theses and Dissertations, Department of Civil and Environmental Engineering, Iowa State University, Ames, Iowa, 2010.

Yurdakul, E., Taylor, P. C., Ceylan, H., and Bektas, F., "Effect of water-to-binder ratio, air content, and type of cementitious materials on fresh and hardened properties of binary and ternary blended concrete," *Journal of Materials in Civil Engineering*, Vol. 26(6), p. 04014002 2014.

Zapata, P. and Gambatese, J.A.," Energy consumption of asphalt and reinforced concrete," Journal of Infrastructure Systems, Vol. 11(1), p.9-20, 2005.

Zhang, J., Han, Y. D., and Gao, Y., "Effects of water-binder ratio and coarse aggregate on interior humidity, autogenous shrinkage, and drying shrinkage of concrete," *Journal of Materials in Civil Engineering*, vol. 26(1), p.184-189, 2014.

APPENDIX A
THE FRESH CONCRETE PROPERTIES OF THE TRIAL BATCH CONCRETE MIXES

(1) Class II Concrete

The results of fresh concrete tests on the Class II concrete are shown in Table A-1. The following section presents the fresh concrete properties of the Class II concrete mixtures.

The slump test results showed that the concrete mixes with lower CPV have relatively lower slump. Figure A-1 shows that the slump of the fresh concrete improves with increasing CPV of concrete. OAG mixes have higher slump than the typical FSC mixes. Thus, the CPV can affect the slump of the fresh concrete. OAG technique can be used to improve the slump of the fresh concrete.

The air content of all mixtures was in the range of 1-6%. For the concretes with a higher CPV, more bleeding was observed. Figure A-2 shows the air content of the FSC and OAG mixes. At the same CPV, the OAG mixes have lower air content than the FSC mixes. Because the OAG mixes have better packing density than the FSC mixes, the air content of the OAG mixes is lower.

Table A-1 Fresh concrete properties of Class II concrete evaluated in this research study

Concrete Mix (w/cm=0.50)	CPV %	Slump ASTM C143 (inches)	Air Content ASTM C231 (%)	Temp. ASTM C1064 (°F)	Density ASTM C138 (lb/ft^3)	Final Bleeding ASTM C232 (ml/inches^2)
FSC225	22.5	0.50	3.9%	71	139.28	0.00
FSC250	25.0	1.00	3.3%	69	142.40	0.07
FSC275	27.5	1.50	2.0%	74	143.28	0.04
FSC300	30.0	3.25	2.3%	76	142.32	0.08
OAG225	22.5	1.00	3.1%	71	141.12	0.02
OAG250	25.0	1.25	1.9%	69	143.76	0.06
OAG275	27.5	1.75	1.9%	74	143.20	0.05
OAG300	30.0	3.50	1.5%	76	143.20	0.10

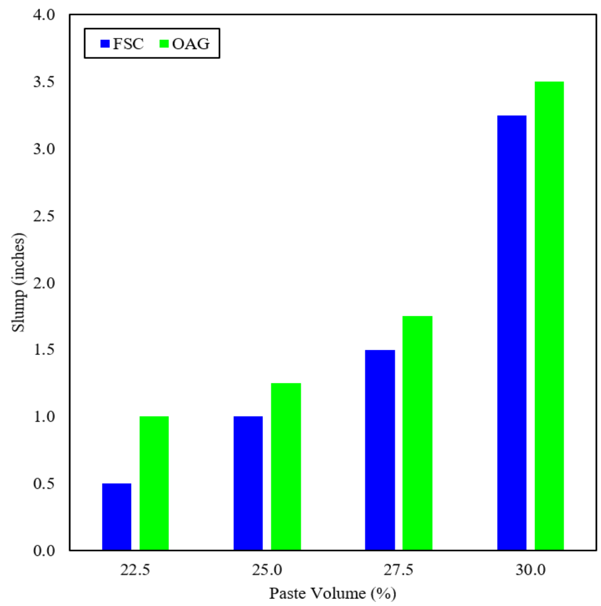


Figure A-1 Slump of FSC and OAG mixes in Class II concrete

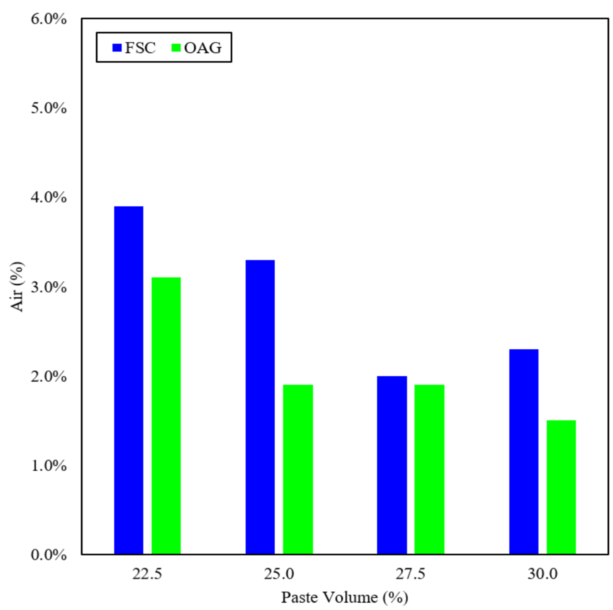


Figure A-2 Air content of FSC and OAG mixes in Class II concrete

(2) Class II - Bridge Deck Concrete

The results of fresh concrete tests on Class II - Bridge Deck mixes are shown in Table A-2. The following section presents the fresh concrete properties of the Class II - Bridge Deck concrete mixtures.

The slump test results showed that the concrete mixes with lower CPV have lower slump. Figure A-3 shows that the slump of the fresh concrete improves with increasing CPV of concrete. The

OAG mixes have higher slump than typical FSC mixes. Thus, the CPV can affect the slump of the fresh concrete. OAG technique can be used to improve the slump of the fresh concrete.

The air content of all mixtures was in the range of 1-6%. For the concretes with a higher CPV, more bleeding was observed. Figure A-4 shows the air content of the FSC and OAG mixes. At the same CPV, the OAG mixes have lower air content than the FSC mixes.

Table A-2 Fresh concrete properties of Class II - Bridge Deck concrete evaluated in this research study

Concrete Mix (w/cm=0.44)	CPV (%)	Slump ASTM C143 (inches)	Air Content ASTM C231 (%)	Temp. ASTM C1064 (°F)	Density ASTM C138 (lb/ft^3)	Final Bleeding ASTM C232 ($ml/inches^2$)
FSC225	22.5	2.25	3.5%	72	139.68	0.09
FSC250	25.0	1.50	3.4%	71	141.76	0.00
FSC275	27.5	2.00	3.8%	72	140.88	0.00
FSC300	30.0	3.00	3.8%	73	139.60	0.55
OAG225	22.5	2.50	2.8%	73	141.68	0.10
OAG250	25.0	1.75	2.7%	72	142.40	0.02
OAG275	27.5	2.50	3.2%	72	141.04	0.00
OAG300	30.0	3.25	3.1%	73	140.40	0.57

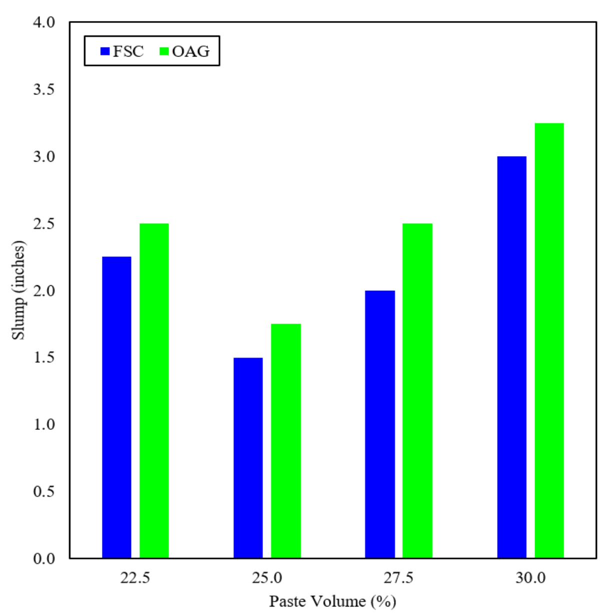


Figure A-3 Slump of FSC and OAG mixes in Class II - Bridge Deck concrete

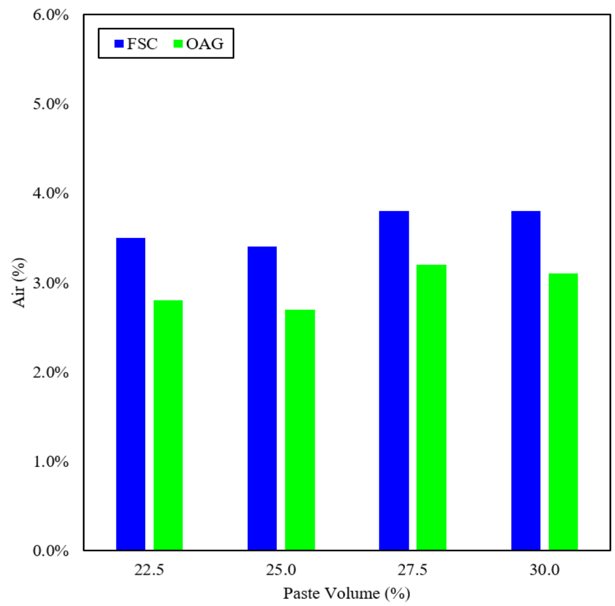


Figure A-4 Air content of FSC and OAG mixes in Class II - Bridge Deck concrete

(3) Class IV Concrete

The results of fresh concrete tests on Class IV mixes are shown in Table A-3. The following section presents the fresh concrete properties of the Class IV concrete mixtures.

The slump test results showed that the concrete mixes with a lower CPV have lower slump. However, the OAG mixes show better slump for the mixes with low CPV. Figure A-5 shows that the slump of the fresh concrete generally improves while increasing the CPV of concrete. OAG mixes have higher slump than the typical FSC mixes. Thus, the CPV can affect the slump of the fresh concrete. OAG technique can be used to improve the slump of the fresh concrete.

The air content of all mixtures was in the range of 1-6%. For the concretes with a higher CPV, more bleeding was observed. Figure A-6 shows the air content of the FSC and OAG mixes. At the same CPV, the OAG mixes have lower air content than the FSC mixes.

Table A-3 Fresh concrete properties of Class IV concrete evaluated in this research study

Concrete Mix (w/cm=0.38)	CPV (%)	Slump ASTM C143 (inches)	Air Content ASTM C231 (%)	Temp. ASTM C1064 (°F)	Density ASTM C138 (lb/ft^3)	Final Bleeding ASTM C232 (ml/inches^2)
FSC250	25.0	2.00	3.3%	74	142.40	0.00
FSC275	27.5	2.00	2.8%	73	144.48	0.00
FSC300	30.0	3.00	2.8%	75	141.92	0.03
FSC325	32.5	2.75	2.4%	76	142.48	0.04
OAG250	25.0	3.50	1.7%	74	146.00	0.00
OAG275	27.5	3.25	3.9%	73	142.80	0.00
OAG300	30.0	2.25	2.1%	75	142.96	0.04
OAG325	32.5	3.25	2.1%	76	142.88	0.06

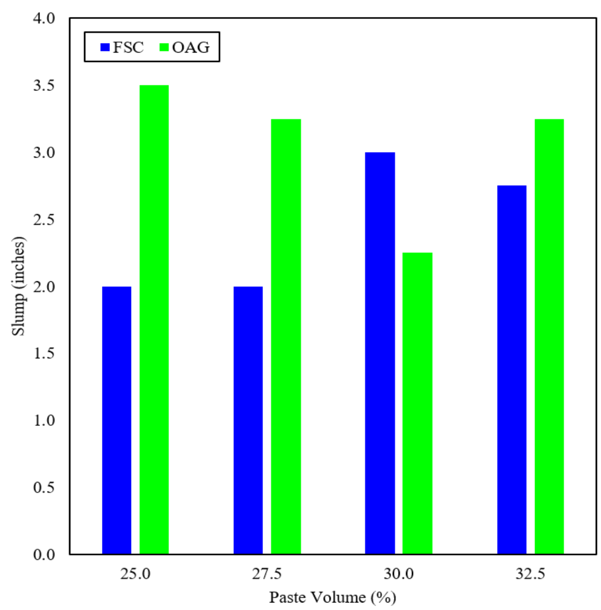


Figure A-5 Slump of FSC and OAG mixes in Class IV concrete

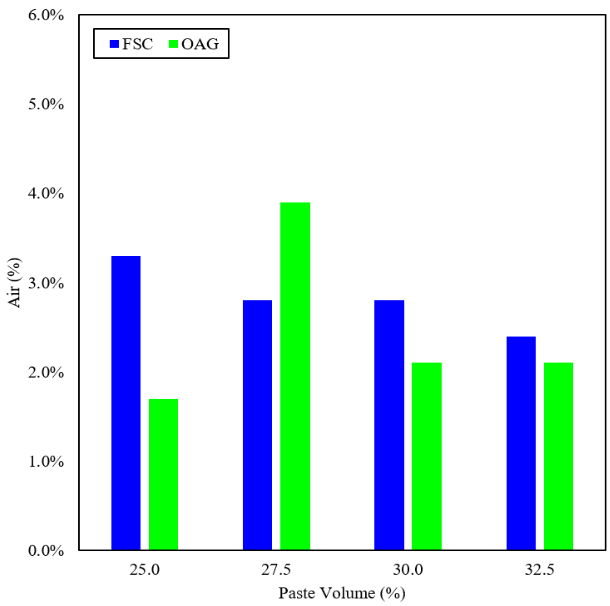


Figure A-6 Air content of FSC and OAG mixes in Class IV concrete

(4) Class V concrete

The results of fresh concrete tests on the Class V mixes are shown in Table A-4. The following section presents the fresh concrete properties of the Class V concrete mixtures.

The slump test results showed that the concrete mixes with a lower CPV have lower slump. Figure A-7 shows that the slump of the fresh concrete generally improves while increasing the CPV of concrete. OAG mixes have similar slump as the typical FSC mixes. Thus, the CPV can affect the slump of the properties. The use of OAG technique did not significantly improve the slump of the fresh concrete in Class V concrete.

The air content of all mixtures was in the range of 1-6%. For the concretes with a higher CPV, more bleeding was observed. Figure A-8 shows the air content of the FSC and OAG mixes. At the same CPV, the OAG mixes have a lower air content than the FSC mixes.

Table A-4 Fresh concrete properties of Class V concrete evaluated in this research study

Concrete Mix (w/cm=0.32)	CPV (%)	Slump ASTM C143 (inches)	Air Content ASTM C231 (%)	Temp. ASTM C1064 (°F)	Density ASTM C138 (lb/ft^3)	Final Bleeding ASTM C232 (ml/inches^2)
FSC250	25.0	1.75	2.0%	74	145.12	0.00
FSC275	27.5	3.00	4.1%	73	141.92	0.00
FSC300	30.0	1.75	3.0%	74	143.12	0.00
FSC325	32.5	3.75	2.6%	74	143.20	0.00
OAG250	25.0	1.75	2.0%	75	145.52	0.00
OAG275	27.5	2.50	2.9%	74	143.36	0.00
OAG300	30.0	2.00	2.0%	74	143.92	0.00
OAG325	32.5	3.50	3.7%	75	140.24	0.00

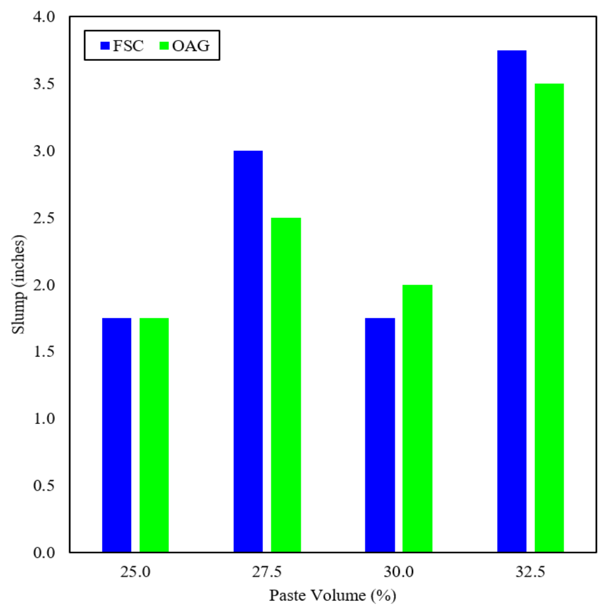


Figure A-7 Slump of FSC and OAG mixes in Class IV concrete

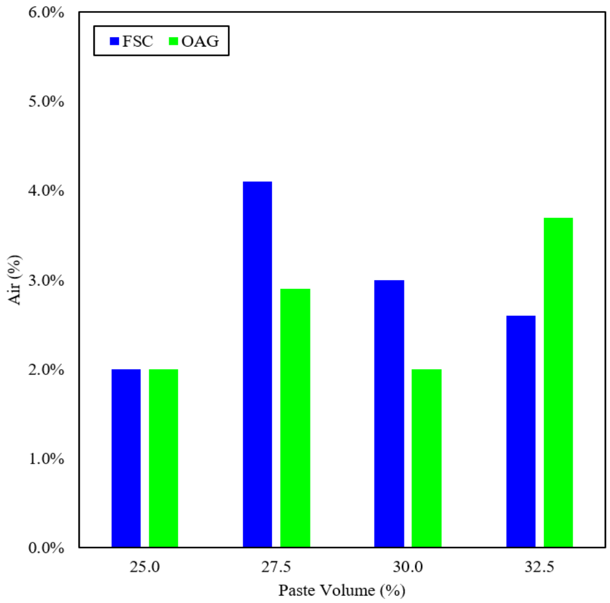


Figure A-8 Air content of FSC and OAG mixes in Class V concrete

APPENDIX B
THE HARDENED CONCRETE PROPERTIES OF THE TRIAL BATCH CONCRETE MIXES

(1) Class II concrete

According to FDOT Standard Specifications for Road and Bridge Construction, Section 346, the specified minimum strength of Class II concrete is 3,400 psi and the required minimum strength is 4,600 psi at 28 days. Figures B-1 and B-2 present the compressive strength development of the FSC and OAG mixes at 7, 28, and 56 days. It can be seen that, for the FSC and OAG mixes, the compressive strengths were about the same at different CPV at different ages. Figure B-3 shows the plots of compressive strength of the FSC and OAG mixes at 28 days. The strength of the FSC and OAG mixes all passed the required minimum strength (4,600 psi). Thus, the minimum CPV of the Class II concrete can be determined to be 22.5% to achieve the target strength.

Figures B-4 and B-5 show the development of the surface resistivity of the FSC and OAG mixes. The surface resistivity of concrete increased when the CPV decreased. Figure B-6 shows the surface resistivity of concrete of the FSC and OAG mixes at 28 days. The better packing in the concrete (OAG) cannot improve the surface resistivity. According to the results of strength and surface resistivity tests, it can be concluded that increasing the CPV of concrete cannot improve the properties of Class II concrete. The concrete with optimized CPV will provide the proper properties of hardened concrete.

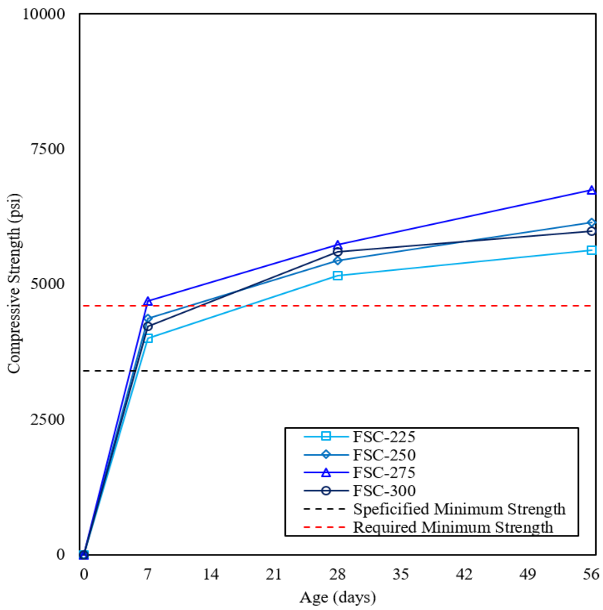


Figure B-1 Strength development of FSC mixes in Class II concrete

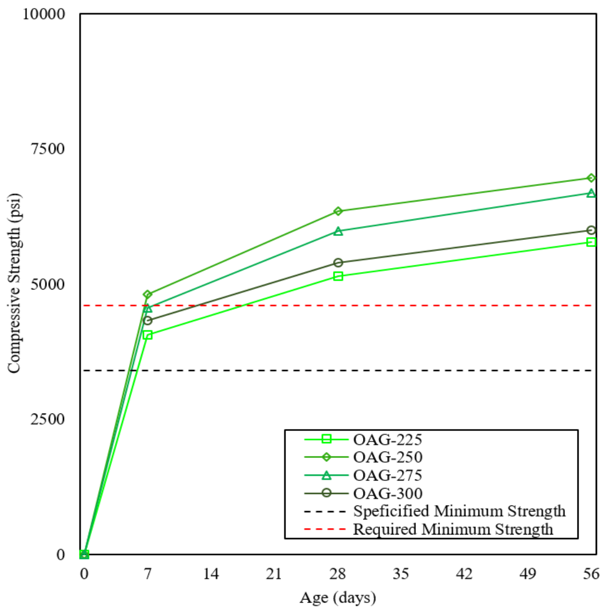


Figure B-2 Strength development of OAG mixes in Class II concrete

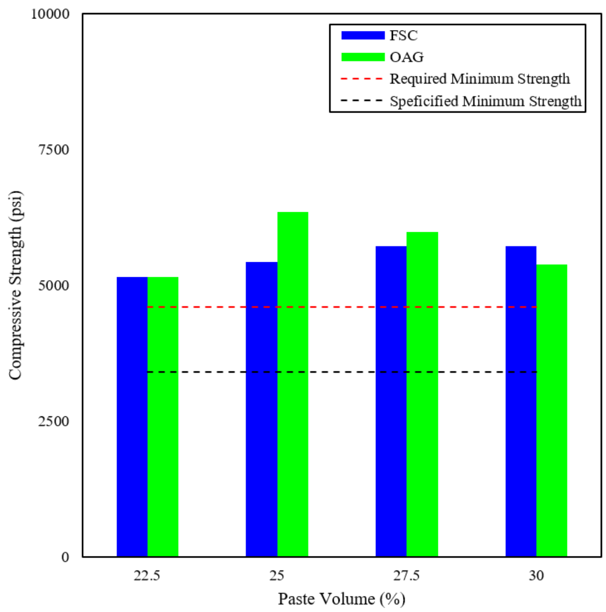


Figure B-3 Compressive strength of FSC and OAG mixes at 28 days in Class II concrete

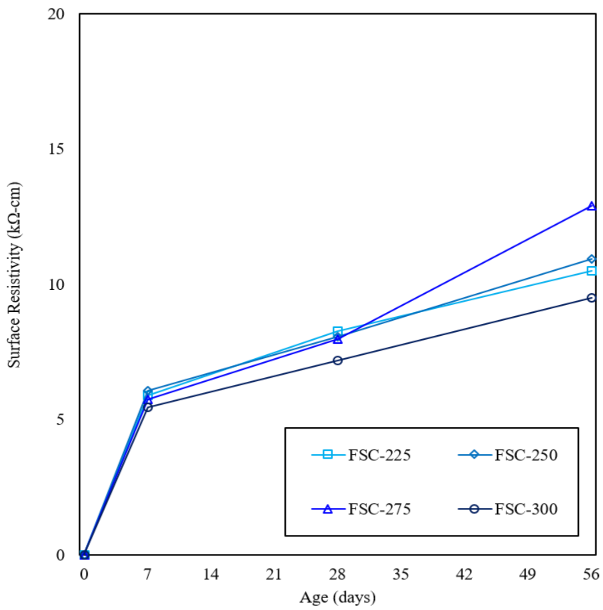


Figure B-4 Surface resistivity of FSC mixes in Class II concrete

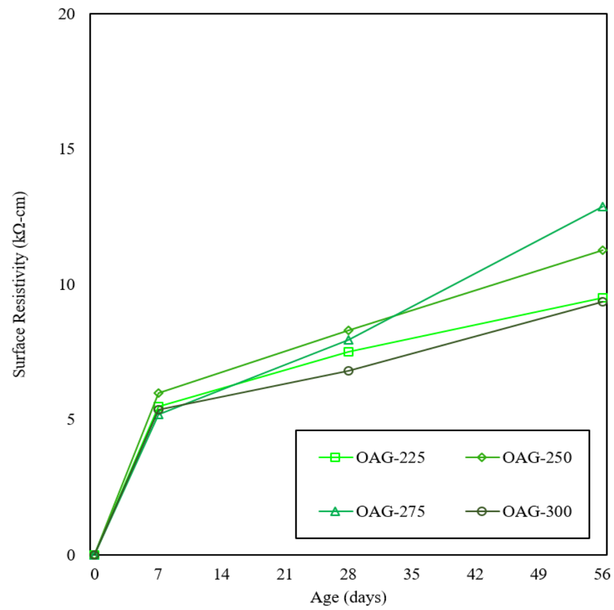


Figure B-5 Surface resistivity of OAG mixes in Class II concrete

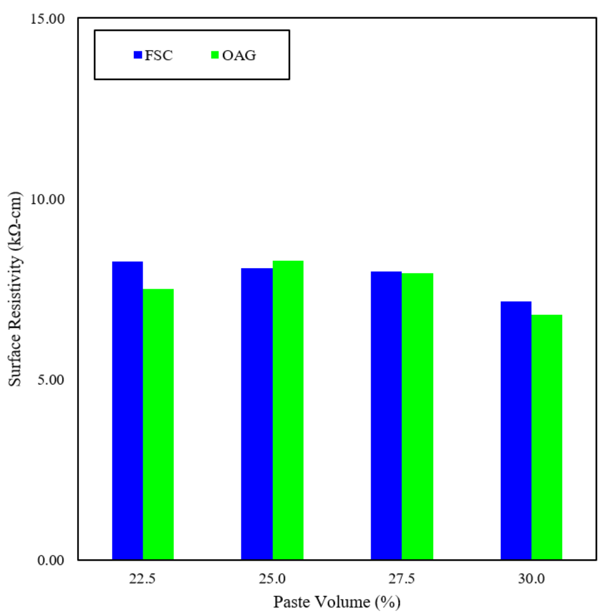


Figure B-6 Surface resistivity of FSC and OAG mixes at 28 days in Class II concrete

(2) Class II - Bridge Deck concrete

According to FDOT Standard Specifications for Road and Bridge Construction, Section 346, the specified minimum strength of Class II - Bridge Deck concrete is 4,500 psi and the required minimum strength is 5,700 psi at 28 days. Figures B-7 and B-8 present the compressive strength development of the FSC and OAG mixes at 7, 28, and 56 days. It can be seen that, for the FSC and OAG mixes, the compressive strengths were about the same at different CPV at different ages. Figure B-9 shows the plots of compressive strength of the FSC and OAG mixes at 28 days. The strength of most of the FSC and OAG mixes (25.0%, 27.5% and 30.0%) passed the required minimum strength (5,700 psi). The concrete with CPV of 22.5% passed the specified minimum strength (4,500 psi), but it did not pass the required minimum strength (5,700 psi). Thus, the minimum CPV of the Class II concrete can be determined to be 25.0% to achieve the target strength.

Figures B-10 and B-11 show the development of the surface resistivity of the FSC and OAG mixes. The surface resistivity of concrete increased when the CPV decreased. Figure B-12 shows the surface resistivity of the FSC and OAG mixes at 28 days. The better packing of concrete (OAG) cannot improve the surface resistivity. According to the results of strength and surface resistivity tests, it can be concluded that increasing the CPV of concrete cannot improve the properties of concrete in Class II - Bridge Deck concrete. The concrete with optimized CPV will provide the proper properties of hardened concrete.

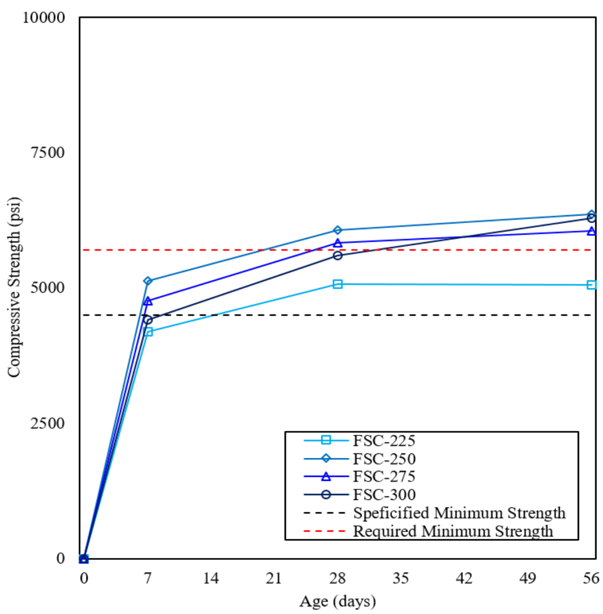


Figure B-7 Strength development of FSC mixes in Class II - Bridge Deck concrete

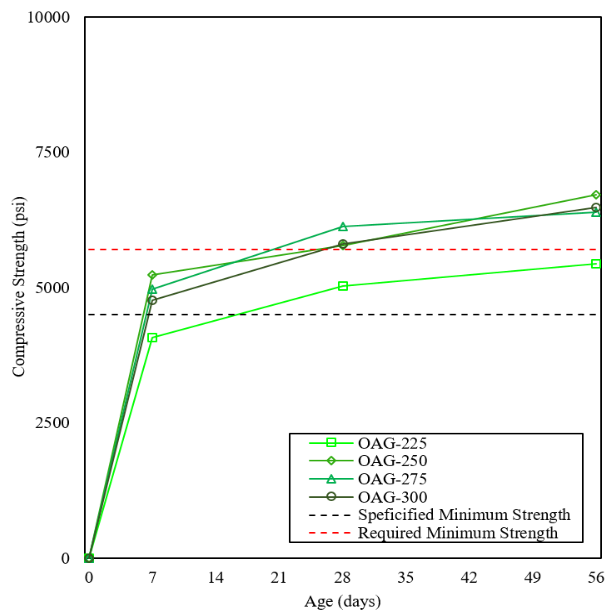


Figure B-8 Strength development of OAG mixes in Class II - Bridge Deck concrete

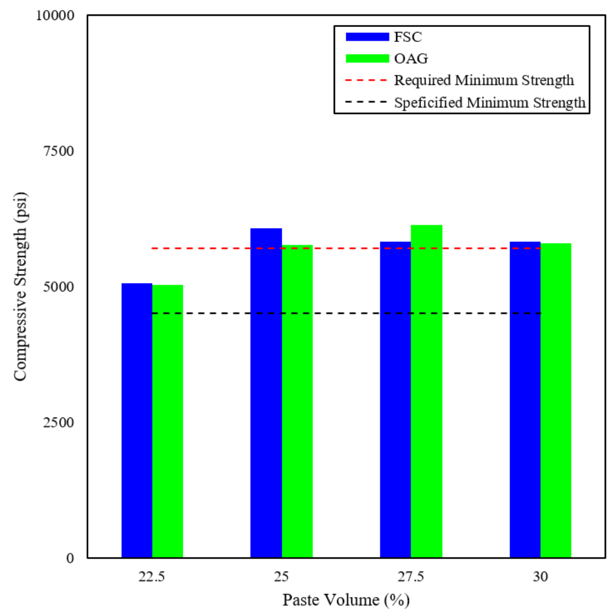


Figure B-9 Compressive strength of FSC and OAG mixes at 28 days in Class II - Bridge Deck concrete

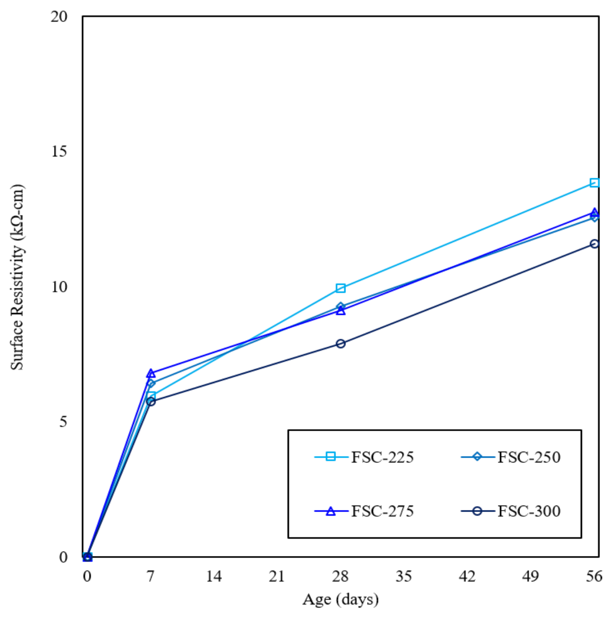


Figure B-10 Surface resistivity of FSC mixes in Class II - Bridge Deck concrete

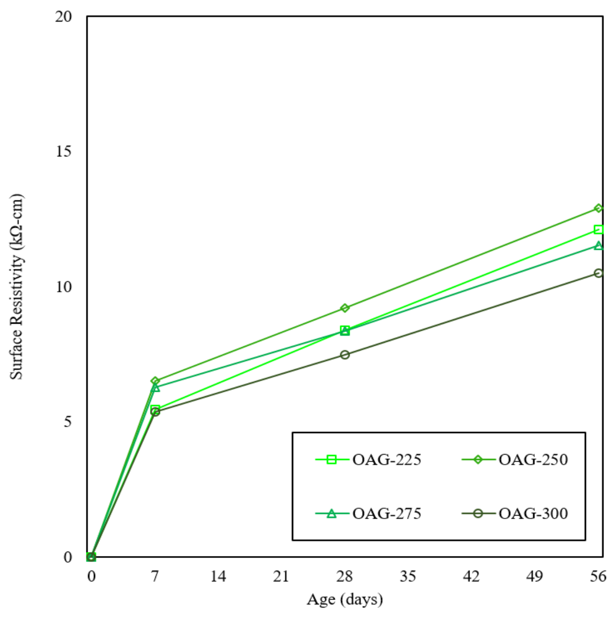


Figure B-11 Surface resistivity of OAG mixes in Class II - Bridge Deck concrete

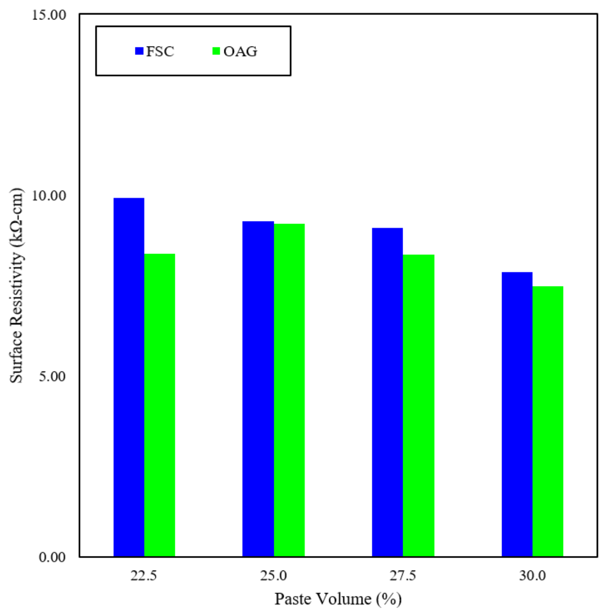


Figure B-12 Surface resistivity of FSC and OAG mixes at 28 days in Class II - Bridge Deck concrete

(3) Class IV concrete

According to FDOT Standard Specifications for Road and Bridge Construction, Section 346, the specified minimum strength of Class IV deck concrete is 5,500 psi and the required minimum strength is 6,750 psi at 28 days. Figures B-13 and B-14 present the compressive strength development of the FSC and OAG mixes at 7, 28, and 56 days. It can be seen that, for FSC and OAG mixes, the compressive strengths were about the same at different CPV at different ages. Figure B-15 shows the plots of compressive strength of FSC and OAG mixes at 28 days. The strength of all FSC and OAG mixes passed the required minimum strength (6,750 psi). Thus, the minimum CPV of Class IV concrete can be determined to be 25.0% to achieve the target strength. Figures B-16 and B-17 show the development of the surface resistivity of the FSC and OAG mixes. The surface resistivity of concrete increased when the CPV decreased. Figure B-18 shows the surface resistivity of concrete of FSC and OAG mixes at 28 days. The better packing of concrete (OAG) cannot improve the surface resistivity. According to the results of strength and surface resistivity tests, it can be concluded that increasing the CPV of concrete cannot improve the properties of concrete in Class IV concrete. The concrete with optimized CPV will provide the proper properties of hardened concrete.

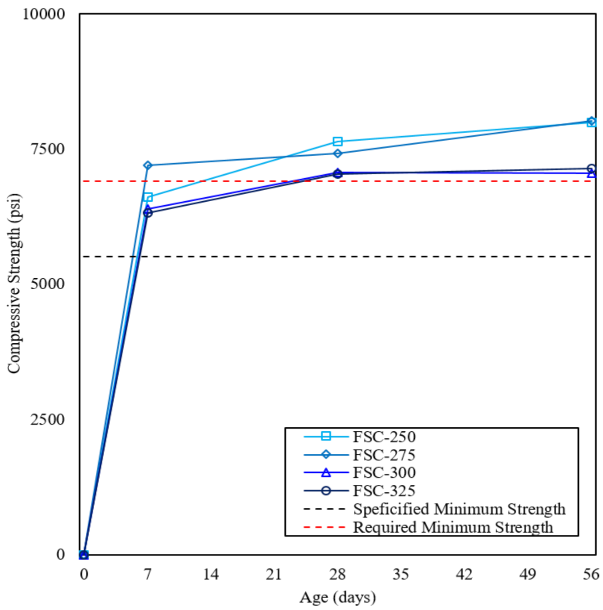


Figure B-13 Strength development of FSC mixes in Class IV concrete

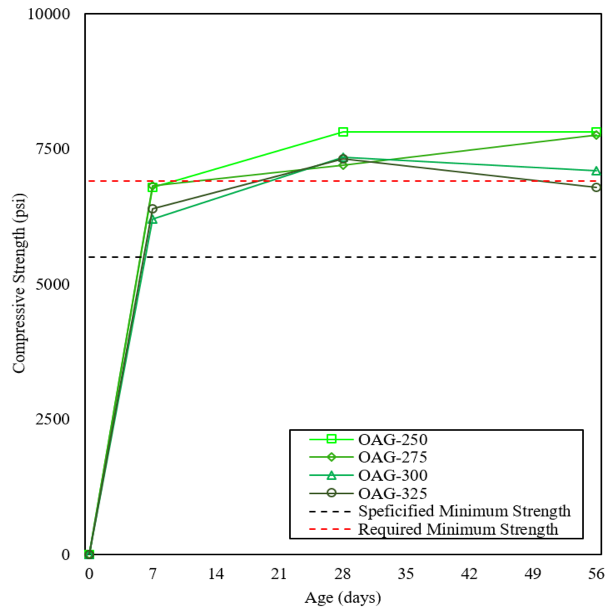


Figure B-14 Strength development of OAG mixes in Class IV concrete

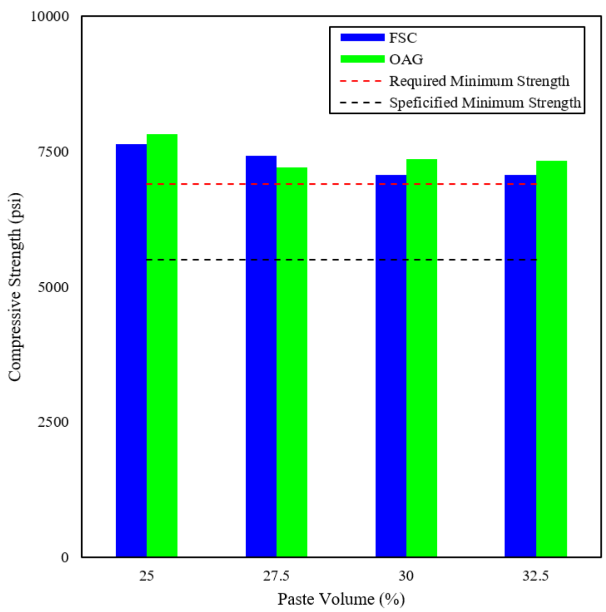
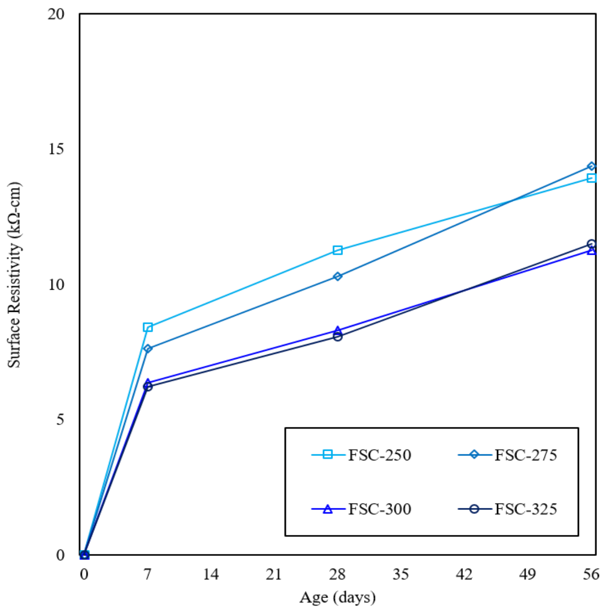


Figure B-15 Compressive strength of FSC and OAG mixes at 28 days in Class IV concrete



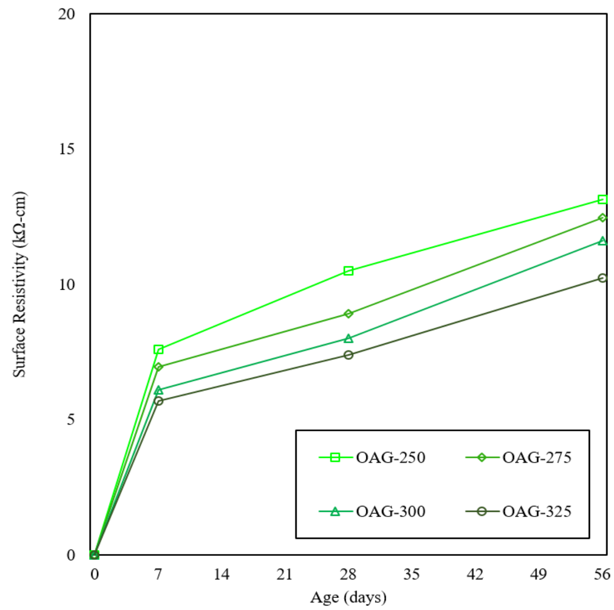


Figure B-16 Surface resistivity of FSC mixes in Class IV concrete

Figure B-17 Surface resistivity of OAG mixes in Class IV concrete

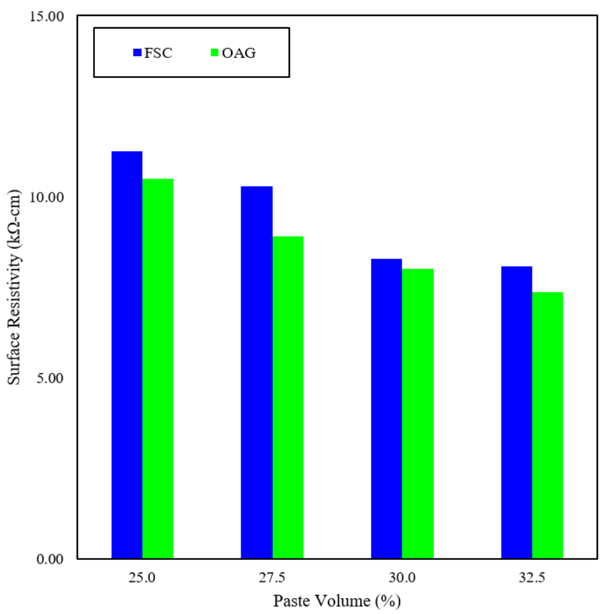


Figure B-18 Surface resistivity of FSC and OAG mixes at 28 days in Class IV concrete

(4) Class V concrete

According to FDOT Standard Specifications for Road and Bridge Construction, Section 346, the specified minimum strength of Class V deck concrete is 6,500 psi and the required minimum strength is 7,850 psi at 28 days. Figures B-19 and B-20 present the compressive strength development of the FSC and OAG mixes at 7, 28, and 56 days. It can be seen that, for the FSC and OAG mixes, the compressive strengths were about the same at different CPV at different ages. Figure B-21 shows the plots of compressive strength of the FSC and OAG mixes at 28 days. The strength of most of the FSC and OAG mixes (25.0%, 27.5% and 30.0%) passed the required minimum strength (7,850 psi). The CPV of concrete with 32.5% only passed the specified minimum strength (6,500 psi), but not the required minimum strength (7,850 psi). Thus, the minimum CPV of Class V concrete can be determined to be 25.0% to achieve the target strength. Figures B-22 and B-23 show the development of the surface resistivity of FSC and OAG mixes. The surface resistivity of concrete increased when the CPV decreased. Figure B-24 shows the surface resistivity of the FSC and OAG mixes at 28 days. The better packing of concrete (OAG) cannot improve the surface resistivity. According to the results of strength and surface resistivity, it can be concluded that increasing the CPV of concrete cannot improve the properties of concrete in Class V concrete. The concrete with optimized CPV will provide the proper properties of hardened concrete.

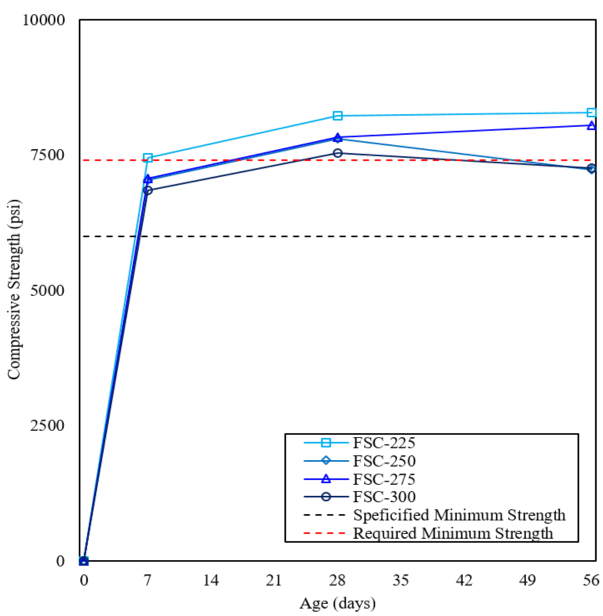


Figure B-19 Strength development of FSC mixes in Class V concrete

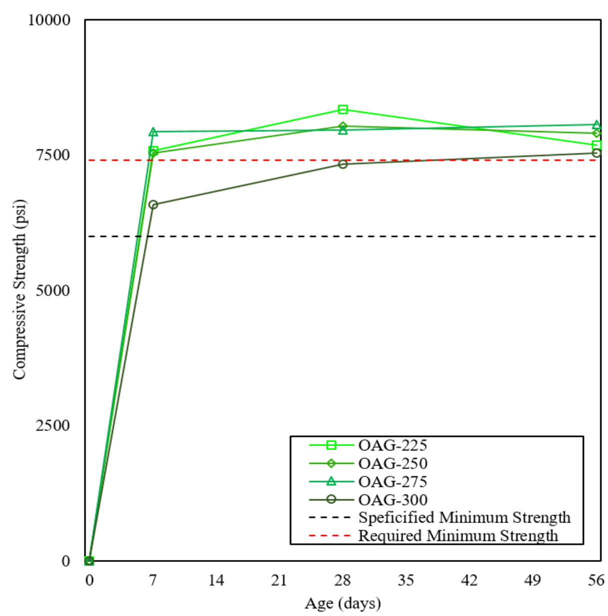


Figure B-20 Strength development of OAG mixes in Class V concrete

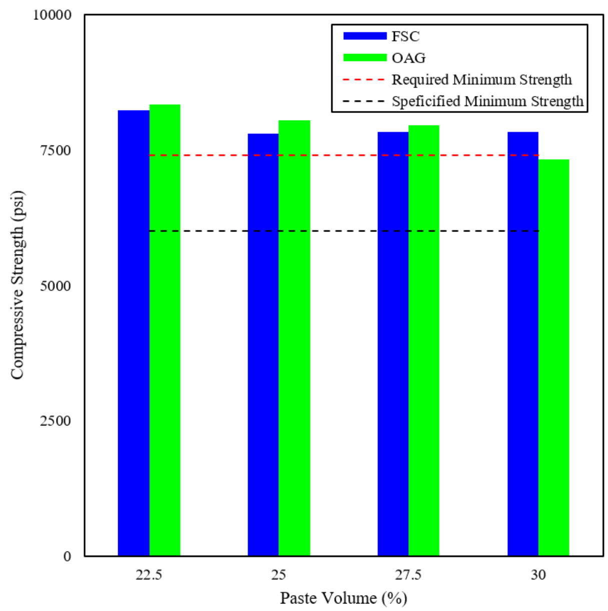


Figure B-21 Compressive strength of FSC and OAG mixes at 28 days in Class V concrete

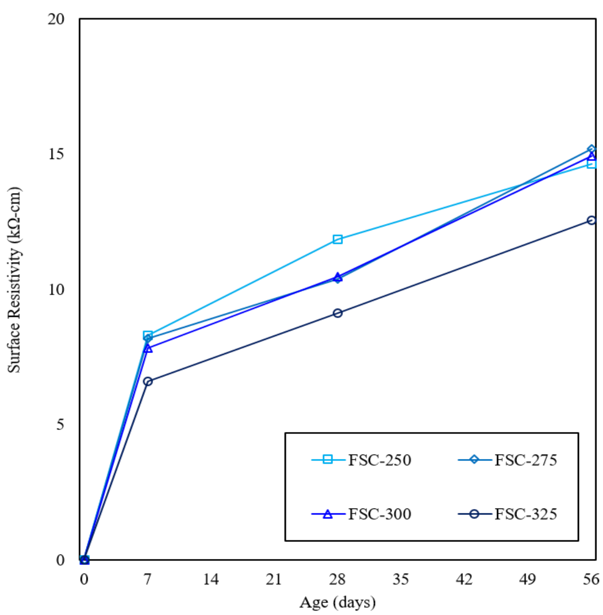


Figure B-22 Surface resistivity of FSC mixes in Class V concrete

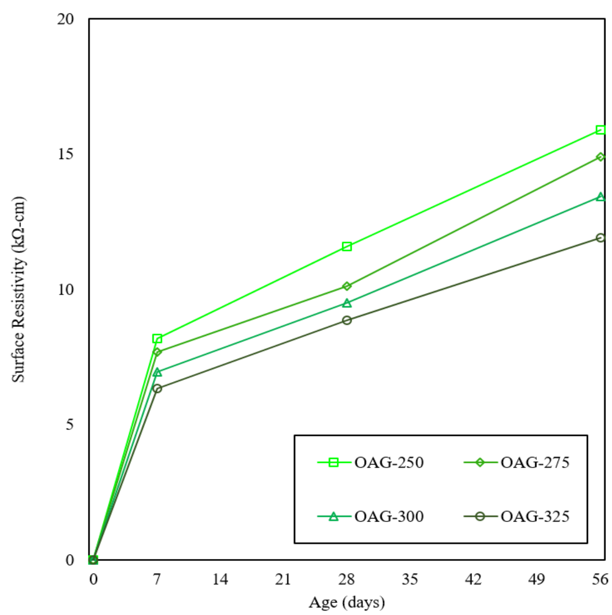


Figure B-23 Surface resistivity of OAG mixes in Class V concrete

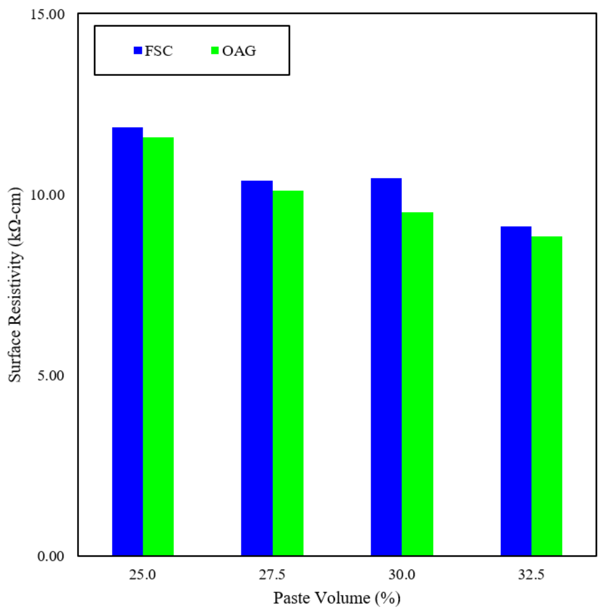


Figure B-24 Surface resistivity of FSC and OAG mixes at 28 days in Class V concrete