FINAL REPORT

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DEVELOPMENT OF TIERED AGGREGATE SPECIFICATIONS FOR FDOT USE

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The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the State of Florida Department of Transportation or the U.S. Department of Transportation.

Prepared in cooperation with the State of Florida Department of Transportation and the U.S. Department of Transportation.

SI (MODERN METRIC) CONVERSION FACTORS (from FHWA)

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

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		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

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		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				

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		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")

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
kip	kilo poundforce	4.45	kilo newtons	kN
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS TO 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

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		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

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		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

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		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

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	$candela/m^2$	0.2919	foot-Lamberts	fl

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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16. Abstract

At present, all limestone aggregates to be used in Florida Department of Transportation (FDOT) projects fall under a single category and must meet the same set of minimum durability requirements. For example, a limestone aggregate mine which could not produce aggregates with a maximum LA Abrasion Loss of 45% would not be approved by FDOT. This research project was conducted to investigate the feasibility of using some aggregates which do not meet the current FDOT aggregate specifications in some concrete applications and to develop tiered aggregate specifications for use by FDOT.

A laboratory testing program was conducted to evaluate the properties of concrete made with aggregates which do not meet current FDOT aggregate specifications and to compare them with those made with aggregates which meet the current FDOT specifications. Ten different aggregate sources were selected for use in this study. They included two control aggregates which met the 2010 FDOT aggregate specifications and eight aggregates which did not meet at least one of the aggregate specification requirements. The required aggregate properties which were not met included the maximum allowable LA abrasion loss, percent passing No. 200 sieve, and shell content.

Based on the results from this study, a developmental three-tier aggregate specification is recommended for use by FDOT. The first tier (Category A) aggregate would have the same requirements as the current FDOT aggregate specifications. The second tier (Category B) and third tier (Category C) aggregates have relaxed requirements in maximum allowable LA abrasion loss, sodium sulfate soundness loss, and materials passing No. 200 sieve. Possible uses for Category B aggregates include all nonstructural concrete applications, manholes, inlets, and junction boxes. Category C aggregates are recommended only for non-structural concrete applications. Results of analysis in this study indicate that Category B aggregates can possibly be used in concrete pavement application. Further study to evaluate the suitability of using Category B aggregates for concrete pavement and precast concrete pipe applications is recommended.

17. Key Words	18. Distribution Statement
Tiered Aggregate Specification, LA Abrasion, Micro-Deval Abrasion, sodium sulfate soundness Loss, Material Passing No. 200 Sieve, Shell Content, Compressive Strength, Water Absorption, FDOT Specifications, FEACONS.	No restrictions.

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EXECUTIVE SUMMARY

Objective of Study

At present, all limestone aggregates to be used in Florida Department of Transportation (FDOT) projects fall under a single category and must meet the same set of minimum durability requirements. For example, a limestone aggregate mine which could not produce aggregates with a maximum Los Angeles (LA) abrasion loss of 45% would not be approved by FDOT. This research project was conducted to investigate the feasibility of using some aggregates which do not meet the current FDOT aggregate specifications in some concrete applications and to develop tiered aggregate specifications to allow for the use of these aggregates in some FDOT projects. Implementing such a system of aggregate specification will induce the opening of new mines that do not currently meet FDOT specifications and would extend the reserves in existing mines that do. Existing mines may also benefit by adding a new production process dedicated to meeting the new tiered specifications.

Laboratory Study

A laboratory testing program was conducted to evaluate the properties of concrete made with non-standard aggregates which do not meet current FDOT aggregate specifications and to compare them with those made with standard aggregates which meet the current FDOT specifications. Ten different aggregate sources were selected for use in this study. They included two control aggregates which met the 2010 FDOT aggregate specifications and eight aggregates which did not meet at least one of the aggregate specification requirements. The required aggregate properties which were not met included the maximum allowable LA abrasion loss, percent passing No. 200 sieve, and shell content. The properties of the aggregate evaluated include (1) gradation, (2) materials passing No. 200 sieve, (3) specific gravity, (4) water absorption, (5) LA abrasion loss, (6) Micro-Deval abrasion loss, (7) sodium sulfate soundness loss, (8) shell content, and (9) unit weight.

The concrete mixes were proportioned using Shilstone and American Concrete Institute (ACI) mix design methods. In the first experimental design, the concrete mixes were proportioned to have water/cement (w/c) ratios of 0.5 and 0.6, with cement contents of 600 and 545 lb/yd^3, respectively. The concrete mixes in the second experimental design had water/cement ratios of 0.6 and 0.7, with cement contents of 470 and 403 lb/yd^3, respectively. The properties of the fresh concrete evaluated include: (1) slump, (2) unit weight, (3) air content, and (4) temperature. The properties of the hardened concrete evaluated include: (1) compressive strength, (2) elastic modulus, (3) splitting tensile strength, (4) flexural strength, (5) drying shrinkage, (6) coefficient of thermal expansion, and (7) water absorption.

Analyses of the test results from the laboratory study include regression analyses to relate (1) different aggregate properties to one another, (2) different properties of hardened concrete to one another, and (3) different aggregate properties to the compressive strength of concrete.

Summary of Findings from the Laboratory Study

The main findings from the laboratory study are summarized as follows:

(1) The workability and air content of the concrete mixes using the nonstandard aggregates were similar to those using the standard aggregates, though most of the non-standard aggregates had higher percentages of material passing the No. 8, No. 4, and No. 200 sieves. This was attributed partly to the use of the Shilstone mix design method, which allowed for optimization of the aggregate gradation.

(2) Though the non-standard aggregates had relatively higher water absorption than the standard aggregates, this difference did not affect the control of the fresh concrete

properties. This was possibly helped by the use of a 45-minute soak time for the aggregates during concrete mixing.

(3) LA abrasion loss, Micro-Deval abrasion loss, sodium sulfate soundness loss, and percent passing No. 200 sieve of the aggregates were found to be positively correlated with one another. However, shell content did not correlate well with the other aggregate properties.

(4) The forms of the regression equation used by ACI were found to work well in interrelating the compressive strength, splitting tensile strength, and modulus of rupture of the concrete using non-standard aggregate. These developed regression equations are presented in this report.

(5) In spite of the high water cement ratios and low cement contents, most of the concrete mixes made with the non-standard aggregates were found to have compressive strength exceeding 2,500 psi, which is the minimum required strength for non-structural concrete applications according to the current FDOT specifications.

(6) For a fixed water/cement ratio of 0.6 or 0.7, the compressive strength of concrete was found to correlate well with LA abrasion loss, Micro-Deval abrasion loss, and sodium sulfate soundness loss. Thus, back prediction models were developed to determine the required values for these aggregate properties to achieve certain required compressive strength of concrete. These back prediction models were used to develop the recommended tiered aggregate specifications.

Production of Nonstandard Aggregates in Florida

Some aggregate mines in Florida are already producing nonstandard aggregates which are used in non-FDOT concrete projects. The nonstandard aggregates used in this study were obtained from some of these mines. An effort was made to collect information on the total amount of nonstandard aggregates which are being produced in Florida and the cost of these aggregates. Questionnaires (see Appendix G) were sent out to all the known aggregate producers in Florida to collect this information. Unfortunately, the aggregate producers were not ready to disclose this information at this time.

Recommendations

Based on the results from this study, a three-tier developmental aggregate specification (Table 6-14) is recommended for use by FDOT. The first tier (Category A) aggregate would have the same requirements as the current FDOT aggregate specifications, with maximum allowable LA abrasion loss of 45%, sodium sulfate soundness loss of 12%, and materials passing No. 200 sieve of 3.75% (at point of use). The second tier (Category B) aggregate would have slightly relaxed requirements, with maximum allowable LA abrasion loss of 50%, sodium sulfate soundness loss of 20%, and materials passing No. 200 sieve of 5.5% (at point of use). Possible uses for Category B aggregates include all non-structural concrete applications, precast drainage products, manholes, inlets, and junction boxes. The third tier (Category C) aggregate would have more relaxed requirements, with maximum allowable LA abrasion loss of 60%, sodium sulfate soundness loss of 30%, and materials passing No. 200 sieve of 8.2% (at point of use). Category C aggregate is recommended only for non-structural concrete applications. Results of analysis in this study indicate that Category B aggregates can possibly be used in concrete pavement application. It is recommended that further study be conducted to evaluate the suitability of using Category B aggregate for concrete pavement, parking lot and precast concrete pipe applications.

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CHAPTER 1
INTRODUCTION

1.1 Background and Research Need

At present, all limestone aggregates to be used in Florida Department of Transportation (FDOT) projects fall under a single category and must meet the same set of minimum durability requirements. These requirements include the following (FDOT, 2010):

Los Angeles (LA) abrasion loss	≤ 45%
Soundness (sodium sulfate) loss	≤ 12%
Flat or elongated pieces	≤ 10%
Deleterious substances:	
Coal and lignite	≤ 1.0%
Soft and friable particles	≤ 2.0%
Clay lumps	≤ 2.0%
(Soft and friable particles + clay lumps ≤ 3.0%)	
Plant root matter	≤ 0.005%
Wood and wood matter	≤ 0.005%
Cinders and clinkers	≤ 0.50%
Free shell	≤ 1.0%
Total material passing the No. 200 sieve:	
At source with LA abrasion less than or equal to 30	≤ 2.5%
At source with LA abrasion greater than 30	≤ 1.75%
At point of use	≤ 3.75%
Fine-grained organic matter	≤ 0.03%
Chert (less than 2.40 specific gravity SSD)	≤ 3.0%

Some exceptions to the above requirements are allowed on some non-limestone aggregates. For example, granite used in bituminous mixtures and treatments, cemented coquina rock used in bituminous mixture, air-cooled blast furnace slag not used in concrete, and reclaimed Portland cement concrete (PCC) are allowed to have a maximum Los Angeles (LA) abrasion loss of 50%. However, all limestone aggregates must meet the same set of minimum durability requirements as listed above. A limestone aggregate mine which could not produce aggregates with a maximum allowable LA abrasion loss of 45% would not be approved by FDOT.

Is the requirement of a maximum LA abrasion loss of 45% for limestone aggregates an absolute necessity to ensure performance of the aggregate regardless of the finished products the aggregates go into? The same question can be asked about other requirements, such as a maximum sodium sulfate soundness loss of 12% and maximum material passing the No. 200 sieve. Could it be more cost-effective to have tiered aggregate specifications, such that different requirements could be applicable depending on the finished products the aggregates are to be used in?

This research project was conducted to address the above questions. The main objective of this research is to investigate the effects of the key aggregate properties on the properties of hardened concrete, to evaluate the effects of relaxing a few key aggregate properties, and to recommend tiered aggregate specifications for use by FDOT. Implementing such a system of aggregate specification will induce the opening of new mines that do not currently meet FDOT specifications and would extend the reserves in existing mines that do. Existing mines may also benefit by adding a new production process dedicated to meeting the new tiered specifications.

1.2 Research Approach

In achieving the objectives of this research, the flowchart shown in Figure 1-1 was followed. An experimental design was also developed for the implementation of the laboratory testing. Figure 1-2 shows a schematic representation of the laboratory study.

1.3 Outline of the Report

Chapter 2 presents a literature review on the tiered aggregate specifications adopted by other highway and transportation agencies, the production and uses of aggregates in Florida, and the relationship between aggregate properties and concrete properties. Details of the experimental design used in the laboratory study, including description of the materials and test

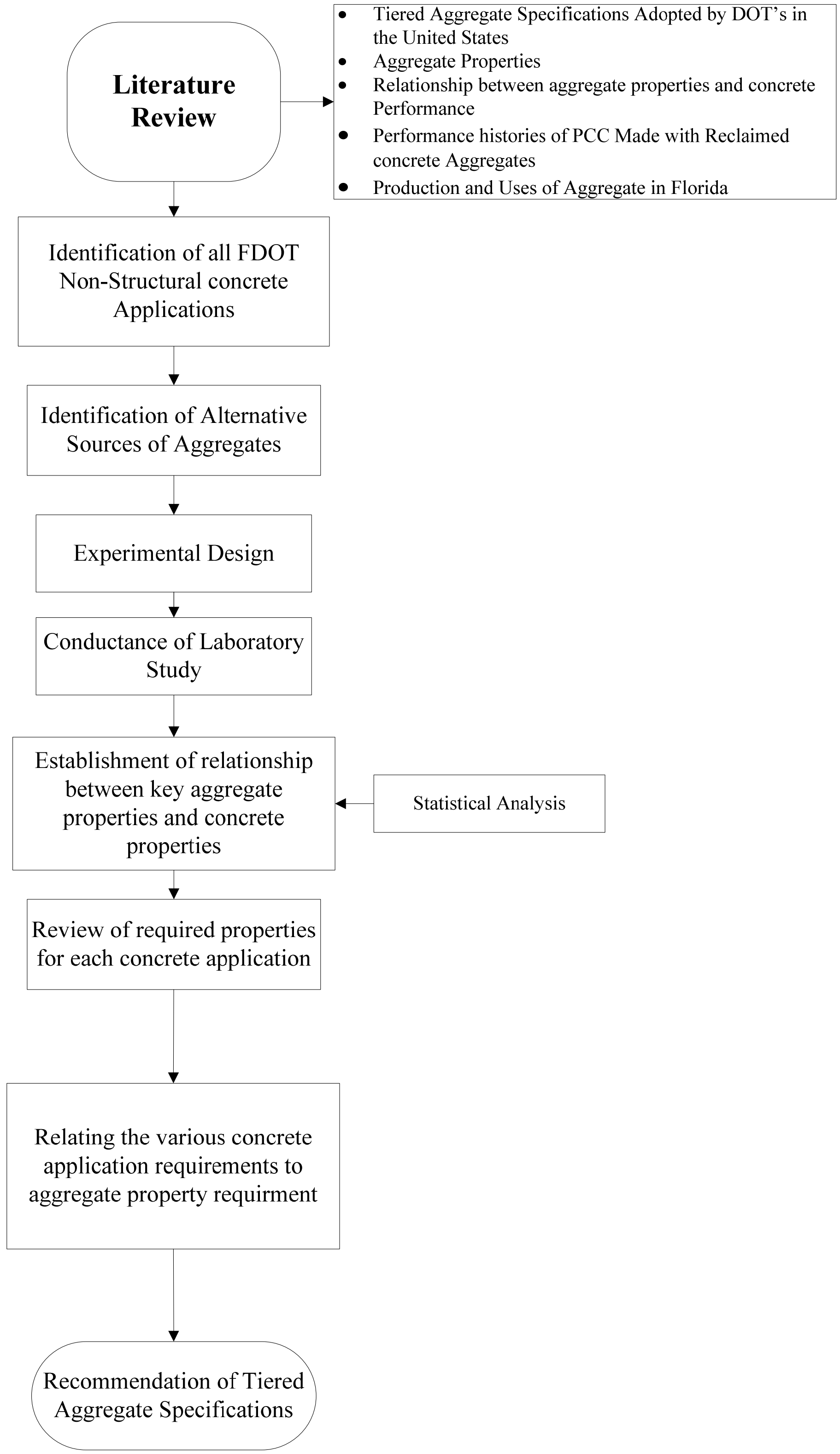


Figure 1-1. Flow chart of methodology.

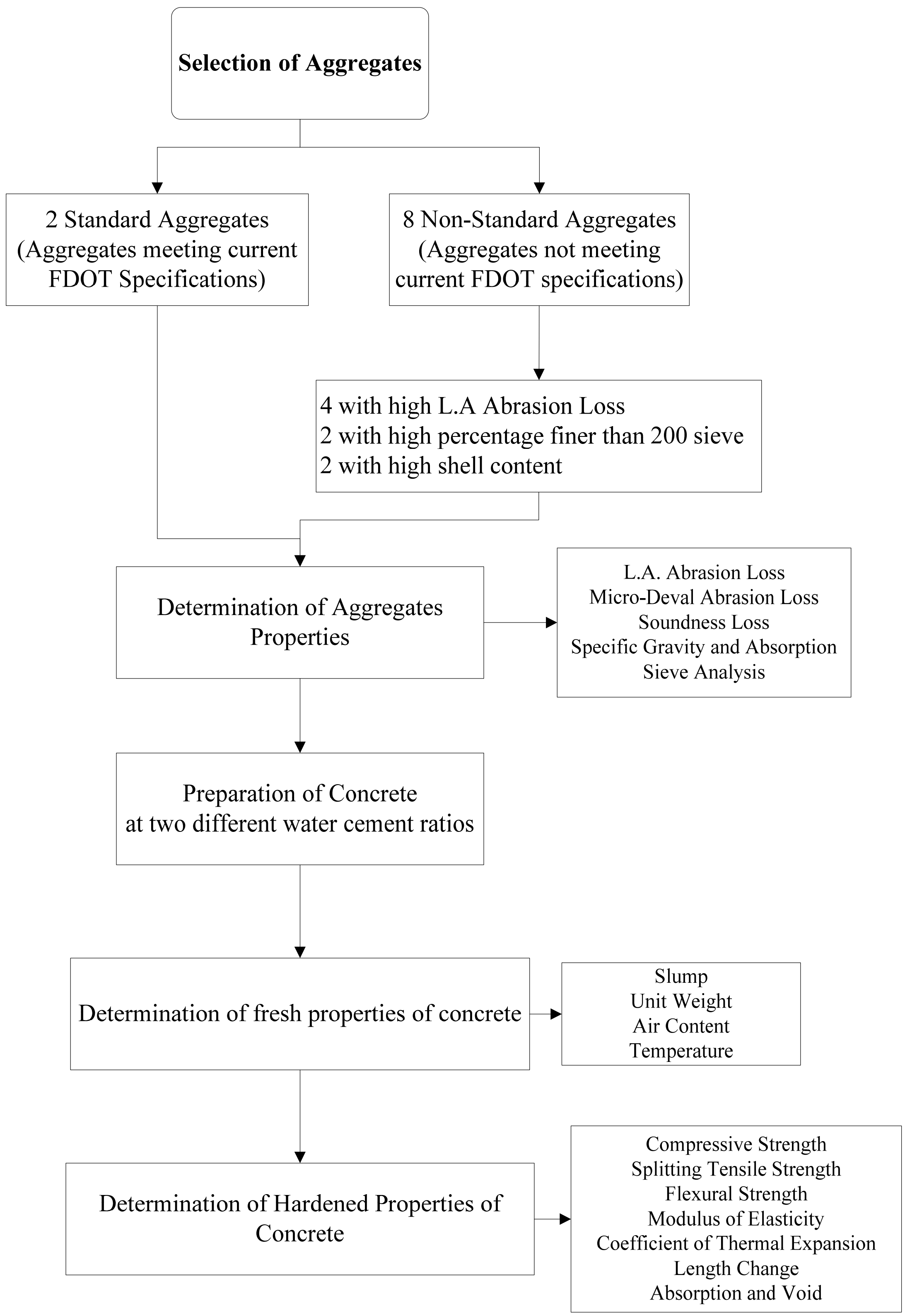


Figure 1-2. Flow chart of laboratory study.

methods used, are presented in Chapter 3. The results of tests on the concrete evaluated in this study and an analysis on the relationship between the various concrete properties are presented in Chapter 4. Statistical analyses were performed to determine the possible relationship between the various physical properties of aggregates and the compressive strength of concrete. The results of these analyses are presented in Chapter 5. Chapter 6 presents the development of the recommended tiered aggregate specifications for use by FDOT. Conclusions and recommendations from this study are presented in Chapter 7.

CHAPTER 2
LITERATURE REVIEW

2.1 Introduction

This chapter first presents a review of the development of tiered aggregate specifications adopted by other highway and transportation agencies. A review of aggregate properties, their relationship with concrete properties, and their effect on the performance of concrete is also presented. The chapter also gives a review of the performance histories of PCC made with reclaimed concrete aggregate (RCA) and the production and uses of aggregates in Florida.

2.2 Tiered Aggregate Specifications Adopted by DOTs in the United States

With the assistance of the aggregate section of the State Materials Office (SMO) of FDOT, a survey was sent to all 50 state departments of transportation (DOTs); 31 of them responded, and 18 of these indicated they had adopted a tiered aggregate specification. Appendix A shows the request sent and details of the responses of the various states. Based on those that responded in the affirmative, a detailed analysis was done to identify the states that specifically had a tiered aggregate specification for only concrete applications. The details of the various tiered aggregate specification of the responding states are presented below.

Indiana DOT (INDOT) indicated eight different classes of aggregates, AP, AS, A, B, C, D, E, and F with AP the highest classification, and Class F the lowest. Different classes are employed depending on the finished product. The maximum allowable LA abrasion loss for these various categories varies from 30% to 50%, while that for sodium sulfate soundness loss varies from 12% to 25%.

Illinois DOT (IDOT) has also adopted a tiered aggregate specification with four different coarse aggregate categories (namely, categories A, B, C and D) based on the quality of the aggregates. These four categories have LA abrasion loss limits ranging from 40% to 45%, sodium sulfate soundness loss from 15% to 25%, and percentage finer than No. 200 sieve from 1% to 2.5%. Aggregate gradation limits also vary depending on the application.

The California Department of Transportation (Caltrans) has a tiered aggregate specification based on aggregate gradation by virtue of relaxing its requirements for minor concrete works.

Georgia Department of Transportation (GDOT) has two groups of aggregates, i.e., Group 1 and Group 2, which are mainly based on their mineralogical compositions. The physical properties of the aggregates are tiered into two classes, i.e., Class A and Class B. For Portland cement concrete applications, the limits on LA abrasion for the two classes are given in Table 2-1. Also, the limit of soundness loss of 15% is waived for concrete to be used as a subbase in pavement applications.

Table 2-1. Georgia DOT Tier Aggregate Specification Based on LA Abrasion Loss

Aggregate Group	LA Abrasion Loss (%)	
	Class A	Class B
Group 1	0-40	41-55
Group 2	0-50	51-60

Kentucky Transportation Cabinet (KYTC) has a tiered aggregate specification based on soundness loss, shale content, LA abrasion loss and gradation. The maximum sodium sulfate soundness loss permitted for coarse aggregates for bridge decks, bridge deck overlays and bridge barrier walls is 9%, while for aggregates in all other concrete uses, the maximum allowed is 12%. The shale content limit for coarse aggregates to be used in bridge decks, bridge deck overlays and bridge barrier walls is 1.0%, while that permitted for all other concrete applications is 2.0%. For LA abrasion loss, the limits are set based on the material composition of the coarse aggregate. Specifically, the maximum allowable LA abrasion loss for all aggregates, with the exception of slag and sandstone is 40%; for sandstone alone, it is 50%; and for slag alone, it is 60%. The gradation requirement is generally one tier; however, the department allows the use of aggregates not meeting the requirement with the contractor paying a penalty through payment reduction.

Missouri Department of Transportation (MoDOT) has adopted a tiered aggregate specification for coarse aggregates to be used in concrete. The specification is mainly based on the source of the aggregates. For crushed stone obtained from rock of uniform quality, Table 2-2 shows the required specification; similarly, Table 2-3 shows the specification for gravel. Also, two tiers on gradation requirements are shown in Table 2-4.

Table 2-2. MoDOT Specification for Crushed Stone

Property	Value (%)
LA Abrasion, max. percent loss	50
Absorption, percent max.:	
Portland Cement Concrete Pavement	–
Portland Cement Concrete Masonry	3.5
Soundness, MoDOT Test Method T14, max. percent loss:	
Portland Cement Concrete Pavement	–
Portland Cement Concrete Masonry	18.0
Durability Factor, AASHTO[a] T 161 Procedure B, min. percent:	
Portland Cement Concrete Pavement Portland Cement Concrete Masonry	75[b]

[a] American Association of State Highway and Transportation Officials.
[b] Approval will be based on maximum aggregate size produced that meets durability requirements.

Table 2-3. MoDOT Specification for Gravel

Property	Value (%)
LA Abrasion, max. percent loss	45
Absorption, max percent	4.5
Soundness, MoDOT Test Method T14, max percent loss	18.0

Table 2-4. MoDOT Coarse Aggregate Gradation Requirements for Concrete Structures

Gradation D	Percent by Weight (%)
Passing 1-inch sieve	100
Passing 3/4-inch sieve	85-100
Passing 3/8-inch sieve	15-55
Passing No. 4 sieve	0-10

Gradation E	Percent by Weight (%)
Passing 3/4-inch sieve	100
Passing 1/2-inch sieve	70-100
Passing 3/8-inch sieve	40-70
Passing No. 4 sieve	0-10
Passing No. 8 sieve	0-6

Nebraska Department of Roads (NDOR) has also developed a tiered aggregate specification for coarse aggregates to be used in concrete structures. Their main tier is based on gradation. There are two coarse aggregate gradation requirements (i.e., E and F) and the various concrete specifications are linked to them.

New Jersey Department of Transportation (NJDOT) tiered specification for coarse aggregates used in concrete is mainly based on the LA abrasion loss. The LA requirement for concrete surfaces and bridge decks is 40% and for other concrete applications is 50%.

Ohio Department of Transportation (ODOT) has developed a tiered aggregate specification for coarse aggregates to be used in concrete. There are three different tiers (i.e., 15%, 12%, and 10%) for sodium sulfate soundness loss depending on the application of the aggregate. Similarly, the requirement for the amount of material passing No. 200 sieve is also based on the application and the type of aggregate. Table 2-5 shows a summary of the requirements for percent passing the No. 200 sieve size.

Table 2-5. ODOT Minus 200 Requirements

Material Type	Percent by Weight (%)	
	Superstructure	All Other Concrete
Crushed carbonate stone and crushed air-cooled blast furnace slag	3.4	3.8
Washed gravel	2.0	2.2

South Dakota Department of Transportation (SDDOT) tiered specification for aggregate to be used in concrete is based on whether the concrete in which it is to be used is structural or for incidental construction. The maximum sodium sulfate soundness loss for aggregates to be used for structural concrete is 10%, while for those to be used in incidental construction is 12%. There is also a tiered specification on gradation and amount of deleterious substances.

Texas Department of Transportation (TxDOT) allows different levels for minus No. 200 and soundness loss, for aggregate to be used in Portland cement concrete. For soundness loss criteria, the different levels are based on the environment in which the aggregate is intended for use, i.e., a freeze-thaw area or non-freeze-thaw area.

North Carolina Department of Transportation (NCDOT) has developed a tiered specification for coarse aggregate based on the required compressive strength of the concrete in which it is to be used. The general requirement for soundness loss is a maximum of 15%. However, when the compressive strength of the concrete in which the aggregate is to be used is more than 6000 psi, the limit for sodium sulfate soundness loss is set to 8%. Similarly, the maximum LA abrasion loss for all aggregates is 55%; however, when the concrete in which it is to be used has a compressive strength of more than 6000 psi, the limit on LA abrasion loss is set to 40%.

In summary, twelve (12) of the respondent states were determined to have tiered aggregate specifications for concrete use. Seven (7) states have a tiered specification on LA abrasion loss, eight (8) on soundness loss, three (3) on minus No. 200, and six (6) on gradation. Table 2-6 shows the different states and the physical properties used in their tiered coarse aggregate specifications for concrete.

Table 2-6. States with Tiered Aggregate Specifications on Coarse Aggregate for Concrete Use

Properties Specified	LA Abrasion Loss	Soundness Loss	Minus 200	Gradation
State	Georgia Indiana Kentucky New Jersey North Carolina Illinois Missouri	Georgia Indiana Kentucky North Carolina Illinois Texas Ohio South Dakota	Illinois Texas Ohio	Kentucky Illinois South Dakota California Missouri Nebraska

2.3 Aggregate Properties

Aggregates generally occupy 70 to 80% of the volume of concrete. Hence, their properties significantly influence the physical and mechanical properties of concrete.

Aggregate characteristics that may affect the performance of concrete include porosity, grading or size distribution, moisture absorption, shape and surface texture, crushing strength, elastic modulus, and types of deleterious substances present (Mehta and Monteiro, 2006, p. 253). Other properties of aggregates that are required for concrete mix design are bulk unit weight and specific gravity. "The essential requirement of an aggregate for concrete is that it remains stable within the concrete and in the particular environment throughout the design life of the concrete" (Smith and Collis, 2001).

"The durability of aggregates is also vital to the overall performance of concrete. The lack of durability of aggregate can be divided into physical and chemical causes. The former is concerned with susceptibility of aggregates to freezing and thawing or wetting and drying, as well as physical wear. Chemical durability problems are concerned with various forms of cement-aggregate reaction" (Mindess, Young and Darwin, 2003, p. 140).

2.3.1 Gradation Characterization

Gradation or particle size distribution of aggregates is an important characteristic because it determines the paste requirements for workable concrete. ASTM C 33 sets grading limits for fine and coarse aggregate based on practical experience; if an aggregate does not conform to the ASTM C33 grading limits, it does not necessarily mean that concrete cannot be made with the aggregate. It does mean that concrete may require more paste and be more liable to segregation during handling and placing (Mindess, Young and Darwin, 2003, p. 128). In 1907, Fuller and Thompson developed an equation to determine the maximum density gradation curve as shown in Equation 2.1. A general form of this equation was developed by Andreasen and Andersen (1930) and is shown in Equation 2.2.

$$\% P = \left(\frac{d}{D}\right)^{\frac{1}{2}} \times 100 \tag{2.1}$$

$$\% P = \left(\frac{d}{D}\right)^{q} \times 100 \tag{2.2}$$

where $\% P$ = percentage passing the sieve;

d = size of sieve;

D = maximum aggregate size; and

q = parameter which adjusts curve for fineness or coarseness (it lies between 0 and 1).

Many modifications have since been made to this equation; Shilstone (1990), Quiroga and Fowler (2003) suggested that the optimum value of q is 0.45. Many other mathematical models based on empirical measurements have been developed to compute packing density. In the early 1960s, the Federal Highway Administration (FHWA) introduced the standard gradation graph used in the Hot Mix Asphalt (HMA) industry today. This graph uses $q = 0.45$ and is convenient for determining the maximum density line and adjusting gradation (Roberts et al., 1996).

The 0.45 power chart is similar to a semi-log graph (See Figure 2-1). The x-axis contains the sieve size plotted on a 0.45 power scale, and the y-axis contains the percent of aggregates passing a given sieve. According to this method, a well-graded aggregate combination, i.e., the grading with the least amount of voids is defined by a straight line. Koehler and Fowler (2007) used a modified 0.45 power chart for sands with high microfine content for optimizing self-consolidating concrete mixtures. The difference between the modified 0.45 chart and the conventional 0.45 power is that the modified 0.45 power chart does not take into account microfines as part of the aggregate gradation (microfines are considered part of the paste portion). Deviations from the 0.45 power line help identify the location of grading problems. "Zigzags" across the line are undesirable. Gap-graded aggregate combinations will form an S-shape curve deviating from the optimum. Figure 2-1 shows an example of a gap-graded aggregate.

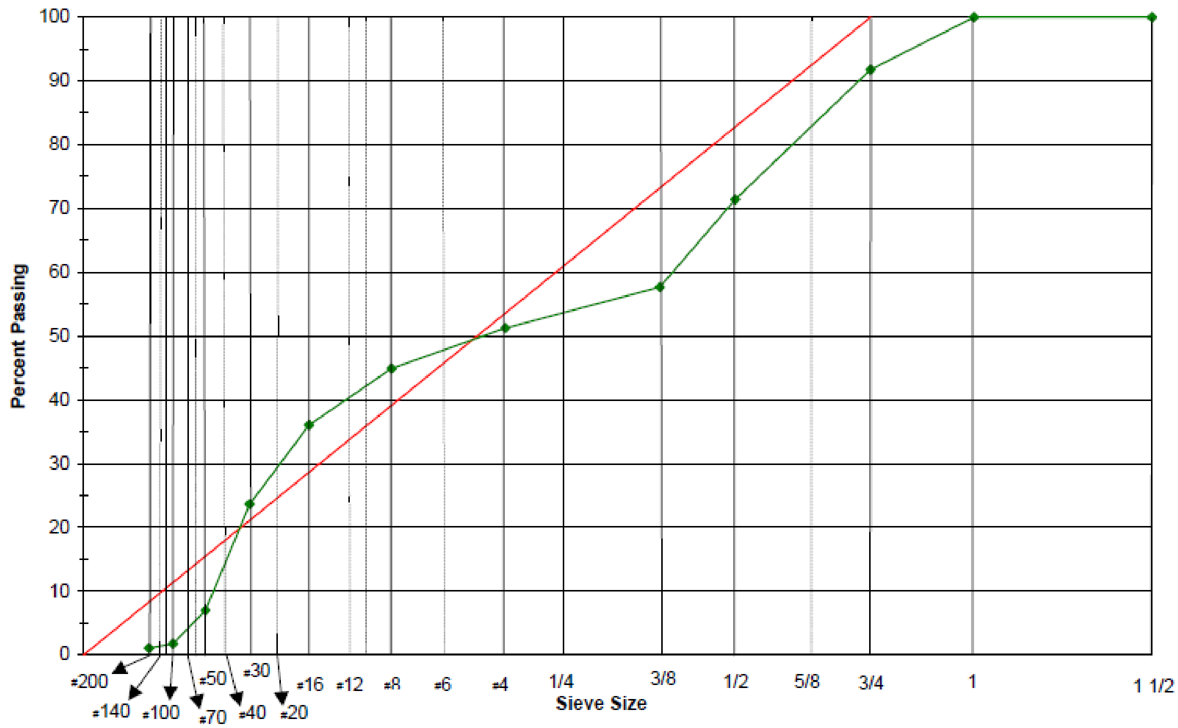


Figure 2-1. Aggregate gradation plot with sieve size on 0.45 power scale.

The percent-retained chart has also evolved from efforts to limit disproportionate amounts of materials retained on any one sieve. The percent-retained chart plots the mathematically combined percent retained for each sieve on a chart having percent retained on the y-axis and sieve sizes on the x-axis. Figure 2-2 shows a typical plot for well-graded aggregate. A well-graded aggregate combination will have no significant peaks and/or dips. A gap-graded aggregate combination will have significant peaks and dips (Figure 2-3). Shilstone (1990) recommends that the sum of the percent retained on two consecutive sieves should be at least 13% to be an optimum gradation.

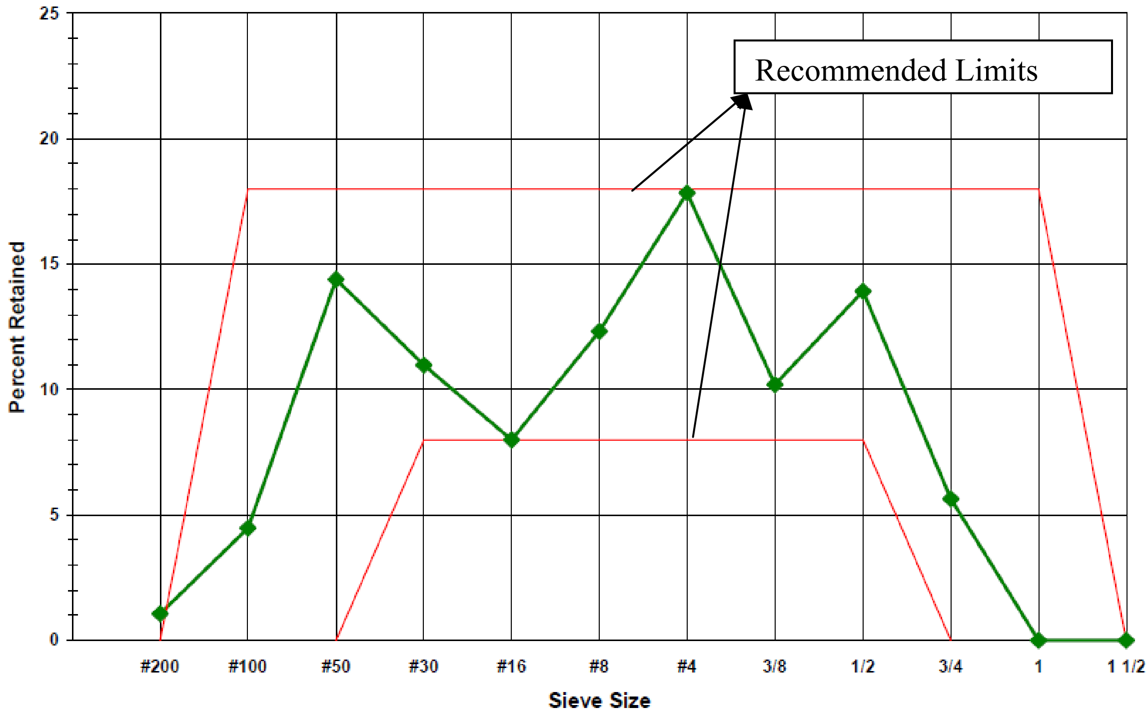


Figure 2-2. Aggregate gradation of a well-graded aggregate.

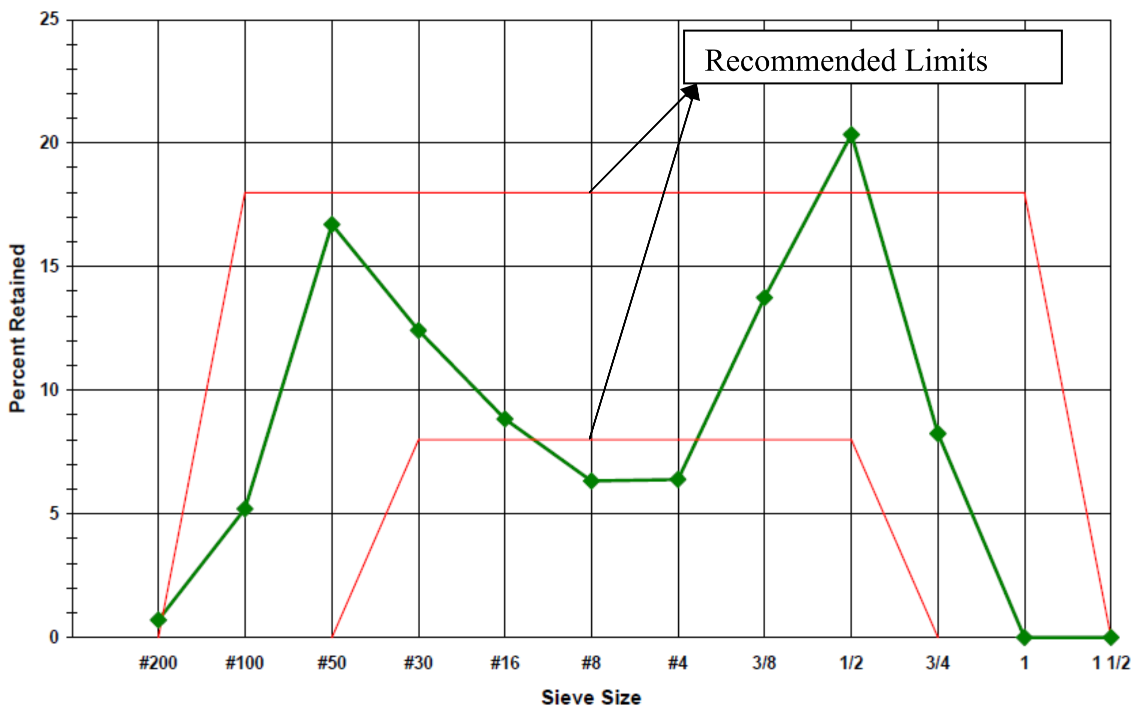


Figure 2-3. Aggregate gradation of a gap-graded aggregate.

Shilstone introduced two factors derived from the aggregate gradation to predict the workability of the concrete mix, i.e., the coarseness factor (CF) and the "workability factor (W)." The coarseness factor, or CF, is the proportion of the plus 3/8 in coarse particles (Q) in relation to the total coarse particles (Q+I), expressed as a percent, as follows:

$$CF = \left[\frac{Q}{Q+I} \right] \times 100 \tag{2.3}$$

A CF of 100 would represent a gap-graded aggregate where there was no #8 to 3/8″. material. A CF of zero would be an aggregate that has no material retained on the 3/8″ sieve.

The workability factor, or W, is the percentage of material passing the #8 sieve. It is also designated as adjusted workability factor (W-adj) to reflect the influence of the amount of cementitious material on workability. Shilstone and Shilstone (2002) showed the relationship, as seen in Figure 2-4, between CF, W (or W-adj), and the characteristics of the mix, such as

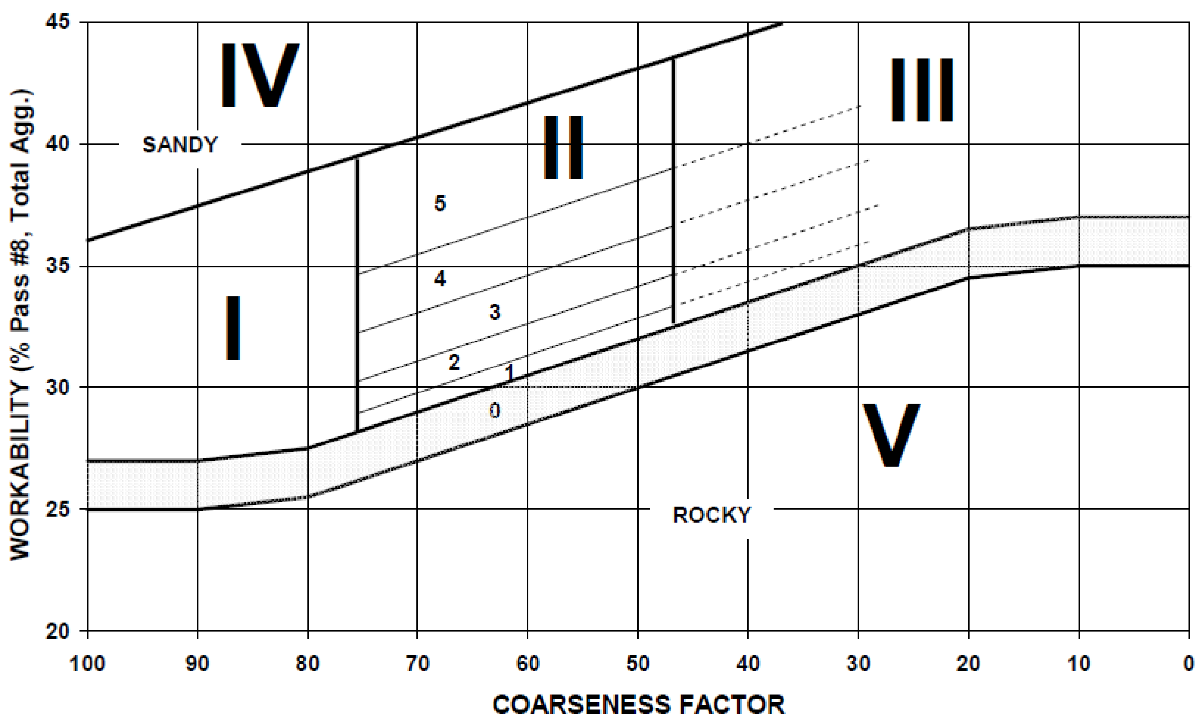


Figure 2-4. Workability-coarseness factor chart (Shilstone and Shilstone, 2002).

harshness, sandiness, excessive shrinkage, pumpability, finishing characteristics, degree of gap-grading, proneness to segregation, and so forth.

Shilstone and Shilstone (2002) included a trend bar which acts as a reference by which to judge a mixture. The diagonal trend bar defines a region where combined rounded or cube-shaped crushed stone and well-graded natural sand are in near perfect balance to fill voids with aggregate. Five zones are used to identify regions above the diagonal trend bar where variation in combined aggregate grading is indicative of certain general characteristics based upon the field experience. Zone I represents mixes that tend to be coarse and gap-graded. Zone II is the optimum zone for mixtures with nominal maximum aggregate size from 1-1/2″. Mixtures in this zone generally produce consistent, high-quality concrete. Zone II is divided into five areas: II-1 excellent but caution, II-2 excellent paving and slipform, II-3 high quality slab, II-4 good general, and II-5 varies to material and construction needs. Zone III represents 3/4″ and finer (pea gravel mixes). Zone IV contains excessive fines which are prone to high potential for segregation during consolidation and finishing. Such mixtures produce variable strength, have high permeability, and exhibit shrinkage, which generally contributes to the development of cracking, curling, spalling, and scaling. Zone V contains too much coarse aggregate that is non-plastic (may be suitable for mass concrete). An increase in fines content is necessary for zone V. In general, mixtures that plot close to the trend bar require close control of the aggregate gradation.

2.3.2 Absorption

Absorption is defined as the increase of mass due to presence of water in the pores of a material, but not including water adhering to the outside surface of a particle, expressed as a percentage of the dry mass (ASTM C127; ASTM C128). The absorption value may be regarded as an aggregate property that is a function of aggregate porosity and pore size (Yzenas, 2006). Water absorption is an indirect measure of the permeability of an aggregate, which in turn, can relate to other physical characteristics, such as mechanical strength, shrinkage, soundness, and to its general durability potential. These relationships are imprecise, although in general, less absorptive aggregates often tend to be more resistant to mechanical forces and to weathering. (Smith and Collis, 2001). A few state departments of transportation, i.e., NJDOT, specify a limit for aggregate absorption.

2.3.3 Specific Gravity

Specific gravity is the ratio of the mass of a given volume of aggregate to the mass of an equal volume of water (National Stone Association, 1991, The Aggregate Handbook, p. 3-8 to 3-9). AASHTO T 85 and ASTM C127 elaborate on the standard test method and procedure for determining the specific gravity and absorption of coarse aggregates while AASHTO T 84 and ASTM C128 are used for the determination of the specific gravity and absorption of fine aggregates. An aggregate with a higher specific gravity of 3.00 is not necessarily better than one with a relatively lower specific gravity of 2.55. However, deleterious particles that are sometimes present in an aggregate are often lighter than the good aggregate. Hence, separation of good from bad particles frequently can be accomplished utilizing this difference in specific gravity (NSA, 1991, The Aggregate Handbook, p. 3-9).

2.3.4 Shell Content

Calcareous shell debris is present in aggregates from marine and coastal land deposits, occurring as broken, whole flat, or whole hollow shells. The primary effect of plate shell fragments in aggregate is to increase the water demand to maintain a given concrete workability.

(Smith and Collis, 2001). Complete hollow shells may render concrete frost susceptible when occurring near an exposed concrete surface (Shirley, 1981).

2.3.5 Aggregate Durability

The overall stability of concrete aggregates may be defined as the ability of individual particles to retain their integrity and not to suffer physical, mechanical or chemical changes to an extent which could adversely affect the properties or performance of concrete. Aggregate stability cannot, therefore, be divorced from concrete durability (Smith and Collis, 2001). The consideration of aggregate stability also depends upon the performance requirements for a concrete in the particular environment. The factors which govern aggregate stability are manifold, but mainly depend on the geological nature and origin of the parent rock.

The mechanical properties of aggregates desirable for concrete should be such that the aggregates do not disintegrate or degrade during handling, transportation, concrete mixing or compaction and that the strength of the concrete in which they are used and subsequent performance are not compromised. In the United States, the major test adopted by most state departments of transportation to determine the mechanical properties of aggregate is the Los Angeles abrasion test. The Micro-Deval abrasion test is also currently used by some states. In the United Kingdom, the aggregate crushing value, aggregate abrasion value, and polished stone value are the major criteria employed.

2.3.5.1 Los Angeles abrasion test

The Los Angeles (LA) abrasion test measures the degradation of construction aggregates of standard grading resulting from a combination of actions including abrasion or attrition, impact, and grinding in a rotating steel drum containing a specified number of steel spheres (ASTM C131-06). It was adopted by ASTM in 1937 because it was felt that the LA abrasion test had a better relationship to field performance (Amirkhanian et al., 1991). Although, there was overwhelming research to show that LA loss is a poor indicator of field performance, the test is widely used as an indicator of the relative quality or competence of various sources of aggregate having similar mineral composition. Two reports have shown that LA loss has good correlations with the British impact value test, and thus, the LA abrasion test should be considered an impact test (Hudec, 1983; Senior and Rogers, 1991). Figure 2-5 shows the correlation between these two tests as shown by Senior and Rogers. As a result, the name of the test was later changed to Los Angeles abrasion and impact test.

2.3.5.2 Micro-Deval test

The Micro-Deval test is a measure of abrasion resistance and durability of mineral aggregates resulting from a combination of actions including abrasion and grinding with steel balls in the presence of water AASHTO T 327, ASTM) D 6928. The Deval test was developed in France in the 1870's to evaluate aggregates to be used for roads, and

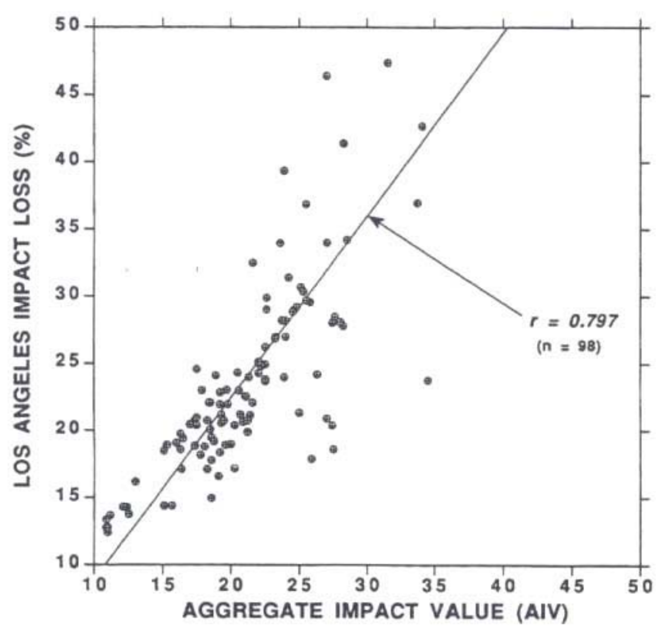


Figure 2-5. Correlation between aggregate impact value and Los Angeles abrasion loss as found by Senior and Rogers.

it was initially adopted by ASTM in 1908 (Amirkhanian et al., 1991). However, the test was abandoned for years by most for all purposes except railroad ballast because it had poor correlations with field performance (Rogers et al., 1991). The Micro-Deval test was adapted from the Deval test in the 1960s in France (Hanna et al., 2003) and was first introduced to North America in Quebec. It was developed to evaluate the wet mechanical strength and abrasion resistance of aggregates (Rogers et al., 1991). The Ontario Ministry of Transportation (OMT) conducted extensive research to refine and characterize the test throughout the 1990's. Research on the Micro-Deval began in the United States in the late 1990s and continues today. In the United States, the standard test method for the resistance of coarse aggregates to degradation by abrasion in the Micro-Deval apparatus is either AASHTO T 327 or ASTM D 6928.

The Micro-Deval test has been reported by several agencies as a good indicator of field performance (Richard and Scarlett 1997). Others have found results that show the Micro-Deval as having poor or mixed correlations with field performance. Early research has shown that the Micro-Deval test can, in fact, successfully determine aggregate performance, but differing specification limits have been used. Richard and Scarlett (1997) reviewed and evaluated the Micro-Deval test and compared the method with the Los Angeles abrasion test and sodium sulfate soundness and water absorption test. The LA abrasion test, was useful in identifying brittle materials which tend to degrade under impact but did not adequately measure inter-particle friction which was generated in cyclical loading. Secondly, as the LA abrasion test was conducted in dry conditions, it usually failed to identify materials which were prone to degrade in a wet condition, for example, materials with argillaceous, schist or shaley particles. Additionally, the LA abrasion test was found to have poor correlation with field performance of marginal granular base coarse aggregates. The Micro-Deval test procedure uses coarse aggregates soaked in water and rotated in a steel jar. As compared with the LA abrasion test, the Micro-Deval test was more effective at separating good from poor granular base aggregates. The Micro-Deval test also had good repeatability and good multi-laboratory precision. Durability and resistance to weathering of construction aggregates were normally evaluated by the sodium sulfate soundness and water absorption tests. However, the sodium sulfate soundness test was believed to suffer from the following disadvantages: lengthy and time consuming; poor multi-laboratory precision; poor repeatability; and inadequate correlation with field performance. The Micro-Deval test had a good correlation with the sodium sulfate soundness test. It required only a fraction of the time for a sodium sulfate soundness test, and had much better multi-laboratory precision.

Cooley et al. (2003) evaluated the toughness/durability of seventy-two aggregates from eight states with respect to their Micro-Deval test results. At the same time, the LA abrasion and sodium sulfate soundness values of each aggregate were also obtained for comparison purposes.

It was found that there was generally no relationship between both the LA abrasion and sodium sulfate soundness test results and the Micro-Deval test results either for an individual state's data or as a whole. For the Florida aggregate sources, the Micro-Deval test showed scattered values for good, fair and poor performers. Therefore, the Micro-Deval test was not considered a good method to distinguish between Florida aggregate sources with different performance histories. Specifications developed for the Micro-Deval test method may need to be based upon the parent aggregate type, rather than on comparisons between various aggregate types.

2.3.4.3 Soundness Test

An aggregate is considered *unsound* when the volume changes in aggregate induced by weather (e.g., alternate cycles of wetting and drying, or freezing and thawing) result in the deterioration of concrete. Unsoundness is shown generally by rocks having a certain characteristic pore structure (Mehta and Monteiro, 2006, p. 270). AASHTO T104 is the most widely used test method and procedure for determining the soundness of aggregates. This test method covers the determination of the resistance to disintegration by saturated solutions of sodium sulfate or magnesium sulfate. This test method furnishes information helpful in judging the soundness of aggregates subject to weathering action, particularly when adequate information is not available from service records of material exposed to actual weathering conditions. Concretes containing some cherts, shales, limestones, and sandstones have been found susceptible to damage by frost action or by salt crystallization within the aggregate particle. Although high moisture absorption is often used as an index for unsoundness, many aggregates such as pumice and expanded clays can absorb large amounts of water but remain sound. Unsoundness is therefore related to pore size distribution rather than to the total porosity of aggregate (Mehta and Monteiro, 2006, p. 270).

2.4 Relationship between Aggregate Properties and Concrete Performance

The relationship between the mechanical properties of aggregates and their subsequent effects on the performance of concrete are generally uncertain (Bloem and Gaynor, 1963).

In the early development study on the LA abrasion test, Woolf (1937) recommended different maximum LA abrasion loss for different applications. Woolf recommended a maximum LA abrasion loss of 50%, 40%, and 40%, respectively, for coarse aggregates to be used in concrete, asphalt surfacing, and surface treatments. It has been reported that the LA abrasion loss is related to the strength of the aggregate (Kilic et al., 2008). The strength of the aggregates generally increases with decreasing LA abrasion loss. Figure 2-6 shows the relationship between the LA abrasion loss and the compressive strength of the rock from Kilic's study. Since the strength of the aggregate can affect the strength of the concrete, the LA abrasion loss can indirectly affect the strength of a concrete. Figure 2-7 shows the relationship between the compressive strength of concrete and the compressive strength of the aggregate rock.

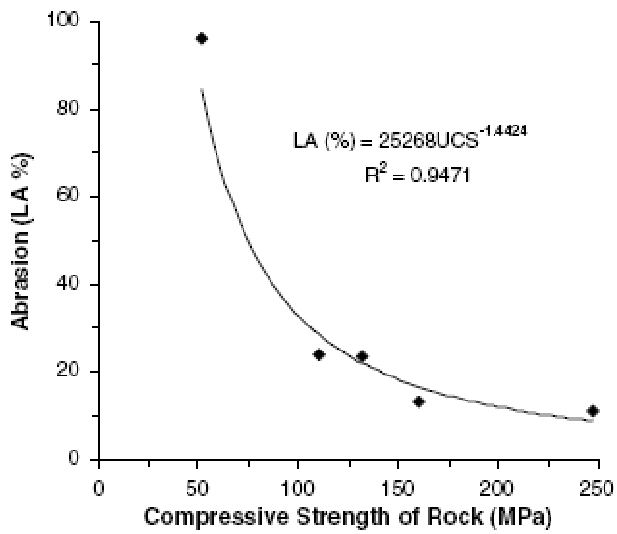


Figure 2-6. Relationship between LA abrasion loss and compressive strength of aggregate (Kilic et al., 2008).

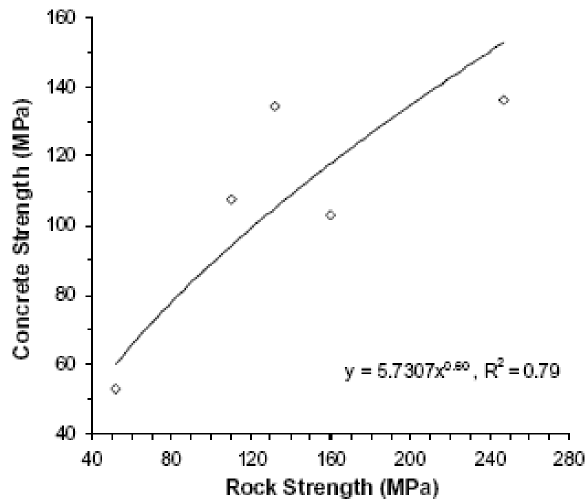


Figure 2-7. Relationship between compressive strength of concrete and compressive strength of aggregate rock (Kilic et al., 2008).

2.5 Performance Histories of Portland Cement Concrete (PCC) Made with Reclaimed Concrete Aggregates

Cho and Yeo (2004) compared the properties of recycled aggregates and natural aggregates. The recycled aggregates were reported to have lower specific gravity, higher water absorption, higher creep, and more drying shrinkage. The absorption problem can be solved by soaking aggregates in water before entering the batch plant. High shrinkage characteristics, however, will cause durability problems and cracking under harsh environment. Concrete made of waste aggregates showed lower indirect tensile strength, flexural strength, compressive strength, and Young's modulus by impact-echo test. Considering the lower strength and higher shrinkage, it was not recommended to use the waste aggregates in general concrete structures. However, it was suggested for use as a material in lean concrete due to the long-term strength stability. In addition, the use of waste aggregates is up to 73% more economical than using natural aggregates.

Similar research was conducted by Sagoe-Crentsil et al. (2001) but different results were found. A single source of commercially graded coarse recycled aggregate was compared with natural virgin coarse aggregate in terms of fresh concrete workability, compressive strength, splitting tensile strength, drying shrinkage, and abrasion resistance. It was found that the workability of recycled aggregate concrete was improved compared to natural basalt aggregate concrete. The 28-day compressive strengths of the concrete made with recycled aggregate had no significant difference from the natural aggregate concrete. Similarly, no statistically significant reduction in tensile strength was found in the recycled aggregate concrete and the natural concrete. However, recycled aggregate concrete displayed higher drying shrinkage values compared with the natural concrete, and the abrasion resistance was reduced by about 12%.

Xiao et al. (2005) investigated the compressive strength and stress-strain curves of concrete made with natural and recycled coarse aggregate (RCA) at different proportions. The compressive strengths decrease with increasing recycled aggregate percentages. Compared with normal concrete, the elastic modulus of RCA is smaller and the peak strain is higher.

Poon et al. (2004) investigated the performance of concrete made with natural and recycled aggregates at different proportions. The moisture states of the aggregates, oven-dry (OD), saturated surface-dried (SSD) and air-dried (AD), were found to impact the slump and compressive strength of the concrete made. OD aggregates led to a higher initial slump and quicker slump loss, while SSD and AD aggregates had normal initial slumps and slump losses. For the concrete mixtures incorporating with recycled concrete aggregates, the AD aggregate concretes exhibited the highest compressive strength. With the increase of recycled aggregate proportion, the strength of AD mixes almost remained unchanged, OD mixes increased in strength but the SSD mixes decreased in strength. The moisture states of the recycled aggregate influenced the strength development of the concrete negatively; therefore the use of the recycled aggregate in the SSD state is not preferable. Aggregates in the AD (as-received) state and containing not more than 50% RCA should be optimum for normal strength RCA concrete production.

Bekoe et al. (2010) evaluated the feasibility of using RCA in concrete pavement application. Concrete containing 0%, 25% and 50% of RCA were produced in the laboratory and their properties vital to the performance of concrete pavement were evaluated. Using the measured properties, a finite element analysis was performed to determine how the concrete containing the different amounts of RCA would perform if they were used in a typical concrete pavement in Florida. From the analysis, they concluded that the use of RCA up to about 50% will not adversely affect the performance of concrete pavement.

Tam et al. (2005) introduced a new approach to mixing concrete, the “Two-Stage Mixing Approach (TSMA),” to improve the compressive strength for concrete made with recycled concrete aggregate. The approach divided the normal mixing into two parts. The mixing water was divided into two portions which were added at two different times. Under the observation of scanning electron microscopy, the cracks within the recycled aggregates were filled after adopting TSMA due to further hydration. It was concluded the new approach was an effective method for enhancing the compressive strength.

2.6 Production and Uses of Aggregates in Florida

FDOT is the single largest consumer of aggregate materials in the state through its construction and maintenance programs. The United States Geological Survey (USGS) estimates that, for year 2010, Florida produced about 39.6 million tons of limestone. The breakdown of the different uses is as follows:

- Concrete aggregates 4.23 million tons
- Bituminous aggregate 4.090 million tons

- Roadstone and covering 3.910 million tons
- Riprap and railroad ballast 66 thousand tons
- Other construction uses 5.6 million tons
- Agriculture uses 631 thousand tons
- Other uses 20.7 million tons

Crushed stone in Florida is produced from limestone, which is mined or extracted from naturally occurring deposits found in 22 counties. Approximately 93% of crushed stone materials used by the road-building and construction industries in Florida are mined within the state. 43% of this total comes from an area known as “The Lake Belt” in Miami-Dade, Southeast Florida, because of the quality characteristics of the rock resource. Other sources of rock materials are imported domestically from Georgia, Alabama, and internationally from Mexico, Canada, and the Bahamas.

The Miami limestone formation found along the southeast coast in the Lake Belt Region of Miami-Dade County is the hardest and most durable geologic formation available in the state. In a 2007 study conducted for the FDOT by Lampl Herbert Consultants who investigated issues related to location and quality of the rock formations that are presently mined throughout Florida, the following issues, among others, were found:

- The quality of rock available outside the Lake Belt Region for many engineering applications is declining;
- Identified aggregate reserves in Florida do not appear adequate to produce 150 million tons per year for a 5- to 10-year growth period and beyond;
- Florida is heavily dependent on resources from one single area, namely, the Lake Belt Region of Miami-Dade County.

CHAPTER 3
MATERIALS AND TEST METHODS

3.1 Introduction

This section gives details of the aggregate properties, concrete mix proportion, and the properties of the fresh concrete used in this study. It also explains the standard method and fabrication procedure for the preparation of the concrete mixture in the laboratory and the standard testing methods performed in this research study.

3.2 Identification and Selection of Aggregates

Ten (10) different sources of aggregates were identified and used for this research. These sources were selected from mines that are currently producing aggregates that either meet or did not meet certain aspects of the 2010 FDOT standard aggregate specification. The mines were selected based on information provided by aggregate producers about the quality of their aggregates and further information provided by the project manager. Table 3-1 shows details of the sources of the aggregates, while Table 3-2 shows the properties of the aggregates initially provided by the aggregate producers.

Table 3-1. Sources of Aggregates

Aggregate I.D.	Aggregate Type	Mine #	Aggregate Source	Contractor	District
11	Standard	87089	Miami Oolite	Cemex	6
12	Standard	12260	Fort Myers	Vulcan	1
1A	Non-Standard	87089	Modified Miami Oolite	Cemex	6
1B	Non-Standard	12260	Modified Fort Myers	Vulcan	1
1C	Non-Standard	N/A	Inglis	Cemex	7
1D	Non-Standard	38228	Cabbage Grove	Martin Marietta	2
1E	Non-Standard	36696	Ocala	Steven Counts Inc.	5
1F	Non-Standard	01011	Punta Gorda	Coral Rock Inc.	1
1G	Non-Standard	N/A	Charlotte County	Weber South	1
1H	Non-Standard	08012	Brooksville	Cemex	7

Table 3-2. Initially Assumed Aggregate Properties

Aggregate I.D.	Mine #	Aggregate Source	Aggregate Properties
11	87089	Miami Oolite	Standard aggregate
12	12260	Fort Myers	Standard aggregate
1A	87089	Modified Miami Oolite	Miami Oolite with addition of pulverized fines to produce a total of 8% minus 200
1B	12260	Modified Fort Myers	Fort Myers with addition of pulverized fines to produce a total of 5% minus 200
1C	N/A	Inglis	High minus 200
1D	38228	Cabbage Grove	High LA abrasion loss
1E	36696	Ocala	High LA abrasion
1F	01011	Punta Gorda	High shell content
1G	N/A	Charlotte County	High shell content
1H	08012	Brooksville	High LA abrasion loss*

* *The researchers acknowledge the cooperation of Cemex in customizing a product to purposefully suit this project*

Of the ten identified aggregates, two (2) were intended to meet current FDOT standard specification while the remaining eight (8) were intended not to meet at least one of the general requirements on aggregates. Figure 3-1 shows a map of the different locations of the aggregates.

For the purpose of this report, the aggregates are mainly identified by their location. Two batches of aggregates were obtained from some of the sources and the different batches are also distinguished. Table 3-3 shows the nomenclature for the aggregates that will be referred to throughout this report.

The Modified Miami Oolite and Modified Fort Myers aggregates were artificially created by adding pulverized fines passing the No. 200 sieve to produce aggregates with a total percentage passing the No. 200 sieve of 5% and 8%, respectively. A pulverizer used to produce the fines is shown in Figure 3-2.

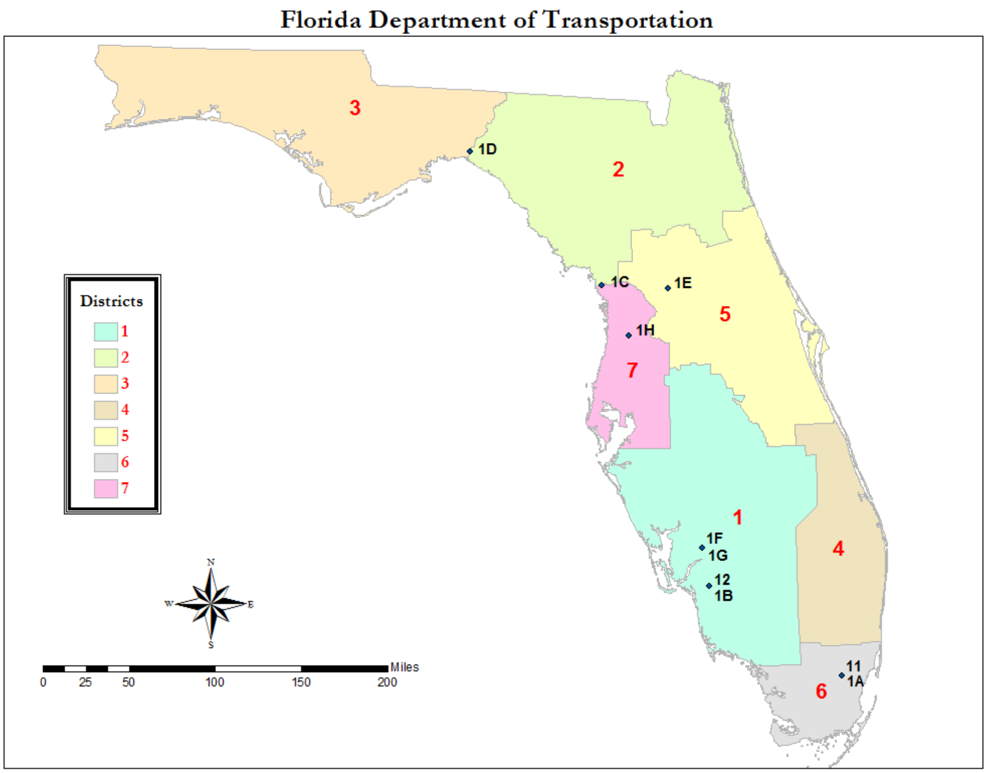


Figure 3-1. Aggregate locations.

Table 3-3. Nomenclature for the Aggregate Sources

Aggregate Source	Nomenclature
First Batch:	
Miami Oolite	Miami Oolite-1
Fort Myers	Fort Myers-1
Inglis	Inglis-1
Cabbage Grove	Cabbage Grove-1
Ocala	Ocala-1
Second Batch:	
Miami Oolite	Miami Oolite
Fort Myers	Fort Myers
Modified Miami Oolite	Modified Miami Oolite
Modified Fort Myers	Modified Fort Myers
Inglis	Inglis
Cabbage Grove	Cabbage Grove
Ocala	Ocala
Punta Gorda	Coral Rock
Charlotte County	Weber South
Brooksville	Brooksville

Figure 3-2. Pulverizer.

3.3 Aggregate Testing

3.3.1 Sampling of Aggregates

After the aggregates were acquired from the various producers, they were stored in bins shown in Figure 3-3. They were remixed with the aid of a loader to ensure that any form of

Figure 3-3. Storage bins for aggregates.

segregation that may have occurred during transportation was minimized. Adequate samples were subsequently taken and dried with an oven (refer to Figure 3-4) at a temperature of 230 ± 9°F for 24±2 hours. The materials were left to cool thereafter before any of the aggregate tests were conducted.

Figure 3-4. Oven.

3.3.2 Sieve Analysis

Sieve analysis was conducted on the coarse aggregates to determine the gradation in accordance with FM 1-T 027. This was done with the aid of a Gilson mechanical screen shaker shown in Figure 3-5. Three (3) different samples were prepared for every aggregate type and the representative gradation obtained from the average of the three (3). Tables 3-4 and 3-5 show the mean gradation for each of the aggregate sources and the standard gradation limit for #57 coarse aggregates as stipulated in Section 901-1.4 of the *2010 FDOT Standard Specification for*

Figure 3-5. Gilson mechanical screen shaker.

*Road and Bridge Cons*truction. Figures 3-6 to 3-8 show the gradation chart for all the aggregate sources. The gradation results for each of the aggregates are also shown in Tables B-1 to B-5 in Appendix B.

For the first batch of the aggregates, all the non-standard aggregates, i.e., Inglis-1, Cabbage Grove-1, Ocala-1 did not meet the requirement on #8 sieve size. Furthermore, Inglis did not meet the requirement on #4 sieve size. Similarly, on the second batch of aggregates, Modified Miami Oolite, Modified Fort Myers, Inglis, Cabbage Grove, Ocala Weber South and Brooksville did not meet the requirement on #8 sieve sizes. Inglis and Brooksville also did not meet the requirement on #4 sieve sizes. Weber South and Brooksville did not meet the requirement on 1/2″ sieve size while Ocala did not meet the requirement on 1″ sieve size.

In general, most of the non-standard aggregates did not meet the gradation requirement for #57 on the # 4 and # 8 sieve sizes.

Table 3-4. Gradation for First Batch of Aggregates

Sieve Size	Gradation Requirement	Aggregate Source				
		Miami Oolite-1	Fort Myers-1	Inglis-1	Cabbage Grove-1	Ocala-1
	Percentage Passing (%)					
1½″	100	100	100	100	100	100
1″	95-100	100	100	96	99	96
½″	25-60	35	30	48	44	36
#4	0-10	3	3	11	8	9
#8	0-5	2	2	8	7	8

Table 3-5. Gradation for Second Batch of Aggregates

Sieve Size	Gradation Requirement	Aggregate Source									
		Miami Oolite	Fort Myers	Modified Miami Oolite	Modified Fort Myers	Inglis	Cabbage Grove	Ocala	Weber South	Coral Rock	Brooksville
	Percentage Passing (%)										
1½″	100	100	100	100	100	100	100	100	100	100	100
1″	95-100	100	100	100	100	96	98	93	100	100	99
½″	25-60	59	30	60	35	48	35	31	66	43	71
#4	0-10	7	3	10	9	11	6	7	8	3	22
#8	0-5	3	2	6	9	8	6	6	6	2	17

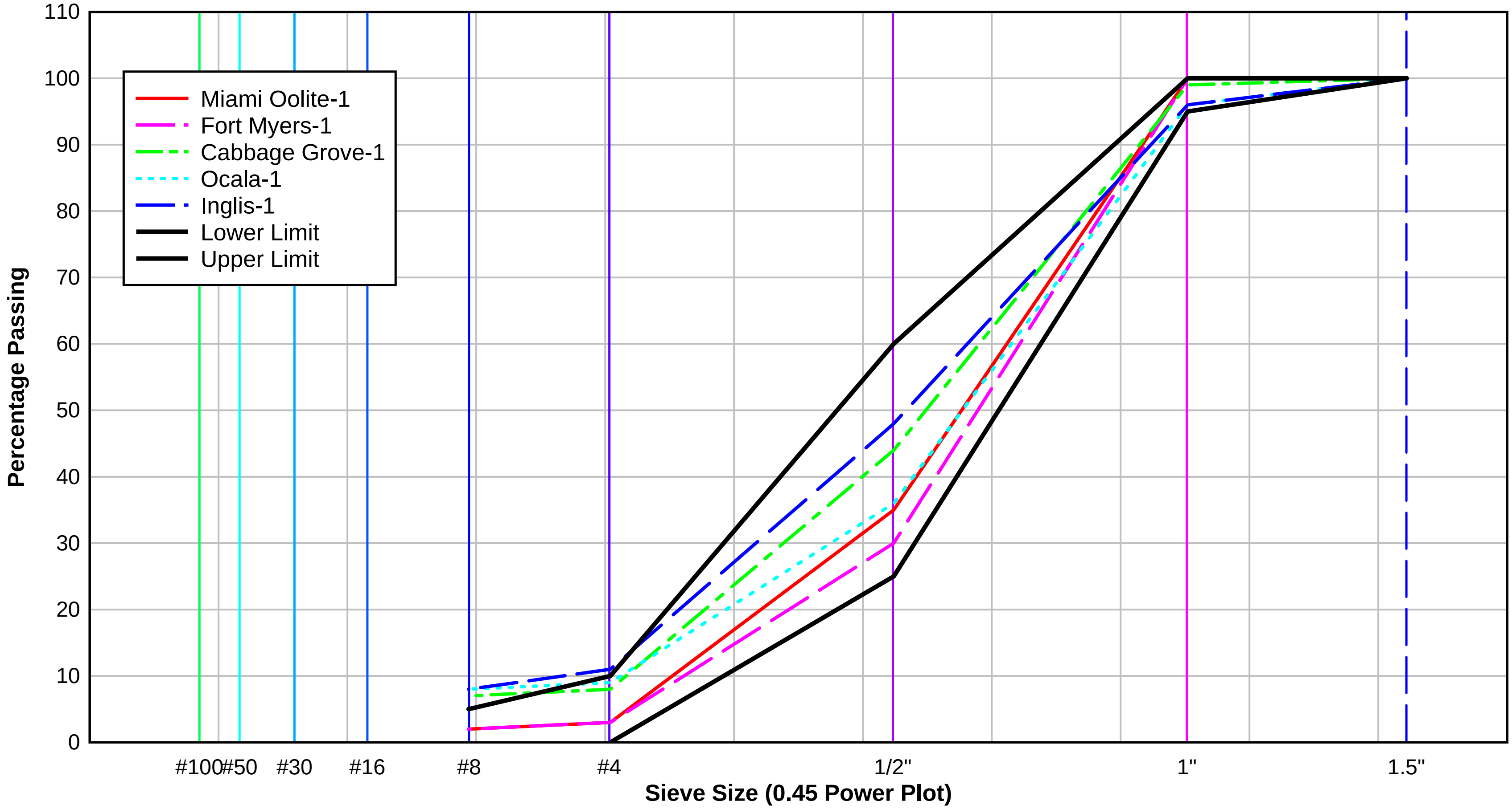


Figure 3-6. Gradation chart for all five aggregates in first batch.

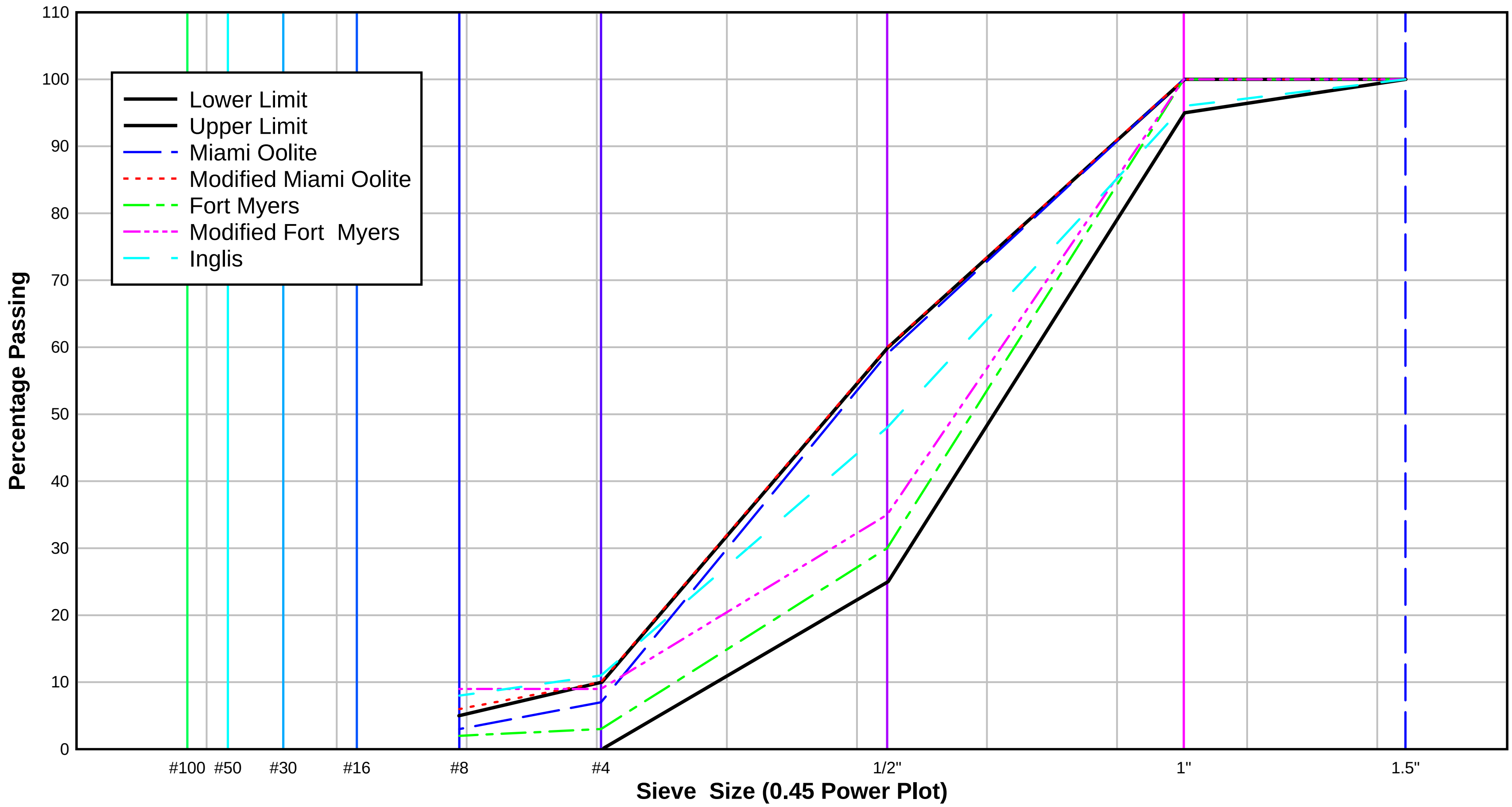


Figure 3-7. Gradation chart for first five aggregates in second batch.

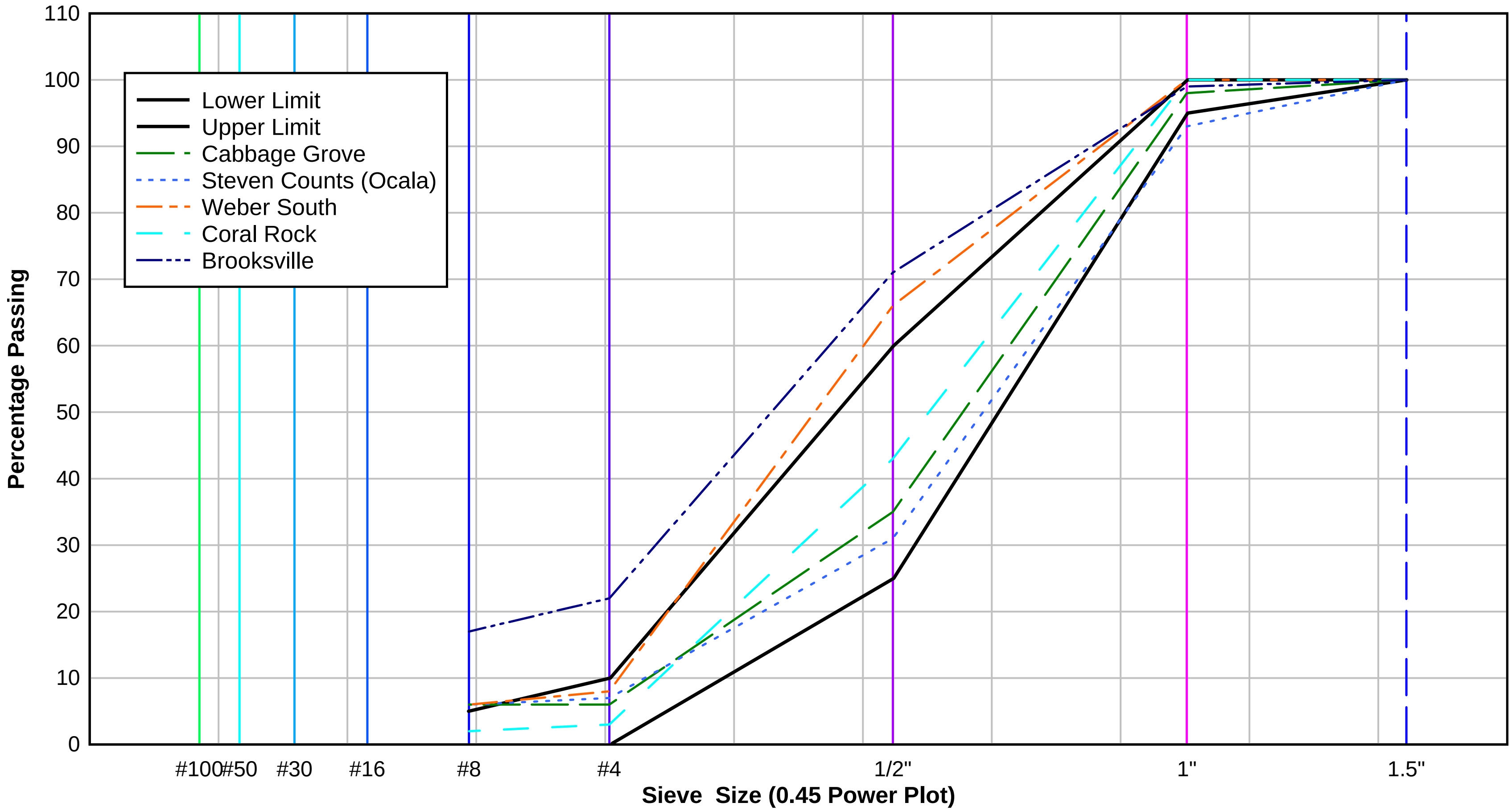


Figure 3-8. Gradation chart for remaining five aggregates in second batch.

3.3.3 Materials Finer than 75µm (No. 200) Sieve in Mineral Aggregate by Washing

The percentage of materials finer than 75µm (No. 200) sieve in the coarse aggregates was determined in accordance with FM 1-T011. Three samples were tested for each aggregate and Table 3-6 shows the mean value for each aggregate. Tables B-1 through B-5 in Appendix B show the individual values for each of the aggregates. Figure 3-9 shows a plot of the data in Table 3-6. Section 901-1.2 of the *2010 FDOT Standard Specification for Road and Bridge Construction* stipulates that the total material passing the No. 200 sieve at the point of use should be less than 3.75%. The researchers assume that the aggregate delivery to the SMO simulates delivery to a point of use such as a concrete plant. From Table 3-2, Ocala and Brookville were not intended to have a high LA abrasion loss, but the results show that their percentages passing the No. 200 sieve were also high.

Table 3-6. Percentage of Materials Finer than Minus 200 Sieve Size

Aggregate Source	Minus 200 (%)
Miami Oolite-1	1.77
Fort Myers-1	0.98
Inglis-1	3.98
Cabbage Grove-1	2.74
Ocala-1	3.56
Miami Oolite	2.20
Fort Myers	0.98
Modified Miami Oolite	5.00
Modified Fort Myers	8.00
Inglis	3.98
Cabbage Grove	3.36
Ocala	4.08
Weber South	2.74
Coral Rock	0.85
Brooksville	9.22

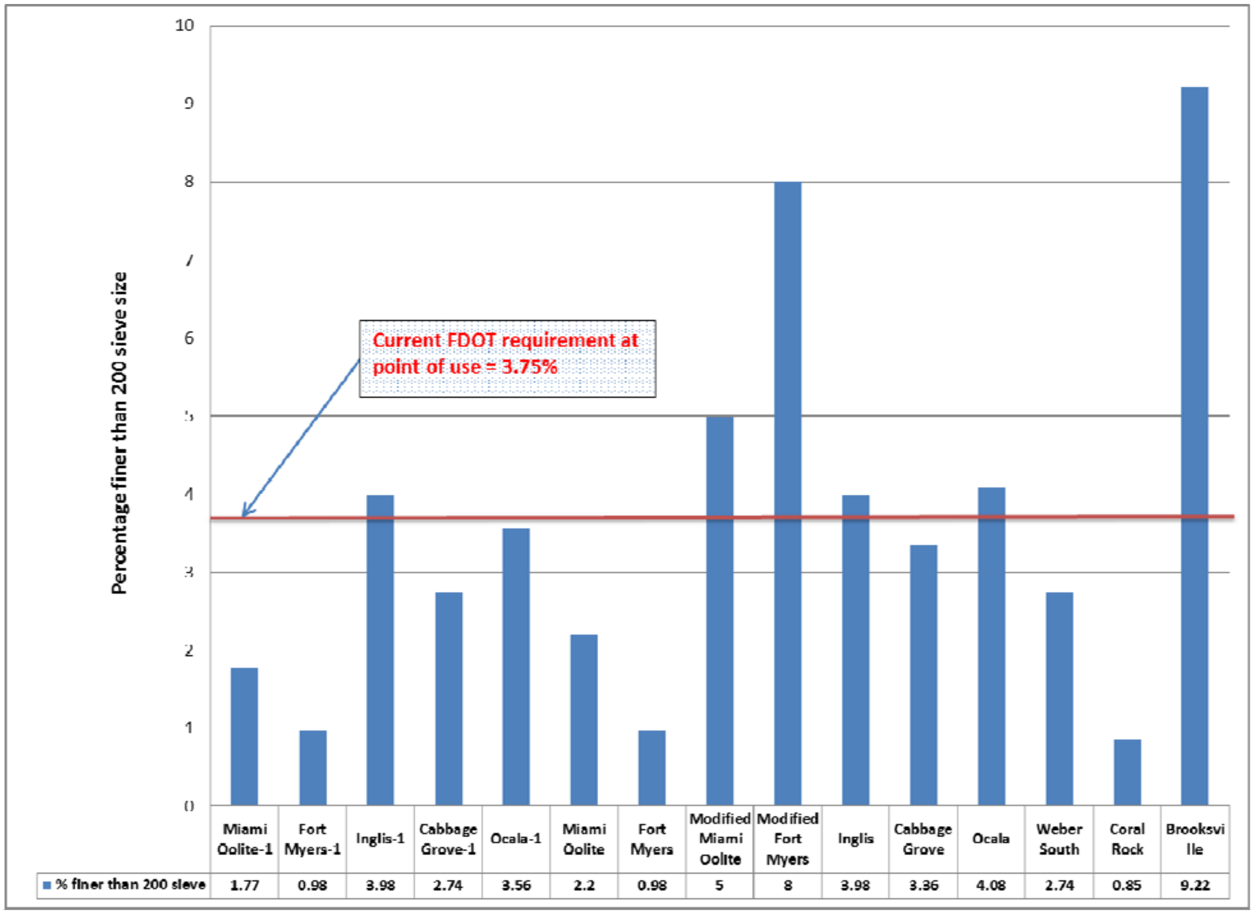


Figure 3-9. Plot of percentage finer than 200 sieve size.

3.3.4 Specific Gravity and Absorption

The specific gravity of the coarse aggregates was determined in accordance with FM 1 T-085. Three main specific gravities were determined: dry bulk specific gravity, SSD bulk specific gravity and apparent specific gravity. Three replicates were tested for each aggregate and the mean values presented in Table 3-7 and Table 3-8. Tables B-6 and B-7 in Appendix B show the individual values for each of the aggregates. Figures 3-10 and 3-11 show plots of the data in Table 3-7 and Table 3-8, respectively. From Figure 3-11, it may be observed that, comparatively, the non-standard aggregates have a higher absorption.

Table 3-7. Specific Gravities and Absorption of First Batch of Aggregates

Aggregate Source	Bulk Specific Gravity (Dry)	Bulk Specific Gravity (SSD)	Apparent Specific Gravity	Absorption (%)
Miami Oolite-1	2.320	2.404	2.534	3.65
Fort Myers-1	2.302	2.394	2.535	3.99
Inglis-1	2.354	2.445	2.589	3.85
Cabbage Grove-1	2.157	2.294	2.499	6.34
Ocala	2.171	2.321	2.554	6.90

Table 3-8. Specific Gravities and Absorption of Second Batch of Aggregates

Aggregate Source	Bulk Specific Gravity (Dry)	Bulk Specific Gravity (SSD)	Apparent Specific Gravity	Absorption (%)
Miami Oolite	2.35	2.45	2.63	4.54
Fort Myers	2.30	2.39	2.54	3.99
Inglis	2.35	2.45	2.59	3.85
Cabbage Grove	2.11	2.27	2.49	7.25
Ocala	2.12	2.29	2.54	7.80
Weber South	2.16	2.32	2.57	7.38
Coral Rock	2.23	2.34	2.52	5.16
Brooksville	1.82	2.08	2.46	14.38

3.3.5 Los Angeles Abrasion Loss

The resistance of the aggregates to degradation by abrasion and impact was determined in accordance with FM 1-T 096. Figure 3-12 shows the LA abrasion machine used in the experiment, while Figure 3-13 shows discharged aggregates from the abrasion machine. Three different tests were done on each aggregate, and Table 3-9 shows the result of the mean abrasion loss. Table B-8 shows the individual results for each aggregate. Figure 3-14 shows a plot of the data in Table 3-9. Section 901-1.3 of the *2010 FDOT Standard Specification for Road and Bridge Construction* stipulates that the maximum Los Angeles abrasion loss should be 45%. The result shows that Weber South, which was intended to be an aggregate with high shell content (refer to Table 3-2), also had a high percentage passing the No. 200 sieve size. From the plot in Figure 3-14, it may be observed that most of the non-standard aggregates do not meet the current FDOT specification.

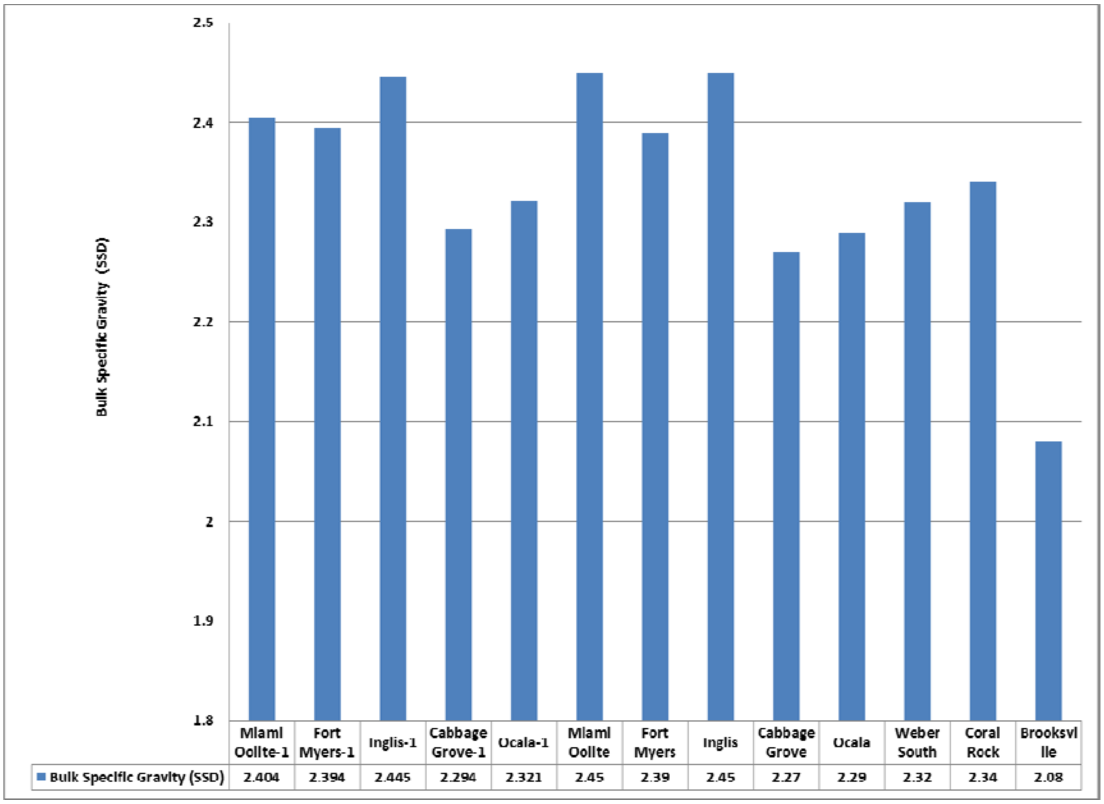


Figure 3-10. Plot of bulk specific gravity (SSD).

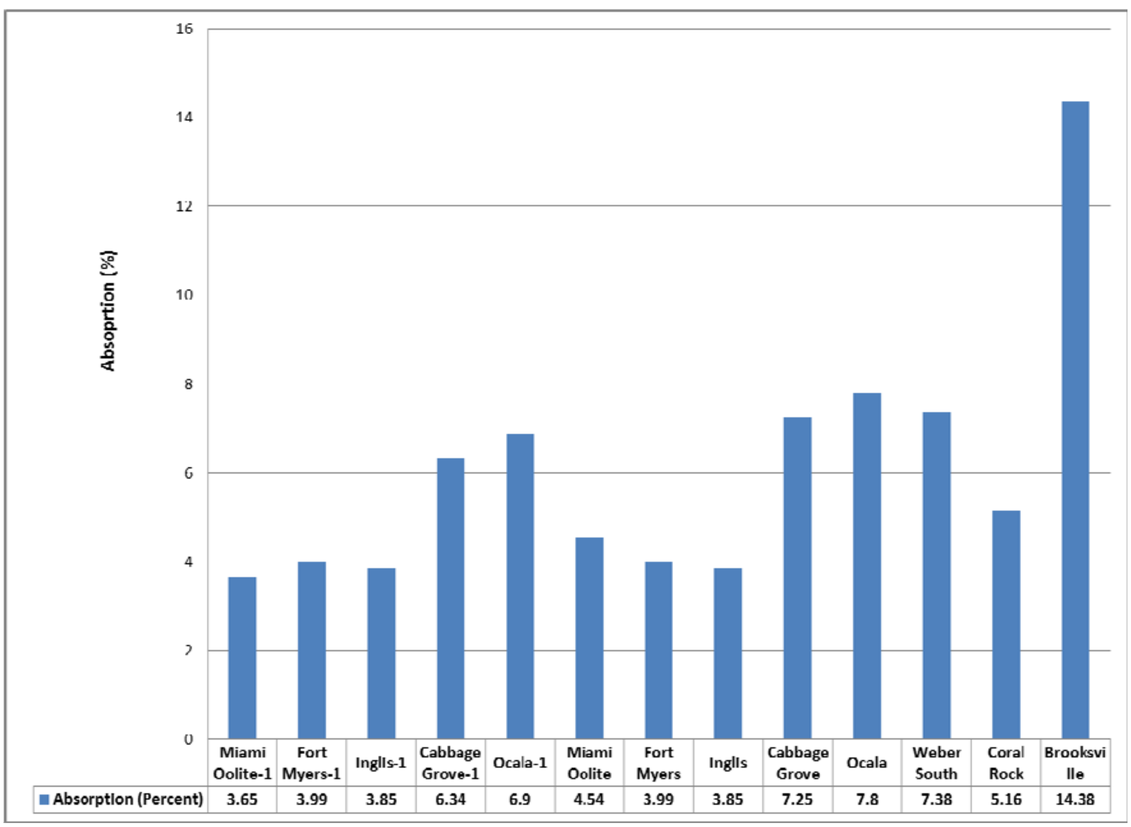


Figure 3-11. Plot of absorption.

Figure 3-12. Los Angeles abrasion machine.

Figure 3-13. Aggregate and steel spheres after Los Angeles abrasion loss test.

Table 3-9. Los Angeles Abrasion Loss

Aggregate Source	Los Angeles Abrasion Loss (%)
Miami Oolite-1	35
Fort Myers-1	36
Inglis-1	42
Cabbage Grove-1	48
Ocala-1	47
Miami Oolite	31
Fort Myers	36
Inglis	42
Cabbage Grove	50
Ocala	46
Weber South	48
Coral Rock	40
Brooksville	67*

** The researchers acknowledge the cooperation of Cemex in customizing a product to purposefully suit this project*

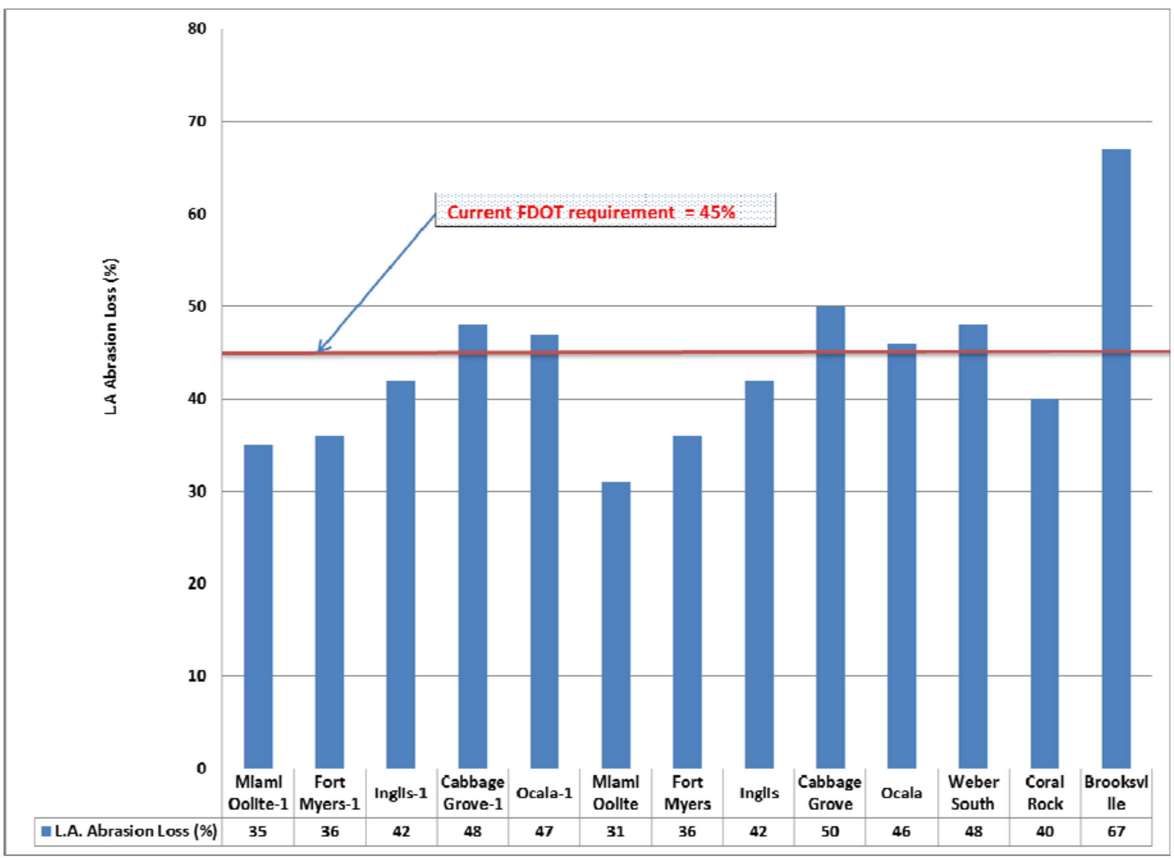


Figure 3-14. Plot of Los Angeles abrasion loss (%).

3.3.6 Micro-Deval Abrasion Loss

The resistance of the aggregates to degradation by abrasion with the use of the Micro-Deval apparatus was determined in accordance with AASHTO T 327-09. Figure 3-15 shows the Mico-Deval machine used in the experiment while Figures 3-16 through 3-18 show the experimental stages. Three different tests were done on each aggregate, and Table 3-10 shows the results of the mean abrasion loss. Table B-9 in Appendix B shows the individual results for each aggregate. Figure 3-19 shows the plot of the data from Table 3-10.

Figure 3-15. Micro-Deval abrasion machine.

Figure 3-16. Soaked aggregate with steel balls before Micro-Deval test.

Figure 3-17. Aggregate being poured after Micro-Deval test.

Figure 3-18. Steel balls being removed from aggregate after Micro-Deval test.

Table 3-10. Micro-Deval Abrasion Loss

Aggregate Source	Micro-Deval Loss (%)
Miami Oolite-1	30
Fort Myers-1	30
Inglis-1	27
Cabbage Grove-1	34
Ocala-1	45
Miami Oolite	26
Fort Myers	29
Inglis	27
Cabbage Grove	38
Ocala	47
Weber South	32
Coral Rock	29
Brooksville	81

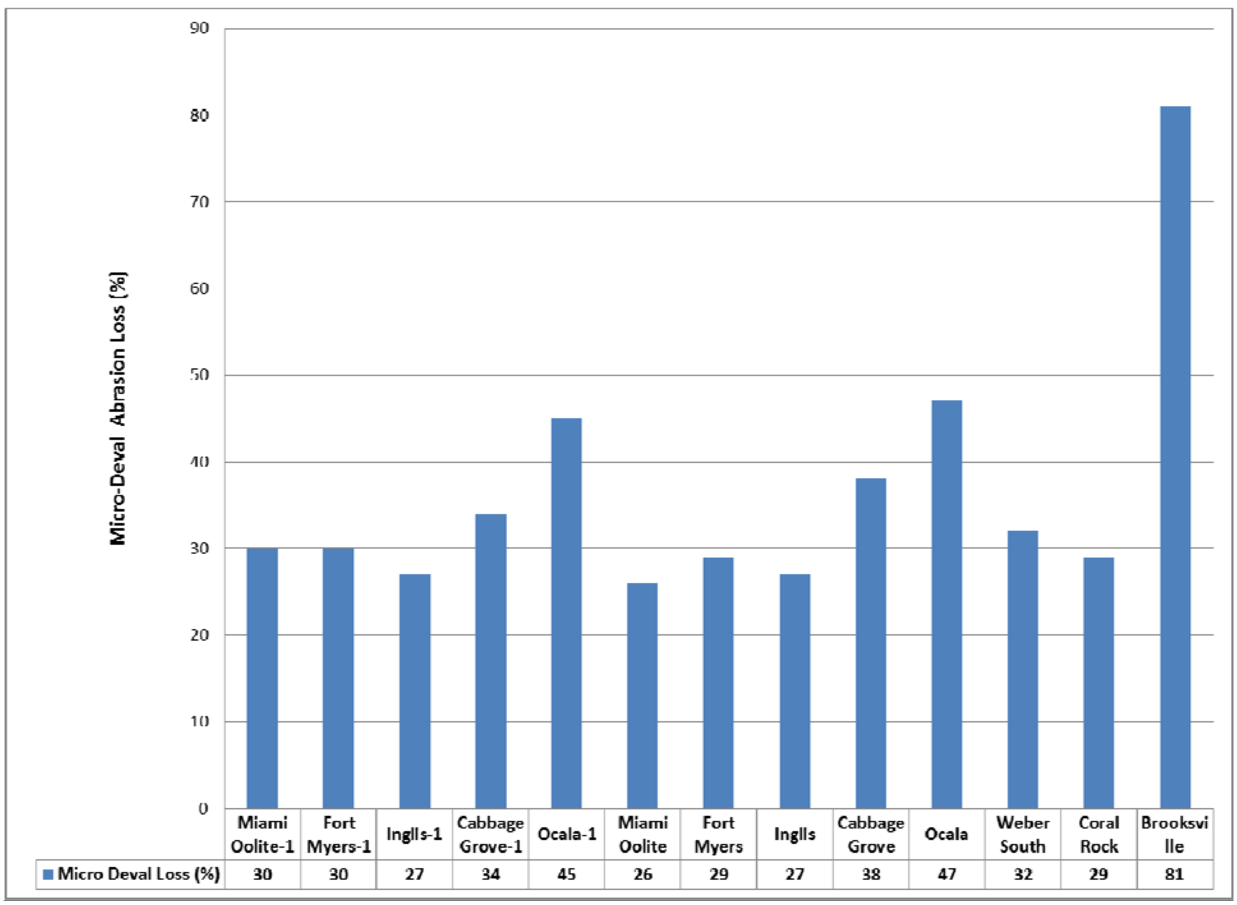


Figure 3-19. Plot of Micro-Deval abrasion loss (%).

3.3.7 Sodium Sulfate Soundness Loss

The resistance of the aggregates to disintegration by soaking in sodium sulfate was determined in accordance with AASHTO T 104. Figure 3-20 shows the incubator in which the aggregates placed in the solution of sodium sulfate were stored, while Figure 3-21 shows the aggregates being dried in the oven after they were removed from the solution of sodium sulfate. Three replicate tests were done on each aggregate and Table 3-11 shows the results of the mean sodium sulfate soundness loss. Table B-10 in Appendix B shows the individual results for each aggregate. Figure 3-22 shows a plot of the data in Table 3-11. Section 901 of the *2010 FDOT Standard Specification for Road and Bridge Construction* stipulates that the maximum allowable sodium sulfate soundness loss should be 12%. From the plot in Figure 3-22, it may be observed that most of the aggregates do not meet the current specification. Also, the Fort Myers limestone which was used in this study did not meet the current specification.

3.3.8 Shell Content

The percentage of free shell in the aggregates was determined in accordance with FM 5-555. Figure 3-23 shows the caliper used to determine the free shell content, while Figure 3-24 shows a sample of the free shells removed from the aggregates. Only two of the aggregates had free shell in them; Table 3-12 shows the mean values of shell content from three replicate tests performed on each aggregate. Table B-11 in Appendix B shows the individual results for each aggregate. Section 901-1.2 of the *2010 FDOT Standard Specification for Road and Bridge Construction* specifies a maximum of 1% free shell for aggregate to be used in concrete applications. From the results, it may be seen that although Coral Rock was intended to have a high shell content (refer to Table 3-2), the results obtained were within specification.

Figure 3-20. Incubator for storing samples.

Figure 3-21. Sodium sulfate samples in oven.

Table 3-11. Sodium Sulfate Soundness Loss

Aggregate Source	Sodium Soundness Loss (%)
Miami Oolite-1	8
Fort Myers-1	12
Inglis-1	13
Cabbage Grove-1	15
Ocala-1	20
Miami Oolite	9
Fort Myers	13
Inglis	13
Cabbage Grove	14
Ocala	20
Weber South	29
Coral Rock	12
Brooksville	38

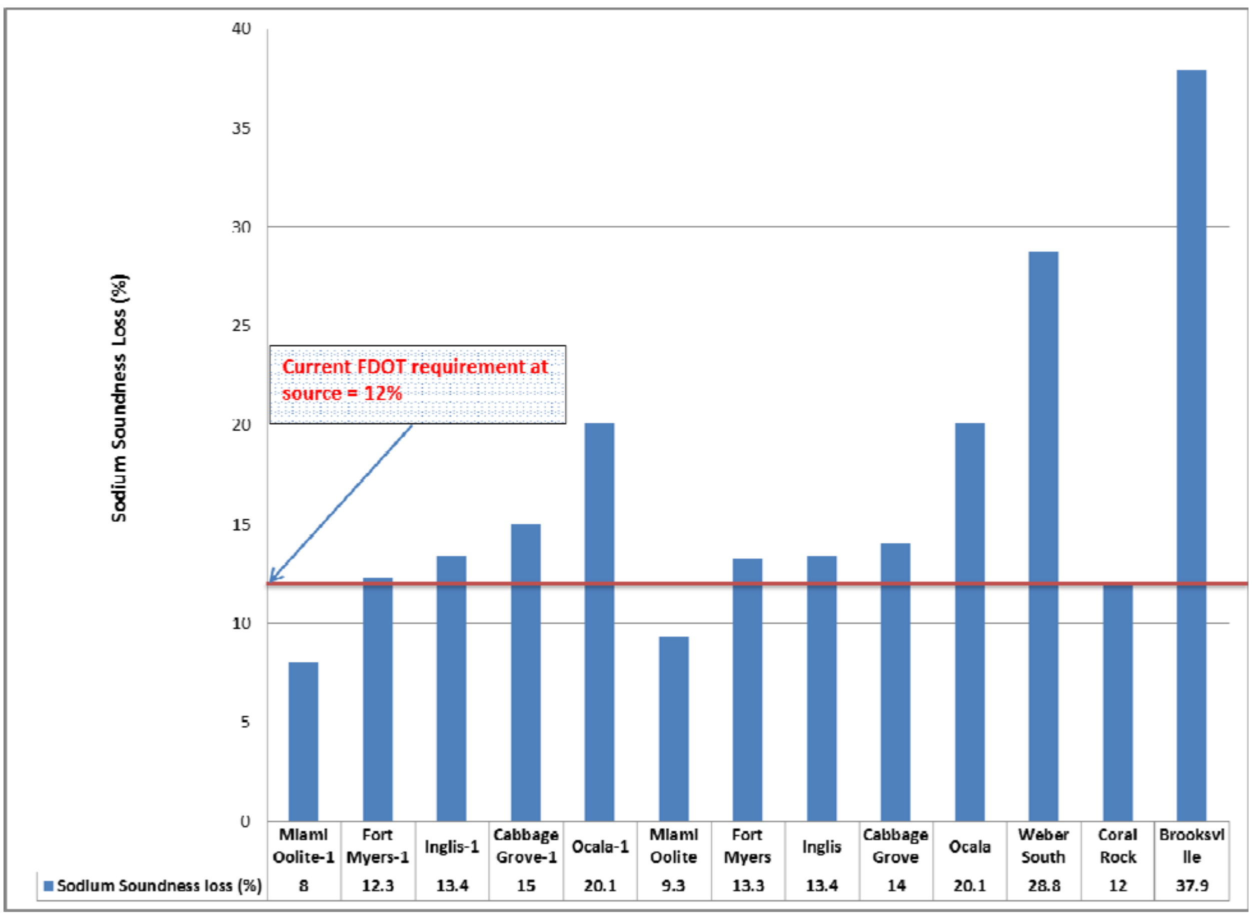


Figure 3-22. Plot of sodium sulfate soundness loss (%).

Figure 3-23. Caliper for determining free shell content.

Figure 3-24. Free shell removed from aggregates.

Table 3-12. Shell Content

Aggregate Source	Shell Content (%)
Weber South	12.1
Coral Rock	0.6

3.3.9 Unit Weight

The unit weight of each of the aggregates was determined in accordance with AASHTO T19. The results are shown in Table 3-13. Figure 3-25 shows the plot of the data from Table 3-13. From Figure 3-25, it may be observed that most of the non-standard aggregates have a lower unit weight as compared with the standard aggregates.

Table 3-13. Unit Weight

Aggregate Source	Unit Weight (lb/ft^3)
Miami Oolite-1	89.95
Fort Myers-1	85.42
Inglis-1	89.95
Cabbage Grove-1	79.42
Ocala-1	80.34
Miami Oolite	82.57
Fort Myers	85.42
Inglis	89.95
Cabbage Grove	79.08
Ocala	80.34
Weber South	75.68
Coral Rock	78.92
Brooksville	74.00

3.4 Concrete Mix Design

Concretes of different proportions were produced using the Shilstone and the American Concrete Institute (ACI) design methods for the non-standard and standard aggregates, respectively. The first batch of aggregates was used to produce concrete at water/cement ratios (w/c) of 0.5 and 0.6 and a cement content of 600 lb/yd3 and 545 lb/yd3, respectively, while the second batch of aggregates was used to produce concrete at 0.6 and 0.7 w/c ratios and a cement content of 470 lb/yd3 and 407 lb/yd3, respectively. The various mix proportions are shown in Tables 3-14 and 3-15. Details of the mixing ingredients used for the concrete are presented in the following section.

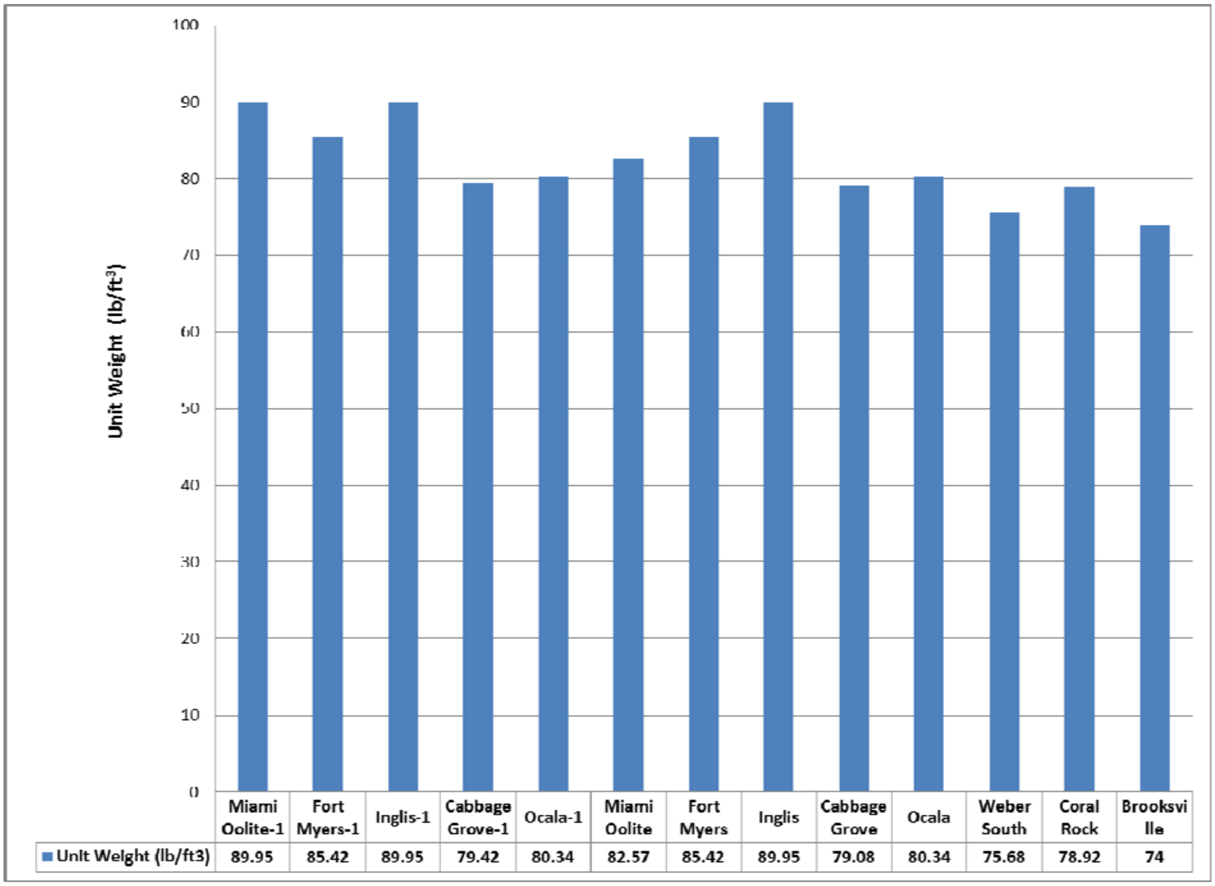


Figure 3-25. Plot of unit weight (lb/ft^3).

Table 3-14. Concrete Mix Proportions for Concrete Containing High Cement Content

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Coarse Aggregate (lb/yd^3)	Fine Aggregate (lb/yd^3)	Cement (lb/yd^3)	Water (lb/yd^3)	WRA (oz/yd^3)
Miami Oolite-1	1	0.5	1678	1218	600	300	–
	2	0.6	1607	1271	545	327	–
Fort Myers-1	1	0.5	1600	1289	600	300	–
	2	0.6	1632	1229	545	327	–
Inglis-1	1	0.5	1709	1219	600	300	–
	2	0.6	1639	1270	545	327	–
Cabbage Grove-1	1	0.5	1556	1270	600	300	–
	2	0.6	1595	1200	545	327	–
Ocala-1	1	0.5	1576	1270	600	300	–
	2	0.6	1616	1200	545	327	–

Table 3-15. Concrete Mix Proportions for Concrete Containing Low Cement Content

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Coarse Aggregate (lb/yd^3)	Fine Aggregate (lb/yd^3)	Cement (lb/yd^3)	Water (lb/yd^3)	WRA (oz/yd^3)
Miami Oolite	1	0.6	1672	1421	470	282	28.2
	2	0.7	1672	1477	403	282	24.2
Fort Myers	1	0.6	1630	1413	470	282	21.2
	2	0.7	1630	1469	403	282	24.0
Modified Miami Oolite	1	0.6	1672	1421	470	282	28.2
	2	0.7	1672	1477	403	282	24.2
Modified Fort Myers	1	0.6	1630	1413	470	282	28.2
	2	0.7	1630	1468	403	282	24.2
Inglis	1	0.6	1667	1421	470	282	28.2
	2	0.7	1667	1477	403	282	24.2
Cabbage Grove	1	0.6	1562	1360	470	282	28.2
	2	0.7	1576	1400	403	282	24.2
Ocala	1	0.6	1512	1477	470	282	28.2
	2	0.7	1532	1510	403	282	24.2
Weber South	1	0.6	1579	1380	470	282	28.2
	2	0.7	1590	1423	403	282	24.2
Coral Rock	1	0.6	1575	1399	470	282	28.2
	2	0.7	1585	1444	403	282	48.4
Brooksville	1	0.6	1360	1450	470	282	28.2
	2	0.7	1388	1470	403	282	24.2

3.5 Mix Ingredients

The properties of the ingredients used for the concrete mixes are as follows:

Water – Water supplied from the City of Gainesville grid was used for the mix. Care was taken to ensure that no foreign impurities got into the water.

Cement – Portland cement type I/II supplied by Florida Rock Industries was used. Tables 3-16 and 3-17 show the physical and chemical properties of the cement determined by Florida Department of Transportation. Appendix C shows the detailed test results.

Aggregates – Silica sand from FDOT source 71132, Vulcan Materials Company's Goldhead mine of Florida was used as fine aggregate. The properties of the fine aggregate are shown in Tables 3-18 and 3-19. The coarse aggregates which were described in Section 3.3 were used for the different mixtures.

Admixture – W.R. Grace & Co. supplied the researchers with WRDA 60 water-reducing admixture complying to ASTM C494 Type A and D was used as necessary to improve on the workability of the mixtures.

Table 3-16. Physical Properties of Portland Cement

Test	Standard Test	Cement
Loss on ignition	ASTM C114	3.0%
Loss on ignition (acid insoluble)	ASTM C114	0.57%
7-day compressive strength	ASTM C109	4580 psi
Time of setting (initial)	ASTM 266	100 min
Time of setting (final)	ASTM 266	300 min

Table 3-17. Chemical Properties of Portland Cement

Constituents	Percentage
Aluminum oxide	5.0%
Ferric oxide	4.0%
Magnesium oxide	1.3%
Sulfur trioxide	2.7%
Tricalcium aluminate	6.0%
Tricalcuim silicate	70.0%
Total alkali as Na_2O	0.35%

Table 3-18. Specific Gravity and Water Absorption of Fine Aggregates (FM 1-T 084)

Property	Fine Aggregates
SSD specific gravity	2.63
Dry bulk specific gravity	2.62
Dry apparent specific gravity	2.65
Absorption	0.5

Table 3-19. Results of Sieve Analysis on the Fine Aggregate

Sieve Size	Sieve Size (mm)	Percentage Passing Fine Aggregates (%)
#4	4.75	100
#8	2.36	99
#16	1.18	91
#30	0.60	70
#50	0.30	32
#100	0.15	5
Fineness modulus		2.03

3.6 Fabrication and Curing of Concrete Specimens

Concrete mixtures were produced in the laboratory using a nine-cubic foot drum mixer as shown in Figure 3-26. For each concrete mix, about four and half cubic feet of fresh concrete was produced to fabricate twenty-eight cylinders (4″ × 8″), thirteen beams (4″ × 4″ × 14″) and three prisms (3″× 3″ × 11.25″). Table 3-20 shows the details of tests performed on concrete samples with various specimen sizes and curing periods.

Figure 3-26. Drum mix.

Table 3-20. Tests Performed on the Concrete Samples

Test	Specimen Size	Curing Period
Compressive strength	4″ × 8″ cylinder	14 and 28 days
Elastic modulus	4″ × 8″ cylinder	14 and 28 days
Flexural strength	4″ × 4″ × 14″ beam	14 and 28 days
Splitting tensile strength	4″ × 8″ cylinder	14 and 28 days
Coefficient of thermal expansion	4″ × 8″ cylinder	28 days
Drying shrinkage	3″ × 3″ × 11.25″ prism	28 days

3.7 Concrete Preparation

3.7.1 Steps Followed in Laboratory

The following steps were followed to produce concrete in the laboratory;

i. Fill cloth bags with fine aggregates required for mix.

ii. Dry the fine aggregates for at least 24 hours in an oven at 230°F, and then let it cool for another 24 hours.

iii. Fill buckets with coarse aggregates required for the mix.

iv. Take a sample of the coarse aggregate to be used to determine the moisture as stipulated by ASTM C 566.

v. Based on the mix design, batch the coarse aggregate, fine aggregate, cement and water using a weighing scale as shown in Figure 3-27.

vi. Place the coarse aggregate and fine aggregate, and add the calculated amount of water to let the aggregates achieve Saturated Surface Dry (SSD) condition in a drum mixer.

vii. Wait for about 45 minutes before starting mixing. Section 3.7.2 explains rationale for this step.

viii. Run the mixer for 30 seconds.

ix. Add more than half of the mixing water and mix it for 1 minute.

x. Place cement and remaining water and mix it for 3 minutes, followed by a 3 minute rest, followed by a 2-minute mixing. (This is done in accordance with ASTM C 192).

xi. Perform fresh concrete property tests as presented in Section 3.9.

3.7.2 Rationale for 45-Minute Wait Time before Mixing

The usual laboratory procedure of placing the coarse aggregates in jute bags and soaking them in water for at least 48 hours before getting them to SSD condition was not implemented because most of the non-standard aggregates had high percentage of materials finer than the 200 sieve size as presented earlier in this chapter. Therefore, soaking them may remove most of the fines from the aggregates and will make it difficult to determine the effect of the fines on the concrete. This method was chosen in other to approximate the SSD aggregate conditions. In the ready-mix plant, producers may have to turn aggregates to avoid similar problem.

Figure 3-27. Scale.

The researcher therefore conducted several experiment to determine the amount of soak time the aggregates takes to absorb water to reach SSD condition. A comparison of this was done with the standard absorption test stipulated by FM 1 T-085. It was found that the 45 minutes wait time was enough time for the aggregates to reach SSD condition. Table 3-21 shows the comparison between the mean of three replicates of absorption after the 45-minutes soak time and standard FM 1 T-085 soak time for two aggregates tested.

Table 3-21. Sample Comparison between 45-Minute and FM T-085 Absorption Values

Aggregate Source	Mean Absorption after 45% of Soaking	Mean Absorption Determined from FM 1 T-085	Percentage Difference (%)
Cabbage Grove-1	6.00	6.34	5.4%
Ocala	8.02	7.80	2.82%

3.8 Sample Preparation

After concrete was produced, some portion of the concrete was immediately used for conducting tests to determine fresh concrete properties as discussed in Section 3.9. The remaining concrete was used to fabricate different samples in accordance with ASTM C192 as follows:

3.8.1 Cylindrical Specimen

- Place concrete in molds such that they are half filled.
- Place the molds on a vibrating table and vibrate for 45 seconds.
- Fill the molds completely and vibrate them for another 45 seconds. Figure 3-28 shows the cylinders on the vibratory table.
- Finish the concrete surface with a hand trowel.
- Cover the concrete with plastic caps.
- Remove the samples from the molds after 24 hours and place them in a moist curing room as shown in Figure 3-29.

3.8.2 Beam Specimen

- Place concrete in molds such that they are half filled.
- Vibrate with a vibrating table shown in Figure 3-30.
- Finish the concrete surface with a hand trowel.
- Cover the concrete with polythene sheets.
- Remove the samples from the molds after 24 hours and place them in a moist curing room as shown in Figure 3-29.

Figure 3-28. Cylinders on vibrating table.

Figure 3-29. Samples in moist curing room.

Figure 3-30. Beams on vibrating table.

3.9 Tests on Fresh Concrete

Table 3-22 provides the list of ASTM standard tests performed on the fresh concrete used in this study. The properties of the fresh concrete mixtures are presented in Tables 3-23 and 3-24.

Table 3-22. Standards for Fresh Concrete Tests

Test	Standard
Slump	ASTM C143
Unit weight	ASTM C138
Air content	ASTM C231
Temperature	ASTM C1064

Slump Test – The test was run in accordance with ASTM C143. The slump is very useful in detecting variations in the uniformity of a mix of given nominal proportions; it is a measure of consistency of the fresh concrete. This test is conducted immediately after the concrete has been made. Figure 3-31 shows a typical determination of slump after mixing.

Unit Weight Test – The test was performed in accordance with ASTM C138. The theoretical unit weight was calculated and compared with the laboratory unit weight to determine whether the mix was properly batched. Figure 3-32 shows a typical determination of unit weight after mixing.

Table 3-23. Properties of Fresh Concrete Containing High Cement Content

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Cement (lb/yd^3)	Slump (inch)	Unit Weight (lb/ft^3)	Air Content (%)	Temper-ature (°F)
Miami Oolite-1	1	0.5	600	6.50	142.32	1.5	79
	2	0.6	545	8.00	140.88	1.2	78
Fort Myers-1	1	0.5	600	8.75	140.10	1.8	75
	2	0.6	545	10.75	139.20	0.6	75
Inglis-1	1	0.5	600	9.25	140.96	2.5	75
	2	0.6	545	10.00	141.68	2.3	75
Cabbage Grove-1	1	0.5	600	7.00	138.64	3.7	75
	2	0.6	545	9.50	138.48	3.0	75
Ocala-1	1	0.5	600	4.25	139.52	3.0	78
	2	0.6	545	9.50	138.56	1.9	79

Table 3-24. Properties of Fresh Concrete Containing Low Cement Content

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Cement (lb/yd^3)	Slump (inch)	Unit Weight (lb/ft^3)	Air Content (%)	Temper-ature (°F)
Miami Oolite	1	0.6	470	3.25	139.28	3.1	76
	2	0.7	403	1.50	137.52	3.7	77
Fort Myers	1	0.6	470	3.00	139.12	2.4	75
	2	0.7	403	2.00	136.80	3.1	76
Modified Miami Oolite	1	0.6	470	1.75	138.16	3.7	77
	2	0.7	403	2.00	139.92	3.3	77
Modified Fort Myers	1	0.6	470	2.25	138.32	2.7	80
	2	0.7	403	1.25	138.80	2.7	78
Inglis	1	0.6	470	2.00	139.04	2.9	76
	2	0.7	403	1.50	139.34	3.2	76
Cabbage Grove	1	0.6	470	2.00	135.28	2.9	74
	2	0.7	403	1.50	135.12	2.9	76
Ocala	1	0.6	470	1.25	135.36	3.8	78
	2	0.7	403	1.25	134.32	3.8	78
Weber South	1	0.6	470	4.50	135.60	3.0	75
	2	0.7	403	2.50	135.60	2.8	75
Coral Rock	1	0.6	470	1.75	138.80	2.4	70
	2	0.7	403	1.00	135.60	4.2	70
Brooksville	1	0.6	470	1.50	131.28	0.4	75
	2	0.7	403	2.00	129.92	0.7	82

Figure 3-31. Determination of slump.

Figure 3-32. Determination of unit weight.

Air Content Test – The entrapped air in the concrete mix was determined by the pressure-meter method in accordance with ASTM C231. The pressure meter was used for this test and an aggregate correction factor determined for each mix. Figure 3-33 shows a typical determination of air content after mixing.

Temperature Test – This test was run in accordance with ASTM C1064. Figure 3-34 shows a typical determination of the temperature of a mixture.

Figure 3-33. Determination of air content.

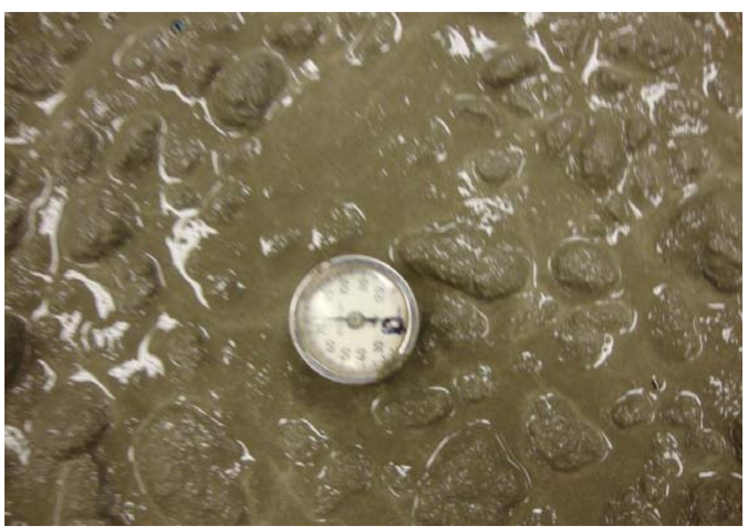


Figure 3-34. Determination of temperature of mixture.

3.10 Tests on Hardened Concrete

3.10.1 Compressive Strength

The compressive strength on each 4″ × 8″ cylindrical specimen was determined in accordance with ASTM C39. The two ends of each specimen were ground before testing to ensure uniform distribution of load during test. Figure 3-35 shows the grinder that was used. The diameter of each specimen was taken before the compressive strength test. The testing machine (see Figure 3-36) was hydraulic controlled with a maximum capacity of 220 kips. Load was applied to the specimen at a constant loading rate of 35 psi/s until complete failure occurred. The outputs of the load cell from the testing machine were connected to a data acquisition system, which records the data during the test. The maximum load is recorded and the compressive stress computed by dividing the maximum load by the cross sectional area of the specimen. The type of fracture was also recorded. Figure 3-37 shows a cylinder in the testing machine before test.

Figure 3-35. Grinder.

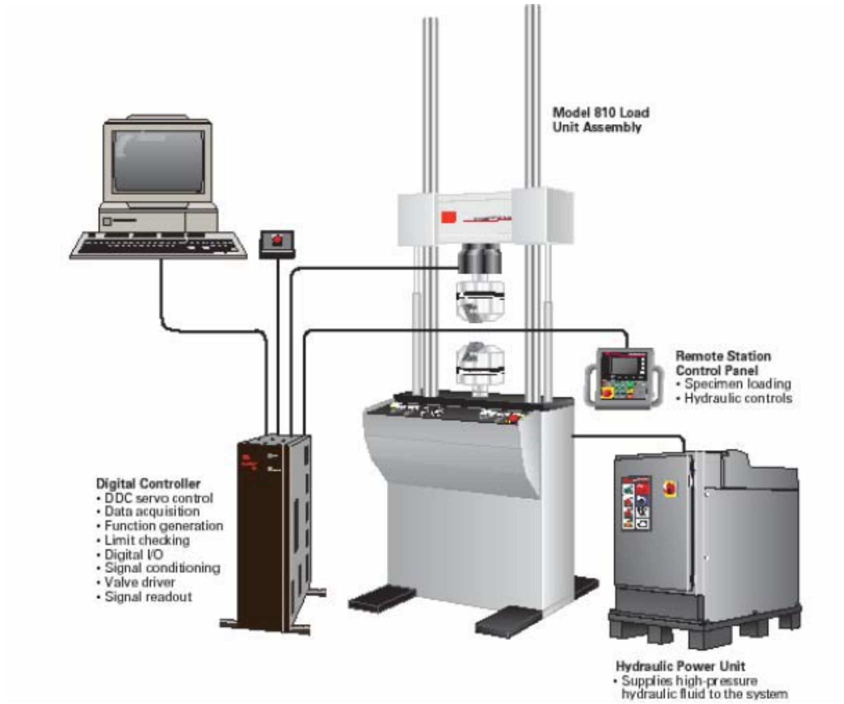


Figure 3-36. Material testing system 810 (Guang Li, 2004).

Figure 3-37. Sample in compressive test.

3.10.2 Elastic Modulus Test

The Elastic Modulus on each 4″ × 8″ cylindrical specimen was determined in accordance with ASTM C469. The ends of the specimens were ground before testing to uniform distribution of load during test. Two 4″ displacement gages attached to the compressometer frame and held by four springs were mounted on the sides of the specimen. The specimen was then placed in a testing system as shown in Figure 3-38. Load was applied to the specimen at a constant loading rate of 35 psi/s until 40% of the average maximum load obtained from the compressive strength test was attained. The outputs of the displacement gages and the load cell from the testing machine were connected to a data acquisition system, which records the data during the test. The average displacement reading was used to calculate the strain, and reading from the load cell was used to calculate the stress.

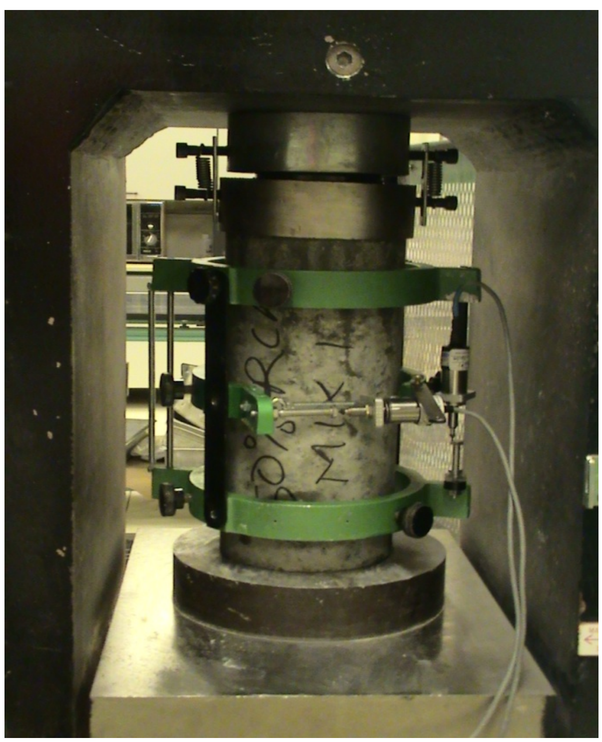


Figure 3-38. Sample in modulus of elasticity test.

The modulus of elasticity was calculated as follows:

$$E = (S_2 - S_1)\frac{1}{(\varepsilon_2 - 0.000050)} \tag{3.1}$$

where E = chord modulus of elasticity (psi);

S_2 = stress corresponding to 40% of ultimate load (psi);

S_1 = stress corresponding to a longitudinal strain, ε_1, of 50 millionths (psi); and

ε_2 = longitudinal strain produced by stress S_2 (in./in.).

3.10.3 Flexural Strength Test

The flexural strength test was run in accordance with ASTM C78 on 4″ × 4″ × 14″ beam specimen at each age and the average strength was computed. Before testing, the two loading surfaces were ground evenly by using a grinding stone to ensure that the applied load was uniform. The flexural strength was calculated according to the type of fracture in the beam as follows:

- If the fracture initiates in the tension surface within the middle third of the span length, then modulus of rupture is calculated as follows:

$$R = \frac{PL}{bd^2} \tag{3.2}$$

where R = modulus of rupture (psi);

P = maximum applied load indicated by the testing machine (lbf);

L = span length (inch or mm);

b = average width of specimen (inch or mm) at the fracture; and

d = average depth of specimen (inch or mm) at the fracture.

- If the fracture occurs in the tension surface outside of the middle third of the span length by not more than 5% of the span length, then modulus of rupture is calculated as follows:

$$R = \frac{3Pa}{bd^2} \tag{3.3}$$

where R = modulus of rupture (psi);

P = maximum applied load indicated by the testing machine (lbf);

a = average distance between line of fracture and the nearest support measured on the tension surface of the beam (inch or mm);

b = average width of specimen (inch or mm) at the fracture; and

d = average depth of specimen (inch or mm) at the fracture.

- If the fracture occurs in the tension surface outside of the middle third span length by more than 5% of the span length, discard the results of the test. Figure 3-39 shows a typical setup of the beam during testing. Figure 3-40 shows a typical failed beam specimen after the flexural test.

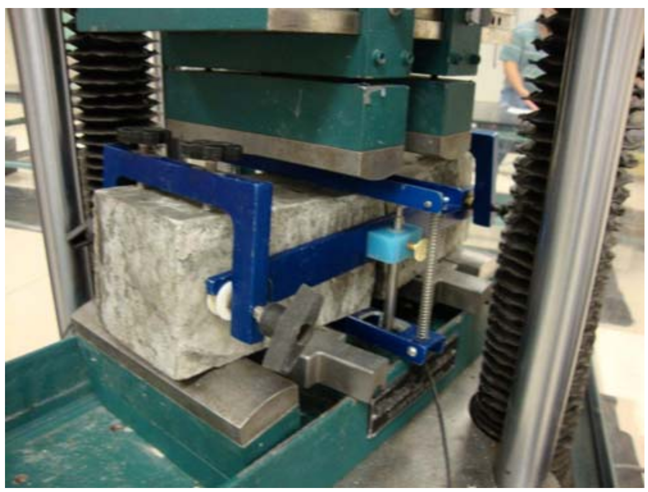


Figure 3-39. Flexural test setup.

Figure 3-40. Failure of sample under flexural test.

3.10.4 Splitting Tensile Strength Test

The splitting tensile strength of concrete was run in accordance with ASTM C496 on cylindrical specimens (4″ × 8″). Four lines were drawn along the center of the cylinder to mark the edges of the loaded plane and to help align the test specimen before the application of load. Figure 3-41 shows a typical setup of the cylinder during testing. A strip of wood, 3-mm thick and 25-mm wide, was inserted between the cylinder and the platens; this helped the applied force to be uniformly distributed. Load was applied and increased under a controlled rate until failure by indirect tension in the form of splitting along vertical diameter took place. Figure 3-42 shows a typical failed sample. The splitting tensile strength of a cylinder specimen was calculated using the following equation:

$$T = \frac{2P}{\pi LD} \tag{3.4}$$

where T = splitting tensile strength of cylinder (psi);

P = maximum applied load (lbf);

L = average length of cylinder (inch); and

D = average diameter of cylinder (inch).

Figure 3-41. Splitting tensile setup.

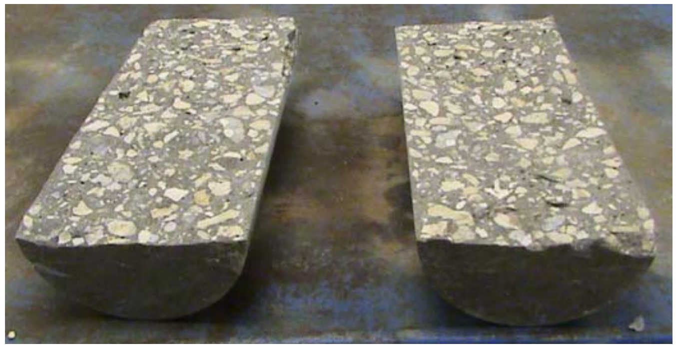


Figure 3-42. Failed sample from splitting tensile test.

3.10.5 Free Shrinkage Test

The free shrinkage measurement was made in accordance with ASTM C157 using 3″ × 3″ × 11.25″ square prism specimens. Steel end plates with a hole at their centers were used to install gage studs at both ends of the specimen. The specimens were removed from the molds at an age of 23 ½ ± ½ h (after the addition of water to cement during the mixing operation) and then placed in lime-saturated water which was maintained at 73.4 ± 1 °F (23.0 ± 0.5°C) for a minimum of 30 min. At an age of 24 ± 1/2 h, the specimens were removed from water storage one at a time, and wiped with a damp cloth. An initial reading was immediately taken with a length comparator. The specimens were then stored in the drying room and maintained at a temperature of 73.4 ± 1 °F (23.0 ± 0.5°C) and a relative humidity of 50%. The comparator readings were taken of each specimen after 28 days. Figure 3-43 shows the test set-up of the free shrinkage test. The length change of a specimen at any age after the initial comparator reading was calculated as follows:

$$\Delta L_x = \frac{\text{CRD} - \text{initial CRD}}{\text{G}} \times 100 \qquad (3.5)$$

where ΔL_x = length change of specimen at any age (%);

CRD = difference between the comparator reading of the specimen and the reference bar; and

G = gage length.

Figure 3-43. Free shrinkage setup.

3.10.6 Coefficient of Thermal Expansion (CTE) Test

The CTE test was run in accordance with AASHTO TP60. The test set-up is shown in Figure 3-44. The samples were sawed using a sawing machine as shown in Figure 3-45 and then ground using a grinding machine as shown previously in Figure 3-35. This helped the samples to be in the desired length (7± 0.1 inch) required for the test.

The procedure for the CTE test is as follows:

- Place the support frame, with LVDT attached, in the water bath and fill the bath with cold tap water. Place the four temperature sensors in the bath at locations that will provide an average temperature for the bath as a whole. To avoid any sticking at the points of contact with specimen, put a very thin film of silicon grease on the end of the support buttons and LVDT tip.

- Remove the specimen from the moisture room and measure its length at room temperature to the nearest 0.1 mm (0.004″). After measuring the length, place the specimen in the support frame located in the controlled temperature bath, making sure that the lower end of the specimen is firmly seated against the support buttons, and the LVDT tip is seated against the upper end of the specimen. Connect the LVDT and temperature sensors to a data acquisition system which is connected to a laptop computer.

Figure 3-44. Coefficient of thermal expansion setup.

Figure 3-45. Sawing machine.

- Set the temperature of the water bath to 10 ± 1°C (50 ± 2°F). When the bath reaches this temperature, allow the bath to remain at this temperature until thermal equilibrium of the specimen has been reached, as indicated by consistent readings of LVDT to the nearest 0.00025 mm (0.00001″) taken every 10 minutes over a one-half hour period. Record the temperature readings from the four sensors to the nearest 0.1°C (0.2°F). Record the LVDT reading to the nearest 0.00025 mm (0.00001″). These are the initial readings. Set the temperature of the water bath to 50 ± 1°C (122 ± 2°F). When the bath reaches this temperature, allow the bath to remain at this temperature until thermal equilibrium of the specimen has been reached, as indicated by consistent readings of LVDT to the nearest 0.00025 mm (0.00001″). Record the temperature readings from the four sensors to the nearest 0.1°C (0.2°F). Record the LVDT reading to the nearest

0.00025 mm (0.00001″). These are the second readings. Set the temperature of the water bath to 10 ± 1°C (50 ± 2°F). When the bath reaches this temperature, allow the bath to remain at this temperature until thermal equilibrium of the specimen has been reached. Record the temperature readings from the four sensors to the nearest 0.1°C (0.2°F). Record the LVDT reading to the nearest 0.00025 mm (0.00001″). These are the final readings. The CTE of one expansion or contraction test segment of a concrete specimen is calculated as follows:

$$CTE = (\Delta L_a / L_0) / \Delta T \tag{3.6}$$

where ΔL_a = actual length change of specimen during temperature change (mm or inch);

L_0 = measured length of specimen at room temperature (mm or inch); and

ΔT = measured temperature change (average of the four sensors) (ºC).

The test result is the average of the two CTE values obtained from the expansion test segment and contraction test segment, and is calculated as follows:

$$CTE = \frac{\text{CTE expansion} + \text{CTE contraction}}{2} \tag{3.7}$$

3.10.7 Concrete Water Absorption Test

The water absorption of the hardened concrete and the volume of voids were determined in accordance with Test Method A of ASTM C 497. In this test, a 4″ × 8″ cylindrical specimen that had been cured for 28 days was cut into thinner pieces to have a wall thickness of about 2″. The testing procedure as stipulated in ASTM C497 was followed to determine the 5-hour water absorption and the volume of voids of the test specimen.

3.11 Summary of Aggregate and Fresh Concrete Test Results

3.11.1 Aggregate Test Results

The results of tests on the aggregates used in this study can be summarized as follows:

- Most of the non-standard aggregates were finer than the standard aggregates; they tended to have a higher percentage of materials passing the No. 8, No. 4 and No. 200 sieves.

- The non-standard aggregates have higher absorption and relatively lower Bulk specific gravity (SSD). Their unit weight was comparatively lower than that of the standard aggregates.

- The non-standard aggregates were generally less durable when you compare their LA abrasion loss, Micro-Deval abrasion loss and sodium sulfate soundness loss with those of the standard aggregates.

3.11.2 Fresh Concrete Test Results

The Shilstone design method was useful in proportioning the mixtures containing the non-standard aggregates. A 45-minute soak time was used to ensure the absorption of water into the aggregates during mixing. In general, the following could be observed about the fresh concrete produced:

- The unit weight for the mixtures containing the non-standard aggregates was about the same as the mixtures containing standard aggregates with the exception of concrete containing Brooksville aggregate, which had lower unit weight.

- The air content for the mixtures containing the non-standard aggregates was about the same as the mixtures containing standard aggregates with the exception of concrete containing Brooksville aggregate, which had lower air content.

CHAPTER 4
HARDENED CONCRETE TEST RESULTS AND ANALYSIS

4.1 Introduction

This chapter presents the results of compressive strength, elastic modulus, flexural strength, splitting tensile strength, free shrinkage and coefficient of thermal expansion tests on the different concrete mixtures evaluated in this study. An analysis of the relationship between the various measured properties is also presented.

4.2 Test Results and Discussion

4.2.1 Compressive Strength Test Results

In the first experimental design, concrete mixes with relatively higher cement content (600 and 645 lb/yd3) were used. In the second experimental design, relatively lower cement contents (403 and 407 lb/yd3) were used. The average compressive strengths at 7 and 28 days of the different concrete mixtures in the first and second experimental designs are presented in Tables 4-1 and 4-2, and a plot of the results is shown in Figures 4-1 and 4-2, respectively. The individual compressive strength values are shown in Tables D-1and D-2 in Appendix D.

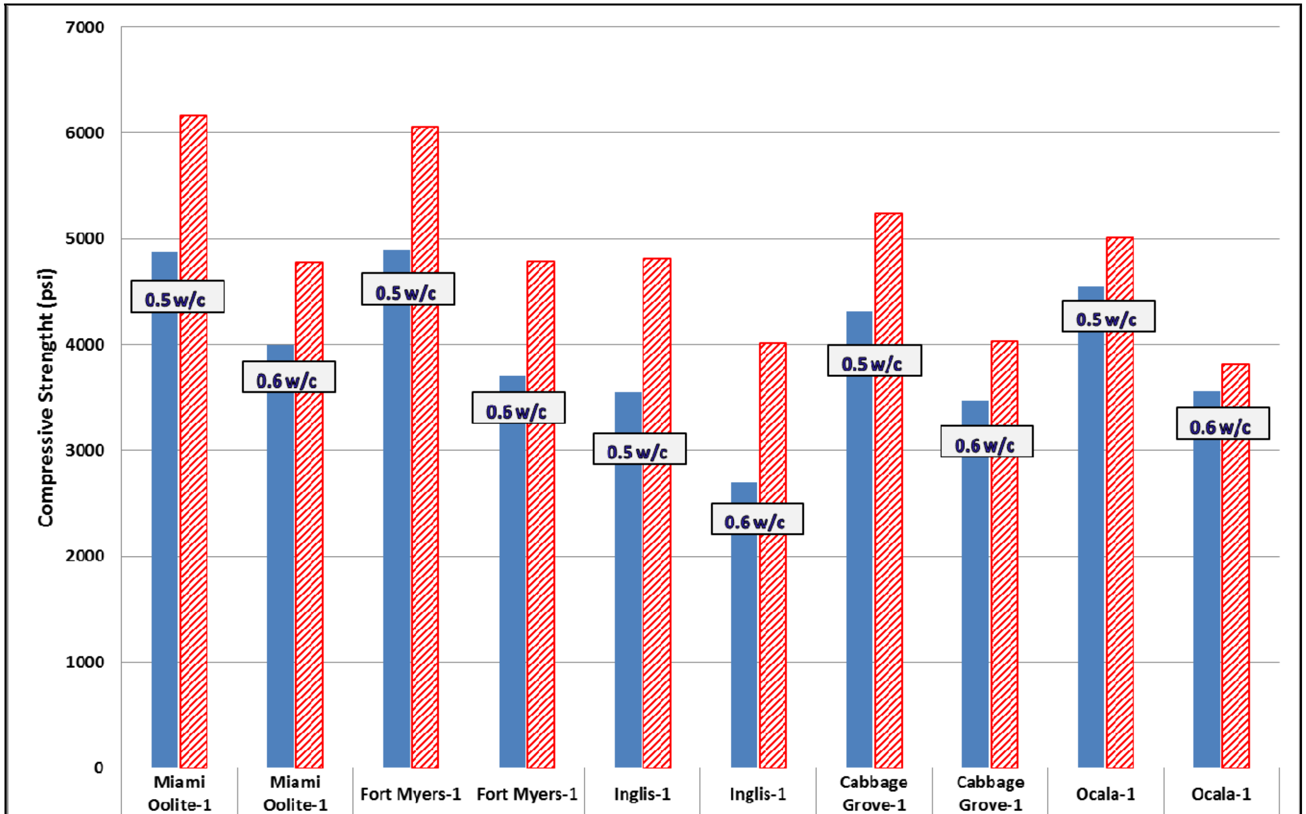
The compressive strength of concrete was also tested on concrete samples used for the modulus of elasticity test. The samples were first loaded to 40% of its ultimate load during the modulus of elasticity test and then unloaded before conducting the compressive strength test. Tables 4-3 and 4-4 show the compressive strengths after preload for the concrete in the first and second experimental designs, respectively. The plots of these results are shown correspondingly in Figures 4-3 and 4-4. Table D-9 and Table D-10 show the details of the results. Table 4-5 also shows the percentage difference in the 28-day compressive strength between concrete tested using standard ASTM C-39 and using samples which have been loaded to 40% of the ultimate load and unloaded before conducting the compressive strength test. The results show that there was not much difference between the two testing procedures. The difference varied from –8% to +5%.

Table 4-1. Compressive Strength of Concrete in the First Experimental Design

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Cement (lb/yd^3)	Testing Age	
				7 days	28 days
				Compressive Strength (psi)	
Miami Oolite-1	1	0.5	600	4870	6170
	2	0.6	545	4000	4780
Fort Myers-1	1	0.5	600	4900	6060
	2	0.6	545	3700	4790
Inglis-1	1	0.5	600	3550	4820
	2	0.6	545	2700	4020
Cabbage Grove-1	1	0.5	600	4310	5250
	2	0.6	545	3470	4030
Ocala-1	1	0.5	600	4550	5010
	2	0.6	545	3560	3820

Table 4-2. Compressive Strength of Concrete in the Second Experimental Design

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Cement (lb/yd^3)	Testing Age	
				7 days	28 days
				Compressive Strength (psi)	
Miami Oolite	1	0.6	470	3780	4720
	2	0.7	403	2490	3270
Fort Myers	1	0.6	470	3590	4590
	2	0.7	403	2270	3120
Modified Miami Oolite	1	0.6	470	3660	4650
	2	0.7	403	2110	2930
Modified Fort Myers	1	0.6	470	3250	4300
	2	0.7	403	2350	3020
Inglis	1	0.6	470	2970	3810
	2	0.7	403	2040	3080
Cabbage Grove	1	0.6	470	2910	3630
	2	0.7	403	2130	2890
Ocala	1	0.6	470	2770	3480
	2	0.7	403	2110	2790
Weber South	1	0.6	470	2170	2760
	2	0.7	403	1660	2320
Coral Rock	1	0.6	470	3150	4160
	2	0.7	403	2170	2940
Brooksville	1	0.6	470	2040	2490
	2	0.7	403	1600	1950



	Miami Oolite-1	Miami Oolite-1	Fort Myers-1	Fort Myers-1	Inglis-1	Inglis-1	Cabbage Grove-1	Cabbage Grove-1	Ocala-1	Ocala-1
7days	4870	4000	4900	3700	3550	2700	4310	3470	4550	3560
28days	6170	4780	6060	4790	4820	4020	5250	4030	5010	3820

Figure 4-1. Compressive strength of concrete in the first experimental design.

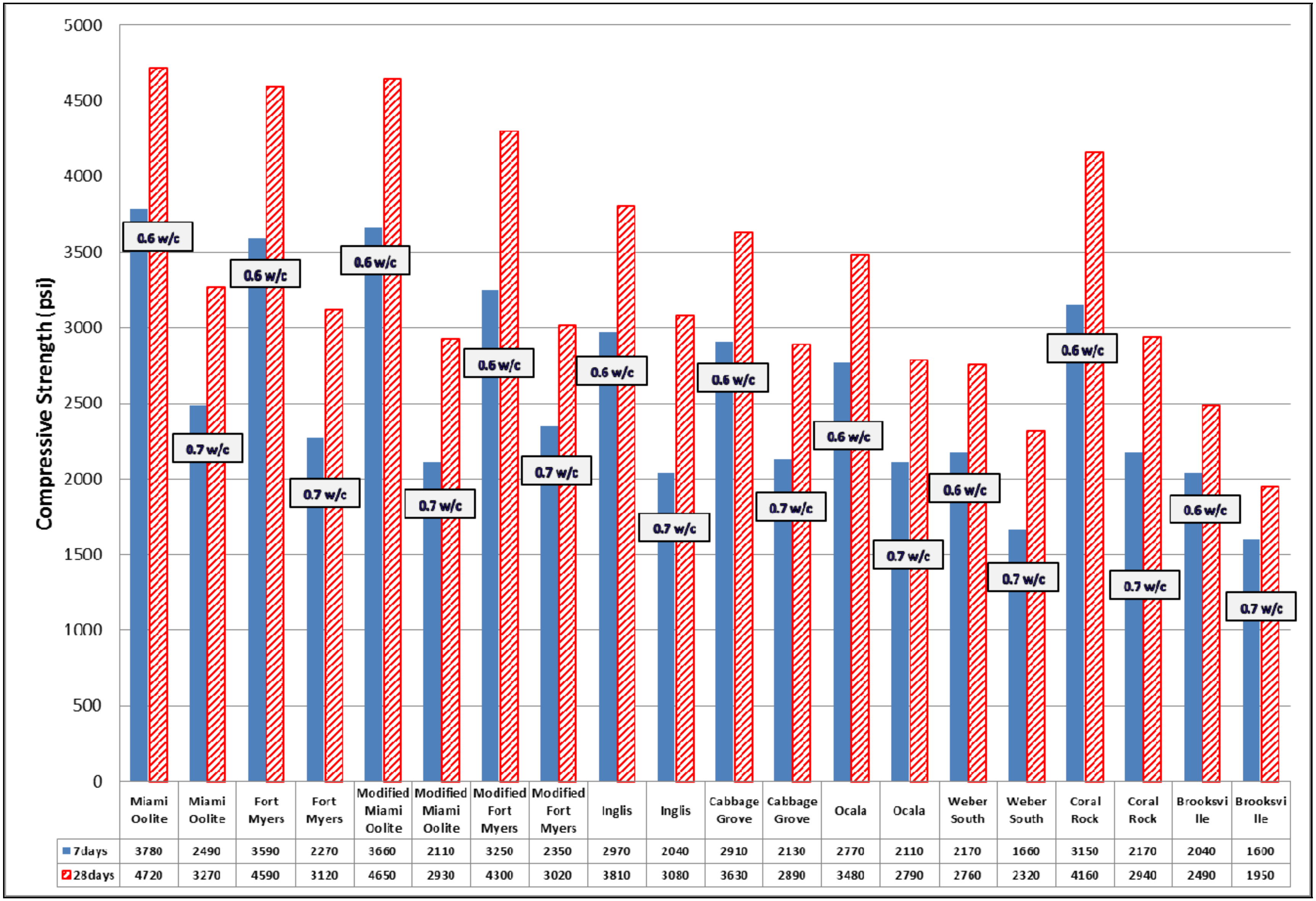


Figure 4-2. Compressive strength in the second experimental design.

Table 4-3. Compressive Strength of Concrete after Loading to 40% of Ultimate Strength and Unloading in the First Experimental Design

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Cement (lb/yd^3)	Testing Age 7 days	Testing Age 28 days
				Compressive Strength (psi)	
Miami Oolite-1	1	0.5	600	4920	5960
	2	0.6	545	3770	4540
Fort Myers-1	1	0.5	600	4740	5910
	2	0.6	545	3720	4950
Inglis-1	1	0.5	600	3510	4880
	2	0.6	545	2840	4010
Cabbage Grove-1	1	0.5	600	4450	5270
	2	0.6	545	2940	4080
Ocala-1	1	0.5	600	4760	5000
	2	0.6	545	3660	3950

Table 4-4. Compressive Strength of Concrete after Loading to 40% of Ultimate Strength and Unloading in the Second Experimental Design

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Cement (lb/yd^3)	Testing Age 7 days	Testing Age 28 days
				Compressive Strength (psi)	
Miami Oolite	1	0.6	470	3680	4840
	2	0.7	403	2300	3430
Fort Myers	1	0.6	470	3600	4720
	2	0.7	403	2360	3110
Modified Miami Oolite	1	0.6	470	3530	4720
	2	0.7	403	2100	2850
Modified Fort Myers	1	0.6	470	3250	4370
	2	0.7	403	2390	3220
Inglis	1	0.6	470	3040	3980
	2	0.7	403	2140	2860
Cabbage Grove	1	0.6	470	3090	3610
	2	0.7	403	2220	2770
Ocala	1	0.6	470	2670	3480
	2	0.7	403	2340	2820
Weber South	1	0.6	470	2130	2990
	2	0.7	403	1680	2280
Coral Rock	1	0.6	470	3250	4190
	2	0.7	403	2190	2930
Brooksville	1	0.6	470	2010	2440
	2	0.7	403	1520	2010

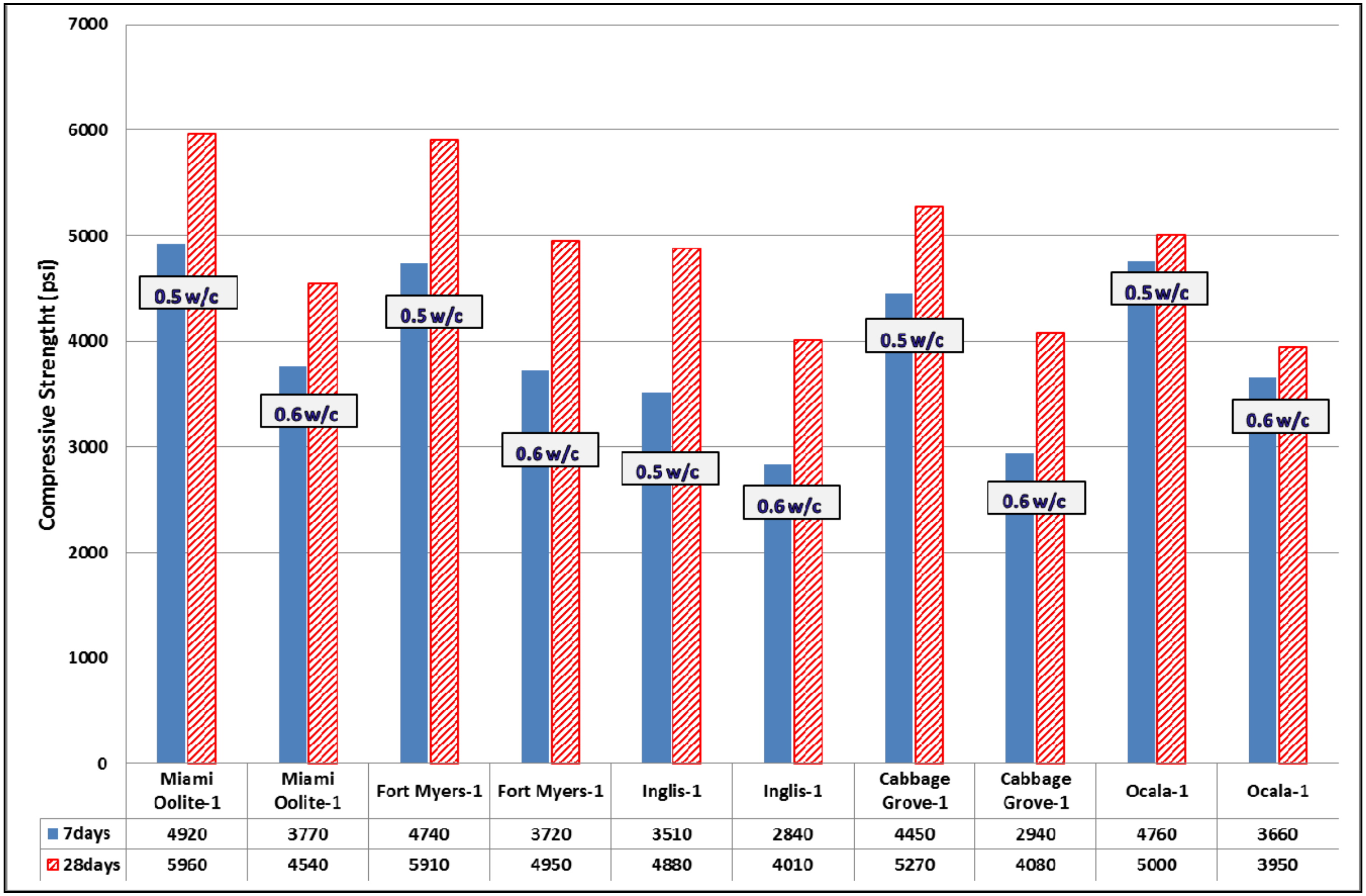
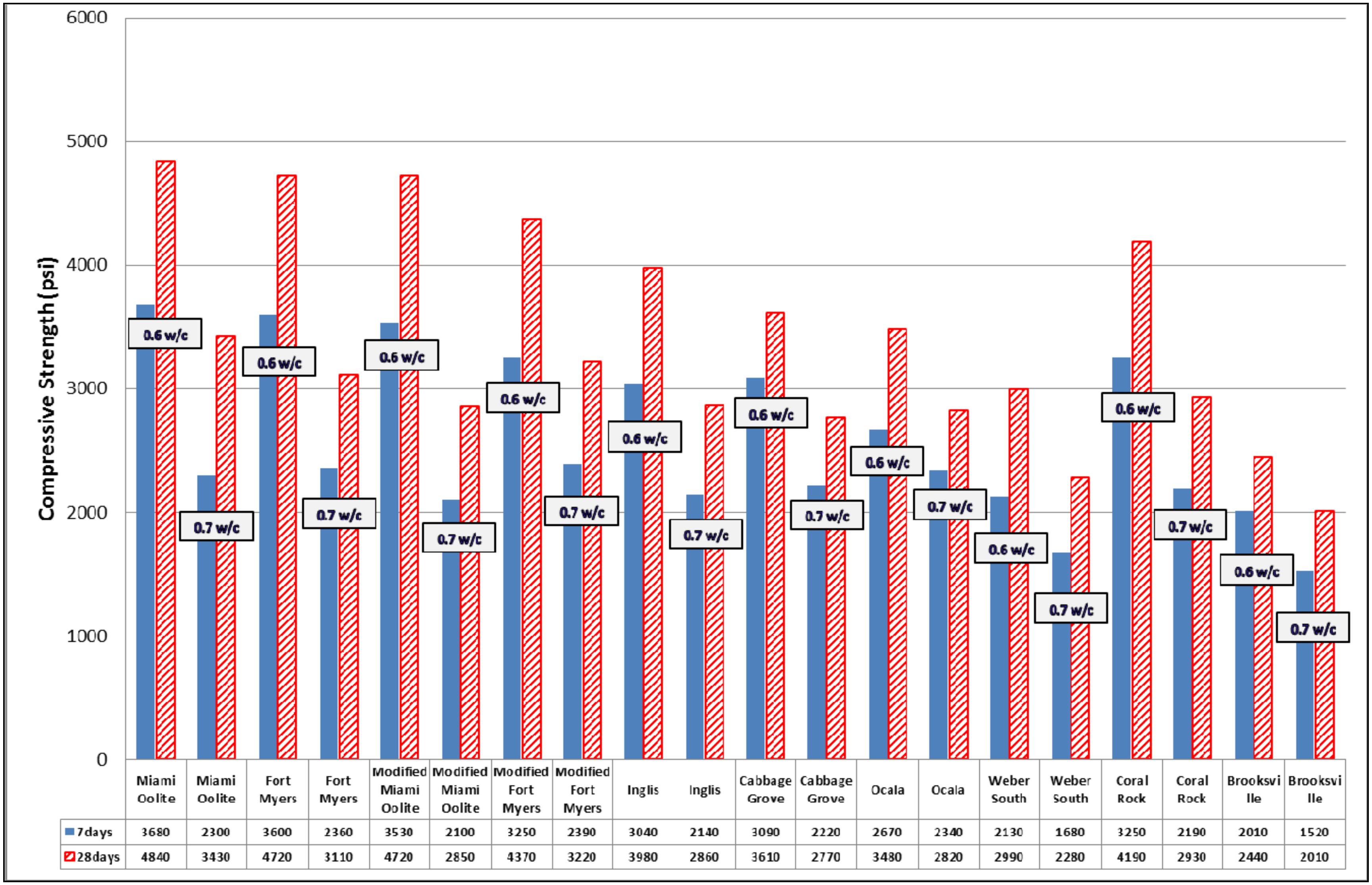


Figure 4-3. Compressive strength of concrete after loading to 40% of ultimate strength and unloading in the first experimental design.



	Miami Oolite	Miami Oolite	Fort Myers	Fort Myers	Modified Miami Oolite	Modified Miami Oolite	Modified Fort Myers	Modified Fort Myers	Inglis	Inglis	Cabbage Grove	Cabbage Grove	Ocala	Ocala	Weber South	Weber South	Coral Rock	Coral Rock	Brooksvi lle	Brooksvi lle
7days	3680	2300	3600	2360	3530	2100	3250	2390	3040	2140	3090	2220	2670	2340	2130	1680	3250	2190	2010	1520
28days	4840	3430	4720	3110	4720	2850	4370	3220	3980	2860	3610	2770	3480	2820	2990	2280	4190	2930	2440	2010

Figure 4-4. Compressive strength of concrete after loading to 40% of ultimate strength and unloading in the second experimental design.

Table 4-5. Comparison between Compressive Strength Determined Using Standard ASTM C-39 Method and Strength Determined after Loading to 40% of the Ultimate Stress for Concrete Containing Low Cement Content (Second Experimental Design)

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Cement (lb/yd^3)	28-day Compressive Strength (psi)	28-day Compressive Strength after Loading to 40% of Ultimate Load and Unloading (psi)	Percentage Difference
Miami Oolite-1	1	0.5	600	6170	5960	3%
	2	0.6	545	4780	4540	5%
Fort Myers-1	1	0.5	600	6060	5910	2%
	2	0.6	545	4790	4950	–3%
Inglis-1	1	0.5	600	4820	4880	–1%
	2	0.6	545	4020	4010	0%
Cabbage Grove-1	1	0.5	600	5250	5270	0%
	2	0.6	545	4030	4080	–1%
Ocala-1	1	0.5	600	5010	5000	0%
	2	0.6	545	3820	3950	–3%
Miami Oolite	1	0.6	470	4720	4840	–3%
	2	0.7	403	3270	3430	–5%
Fort Myers	1	0.6	470	4590	4720	–3%
	2	0.7	403	3120	3110	0%
Modified Miami Oolite	1	0.6	470	4650	4720	–2%
	2	0.7	403	2930	2850	3%
Modified Fort Myers	1	0.6	470	4300	4370	–2%
	2	0.7	403	3020	3220	–7%
Inglis	1	0.6	470	3810	3980	–4%
	2	0.7	403	3080	2860	7%
Cabbage Grove	1	0.6	470	3630	3610	1%
	2	0.7	403	2890	2770	4%
Ocala	1	0.6	470	3480	3480	0%
	2	0.7	403	2790	2820	–1%
Weber South	1	0.6	470	2760	2990	–8%
	2	0.7	403	2320	2280	2%
Coral Rock	1	0.6	470	4160	4190	–1%
	2	0.7	403	2940	2930	0%
Brooksville	1	0.6	470	2490	2440	2%
	2	0.7	403	1950	2010	–3%

4.2.2 Splitting Tensile Strength Results

The average splitting tensile strength at 7 and 28 days of the different concrete mixtures from experimental designs 1 and 2, are presented in Tables 4-6 and 4-7, respectively, while Figures 4-5 and 4-6 show plots of the results. The individual results are shown in Tables D-3 and D-4 in Appendix D.

Table 4-6. Splitting Tensile Strength of Concrete from the First Experimental Design

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Cement (lb/yd^3)	Testing Age: 7 days	Testing Age: 28 days
				Splitting Tensile Strength (psi)	
Miami Oolite-1	1	0.5	600	415	505
	2	0.6	545	410	455
Fort Myers-1	1	0.5	600	500	530
	2	0.6	545	400	465
Inglis-1	1	0.5	600	430	475
	2	0.6	545	320	445
Cabbage Grove-1	1	0.5	600	440	440
	2	0.6	545	355	395
Ocala-1	1	0.5	600	450	555
	2	0.6	545	325	370

Table 4-7. Splitting Tensile Strength of Concrete from the Second Experimental Design

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Cement (lb/yd^3)	Testing Age: 7 days	Testing Age: 28 days
				Splitting Tensile Strength (psi)	
Miami Oolite	1	0.6	470	440	440
	2	0.7	403	385	395
Fort Myers	1	0.6	470	420	460
	2	0.7	403	335	405
Modified Miami Oolite	1	0.6	470	440	450
	2	0.7	403	250	345
Modified Fort Myers	1	0.6	470	380	420
	2	0.7	403	310	360
Inglis	1	0.6	470	375	390
	2	0.7	403	275	330
Cabbage Grove	1	0.6	470	345	425
	2	0.7	403	250	290
Ocala	1	0.6	470	320	370
	2	0.7	403	280	345
Weber South	1	0.6	470	265	360
	2	0.7	403	240	315
Coral Rock	1	0.6	470	345	485
	2	0.7	403	290	360
Brooksville	1	0.6	470	240	275
	2	0.7	403	200	255

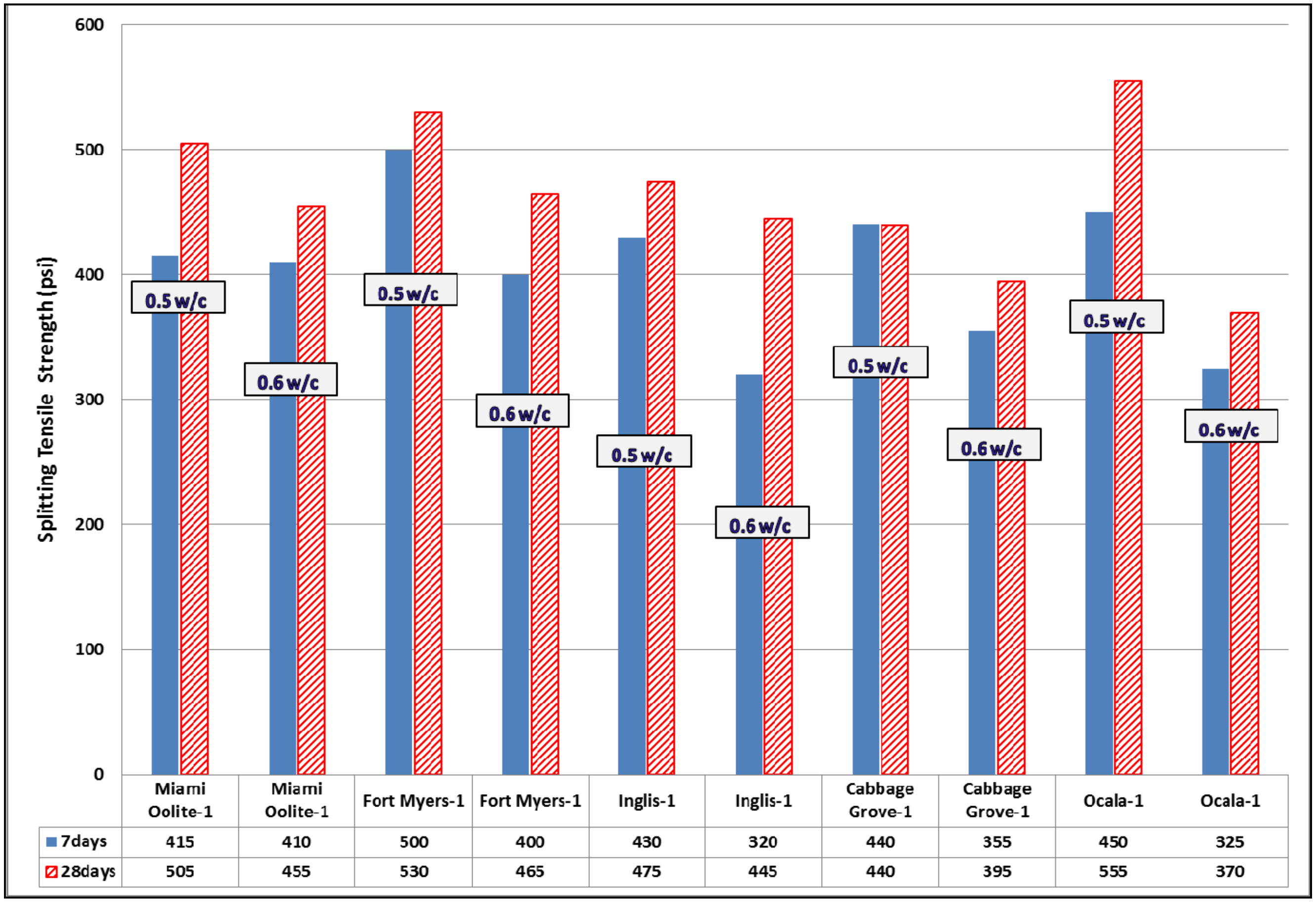


Figure 4-5. Splitting tensile strength of concrete from the first experimental design.

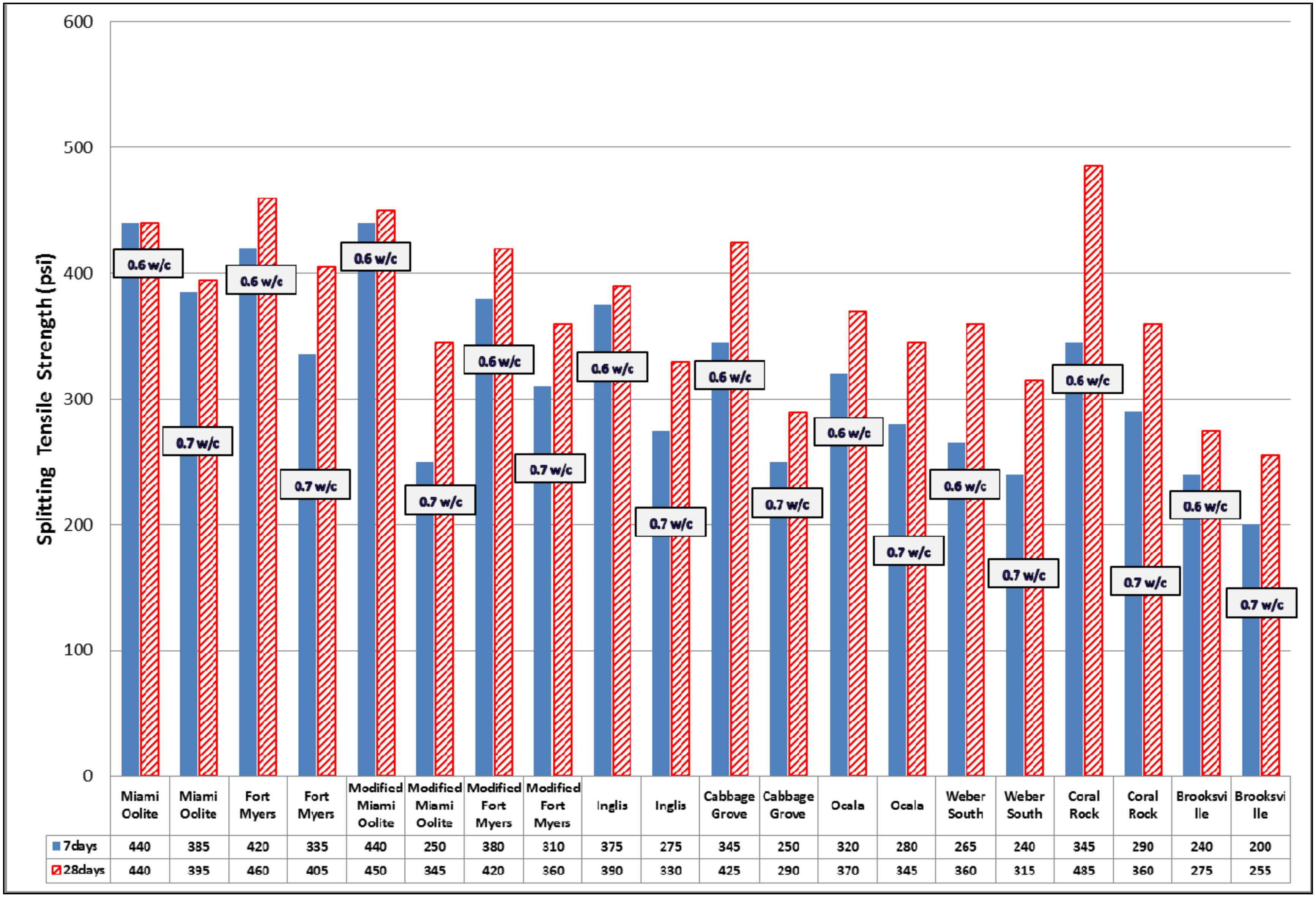


Figure 4-6. Splitting tensile strength of concrete from the second experimental design.

4.2.3 Flexural Strength Results

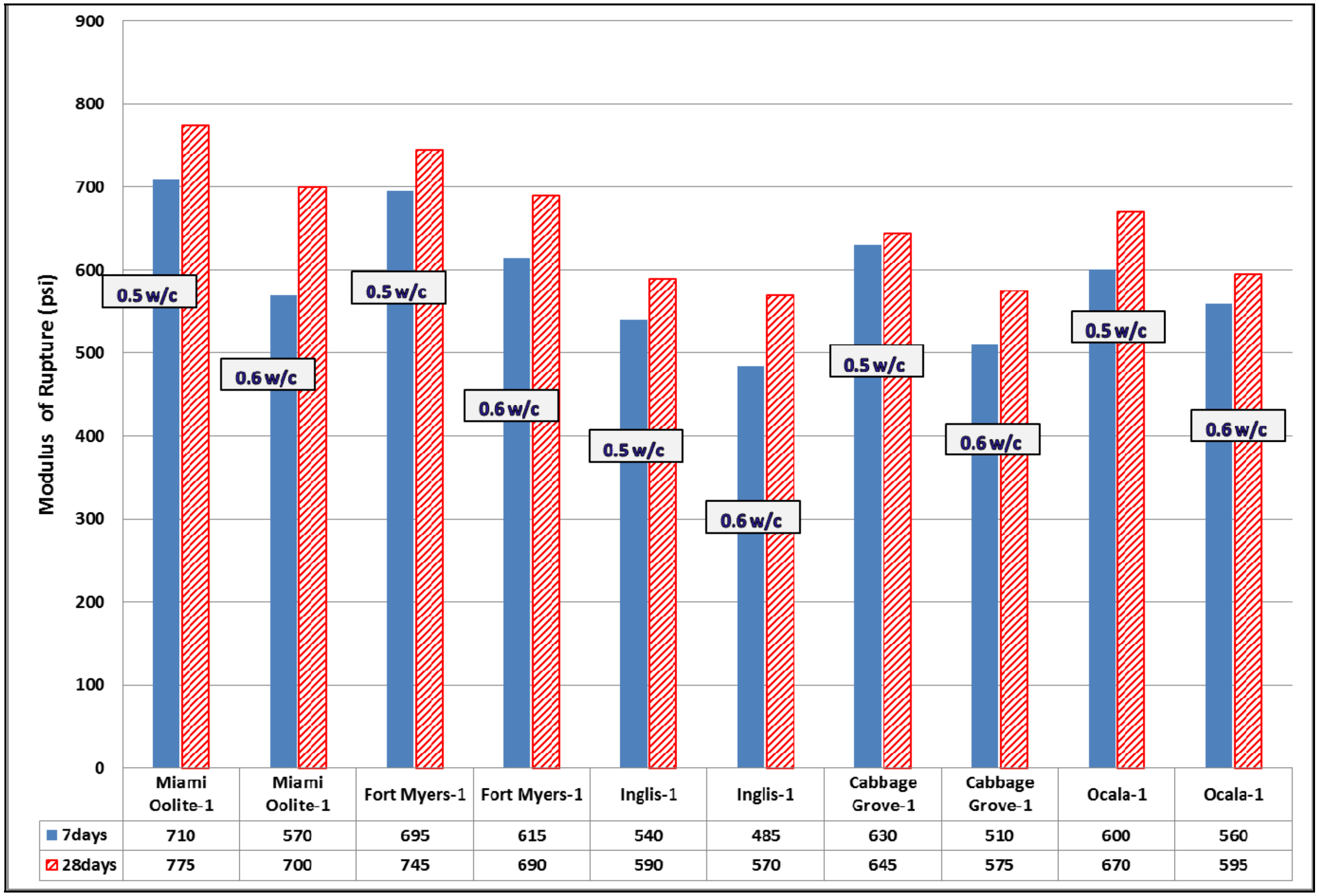
The average flexural strength at 7 and 28 days of the different concrete mixtures from experimental designs 1 and 2 are presented in Tables 4-8 and 4-9, respectively, while Figures 4-7 and 4-8 show plots of the results. The individual results are shown in Tables D-5 and D-6 in Appendix D.

Table 4-8. Flexural Strength of Concrete from the First Experimental Design

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Cement (lb/yd^3)	Testing Age	
				7 days	28 days
				Flexural Strength (psi)	
Miami Oolite-1	1	0.5	600	710	775
	2	0.6	545	570	700
Fort Myers-1	1	0.5	600	695	745
	2	0.6	545	615	690
Inglis-1	1	0.5	600	540	590
	2	0.6	545	485	570
Cabbage Grove-1	1	0.5	600	630	645
	2	0.6	545	510	575
Ocala-1	1	0.5	600	600	670
	2	0.6	545	560	595

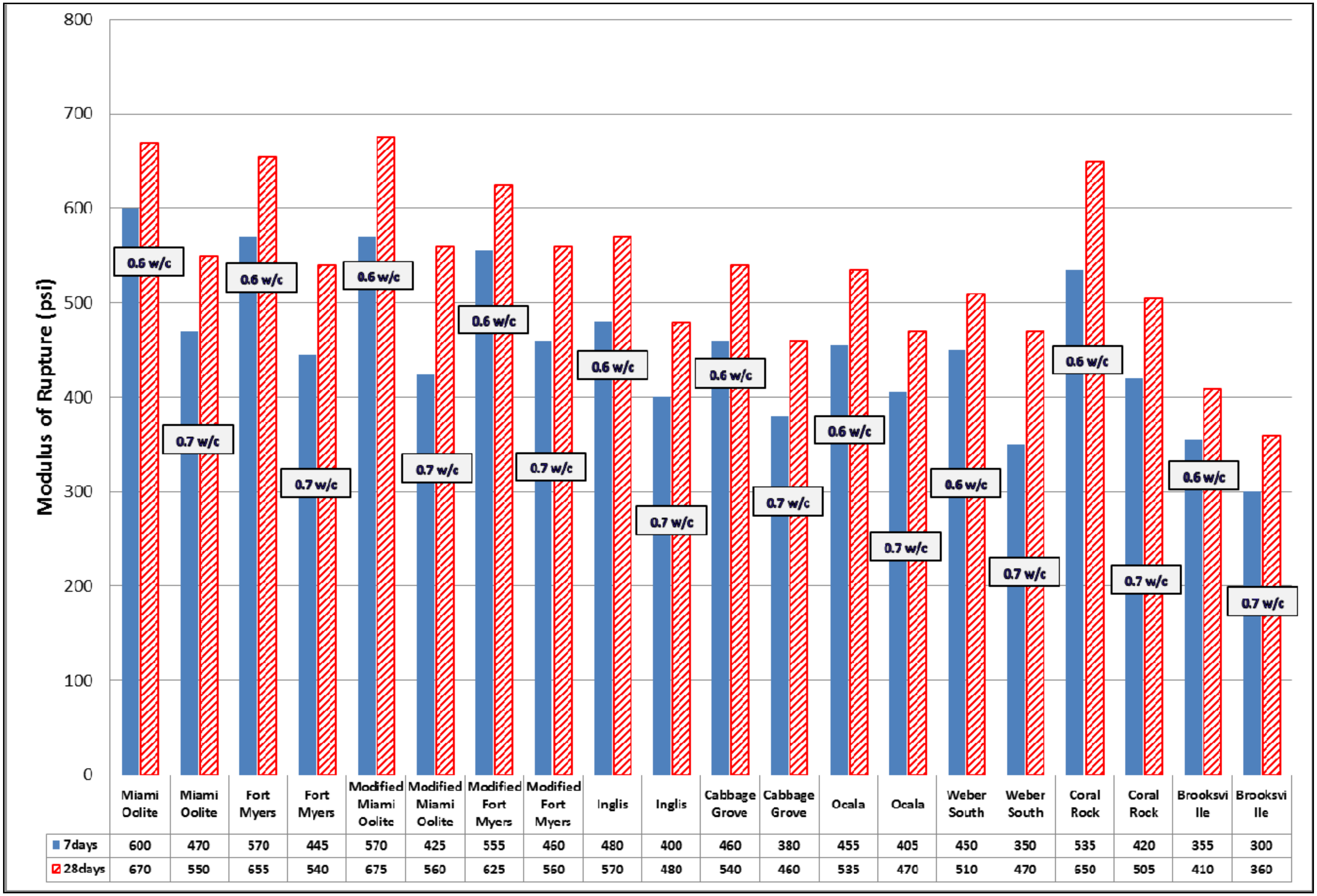
Table 4-9. Flexural Strength of Concrete from the Second Experimental Design

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Cement (lb/yd^3)	Testing Age: 7 days	Testing Age: 28 days
				Flexural Strength (psi)	
Miami Oolite	1	0.6	470	600	670
	2	0.7	403	470	550
Fort Myers	1	0.6	470	570	655
	2	0.7	403	445	540
Modified Miami Oolite	1	0.6	470	570	675
	2	0.7	403	425	560
Modified Fort Myers	1	0.6	470	555	625
	2	0.7	403	460	560
Inglis	1	0.6	470	480	570
	2	0.7	403	400	480
Cabbage Grove	1	0.6	470	460	540
	2	0.7	403	380	460
Ocala	1	0.6	470	455	535
	2	0.7	403	405	470
Weber South	1	0.6	470	450	510
	2	0.7	403	350	470
Coral Rock	1	0.6	470	535	650
	2	0.7	403	420	505
Brooksville	1	0.6	470	355	410
	2	0.7	403	300	360



	Miami Oolite-1	Miami Oolite-1	Fort Myers-1	Fort Myers-1	Inglis-1	Inglis-1	Cabbage Grove-1	Cabbage Grove-1	Ocala-1	Ocala-1
7days	710	570	695	615	540	485	630	510	600	560
28days	775	700	745	690	590	570	645	575	670	595

Figure 4-7. Flexural strength of concrete from the first experimental design.



	Miami Oolite	Miami Oolite	Fort Myers	Fort Myers	Modified Miami Oolite	Modified Miami Oolite	Modified Fort Myers	Modified Fort Myers	Inglis	Inglis	Cabbage Grove	Cabbage Grove	Ocala	Ocala	Weber South	Weber South	Coral Rock	Coral Rock	Brooksville	Brooksville
7days	600	470	570	445	570	425	555	460	480	400	460	380	455	405	450	350	535	420	355	300
28days	670	550	655	540	675	560	625	560	570	480	540	460	535	470	510	470	650	505	410	360

Figure 4-8. Flexural strength of concrete from the second experimental design.

4.2.4 Modulus of Elasticity and Poisson's Ratio Results

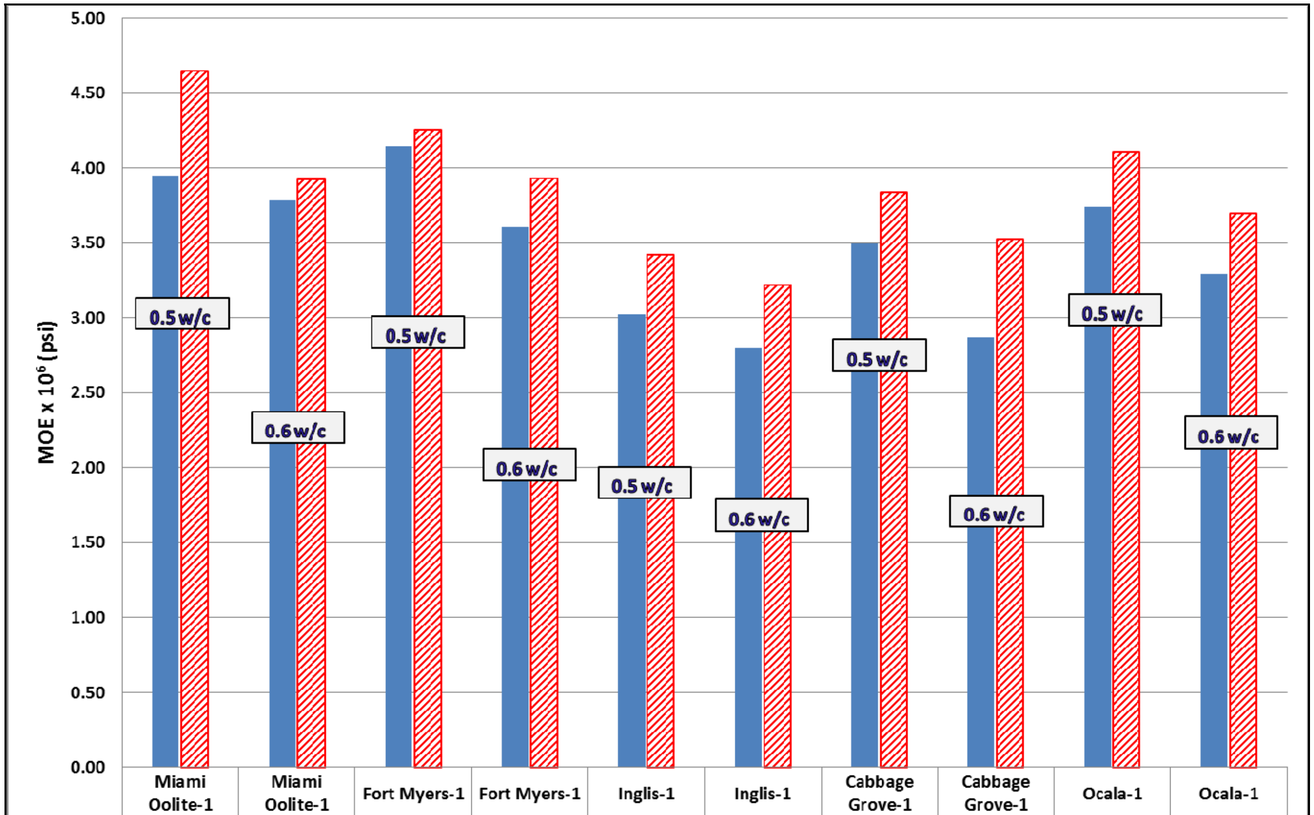
The average modulus of elasticity and Poisson's ratio at 7 and 28 days of the different concrete mixtures from experimental designs 1 and 2 are presented in Tables 4-10 and 4-11, respectively, while Figures 4-9 and 4-10 show plots of the elastic modulus results. Figures 4-11 and 4-12 show plots of the Poisson's ratio. The individual results are shown in Tables D-7 and D-8 in Appendix D.

Table 4-10. Modulus of Elasticity and Poisson's Ratio of Concrete from the First Experimental Design Content

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Cement (lb/yd^3)	Testing Age			
				7 days		28 days	
				MOE ($\times 10^6$ psi)	Poisson's Ratio	MOE ($\times 10^6$ psi)	Poisson's Ratio
Miami Oolite-1	1	0.5	600	3.95	0.25	4.65	0.25
	2	0.6	545	3.78	0.27	3.93	0.27
Fort Myers-1	1	0.5	600	4.14	0.28	4.26	0.28
	2	0.6	545	3.61	0.26	3.93	0.28
Inglis-1	1	0.5	600	3.02	0.28	3.42	0.28
	2	0.6	545	2.80	0.28	3.22	0.28
Cabbage Grove-1	1	0.5	600	3.49	0.23	3.84	0.29
	2	0.6	545	2.87	0.23	3.53	0.31
Ocala-1	1	0.5	600	3.74	0.30	4.11	0.31
	2	0.6	545	3.29	0.29	3.70	0.31

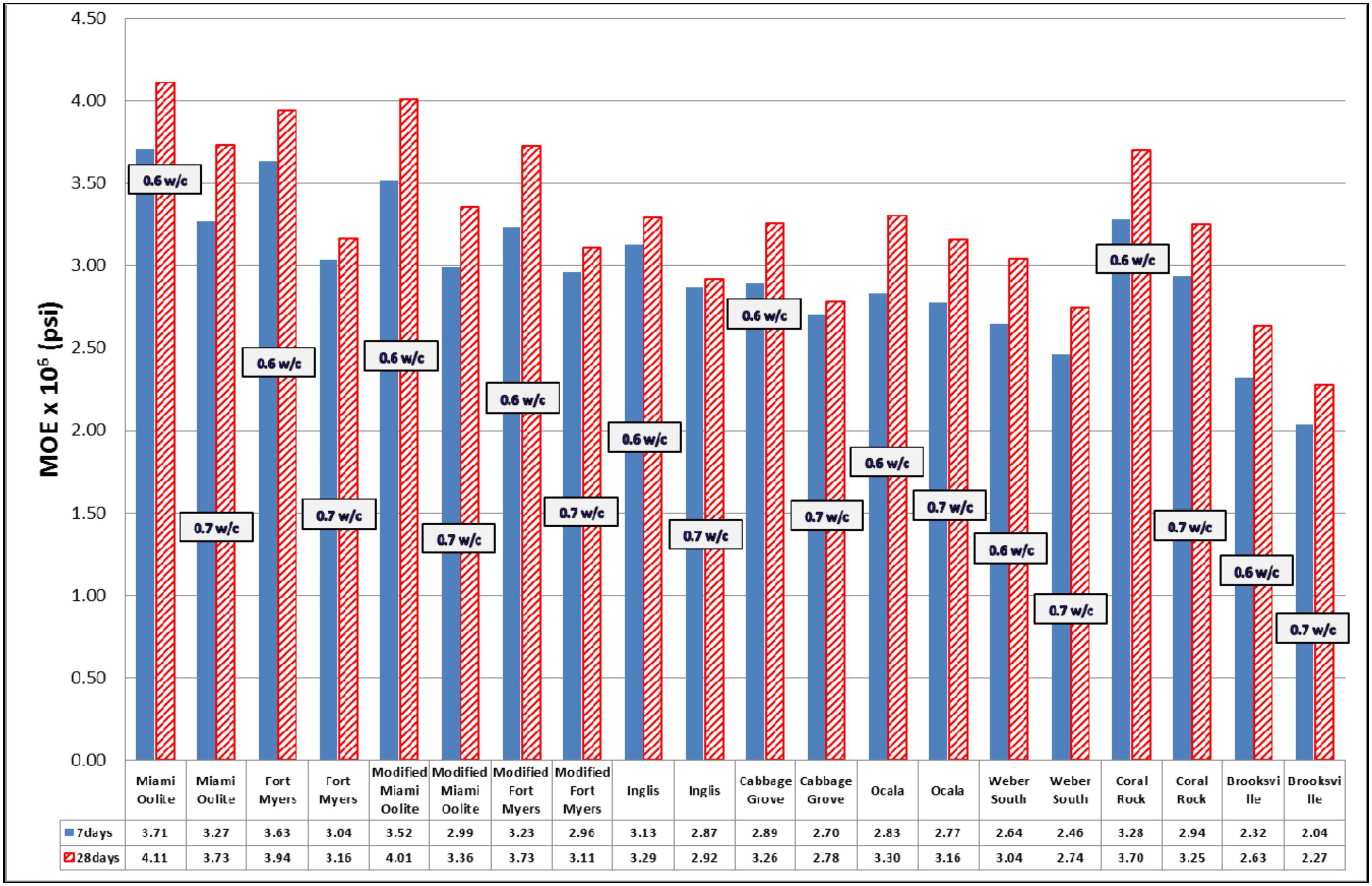
Table 4-11. Modulus of Elasticity and Poisson's Ratio of Concrete from the Second Experimental Design

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Cement (lb/yd^3)	Testing Age: 7 days MOE ($\times 10^6$ psi)	Testing Age: 7 days Poisson's Ratio	Testing Age: 28 days MOE ($\times 10^6$ psi)	Testing Age: 28 days Poisson's Ratio
Miami Oolite	1	0.6	470	3.71	0.23	4.11	0.24
	2	0.7	403	3.27	0.24	3.73	0.25
Fort Myers	1	0.6	470	3.63	0.26	3.94	0.27
	2	0.7	403	3.04	0.21	3.16	0.26
Modified Miami Oolite	1	0.6	470	3.52	0.24	4.01	0.25
	2	0.7	403	2.99	0.24	3.36	0.26
Modified Fort Myers	1	0.6	470	3.23	0.22	3.73	0.26
	2	0.7	403	2.96	0.22	3.11	0.24
Inglis	1	0.6	470	3.13	0.22	3.29	0.22
	2	0.7	403	2.87	0.20	2.92	0.23
Cabbage Grove	1	0.6	470	2.89	0.22	3.26	0.27
	2	0.7	403	2.70	0.21	2.78	0.24
Ocala	1	0.6	470	2.83	0.22	3.30	0.27
	2	0.7	403	2.77	0.22	3.16	0.28
Weber South	1	0.6	470	2.64	0.17	3.04	0.22
	2	0.7	403	2.46	0.22	2.74	0.22
Coral Rock	1	0.6	470	3.28	0.22	3.70	0.25
	2	0.7	403	2.94	0.19	3.25	0.22
Brooksville	1	0.6	470	2.32	0.19	2.63	0.25
	2	0.7	403	2.04	0.22	2.27	0.22



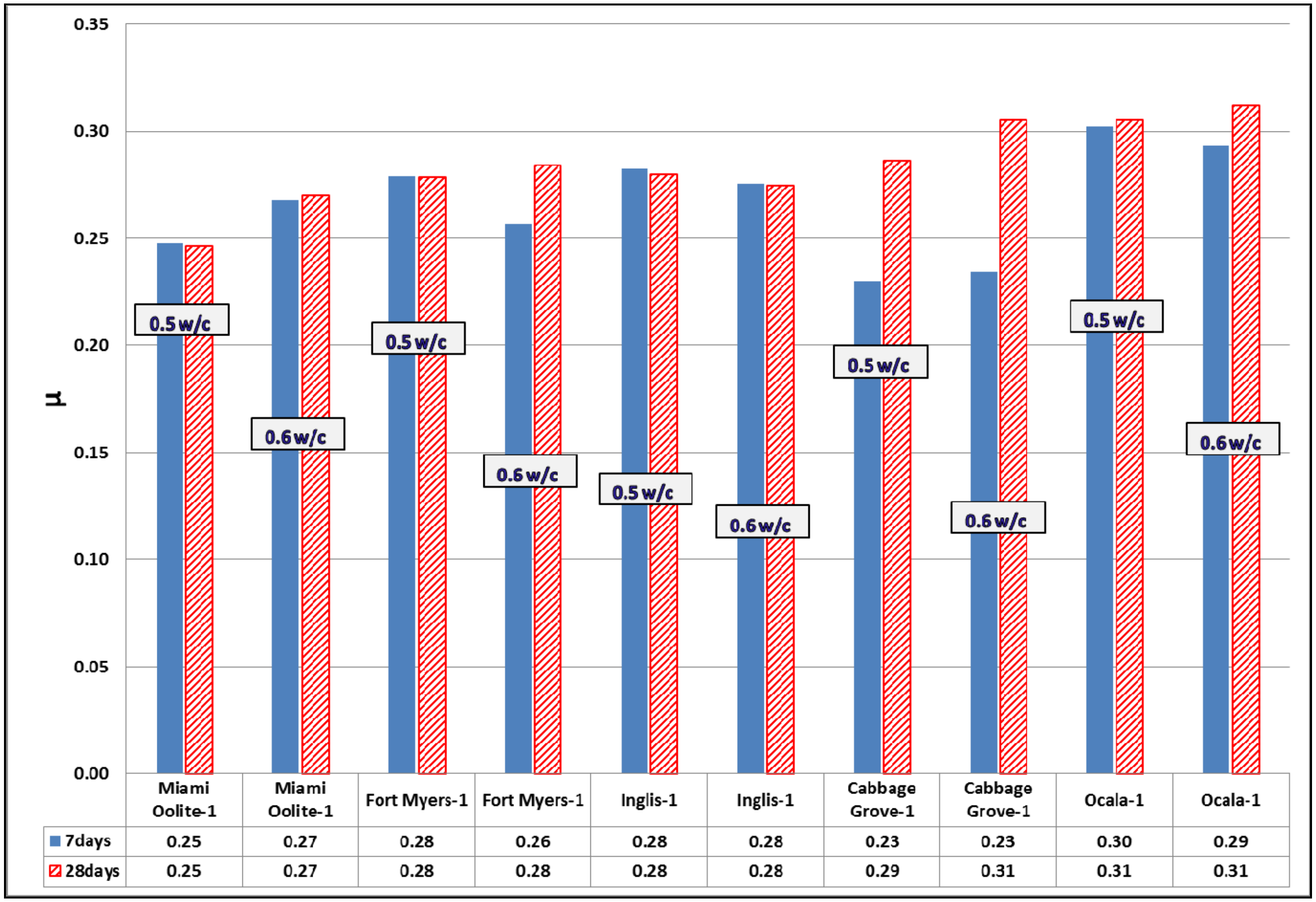
	Miami Oolite-1	Miami Oolite-1	Fort Myers-1	Fort Myers-1	Inglis-1	Inglis-1	Cabbage Grove-1	Cabbage Grove-1	Ocala-1	Ocala-1
7days	3.95	3.78	4.14	3.61	3.02	2.80	3.49	2.87	3.74	3.29
28days	4.65	3.93	4.26	3.93	3.42	3.22	3.84	3.53	4.11	3.70

Figure 4-9. Modulus of elasticity of concrete from the first experimental design.



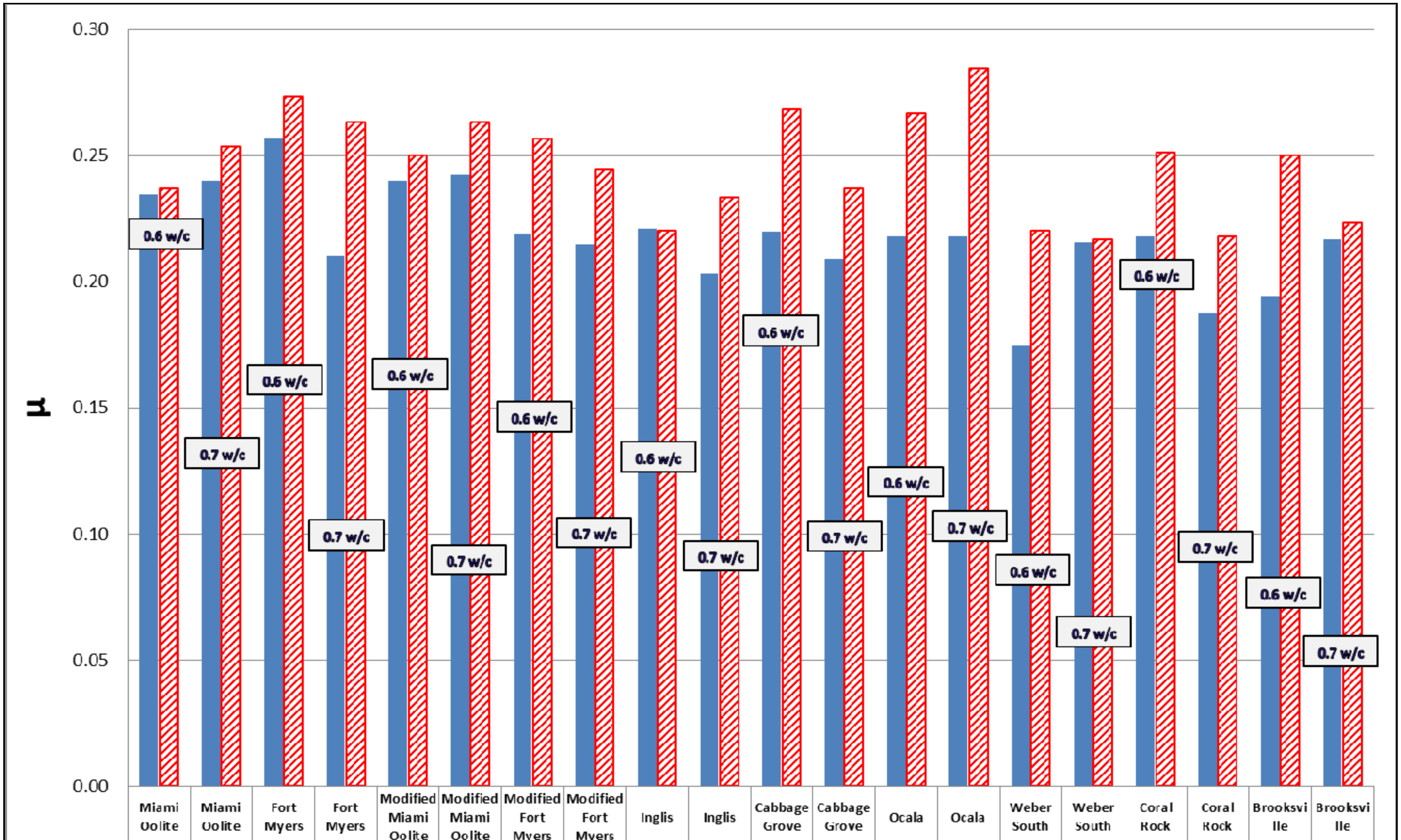
	Miami Oolite	Miami Oolite	Fort Myers	Fort Myers	Modified Miami Oolite	Modified Miami Oolite	Modified Fort Myers	Modified Fort Myers	Inglis	Inglis	Cabbage Grove	Cabbage Grove	Ocala	Ocala	Weber South	Weber South	Coral Rock	Coral Rock	Brooksville	Brooksville
7days	3.71	3.27	3.63	3.04	3.52	2.99	3.23	2.96	3.13	2.87	2.89	2.70	2.83	2.77	2.64	2.46	3.28	2.94	2.32	2.04
28days	4.11	3.73	3.94	3.16	4.01	3.36	3.73	3.11	3.29	2.92	3.26	2.78	3.30	3.16	3.04	2.74	3.70	3.25	2.63	2.27

Figure 4-10. Modulus of elasticity of concrete from the second experimental design.



	Miami Oolite-1	Miami Oolite-1	Fort Myers-1	Fort Myers-1	Inglis-1	Inglis-1	Cabbage Grove-1	Cabbage Grove-1	Ocala-1	Ocala-1
7days	0.25	0.27	0.28	0.26	0.28	0.28	0.23	0.23	0.30	0.29
28days	0.25	0.27	0.28	0.28	0.28	0.28	0.29	0.31	0.31	0.31

Figure 4-11. Poisson's ratio of concrete from the first experimental design.



	Miami Oolite	Miami Oolite	Fort Myers	Fort Myers	Modified Miami Oolite	Modified Miami Oolite	Modified Fort Myers	Modified Fort Myers	Inglis	Inglis	Cabbage Grove	Cabbage Grove	Ocala	Ocala	Weber South	Weber South	Coral Rock	Coral Rock	Brooksville	Brooksville
7days	0.23	0.24	0.26	0.21	0.24	0.24	0.22	0.22	0.22	0.20	0.22	0.21	0.22	0.22	0.17	0.22	0.22	0.19	0.19	0.22
28days	0.24	0.25	0.27	0.26	0.25	0.26	0.26	0.24	0.22	0.23	0.27	0.24	0.27	0.28	0.22	0.22	0.25	0.22	0.25	0.22

Figure 4-12. Poisson's ratio of concrete from the second experimental design.

4.2.5 Coefficient of Thermal Expansion Results

The average coefficient of thermal expansion at 28 days of the different concrete mixtures from experimental designs 1 and 2 are presented in Tables 4-12 and 4-13, respectively, while Figures 4-13 and 4-14 show a plot of the results. The individual results are shown in Tables D-13 and D-14 in Appendix D.

Table 4-12. Coefficient of Thermal Expansion of Concrete from the First Experimental Design

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Cement (lb/yd^3)	Testing Age 28 days CTE ($10^{-6}/°F$)
Miami Oolite-1	1	0.5	600	6.53
	2	0.6	545	7.74
Fort Myers-1	1	0.5	600	7.91
	2	0.6	545	8.42
Inglis-1	1	0.5	600	8.55
	2	0.6	545	9.72
Cabbage Grove-1	1	0.5	600	9.10
	2	0.6	545	9.66
Ocala-1	1	0.5	600	6.43
	2	0.6	545	7.13

Table 4-13. Coefficient of Thermal Expansion of Concrete from the Second Experimental Design

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Cement (lb/yd^3)	Testing Age 28 days CTE ($10^{-6}/°F$)
Miami Oolite	1	0.6	470	7.92
	2	0.7	403	9.73
Fort Myers	1	0.6	470	7.67
	2	0.7	403	7.92
Modified Miami Oolite	1	0.6	470	8.83
	2	0.7	403	8.98
Modified Fort Myers	1	0.6	470	8.96
	2	0.7	403	9.24
Inglis	1	0.6	470	10.70
	2	0.7	403	11.10
Cabbage Grove	1	0.6	470	9.96
	2	0.7	403	10.20
Ocala	1	0.6	470	7.59
	2	0.7	403	8.17
Weber South	1	0.6	470	9.60
	2	0.7	403	10.00
Coral Rock	1	0.6	470	9.09
	2	0.7	403	9.74
Brooksville	1	0.6	470	8.82
	2	0.7	403	10.40

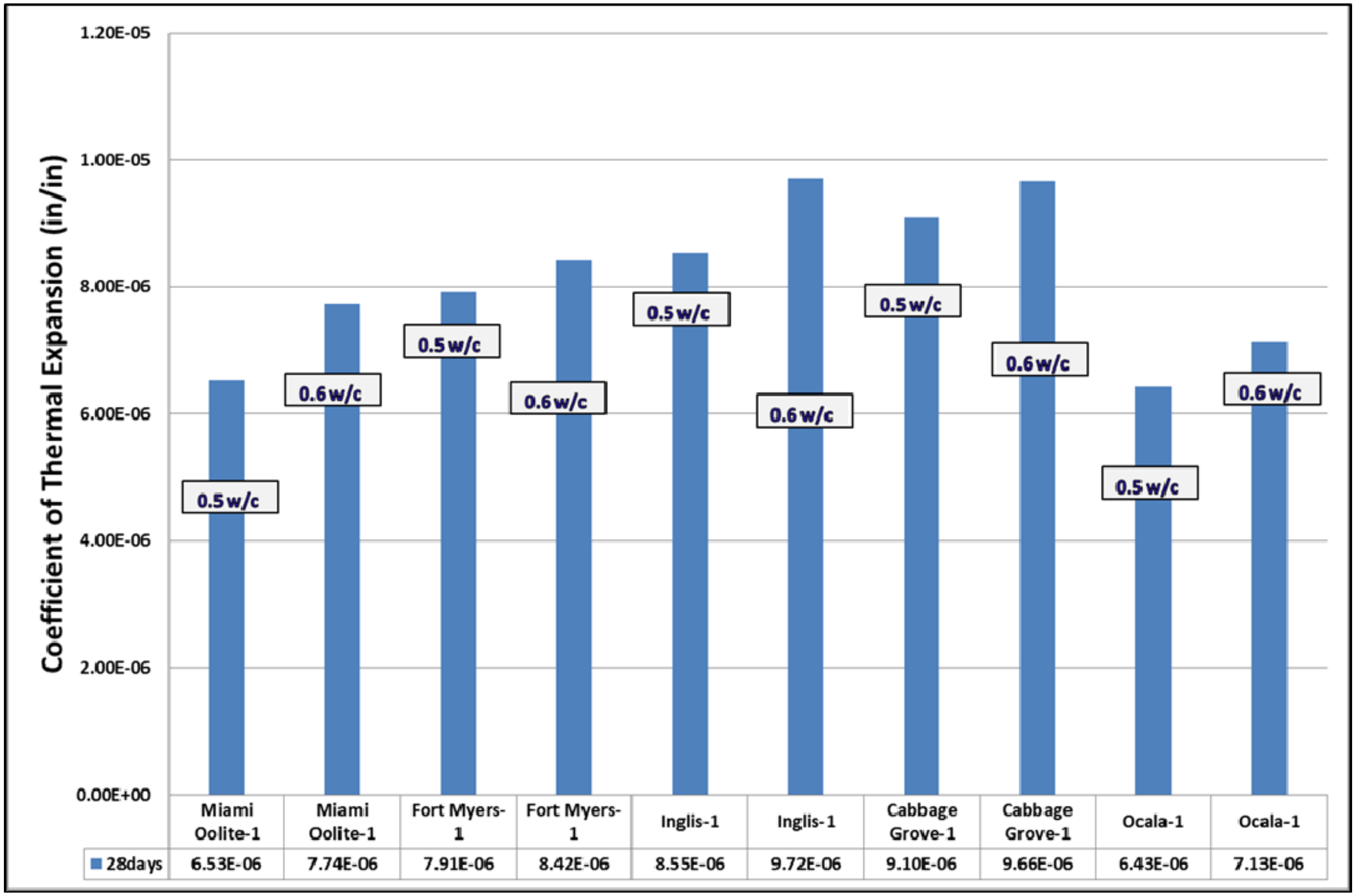


Figure 4-13. Coefficient of thermal expansion of concrete from the first experimental design.

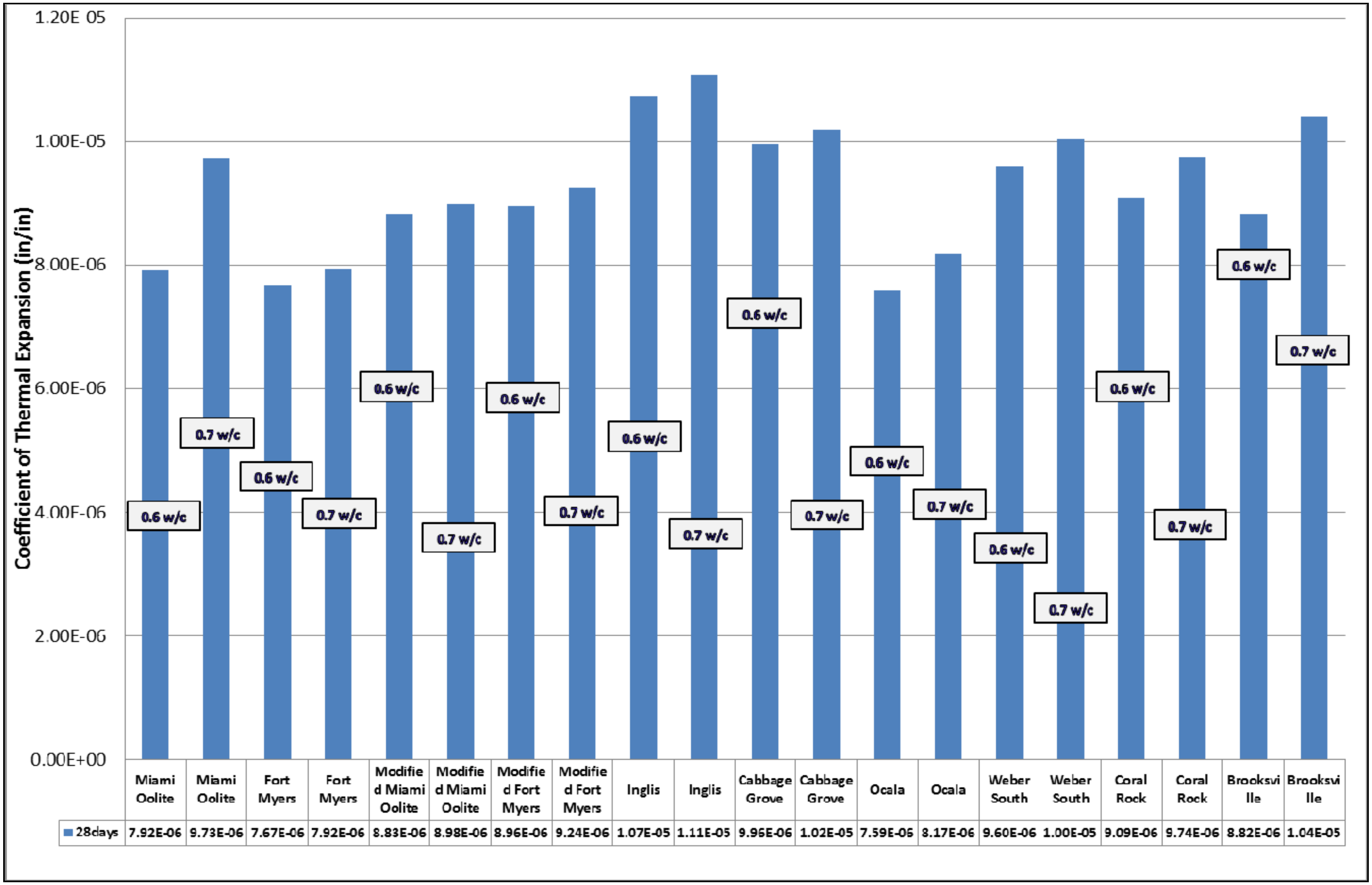


Figure 4-14. Coefficient of thermal expansion of concrete from the second experimental design.

4.2.6 Concrete Absorption Results

The average 5-hour water absorption at 28 days of the different concrete mixtures from experimental designs 1 and 2 are presented in Tables 4-14 and 4-15, respectively, while Figures 4-15 and 4-16 show plots of the results. The volumes of voids are presented in Tables 4-16 and 4-17; plots of the volume of voids are shown in Figures 4-17 and 4-18. The individual results are shown in Tables D-11 and D-12 in Appendix D.

Table 4-14. Five-Hour Absorption of Concrete from the First Experimental Design

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Cement (lb/yd^3)	Testing Age 28 days Percent (%)
Miami Oolite-1	1	0.5	600	7.46
	2	0.6	545	8.52
Fort Myers-1	1	0.5	600	8.83
	2	0.6	545	10.67
Inglis-1	1	0.5	600	10.17
	2	0.6	545	11.37
Cabbage Grove-1	1	0.5	600	10.22
	2	0.6	545	12.27
Ocala-1	1	0.5	600	9.61
	2	0.6	545	10.56

Table 4-15. Five-Hour Absorption of Concrete from the Second Experimental Design

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Cement (lb/yd^3)	Testing Age 28 days Percent (%)
Miami Oolite	1	0.6	470	8.15
	2	0.7	403	8.92
Fort Myers	1	0.6	470	8.17
	2	0.7	403	10.06
Modified Miami Oolite	1	0.6	470	9.18
	2	0.7	403	9.21
Modified Fort Myers	1	0.6	470	8.56
	2	0.7	403	8.72
Inglis	1	0.6	470	9.25
	2	0.7	403	9.76
Cabbage Grove	1	0.6	470	10.54
	2	0.7	403	10.95
Ocala	1	0.6	470	10.24
	2	0.7	403	10.28
Weber South	1	0.6	470	10.75
	2	0.7	403	11.09
Coral Rock	1	0.6	470	9.03
	2	0.7	403	9.28
Brooksville	1	0.6	470	13.38
	2	0.7	403	13.58

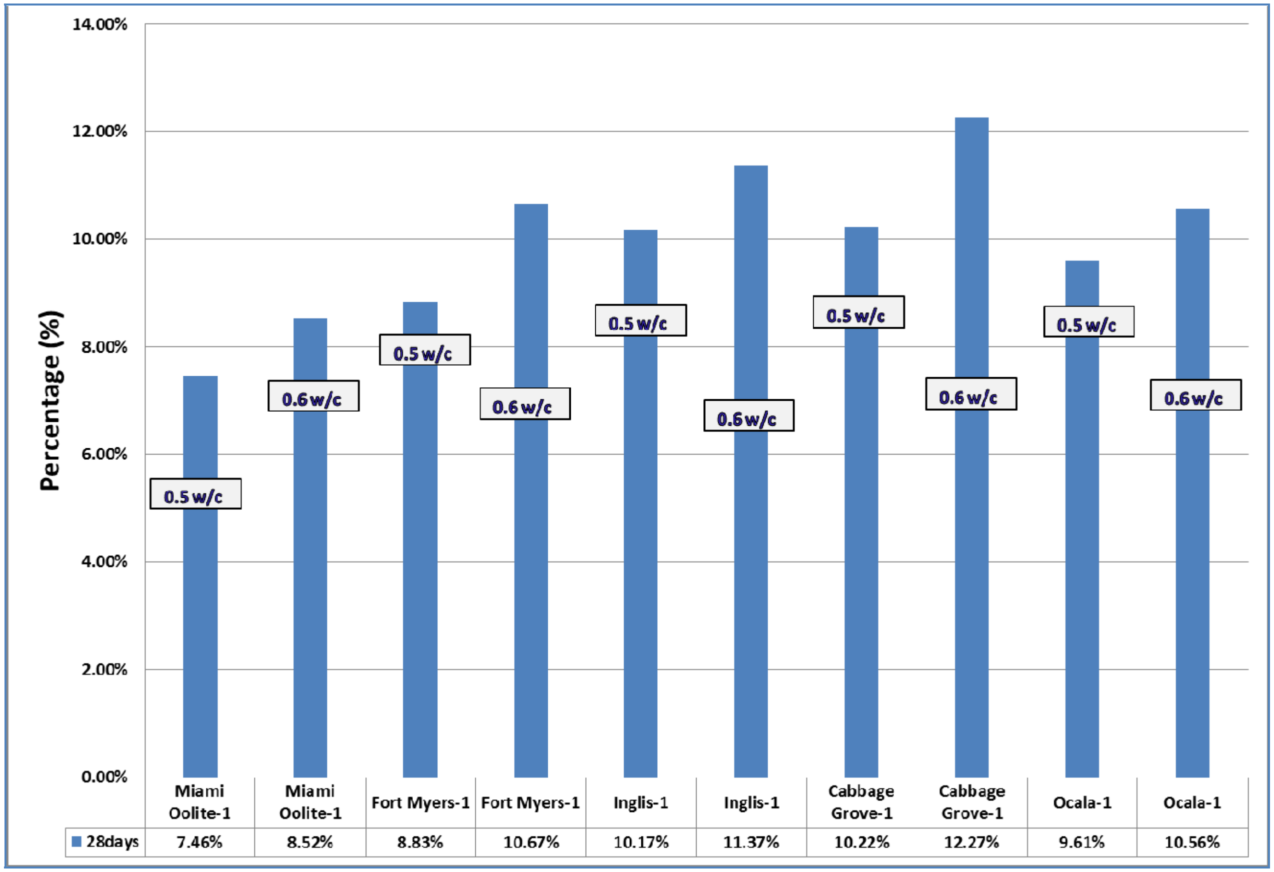


Figure 4-15. Five-hour absorption of concrete from the first experimental design.

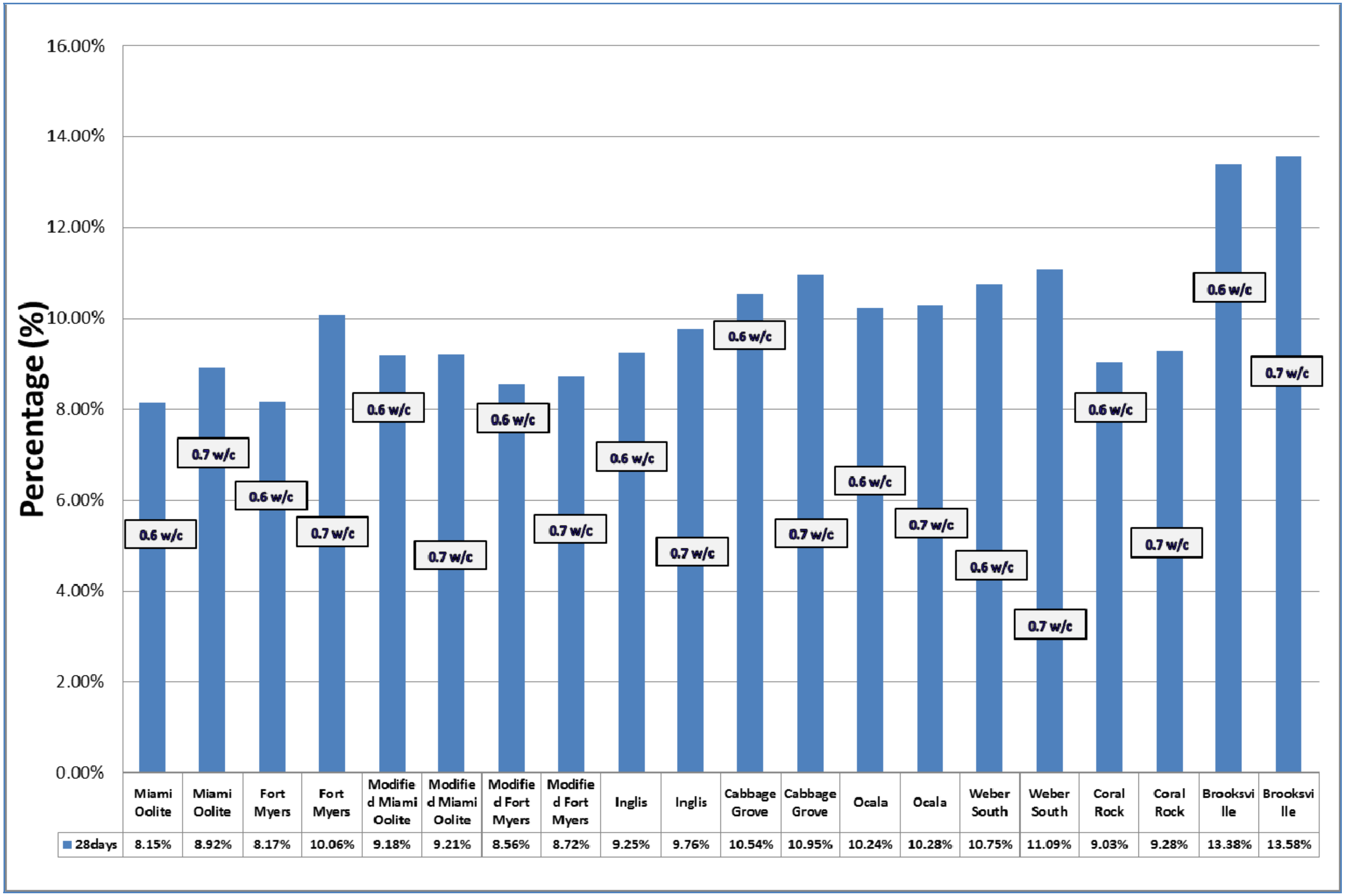


Figure 4-16. Five-hour absorption of concrete from the second experimental design.

Table 4-16. Voids of Concrete from the First Experimental Design

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Cement (lb/yd^3)	Testing Age 28 days Percent (%)
Miami Oolite-1	1	0.5	600	16.09
	2	0.6	545	17.99
Fort Myers-1	1	0.5	600	18.74
	2	0.6	545	22.16
Inglis-1	1	0.5	600	21.10
	2	0.6	545	23.07
Cabbage Grove-1	1	0.5	600	20.76
	2	0.6	545	24.13
Ocala-1	1	0.5	600	19.87
	2	0.6	545	21.56

Table 4-17. Voids of Concrete from the Second Experimental Design

Aggregate Source	Mix Number	Water/ Cement Ratio (w/c)	Cement (lb/yd^3)	Testing Age 28 days Percent (%)
Miami Oolite	1	0.6	470	17.26
	2	0.7	403	18.48
Fort Myers	1	0.6	470	17.32
	2	0.7	403	20.48
Modified Miami Oolite	1	0.6	470	18.99
	2	0.7	403	18.87
Modified Fort Myers	1	0.6	470	17.84
	2	0.7	403	18.10
Inglis	1	0.6	470	19.11
	2	0.7	403	20.04
Cabbage Grove	1	0.6	470	20.94
	2	0.7	403	21.68
Ocala	1	0.6	470	20.58
	2	0.7	403	20.67
Weber South	1	0.6	470	21.72
	2	0.7	403	21.94
Coral Rock	1	0.6	470	18.76
	2	0.7	403	18.96
Brooksville	1	0.6	470	25.08
	2	0.7	403	25.65

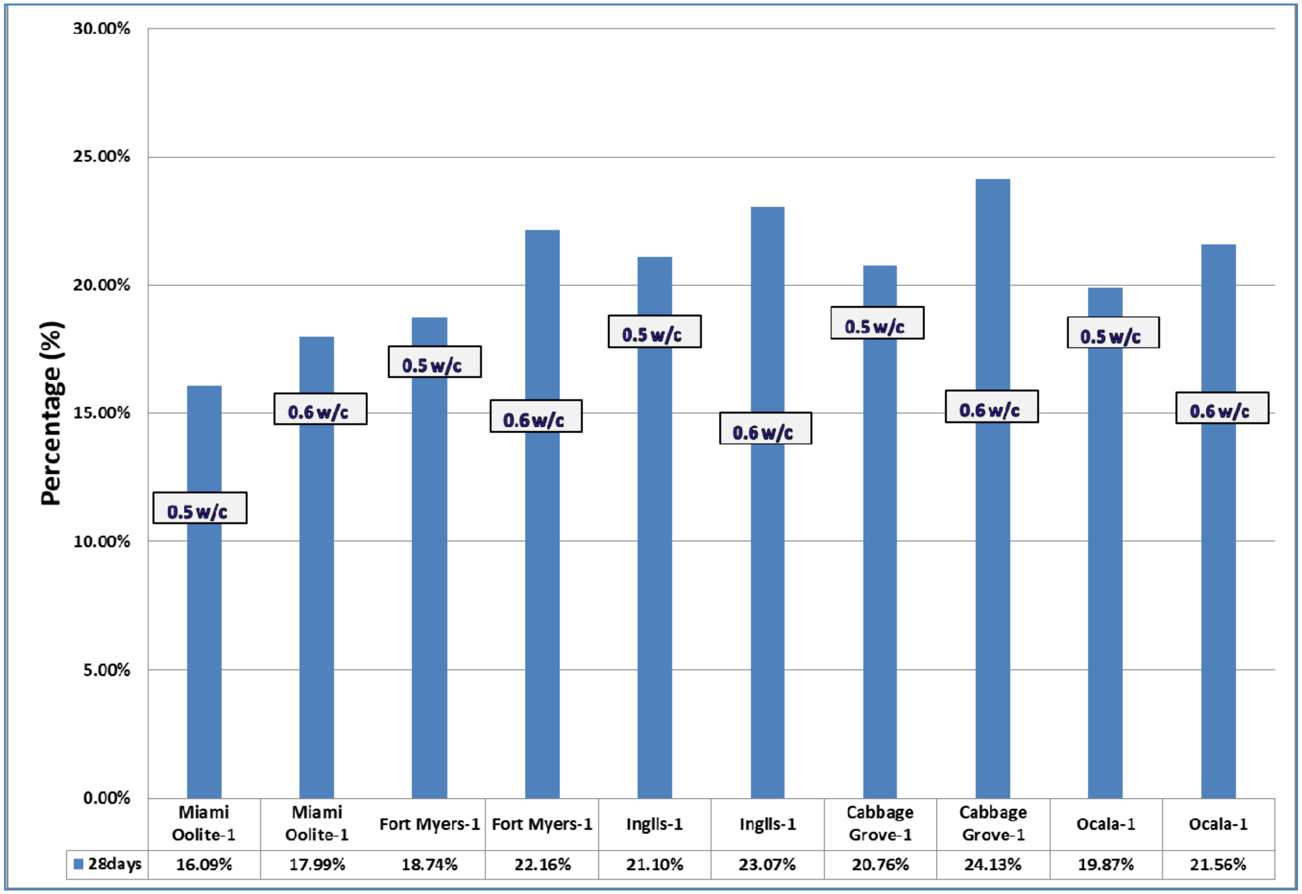


Figure 4-17. Volume of voids in concrete from the first experimental design.

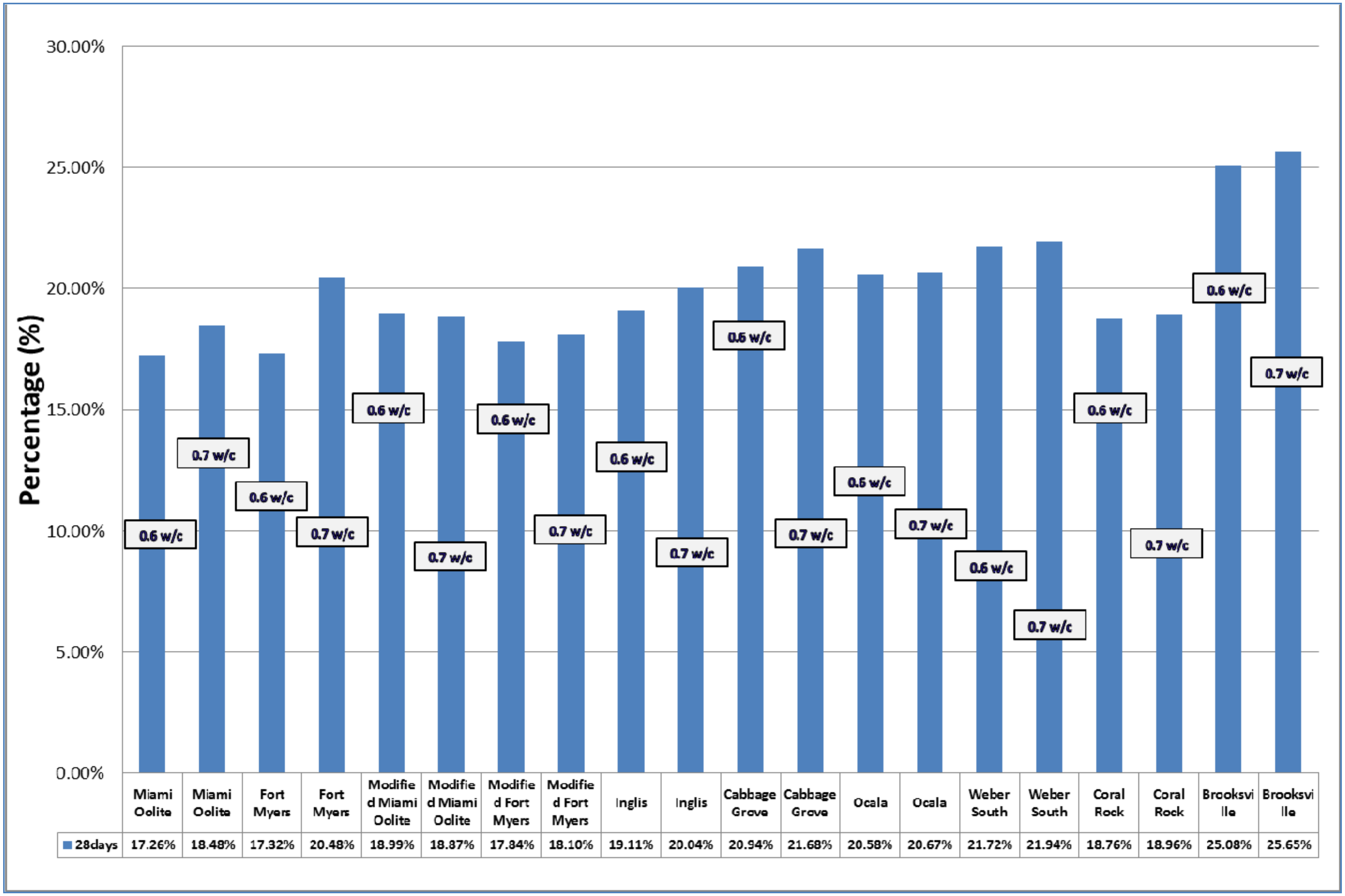


Figure 4-18. Volume of voids in concrete from the second experimental design.

4.3 Relationship between Properties of Hardened Concrete

The relationship between key properties of hardened concrete were developed using the data from the second experimental design.

4.3.1 Relationship between Compressive Strength and Modulus of Rupture

There are many relationships that have been developed to establish the relationship between the compressive strength and the modulus of rupture of concrete. However, by far, the most commonly used one is the ACI equation, which is expressed as follows:

$$f_r = 7.5\sqrt{f'_c} \tag{4.1}$$

where f_r = modulus of rupture (psi); and
f'_c = compressive strength (psi).

Neville (1981) stated that the relation between these two properties depends on the type of coarse aggregate used. This is because the properties of an aggregate affect the modulus of rupture more than the compressive strength.

Regression analysis using an equation of the ACI form, i.e., $f_r = A\sqrt{f'_c}$, was performed to determine whether the above relation holds true for the data in our study. This regression analysis was performed for the two different water/cement ratios and further verified for only the concrete containing the non-standard aggregates. Figures 4-19 to 4-22 show plots of the various relationships developed. Table 4-18 shows the determined values of A.

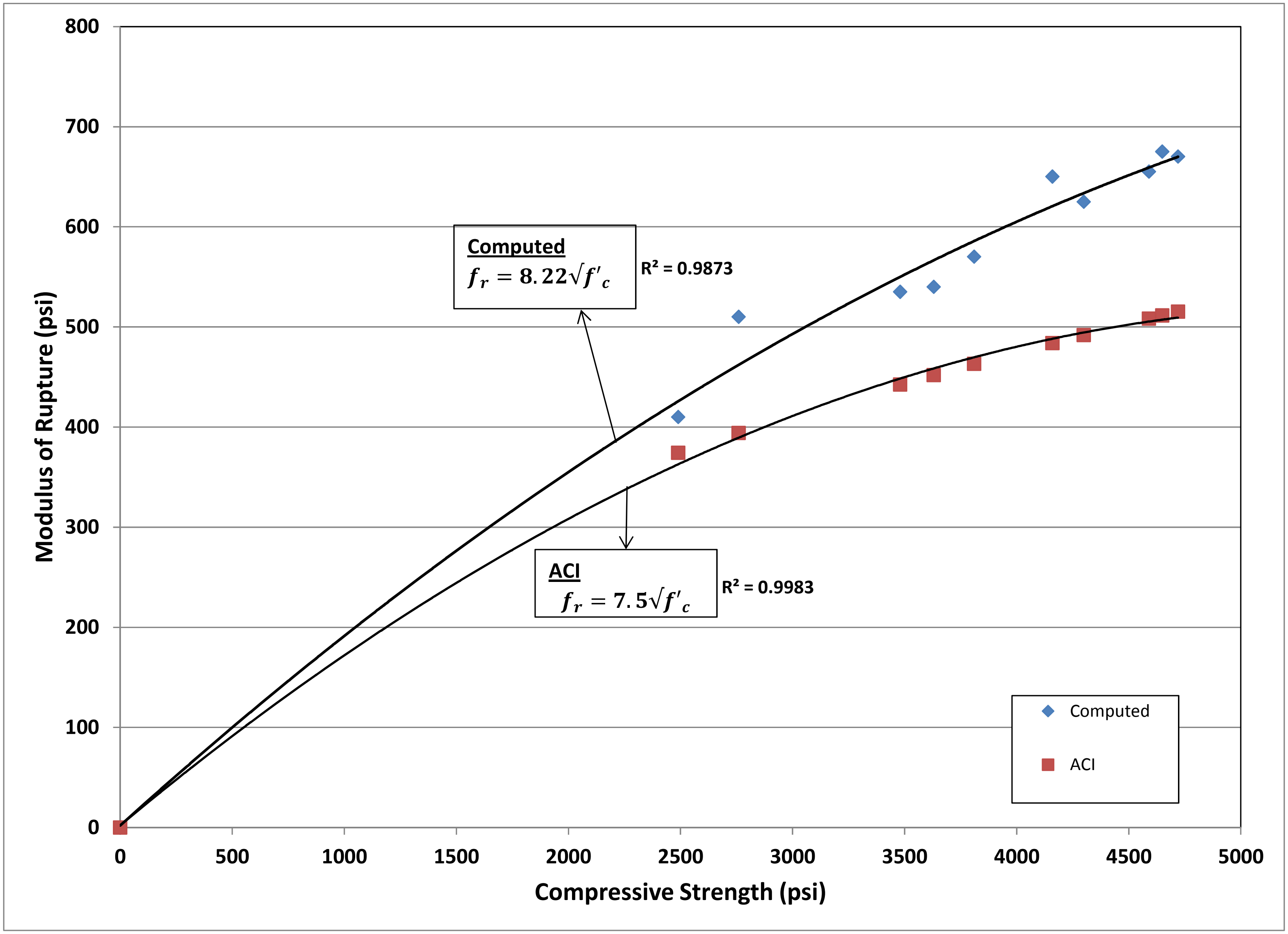


Figure 4-19. Relationship between compressive strength and modulus of rupture for concrete with w/c ratio of 0.6 at 28-day moist curing using all the data.

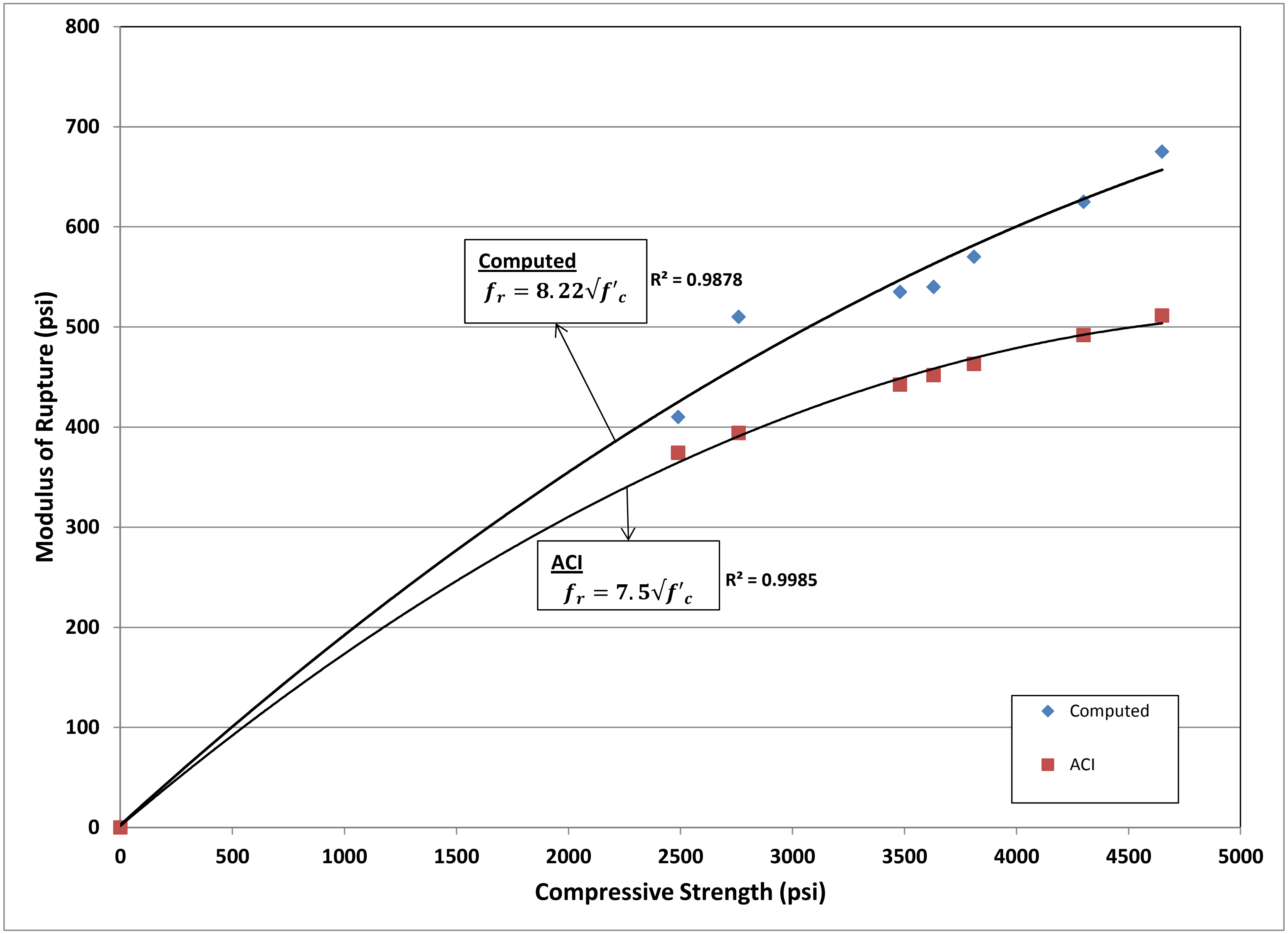


Figure 4-20. Relationship between compressive strength and modulus of rupture for concrete with w/c ratio of 0.6 at 28-day moist curing using data for non-standard aggregates.

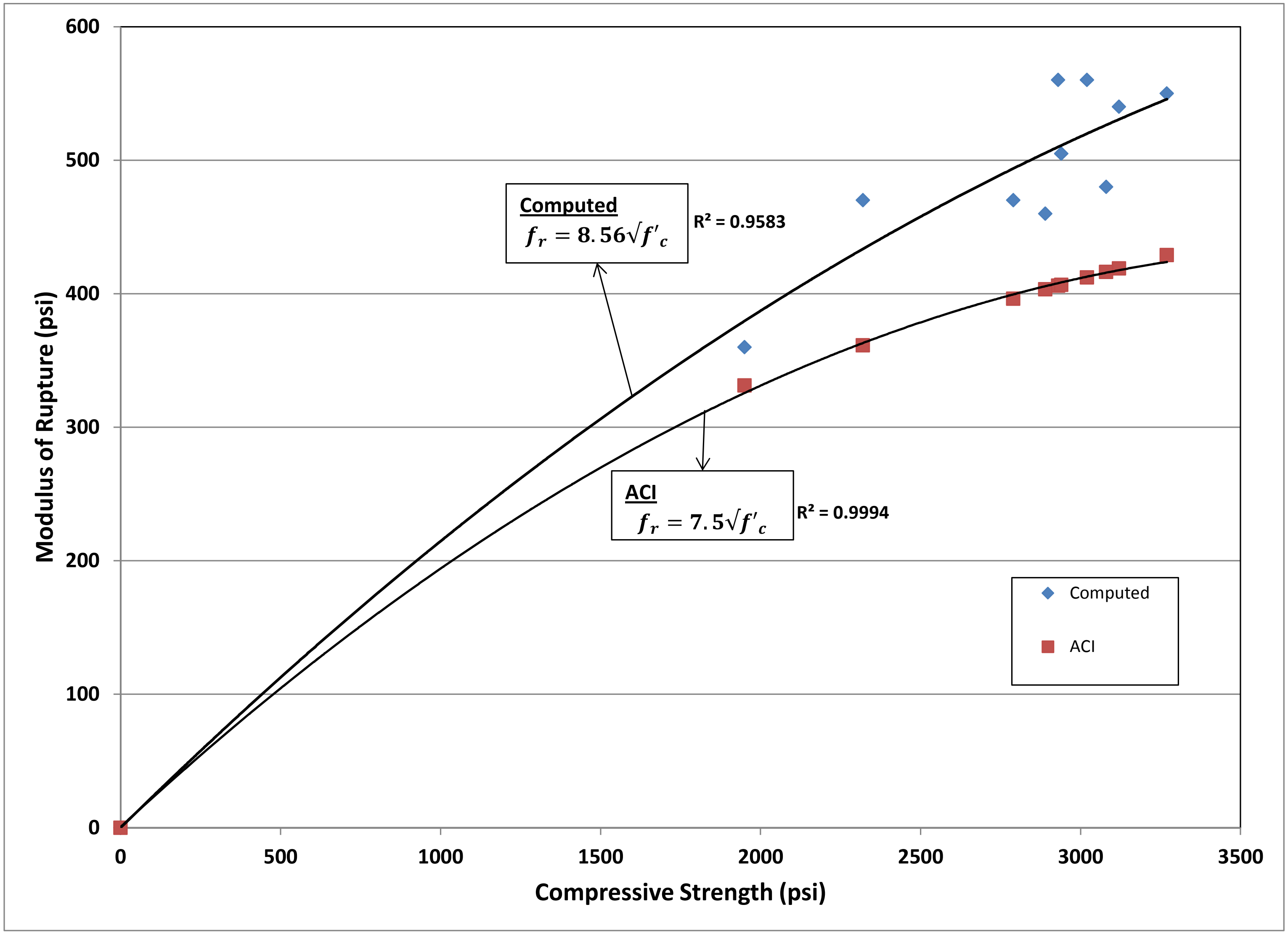


Figure 4-21. Relationship between compressive strength and modulus of rupture for concrete with w/c ratio of 0.7 at 28-day moist curing using all the data.

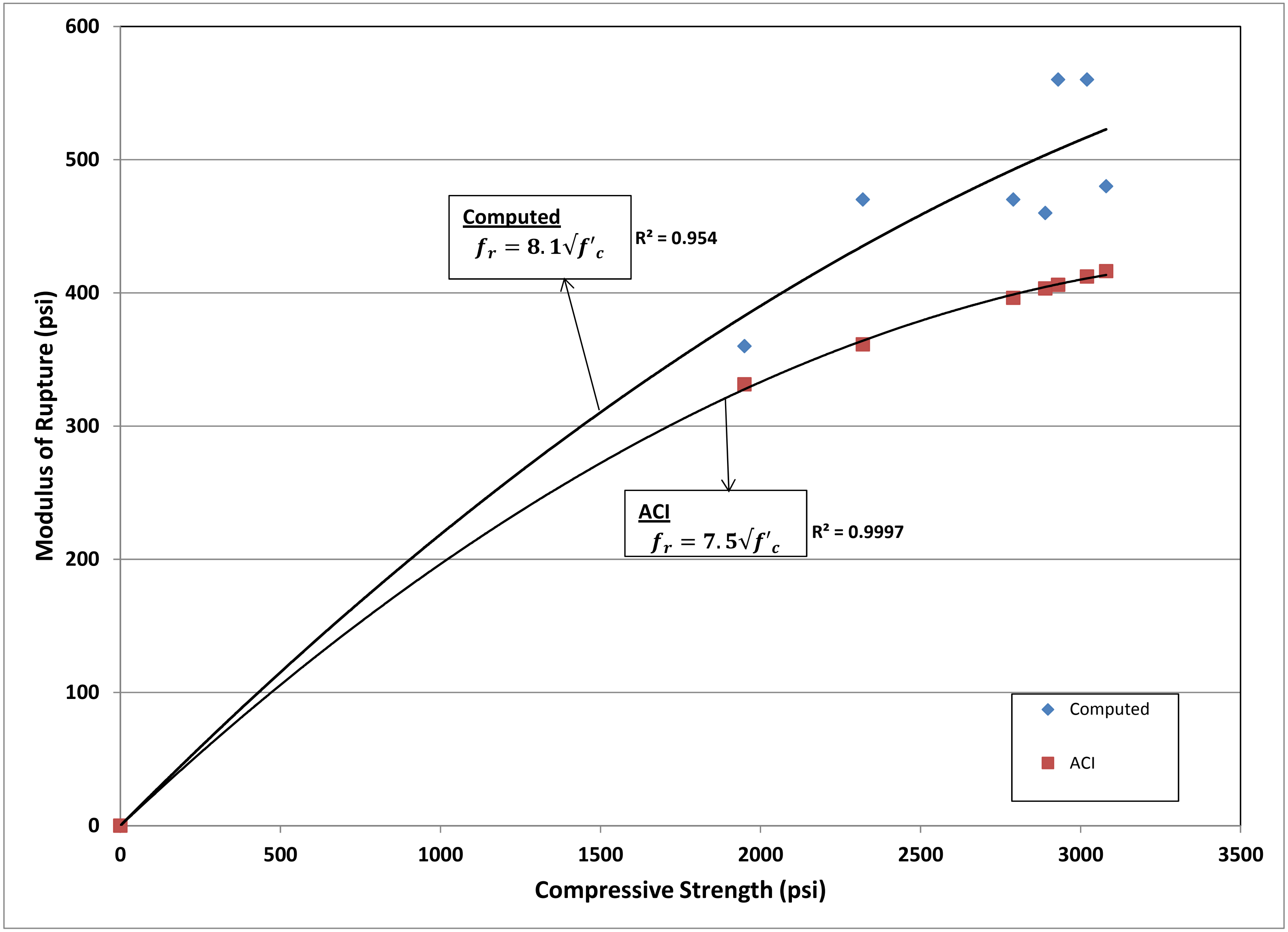


Figure 4-22. Relationship between compressive strength and modulus of rupture for concrete with w/c ratio of 0.7 at 28-day moist curing using data for non-standard aggregates.

Table 4-18. Regression Coefficient Determined for Various Data Categories for Compressive Strength vs. Modulus of Rupture

Data Category	A
Regression Equation	$f_r = 7.5\sqrt{f'_c}$
W/C = 0.6:	
Entire data	8.22
Concrete containing non-standard aggregates	8.22
W/C = 0.7:	
Entire data	8.56
Concrete containing non-standard aggregates	8.10

From the above values of A, it may be seen that the ACI formulation underpredicts the modulus of rupture of the concrete. Also, there is no significant difference in the A value for the different categories. Thus, a mean value of about 8.28 can be used for the A value in this prediction equation for these concretes. In research conducted by Tia et al. (1989), an A value of 8.12 for Brooksville aggregate, 9.71 for Calera, aggregate and 9.06 for River Gravel were obtained.

4.3.2 Relationship between Compressive Strength and Splitting Tensile Strength

A regression analysis was used to establish the relationship between the compressive strength and the splitting tensile strength. ACI uses an empirical relationship of the form shown in Equation 4.2 for lightweight concrete, where A = 6.7. Other researchers (Carino and Lew, 1982) have also found that the empirical relationship of the form as shown in Equation 4.3 gives a better prediction than that shown in Equation 4.2

$$f_{ct} = A\sqrt{f'_c} \tag{4.2}$$

$$f_{ct} = f'^{B}_c \tag{4.3}$$

where f_{ct} = splitting tensile strength, (psi); and

f'_c = compressive strength (psi); and

A and B = coefficients.

The two empirical relationships shown above were used to determine the respective coefficients for our data. Figures 4-23 to 4-26 show plots of the various relationships. Table 4-19 shows the obtained coefficients from the regression analysis.

Table 4-19. Regression Coefficient Determined for Various Data Categories for Compressive Strength vs. Splitting Tensile Strength

Data Category	A	B
Regression Equation	$f_{ct} = A\sqrt{f'_c}$	$f_{ct} = f'^{B}_c$
W/C = 0.6:		
Entire Data	5.51	0.72
Concrete containing non-standard aggregates	5.51	0.74
W/C = 0.7:		
Entire Data	5.77	0.71
Concrete containing non-standard aggregates	6.54	0.71

Carino and Lew (1982) also used the ACI formulation and determined A to be approximately 6.49. They suggested that the estimation of splitting tensile strength using Equation 4.3 was better than Equation 4.2. Also, they found out that for low compressive strengths, Equation 4.2 overestimates the splitting tensile strength, while it underestimates it for high strength concrete. They determined the coefficient B to be approximately 0.73

Our results show an average value of A as 5.83 and B as 0.72. It must be noted, however, that when using the ACI formulation for concrete containing water/cement ratio of 0.7, the value of A when data involved only the non-standard aggregate was much different from when all the data were used. This could be due to the variability in the result of the splitting tensile strength.

Tia et al. (1989) also determined A as 6.77, 7.62, 6.97 and B as 0.712,0.721,0.714 for concrete containing Brooksville, Calera and River Gravel aggregates, respectively.

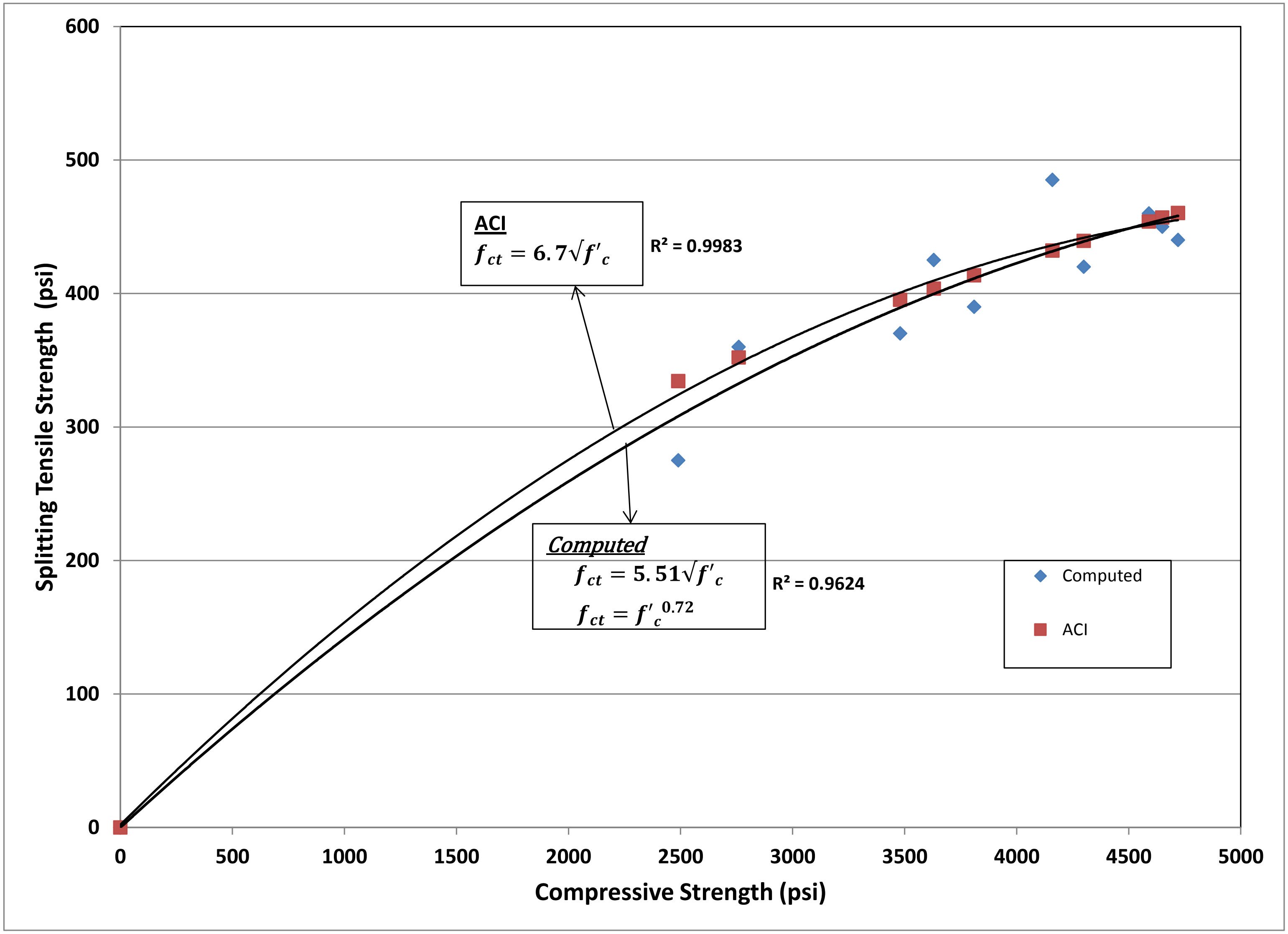


Figure 4-23. Relationship between compressive strength and splitting tensile strength for concrete with w/c ratio of 0.6 at 28-day moist curing using all the data.

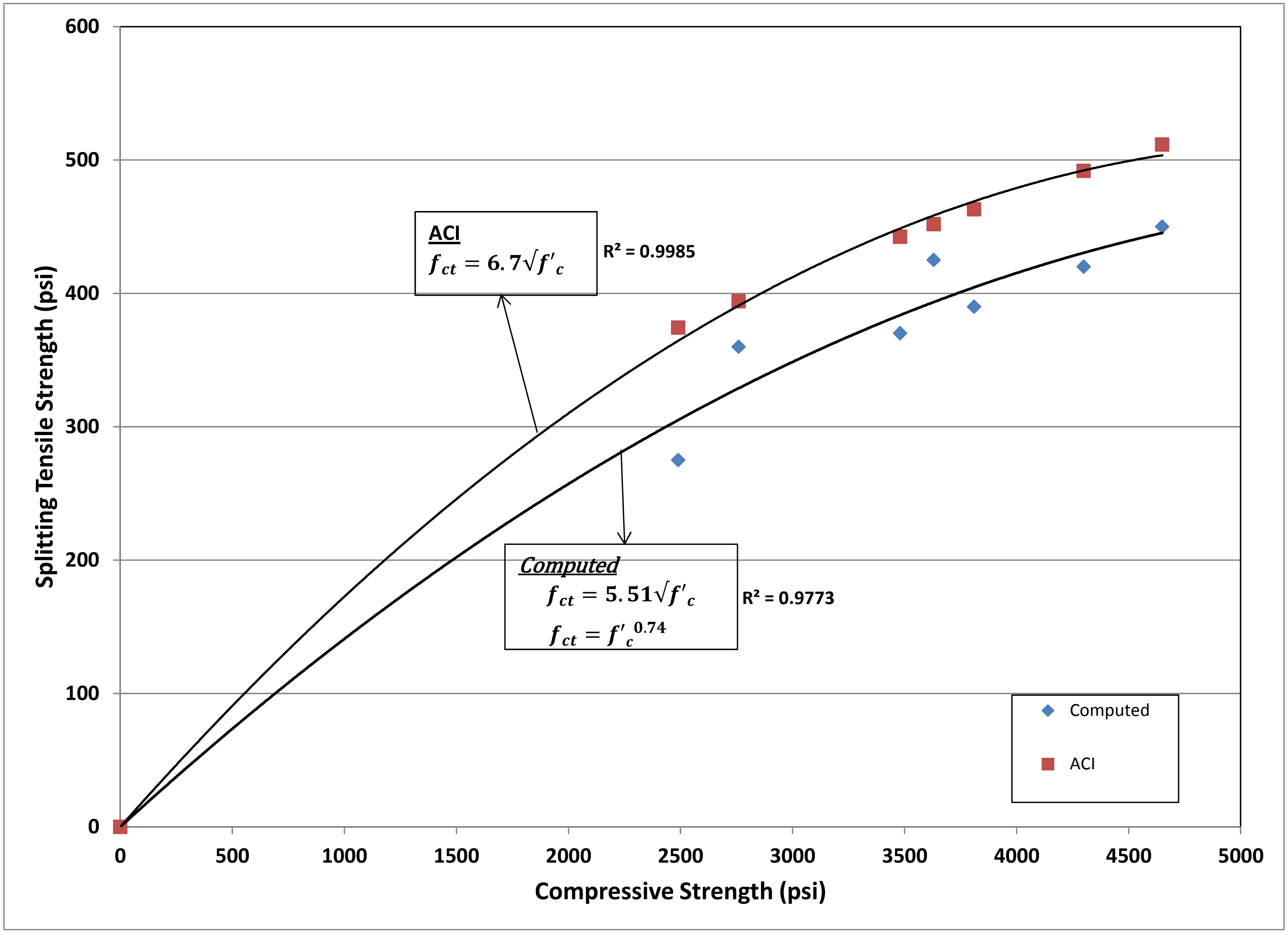


Figure 4-24. Relationship between compressive strength and splitting tensile strength for concrete with w/c ratio of 0.6 at 28-day moist curing using data for non-standard aggregates.

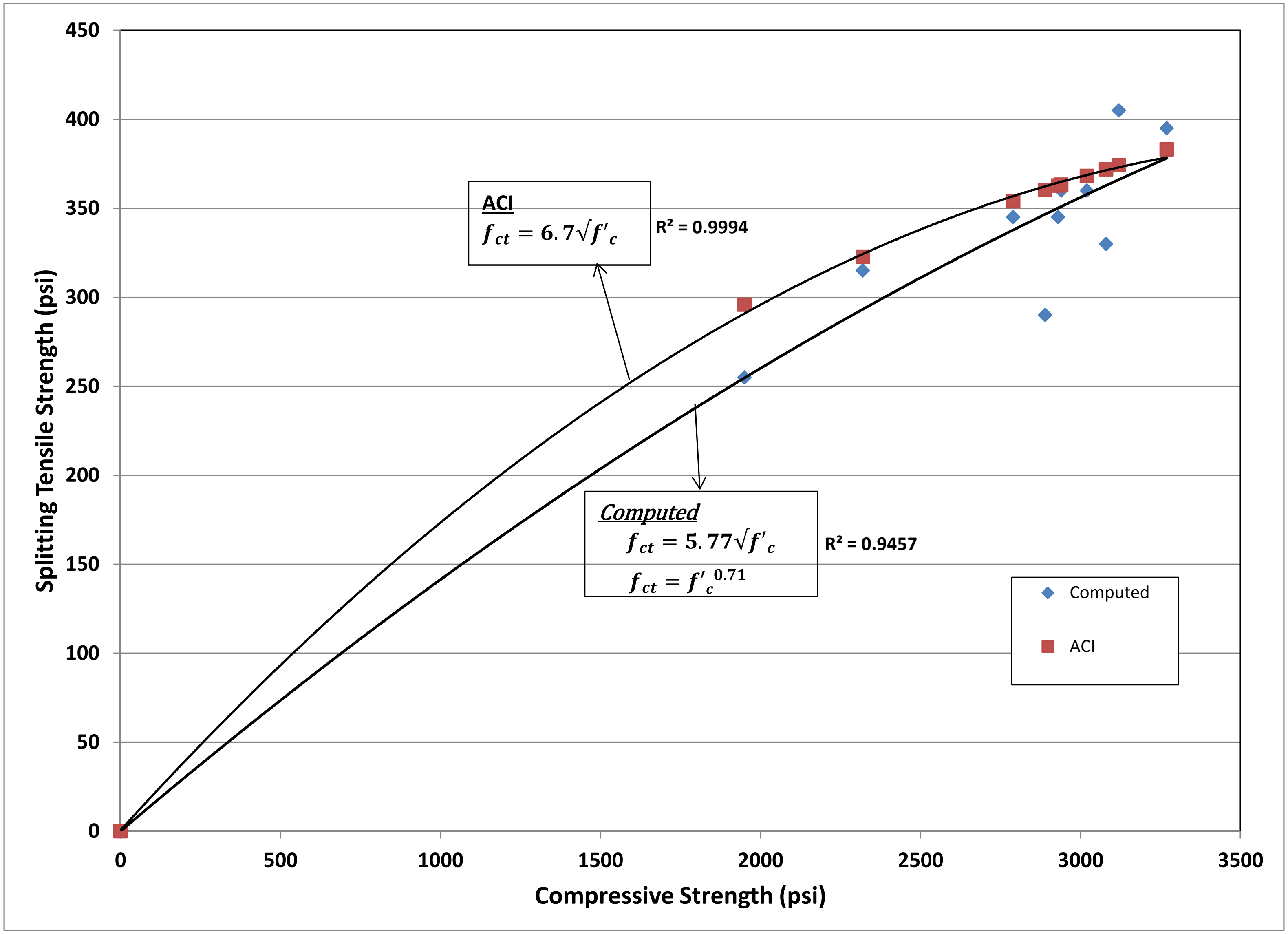


Figure 4-25. Relationship between compressive strength and splitting tensile strength for concrete with w/c ratio of 0.7 at 28-day moist curing using all the data.

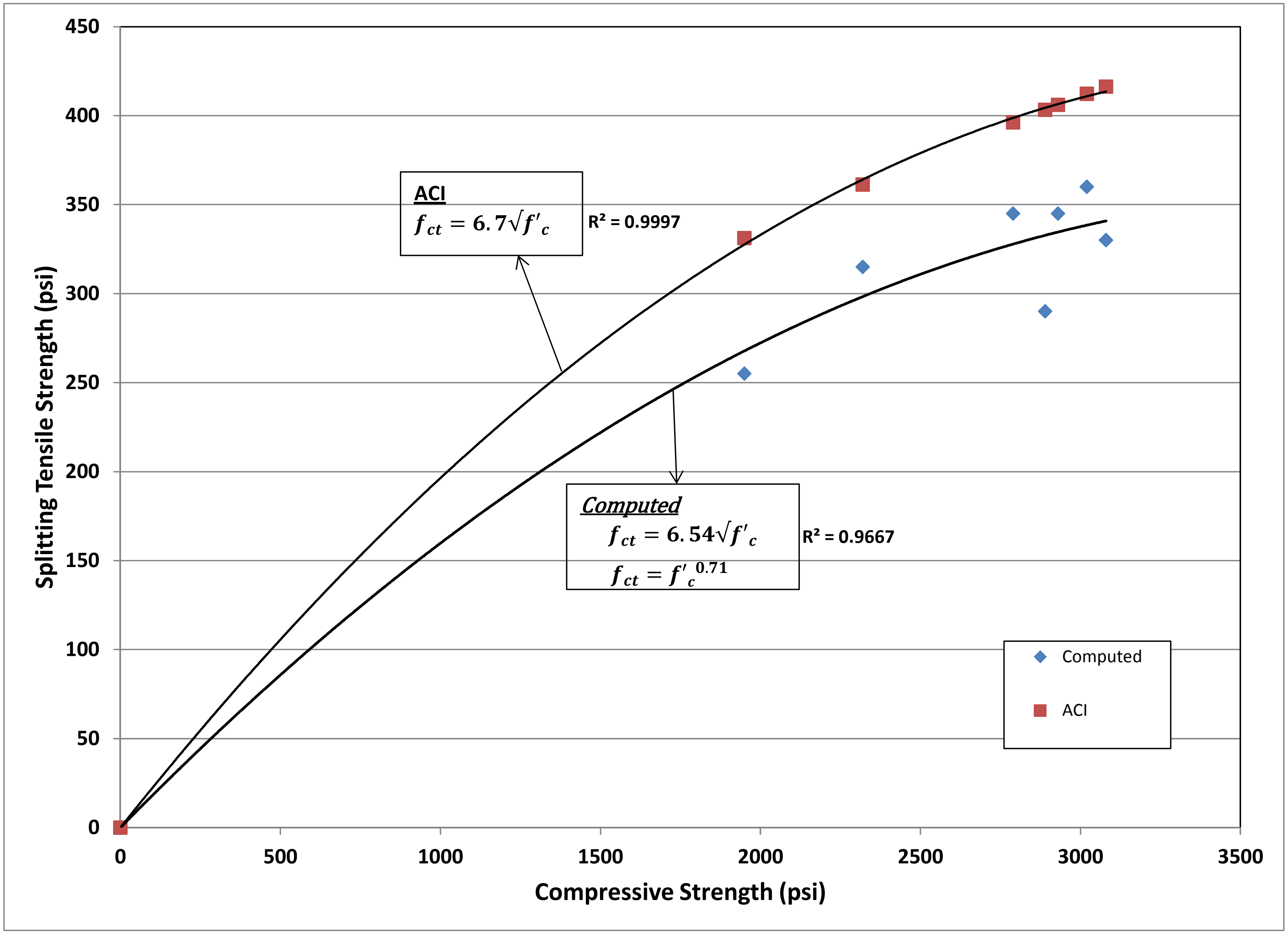


Figure 4-26. Relationship between compressive strength and splitting tensile strength for concrete with w/c ratio of 0.7 at 28-day moist curing using data for non-standard aggregates.

4.3.3 Relationship between Compressive Strength and Modulus of Elasticity

According to ACI, the modulus of elasticity can be determined from the expression shown in Equation 4.5 where A is 33. This empirical specification was applied to our study and from the regression analysis the value of A was determined as shown in Table 4-20. Figures 4-27 to 4-30 show plots of the various relationships.

$$E = Aw^{1.5}\sqrt{f'_c} \tag{4.4}$$

where w = unit weight of concrete (lb/ft^3);

f'_c = compressive strength (psi);

E = modulus of elasticity (psi); and

A = coefficient.

Table 4-20. Regression Coefficient Determined for Various Data Categories for Compressive Strength vs. Modulus of Elasticity

Data Category	A
Regression Equation	$E = Aw^{1.5}\sqrt{f'_c}$
W/C = 0.6:	
Entire data	36.64
Concrete containing non-standard aggregates	36.64
W/C = 0.7:	
Entire data	40.49
Concrete containing non-standard aggregates	38.93

From the results obtained for these data, the values of A determined were all higher than that proposed by ACI, and this agrees with what was obtained by Tia et al. (1998) where they obtained A as 34.05, 40.331 and 34.23 for concrete containing Brooksville, Calera and River Gravel aggregates, respectively. On the average, the value of A for our data is 38.18.

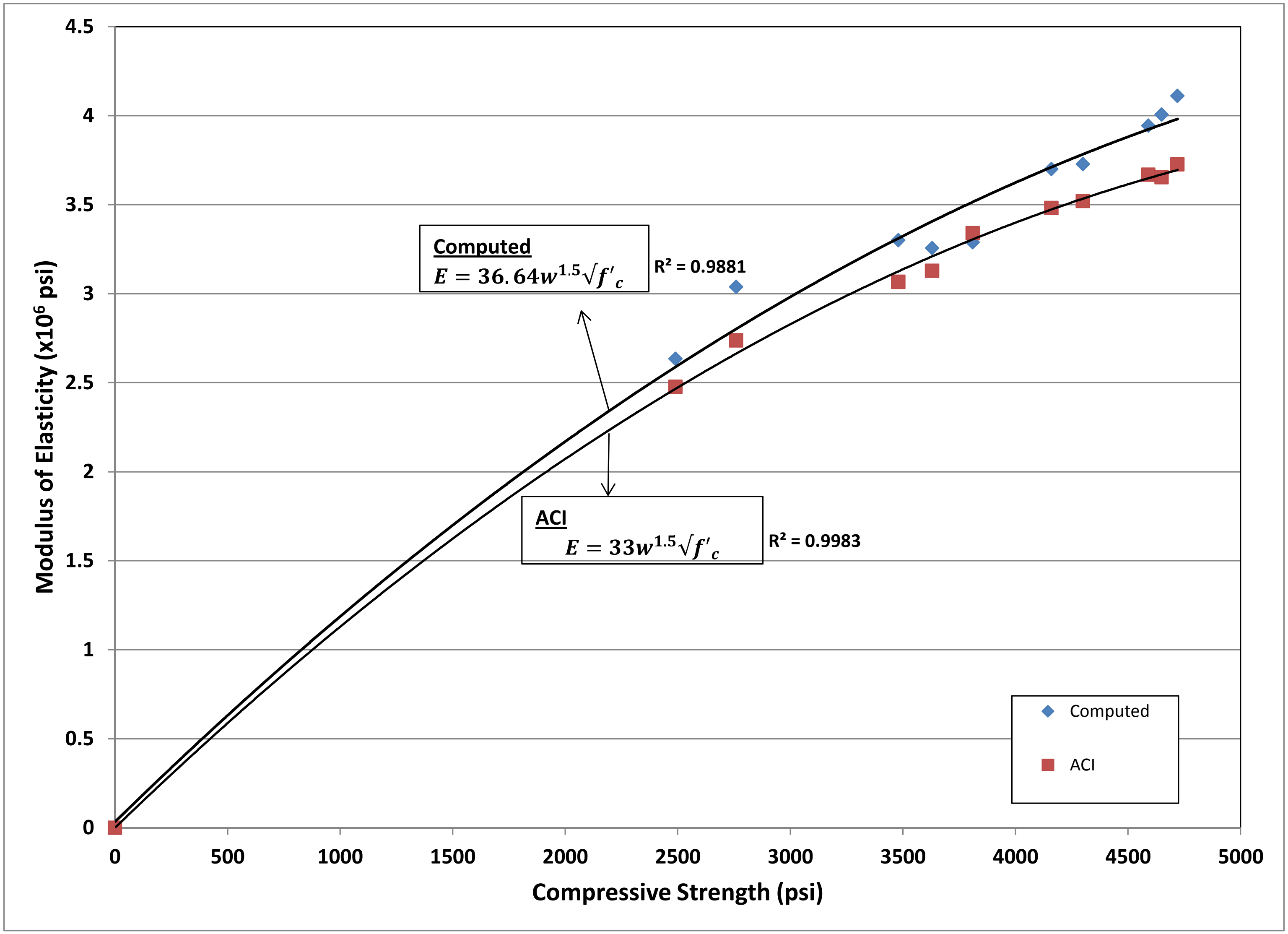


Figure 4-27. Relationship between compressive strength and modulus of elasticity for concrete with w/c ratio of 0.6 at 28-day moist curing using all the data.

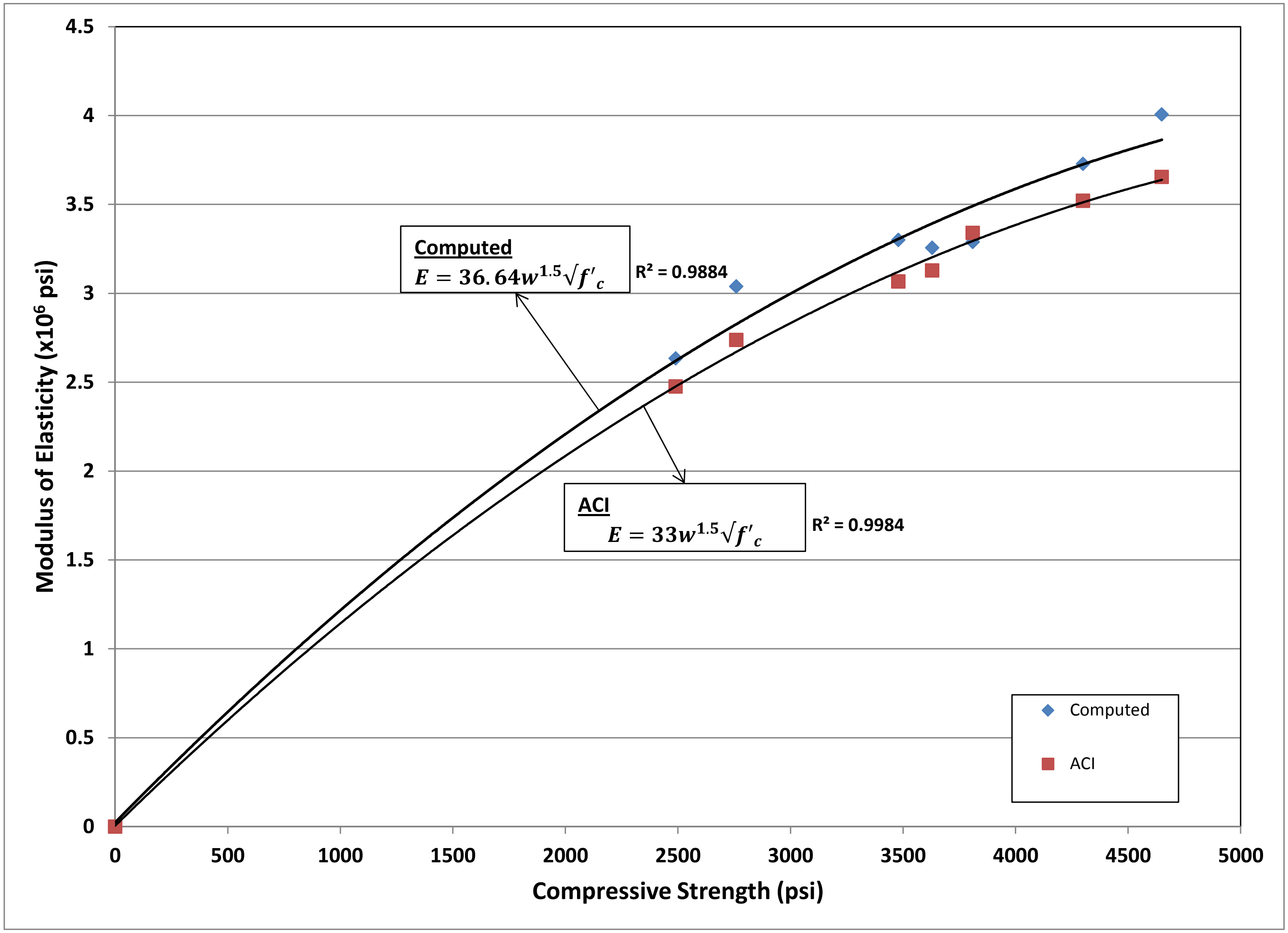


Figure 4-28. Relationship between compressive strength and modulus of elasticity for concrete with w/c ratio of 0.6 at 28-day moist curing using data for non-standard aggregates.

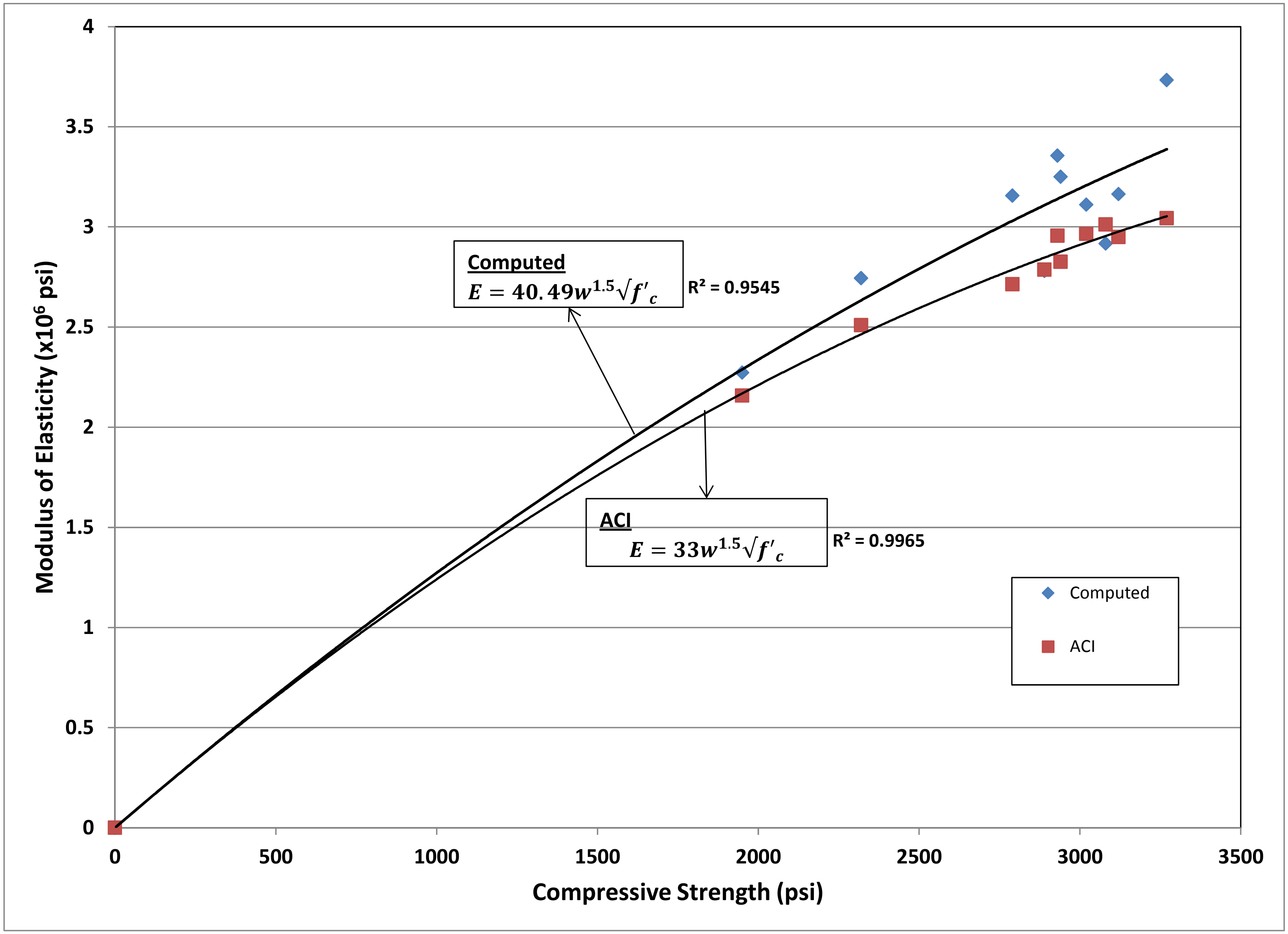


Figure 4-29. Relationship between compressive strength and modulus of elasticity for concrete with w/c ratio of 0.7 at 28-day moist curing using all the data.

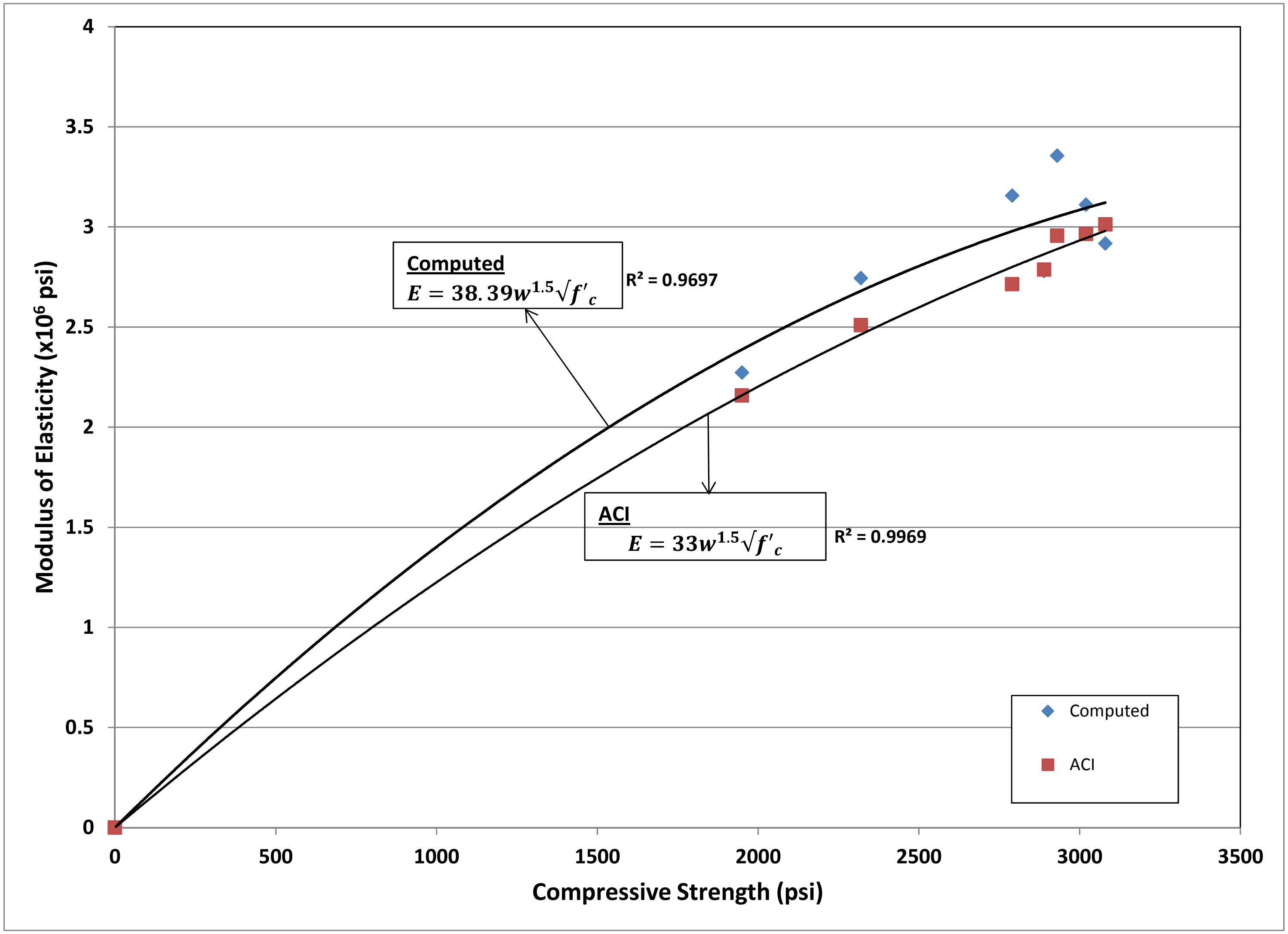


Figure 4-30. Relationship between compressive strength and modulus of elasticity for concrete with w/c ratio of 0.7 at 28-day moist curing using data for non-standard aggregates.

4.3.4 Relationship between Modulus of Rupture and Modulus of Elasticity

There is no direct relationship between the modulus of rupture and the modulus of elasticity specified by ACI. However, an approximate relationship can be inferred from Equations 4.1 and 4.4. The combined relationship is shown in Equation 4.5 where A is 4.4 using the ACI values.

$$E = Aw^{1.5} f_r \tag{4.5}$$

where w = unit weight of concrete (lb/ft^3);

f_r = modulus of rupture (psi);

E = modulus of elasticity (psi); and

A = coefficient.

A regression analysis of the above specification was used in our study to determine the values of A. Graphs of the various relationships are shown in Figures 4-31 to 4-34. Table 4-21 shows the determined values A.

Table 4-21. Regression Coefficient Determined for Various Data Categories for Modulus of Rupture vs. Modulus of Elasticity

Data Category	A
Regression Equation	$E = Aw^{1.5} f_r$
W/C = 0.6:	
Entire data	3.92
Concrete containing non-standard aggregates	4.27
W/C = 0.7:	
Entire data	4.31
Concrete containing non-standard aggregates	4.31

From Table 4-21, the average value of A is 4.20 which compares to the value of 4.4 derived from the two ACI specifications stated above. Tia et al. (1998) also obtained A as 4.20, 4.09 and 3.69 for concrete containing Brooksville, Calera and River Gravel aggregates, respectively.

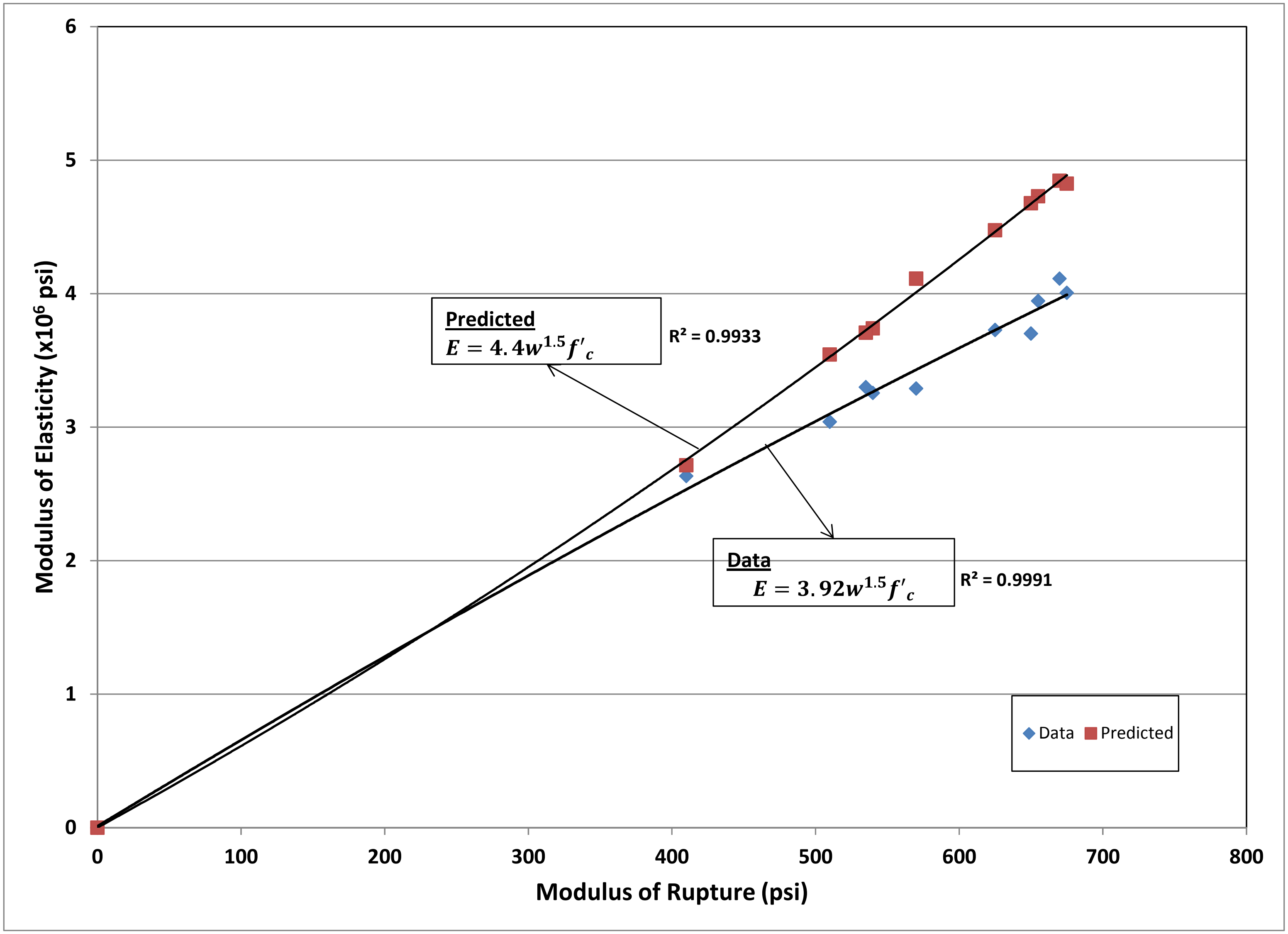


Figure 4-31. Relationship between modulus of rupture and modulus of elasticity for concrete with w/c ratio of 0.6 at 28-day moist curing using all the data.

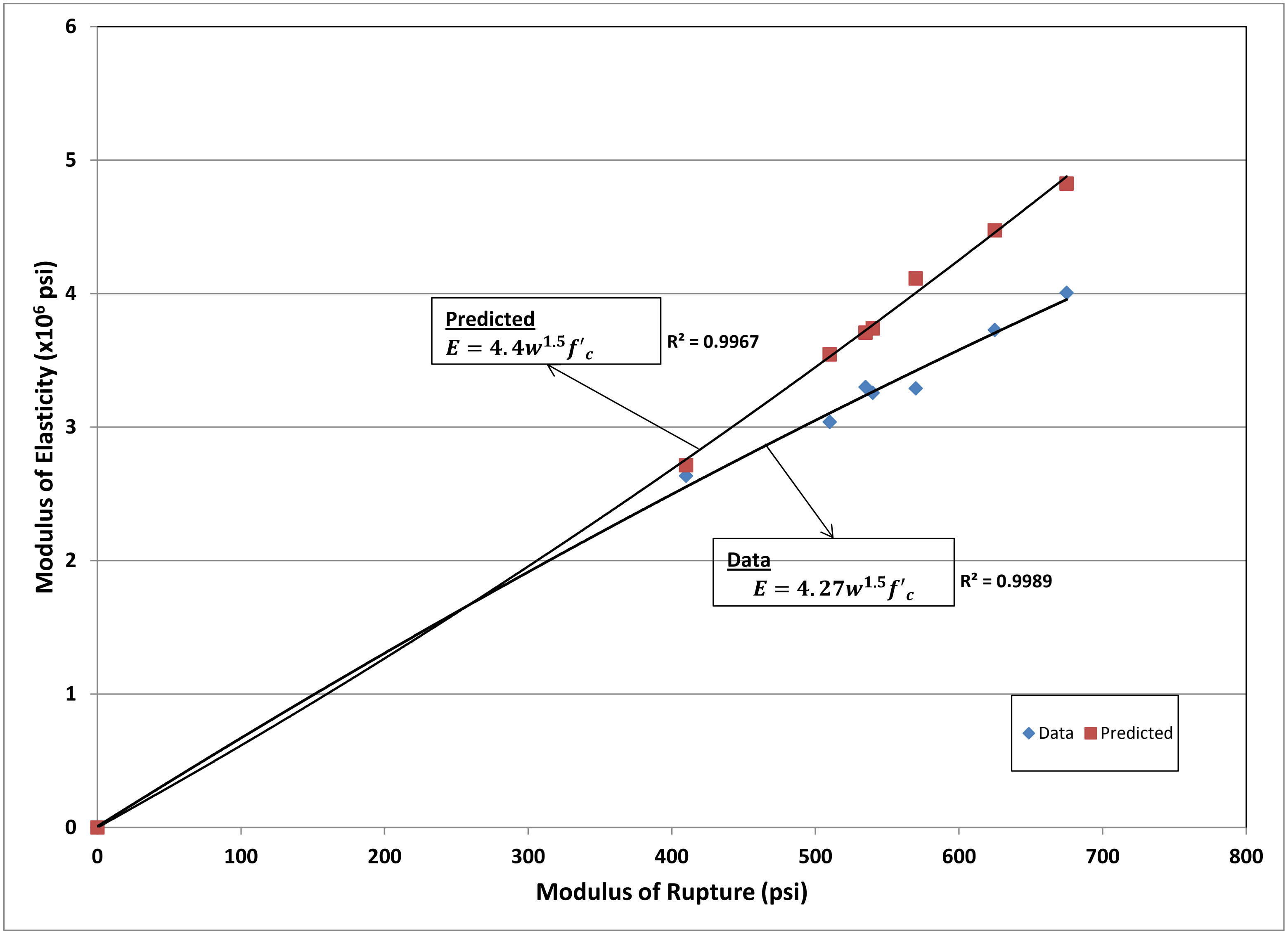


Figure 4-32. Relationship between modulus of rupture and modulus of elasticity for concrete with w/c ratio of 0.6 at 28-day moist curing using data for non-standard aggregates.

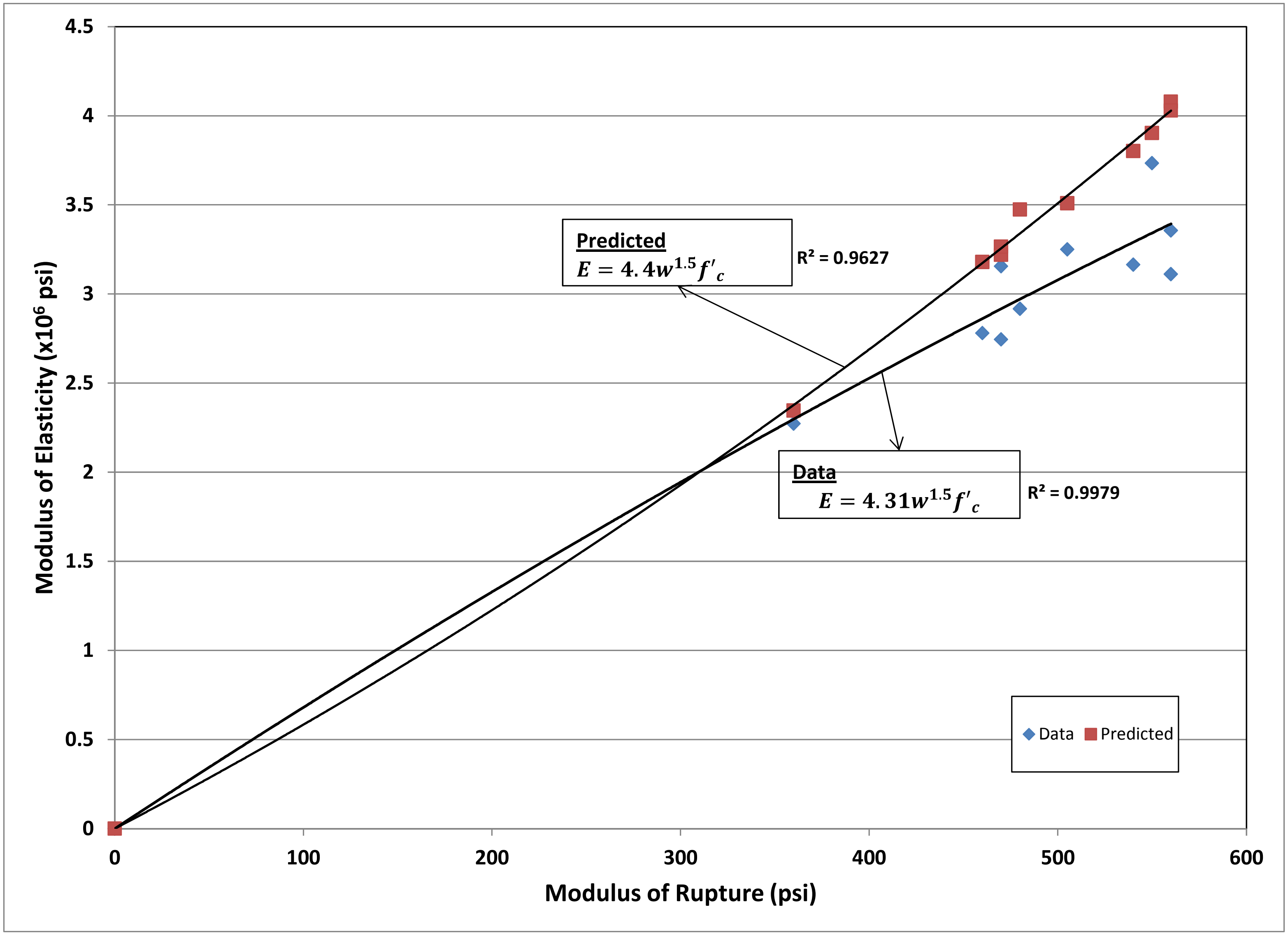


Figure 4-33. Relationship between modulus of rupture and modulus of elasticity for concrete with w/c ratio of 0.7 at 28-day moist curing using all the data.

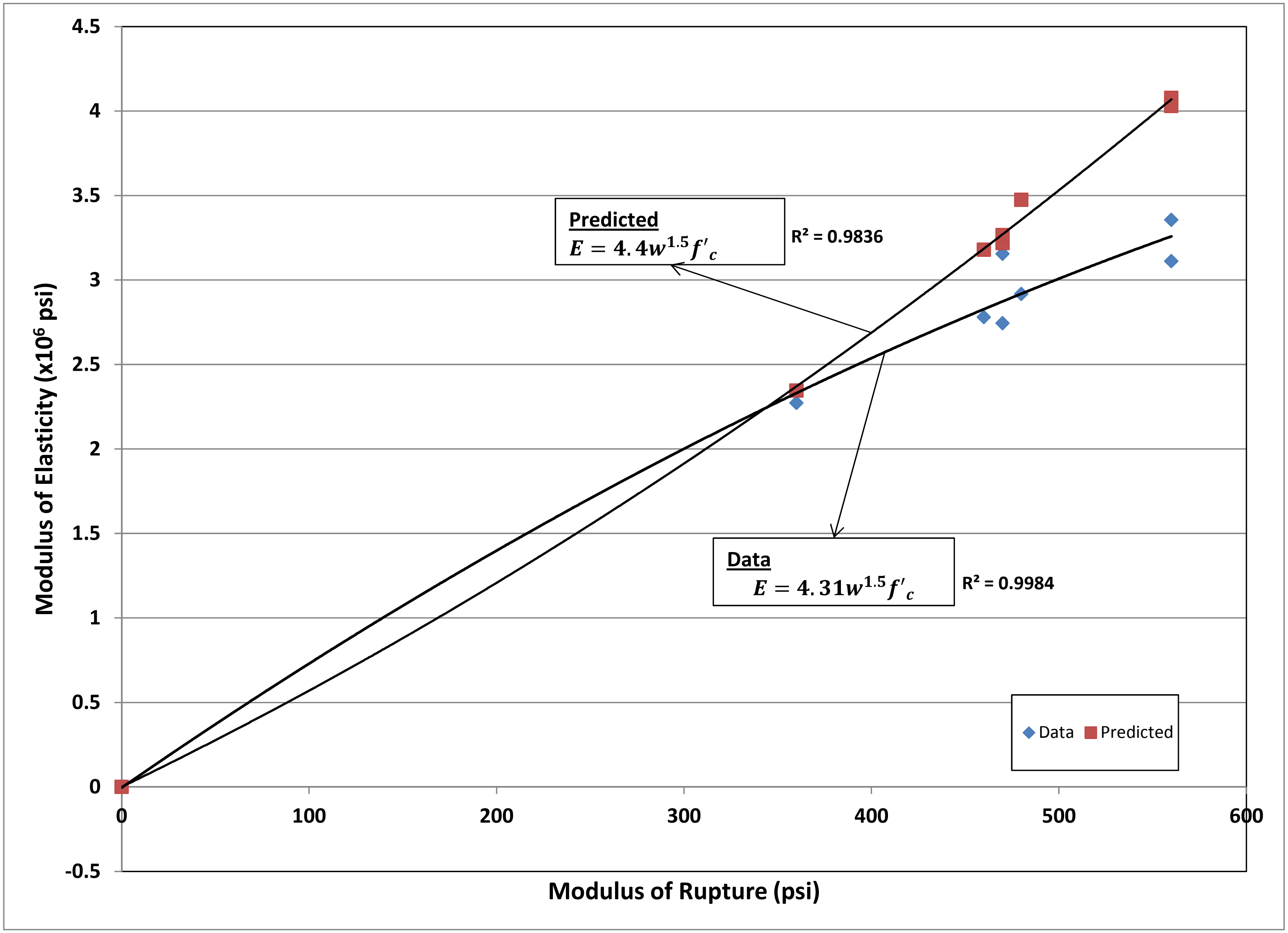


Figure 4-34. Relationship between modulus of rupture and modulus of elasticity for concrete with w/c ratio of 0.7 at 28-day moist curing using data for non-standard aggregates.

4.3.5 Relationship between Modulus of Rupture and Splitting Tensile Strength

The modulus of rupture is known to correlate highly with the splitting tensile strength of concrete. A linear regression of the form in Equation 4.6 was used.

$$f_{ct} = Af_r \qquad (4.6)$$

where f_{ct} = splitting tensile strength (psi);

f_r = modulus of rupture (psi); and

A = coefficient.

This relationship was determined for the two different water/cement ratios and further verified for only the concrete containing the non-standard aggregated. Figures 4-35 to 4-38 show plots of the various relationships. The values of A and the R^2 values are shown in Table 4-22. The average value for A is 0.66.

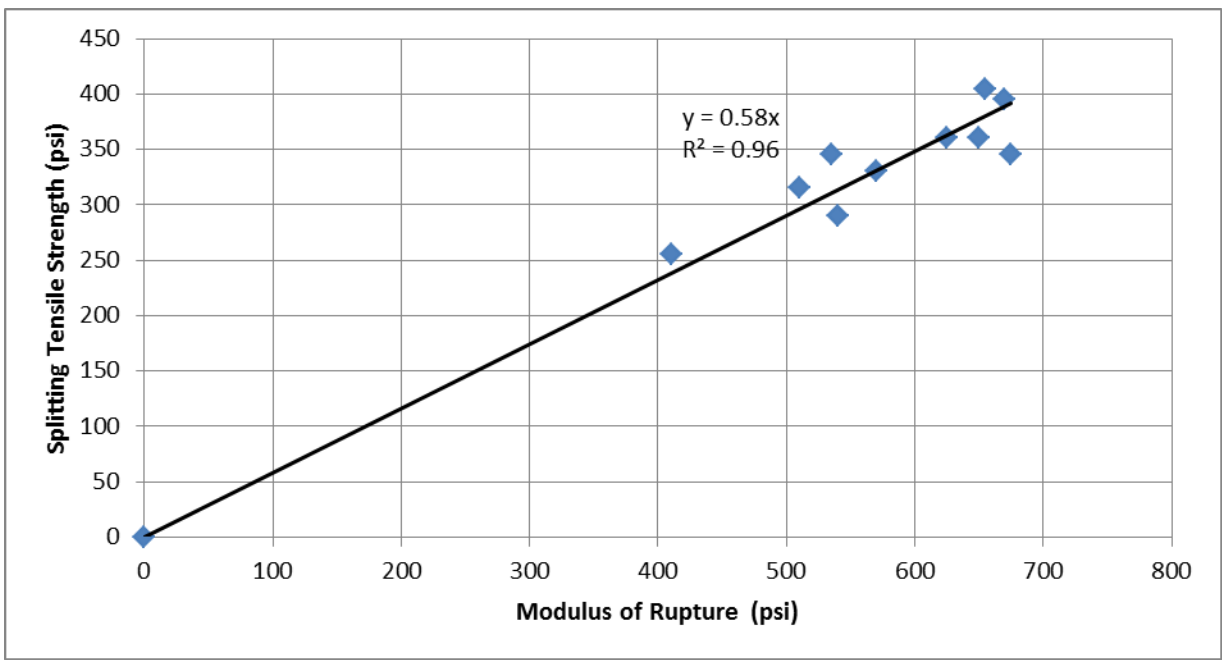


Figure 4-35. Relationship between splitting tensile strength and modulus of rupture for concrete with w/c ratio of 0.6 at 28-day moist curing using all the data.

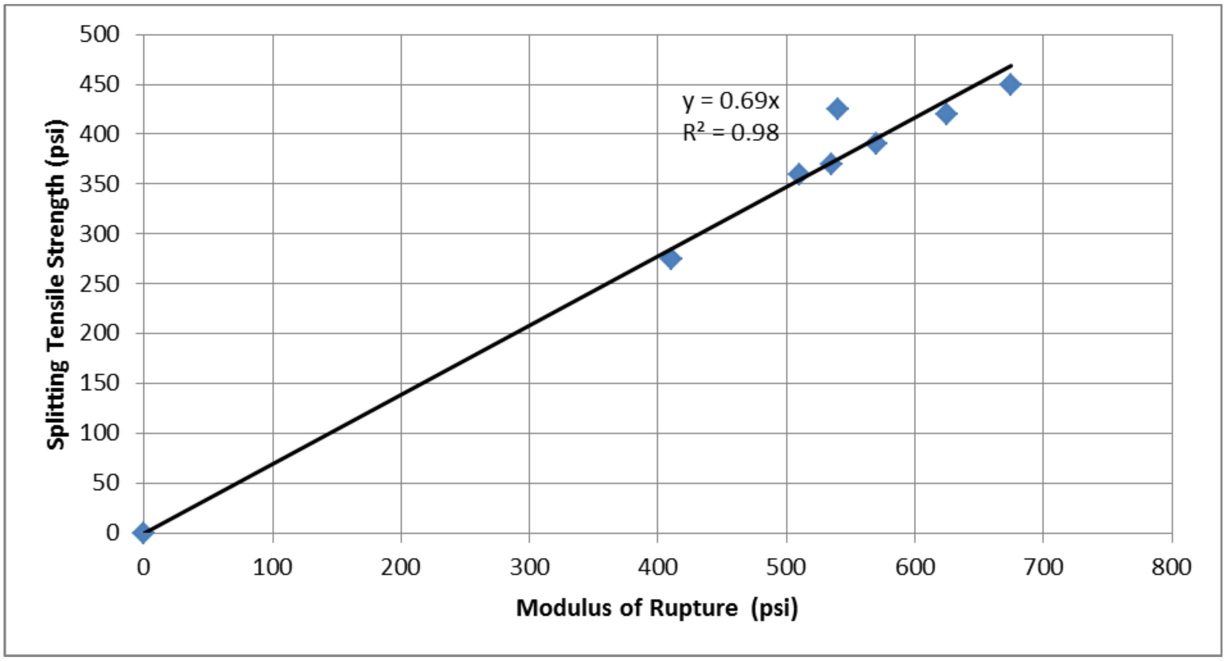


Figure 4-36. Relationship between splitting tensile strength and modulus of rupture for concrete with w/c ratio of 0.7 at 28-day moist curing using all the data.

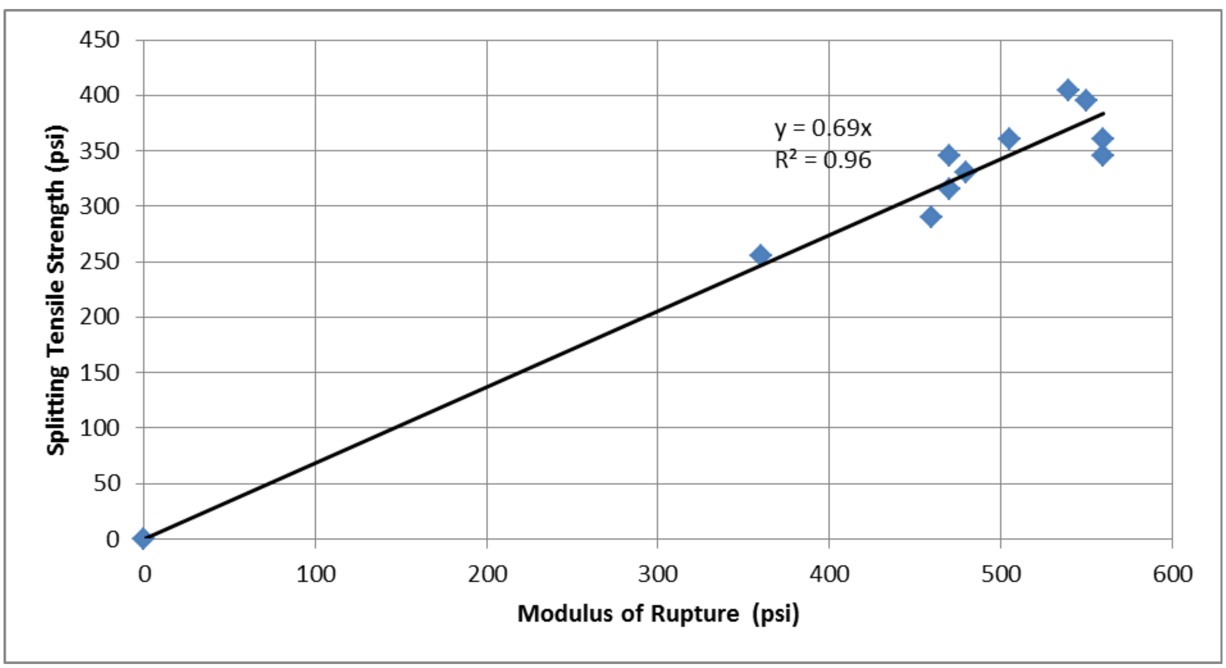


Figure 4-37. Relationship between splitting tensile strength and modulus of rupture for concrete with w/c ratio of 0.7 at 28-day moist curing using all data.

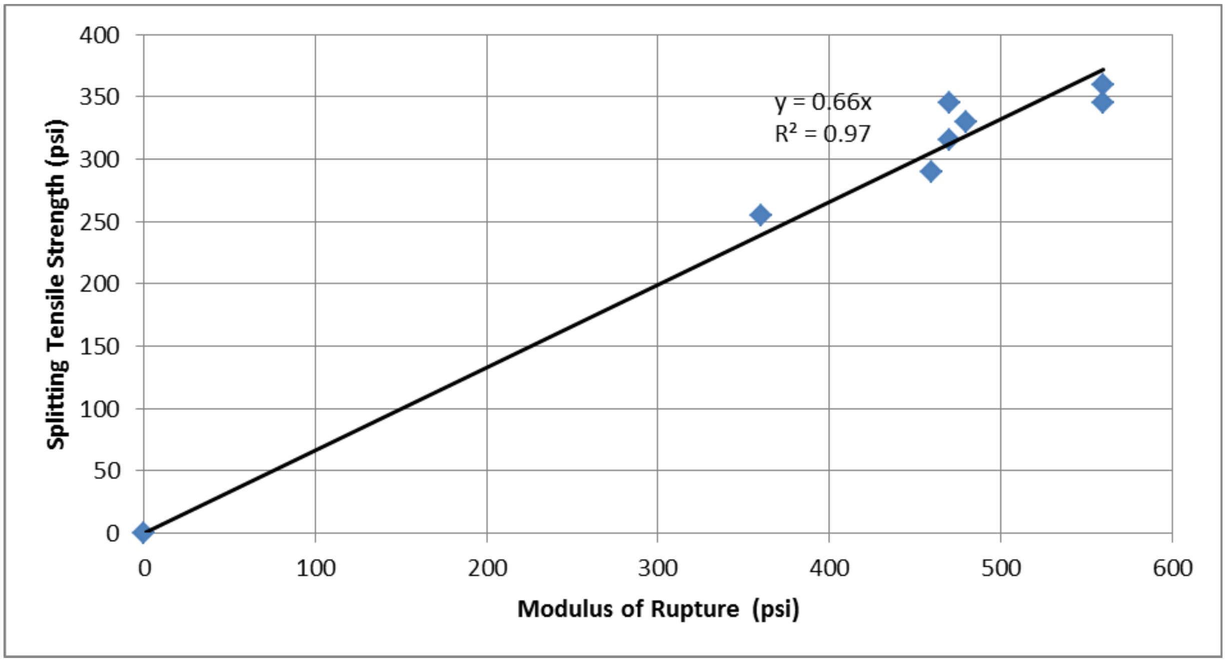


Figure 4-38. Relationship between splitting tensile strength and modulus of rupture for concrete with w/c ratio of 0.7 at 28-day moist curing using data from non-standard aggregates.

Table 4-22. Regression Coefficients and R-Squares Determined for Various Data Categories for Modulus of Rupture vs. Splitting Tensile Strength

Data Category	A	R-Square
Regression Equation	$f_{ct} = Af_r$	$f_{ct} = f_c'^B$
W/C = 0.6:		
Entire Data	0.58	0.96
Concrete containing non-standard aggregates	0.69	0.98
W/C = 0.7:		
Entire Data	0.69	0.96
Concrete containing non-standard aggregates	0.66	0.97

4.4 Summary

The properties of hardened concrete evaluated in this study are presented in this chapter. The compressive strength of the concrete tested using standard ASTM C-39 were found to be almost the same as that of the samples which have been loaded to 40% of the ultimate strength and unloaded before conducting the compressive strength test.

Statistical analyses were performed to relate various concrete properties to one another. The forms of the prediction equations used by ACI were found to work well for these data. The adjusted regression coefficients for these prediction equations to be used for these concretes are presented.

CHAPTER 5
RELATIONSHIP BETWEEN AGGREGATE PROPERTIES AND CONCRETE PROPERTIES

5.1 Introduction

Statistical analyses were performed to determine the possible relationship between the various physical properties of aggregates and the compressive strength of concrete. The statistical analyses were run on the data from the second experimental design where concrete containing low cement content were tested.

5.2 Relationship between Aggregate Properties

5.2.1 Correlation between Different Aggregate Properties

Correlation analyses were performed among different aggregate properties. Table 5-1 shows the Pearson correlation between the different aggregate properties. It can be seen that percent passing No. 200 sieve, LA abrasion loss, Micro-Deval abrasion loss and sodium sulfate soundness loss are positively correlated with one another. The correlation between shell content and the other aggregate properties are low with Pearson correlation varying from –0.187 to 0.433.

Table 5-1. Pearson Correlation between Different Aggregate Properties

Aggregate Properties	LA Loss	M-D Loss	Soundness Loss	Shell Content
% minus No. 200 sieve	0.473	0.623	0.497	–0.187
LA loss		0.893	0.880	0.168
M-D loss			0.841	–0.099
Soundness loss				0.433

5.2.2 Relationship between LA Abrasion Loss and Percentage Finer than No. 200 Sieve

A linear regression analysis was conducted to determine the correlation between the LA abrasion loss and the percentage finer than No. 200 sieve. From the graph shown in Figure 5-1, a positive correlation may be observed between the two properties. An adjusted R-square of 0.78 was determined from the analysis.

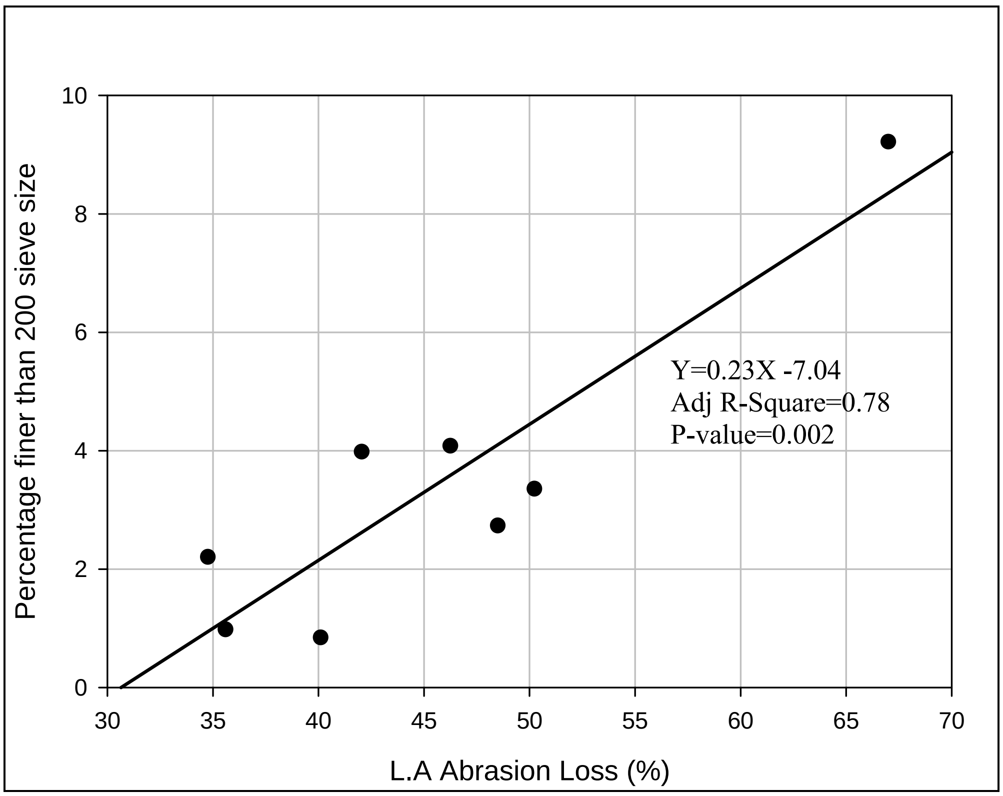


Figure 5-1. Plot of percentage finer than No. 200 sieve vs. Los Angeles abrasion loss (%).

5.2.3 Relationship between Micro-Deval Abrasion Loss and Percentage Finer than No. 200 Sieve

A linear regression analysis was conducted to determine the correlation between the Micro-Deval abrasion loss and the percentage finer than No. 200 sieve size. From the graph shown in Figure 5-2, a positive correlation may be observed between the two properties. An adjusted R-square value of 0.81 was obtained.

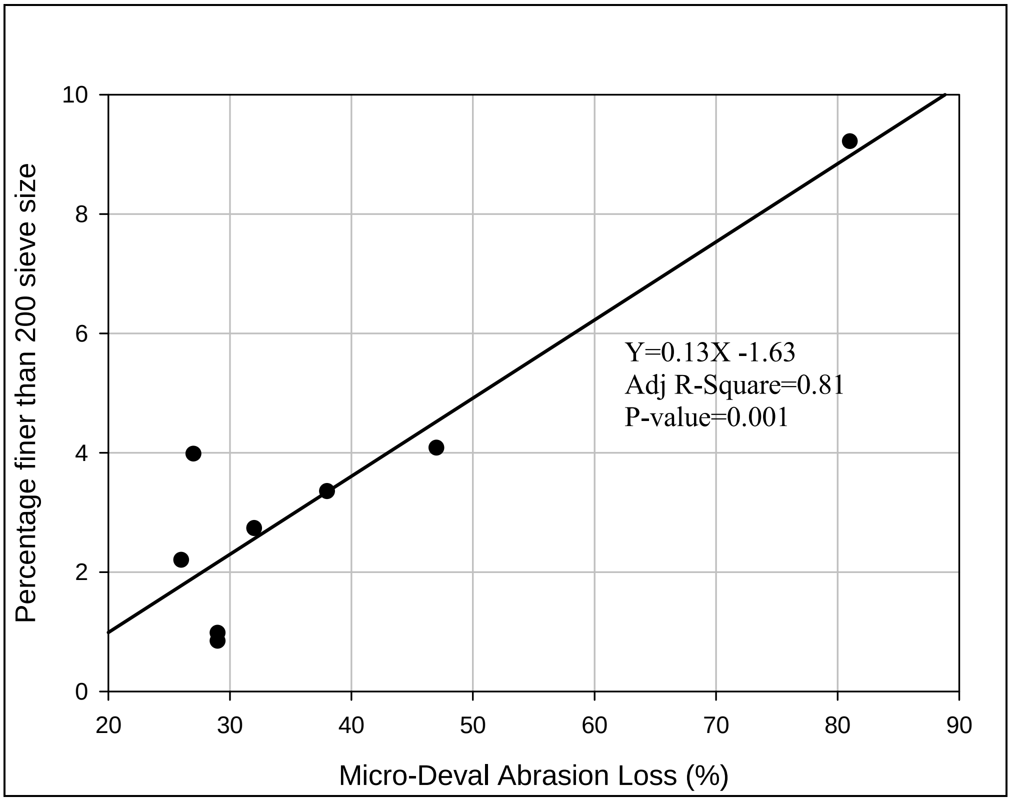


Figure 5-2. Graph of percentage finer than No. 200 sieve vs. Micro-Deval abrasion loss (%).

5.2.4 Relationship between Soundness Loss and Percentage Finer than No. 200 Sieve

A linear regression analysis was conducted to determine the correlation between the soundness loss and the percentage finer than No. 200 sieve. From the graph shown in Figure 5-3, a positive correlation can be seen between the two properties. An adjusted R-square value of 0.56 was obtained.

5.2.5 Relationship between Los Angeles Abrasion Loss and Micro-Deval Abrasion Loss

A linear regression analysis was conducted to determine the correlation between the LA abrasion loss and the Micro-Deval abrasion loss. From the graph shown in Figure 5-4, a positive correlation may be observed between the two properties. An adjusted R-square value of 0.83 was determined.

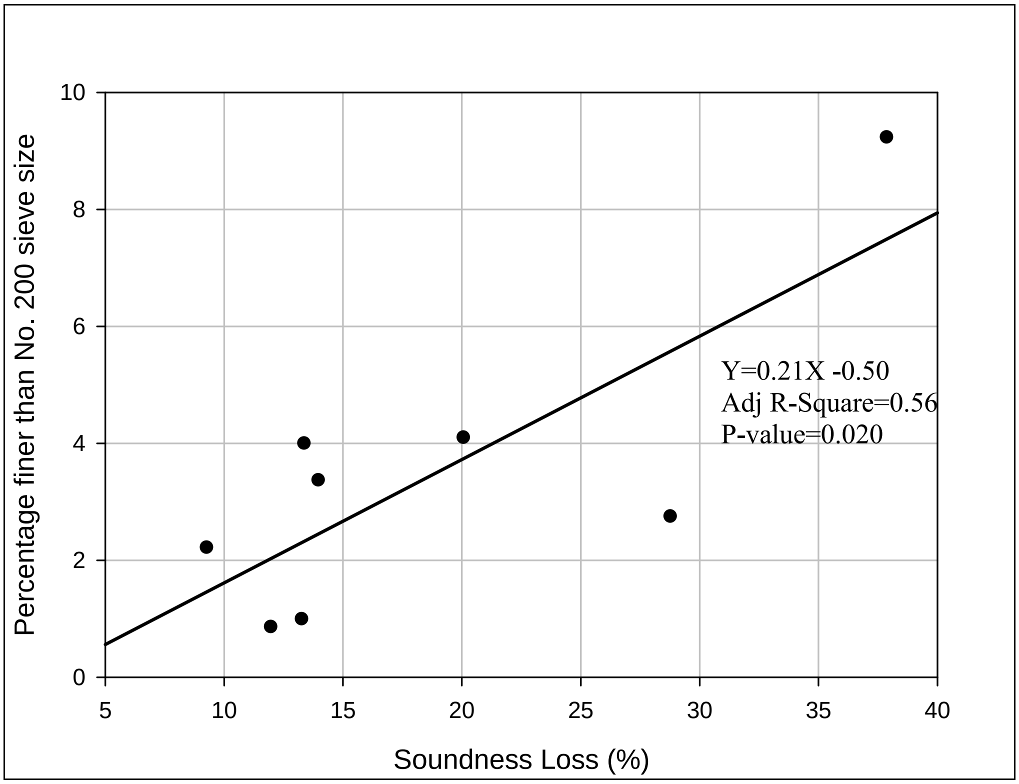


Figure 5-3. Plot of percentage finer than No. 200 sieve vs. soundness loss (%).

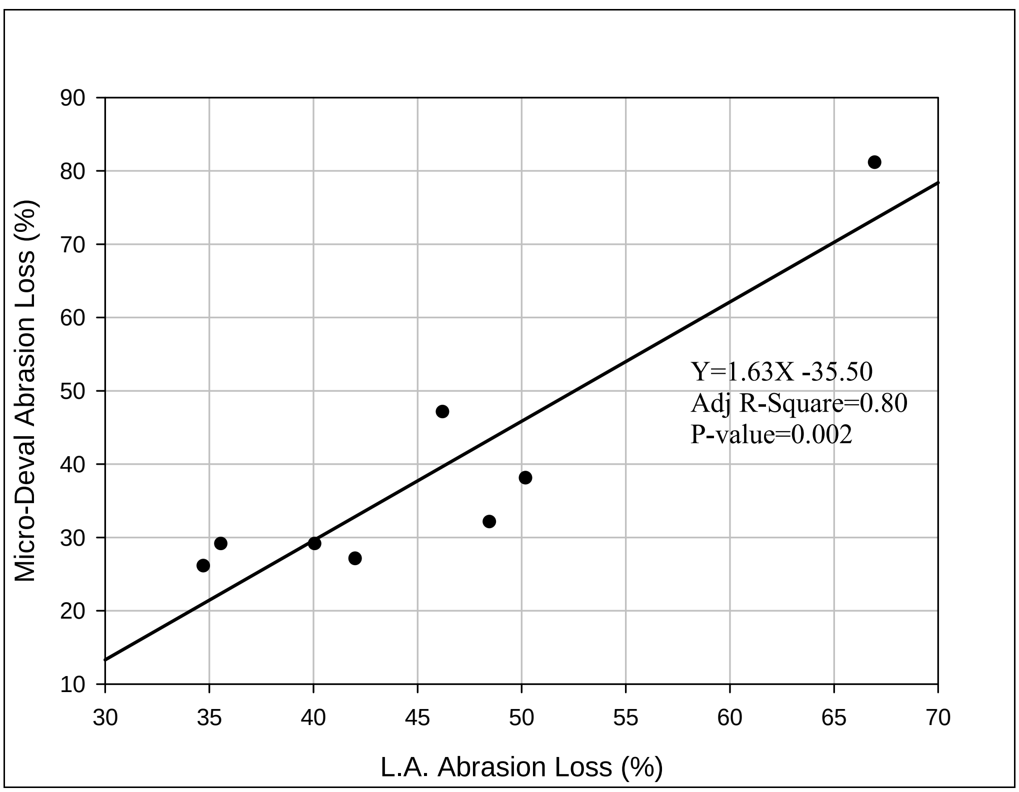


Figure 5-4. Plot of Micro-Deval abrasion loss vs. Los Angeles abrasion loss (%).

5.2.6 Relationship between Los Angeles Abrasion Loss and Soundness Loss

A linear regression analysis was conducted to determine the correlation between the LA abrasion loss and soundness loss. From the graph shown in Figure 5-5, a positive correlation may be observed between the two properties. An adjusted R-square value of 0.74 was obtained.

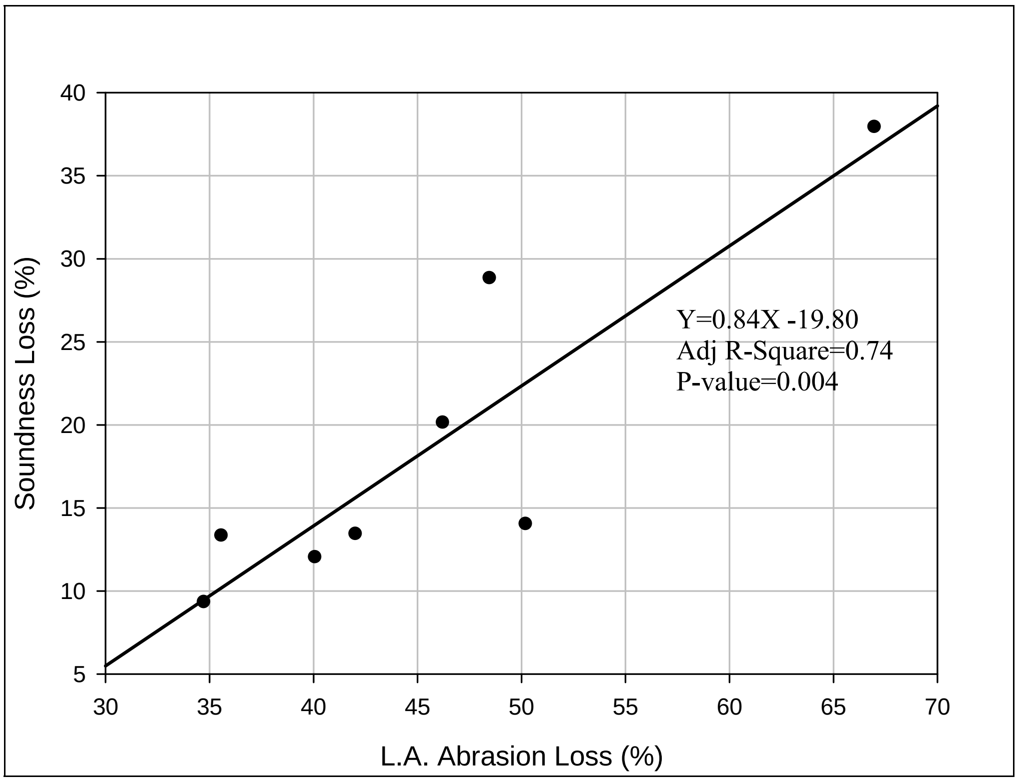


Figure 5-5. Plot of soundness loss (%) vs. Los Angeles abrasion loss (%).

5.2.7 Relationship between Micro-Deval Abrasion Loss and Soundness Loss

A linear regression analysis was conducted to determine the correlation between the Micro-Deval abrasion loss and soundness loss. From the graph shown in Figure 5-6, a positive correlation can be observed between the two properties. An adjusted R-square value of 0.63 was obtained.

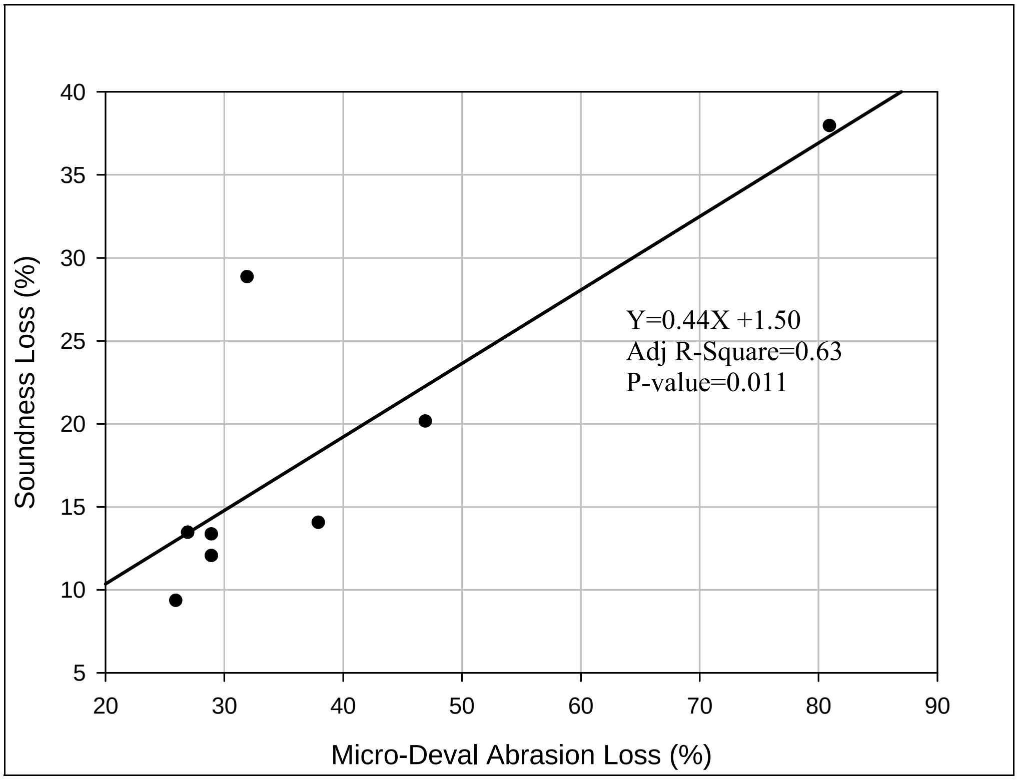


Figure 5-6. Plot of soundness loss (%) vs. Micro-Deval abrasion loss (%).

5.3 Relationship between Physical Properties of Aggregates and Concrete Properties

The relationships between the physical properties of aggregates and the compressive strength of concrete are presented in this section. Statistical analyses were done on the data from the second experimental design where the concrete containing low cement content were tested. Table 5-2 shows a summary of the data used for these analyses.

5.3.1 Correlation between Aggregate Properties and Concrete Properties

Correlation analyses were performed between the different aggregate properties and the compressive strength of concrete at 28 days. Table 5-3 shows the Pearson correlation between the aggregate properties and concrete compressive strengths at 28 days for w/c of 0.6 and 0.7. It can be seen that the compressive strength of concrete was significantly correlated with the LA abrasion loss, Micro-Deval abrasion loss and soundness loss, but not significantly related to percent passing minus No.200 sieve and shell content. In view of the above results, there was no need for further analysis to predict the compressive strength of concrete based on the levels of percentage passing 200 sieve or shell content.

Table 5-2. Summary of Data Used for the Statistical Analyses

Aggregate Source	Compressive Strength at 28 days		Minus No. 200 Sieve (%)	Abrasion Loss (%)		Soundness Loss (%)	Shell Content (%)
	(w/c = 0.6)	(w/c = 0.7)		LA	MD		
Miami Oolite	4720	3270	2.2	31	26	9.3	0
Fort Myers	4590	3120	0.98	36	29	13.3	0
Modified Miami Oolite	4650	2930	5	31	26	9.3	0
Modified Fort Myers	4300	3020	8	36	29	13.3	0
Inglis	3810	3080	3.98	42	27	13.4	0
Cabbage Grove	3630	2890	3.36	50	38	14	0
Ocala	3480	2790	4.08	46	47	20.1	0
Weber South	2760	2320	2.74	48	32	28.8	12.1
Coral Rock	4160	2940	0.85	40	29	12	0.6
Brooksville	2490	1950	9.22	67	81	37.9	0

Table 5-3. Pearson Correlation between Concrete Compressive Strength at 28 Days and Aggregate Properties

Aggregate Properties	% Minus No. 200 Sieve	LA Loss	Micro-Deval Loss	Soundness Loss	Shell Content
Compressive Strength at 28 days (w/c 0.6)	–0.403	–0.933	–0.741	–0.927	–0.491
Compressive Strength at 28 days (w/c 0.7)	–0.519	–0.868	–0.813	–0.959	–0.446

5.3.2 Regression Analysis

A linear regression model was used to establish the exact relationship between the properties of the aggregates and the compressive strength of concrete. Only the properties of the aggregates that had a significant correlation with the compressive strength of concrete were used for this analysis. Thus the LA abrasion loss, Micro-Deval (MD) abrasion loss and soundness loss were used for the analysis. A multiple linear regression was initially planned, however, as shown in Section 5-2, the aggregates properties are highly correlated with one another. Thus a single linear regression analysis was performed instead. The analysis was done for each of the water/cement ratios. Figures 5-7 through 5-12 show plots of compressive strength of concrete versus the various aggregate properties.

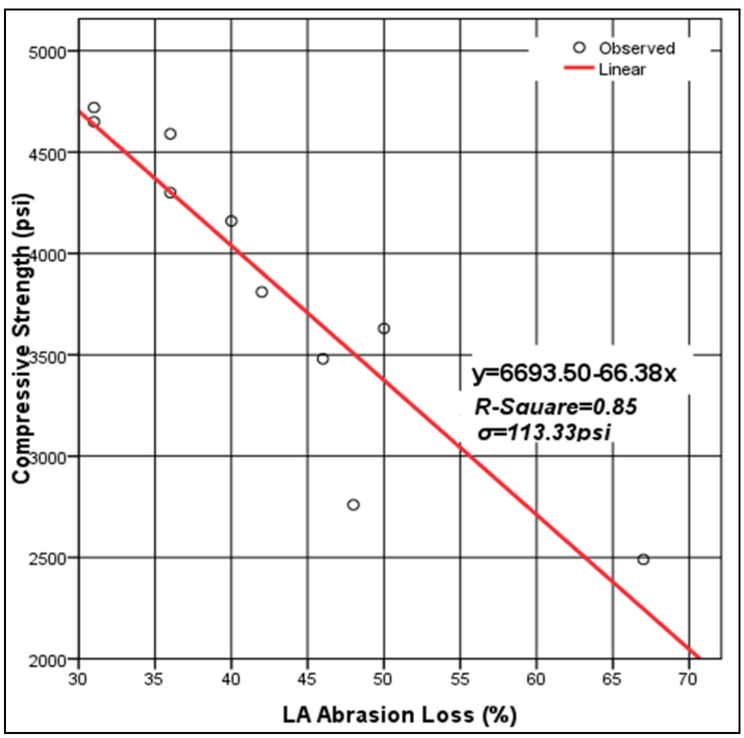


Figure 5-7. Plot of compressive strength of concrete at w/c of 0.6 versus Los Angeles abrasion loss aggregate.

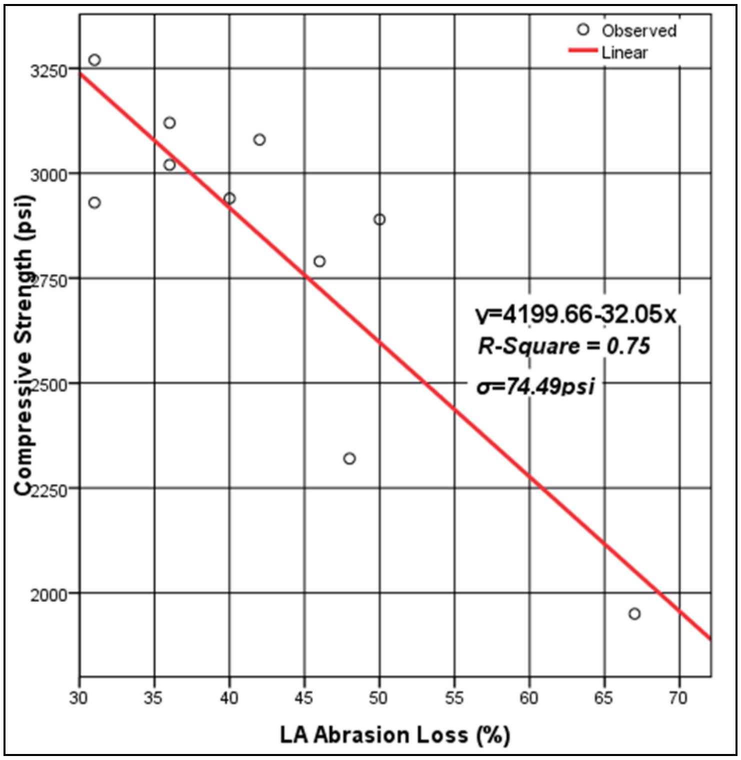


Figure 5-8. Plot of compressive strength of concrete at w/c of 0.7 versus Los Angeles abrasion loss of aggregate.

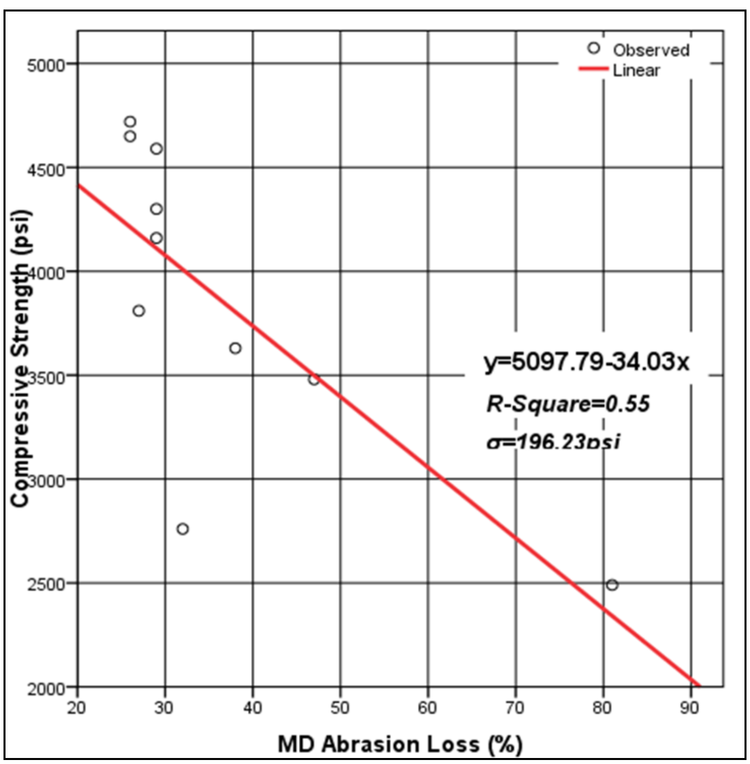


Figure 5-9. Plot of compressive strength of concrete at w/c of 0.6 versus Micro-Deval abrasion loss of aggregate.

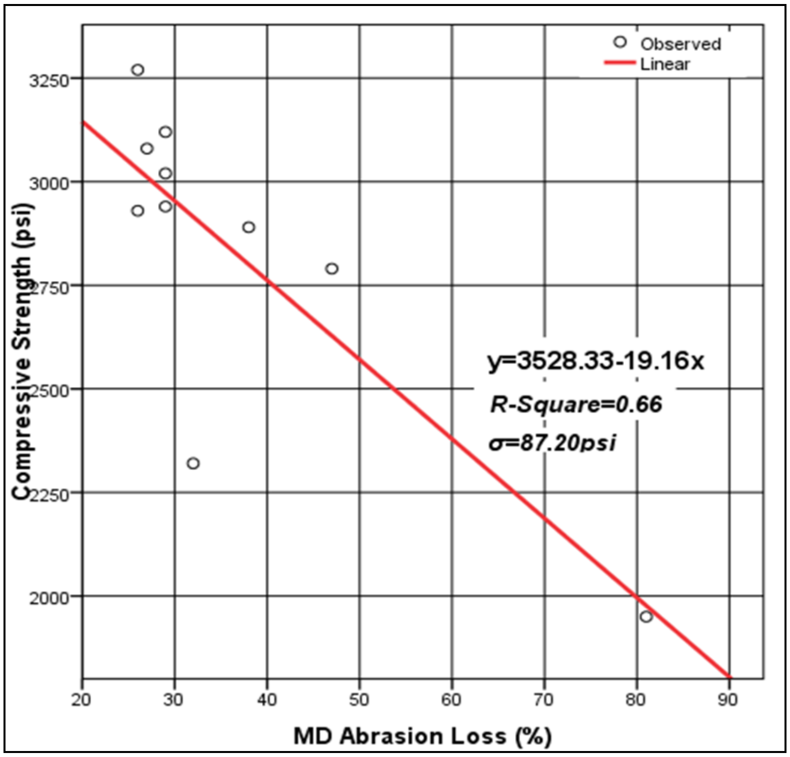


Figure 5-10. Plot of compressive strength of concrete at w/c of 0.7 versus Micro-Deval abrasion loss of aggregate.

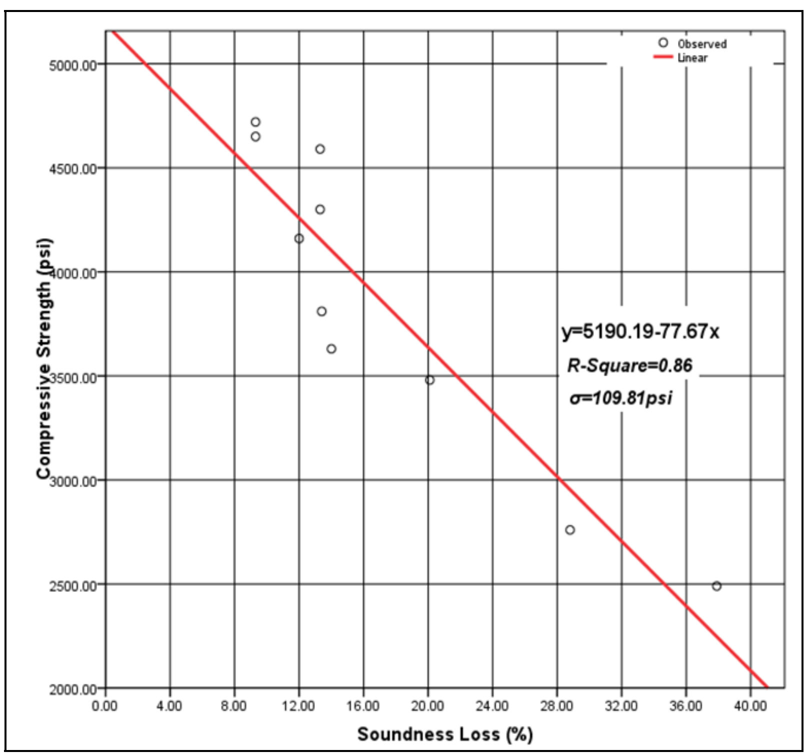


Figure 5-11. Plot of compressive strength of concrete at w/c of 0.6 versus soundness loss of aggregate.

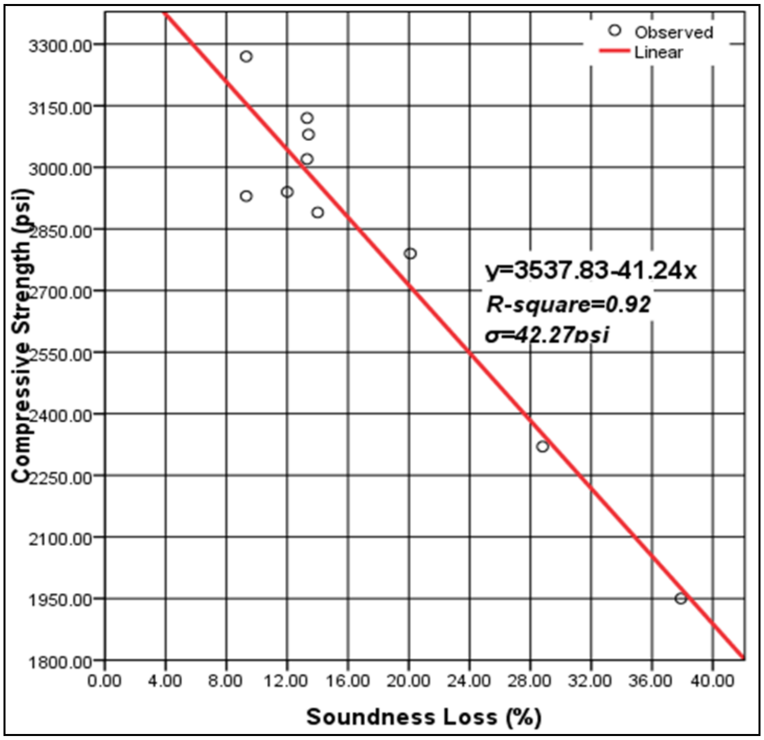


Figure 5-12. Plot of compressive strength of concrete at w/c of 0.7 versus soundness loss of aggregate.

Equations 5.1 and 5.2 show the relationship between compressive strength and LA abrasion loss for concrete with w/c of 0.6 and 0.7, respectively. The coefficients of determination (R^2) of the two equations are 0.85 and 0.75, respectively and the standard errors for the prediction are 113.33 psi and 74.49 psi, respectively.

For w/c = 0.6:

$$\text{compressive strength} = 6693.50 - 66.38 * \text{LA abrasion loss} \tag{5.1}$$

For w/c = 0.7:

$$\text{compressive strength} = 4199.66 - 32.05 * \text{LA abrasion loss} \tag{5.2}$$

Equations 5.3 and 5.4 show the relationship between compressive strength and MD abrasion loss for concrete with w/c of 0.6 and 0.7, respectively. The coefficients of determination (R^2) of these two equations are 0.55 and 0.75, respectively, and the standard errors for the prediction are 196.23 psi and 87.20 psi, respectively.

For w/c = 0.6:

$$\text{compressive strength} = 5097.79 - 34.03 * \text{MD abrasion loss} \tag{5.3}$$

For w/c = 0.7:

$$\text{compressive strength} = 3528.33 - 19.16 * \text{MD abrasion loss} \tag{5.4}$$

Equations 5.5 and 5.6 show the relationship between compressive strength and soundness loss for concrete with w/c = 0.6 and 0.7, respectively. The coefficients of determination (R^2) of the two equations are 0.86 and 0.92, respectively, and the standard errors for the prediction are 109.81 psi and 42.27 psi, respectively.

For w/c = 0.6:

$$\text{compressive strength} = 5190.19 - 77.67 * \text{soundness loss} \tag{5.5}$$

For w/c = 0.7:

$$\text{compressive strength} = 3537.83 - 41.24 * \text{soundness loss} \tag{5.6}$$

Although the above relationships have been determined for specific water/cement ratios, the significance of each expression shows that the compressive strength can be inferred from these aggregate properties. Thus for a specific water/cement ratio, a relationship can be established and this can be used to predict the suitability of new aggregates for used by FDOT.

5.3.3 Analysis on the Effects of Percent Passing No. 200 Sieve

In the experimental design, the amount of minus 200 for Miami Oolite and Fort Myers were increased by pulverizing the rocks. This controlled experiment was done to evaluate the effects of minus # 200 materials on the compressive strength of concrete with the aggregates having low LA abrasion values. Figures 5-13 and 5-14 show plots of compressive strength of concrete versus percent passing No. 200 sieve of aggregate for w/c of 0.6 and 0.7, respectively. It can be seen from these two figures that an increase in the percent passing No. 200 sieve was accompanied by a significant decrease in the compressive strength of the concrete.

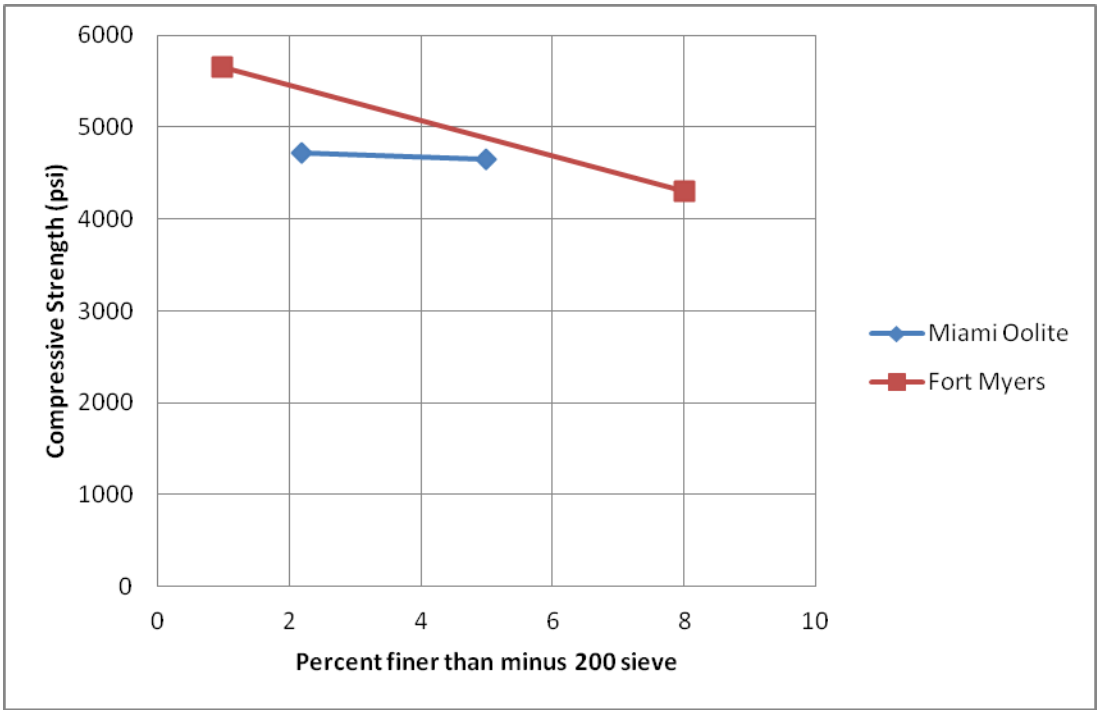


Figure 5-13. Plot of the effect of minus 200 on compressive strength (w/c = 0.6).

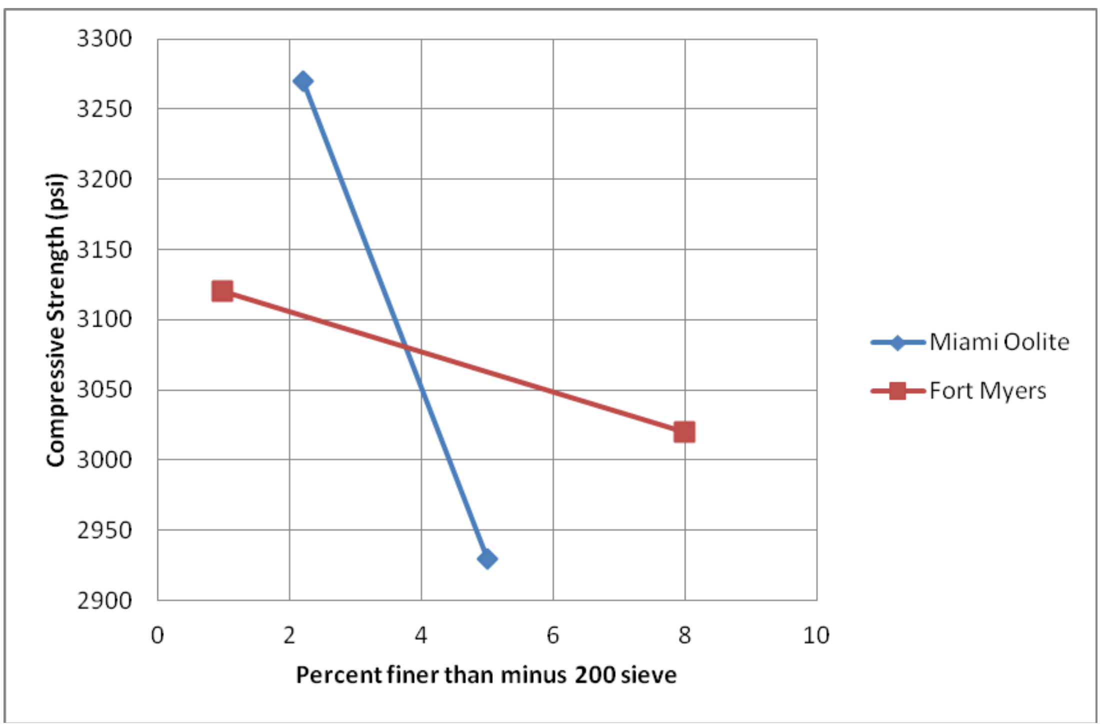


Figure 5-14. Plot of the effect of minus 200 on compressive strength (w/c = 0.7).

5.4 Summary

Linear regression analyses were performed to determine the relationships between the different properties. LA abrasion loss, Micro-Deval abrasion loss, soundness loss and percent passing No. 200 sieve were found to be positively correlated with one another. For the aggregates used in this study, the shell content was not significantly correlated to the other aggregate properties.

Linear regression analyses were performed to relate LA abrasion loss, MD abrasion loss, soundness loss to the compressive strength of concrete with w/c of 0.6 and 0.7. It was found that for a specific w/c, the compressive strength of concrete can be related to the LA abrasion loss, MD abrasion loss and soundness loss of aggregate. The shell content and percentage finer than the minus No. 200 sieve were found not to be significantly correlated with the compressive strength of concrete.

CHAPTER 6
DEVELOPMENT OF A TIERED AGGREGATE SPECIFICATION

6.1 Introduction

The compressive strength of the concrete was used as a basis in developing the tiered aggregate specification. It is postulated that for a given water/cement ratio, the differences in compressive strength are due to the differences in the properties of aggregates. The key properties of the aggregates that were used for the analysis include the percent passing No. 200 sieve, LA abrasion loss, MD abrasion loss, soundness loss and shell content. In Chapter 5, the relationships between these key properties of aggregates and the compressive strength of concrete were established. These relationships were used to relate the specified compressive strength of concrete to the required aggregate properties and to set limits for the aggregate properties in the tiered specification. Possible applications of the different categories of the aggregate in the proposed tiered aggregate specification are also addressed in this chapter.

6.2 Back Calculation Models for Prediction of Compressive Strength

Using the regression equations relating the compressive strength of concrete to aggregate properties as presented in Chapter 5, a 90% confidence interval (CI) was estimated for each relationship and a back prediction estimation was conducted for each relationship. The lower boundary of the 90% CI was selected as the minimum strength requirement for each corresponding aggregate property.

6.2.1 Back Prediction of Compressive Strength from Los Angeles Abrasion Loss

Figure 6-1 shows the plot of compressive strength against LA abrasion loss for concrete with a water/cement ratio of 0.6. The 95% CI is also shown on the plot. Three categories of LA abrasion loss are used to predict the compressive strength. Table 6-1 shows a summary of the predicted compressive strength. A similar analysis was done for concrete with a water/cement ratio of 0.7, as shown in Figure 6-2. The results of this analysis are shown in Table 6-2.

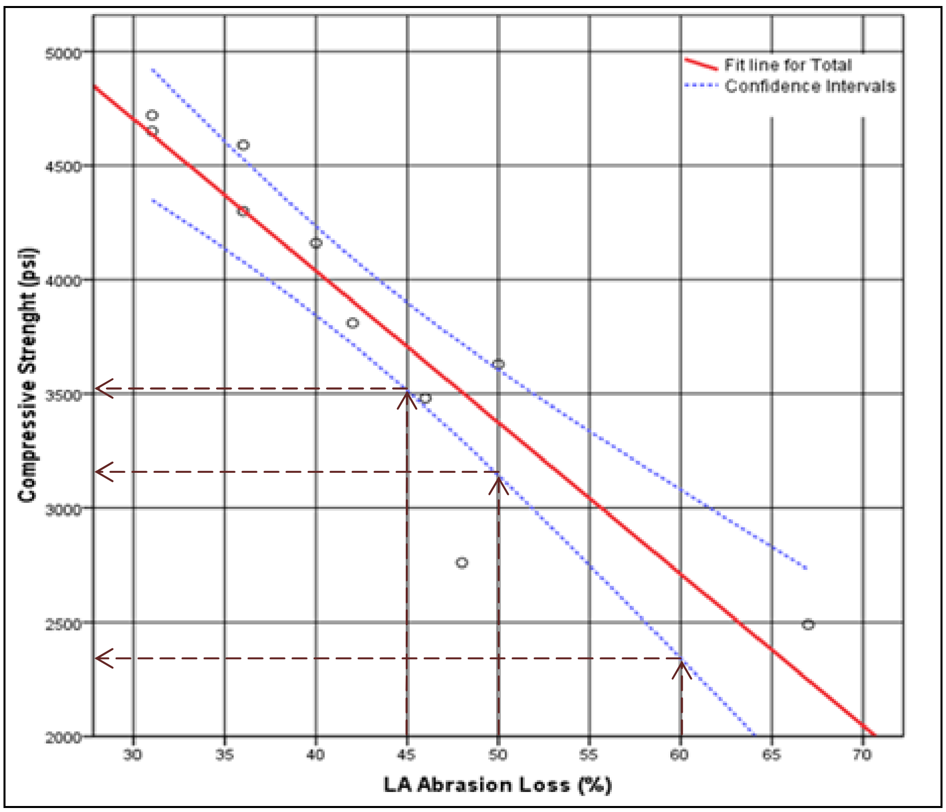


Figure 6-1. Plot of 90% CI for compressive strength vs. Los Angeles abrasion loss (w/c = 0.6).

Table 6-1. LA Abrasion Loss Prediction of Compressive Strength for Concrete of w/c = 0.6

LA Abrasion Loss Category	90% CI of Predicted Compressive Strength (psi)
LA ≤ 45	Comp ≥ 3520
45 < LA ≤ 50	3180 ≤ Comp < 3520
50 < LA ≤ 60	2300 ≤ Comp < 3180

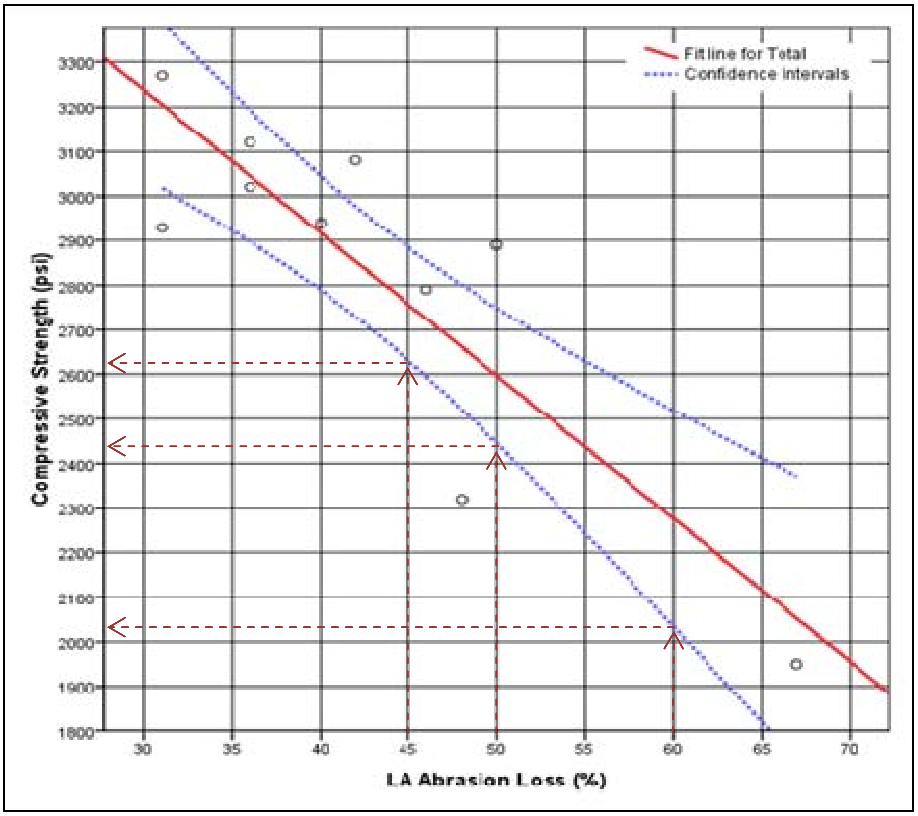


Figure 6-2. Plot of 90% CI for compressive strength vs. Los Angeles abrasion loss (w/c = 0.7).

Table 6-2. LA Abrasion Loss Prediction of Compressive Strength for Concrete of w/c = 0.7

LA Abrasion Loss Category	90% CI of Predicted Compressive Strength (psi)
LA ≤ 45	Comp ≥ 2630
45 < LA ≤ 50	2450 ≤ Comp < 2630
50 < LA ≤ 60	2045 ≤ Comp < 2450

6.2.2 Back Prediction of Compressive Strength from Micro-Deval Abrasion Loss

Figure 6-3 shows the plot of compressive strength against MD abrasion loss for concrete with a water/cement ratio of 0.6. The 95% CI is also shown on the plot. Three categories of MD abrasion loss are used to predict the compressive strength. Table 6-3 shows a summary of the predicted compressive strength. A similar analysis was done for concrete with a water/cement ratio 0.7 as shown in Figure 6-4. The results of this analysis are shown in Table 6-4.

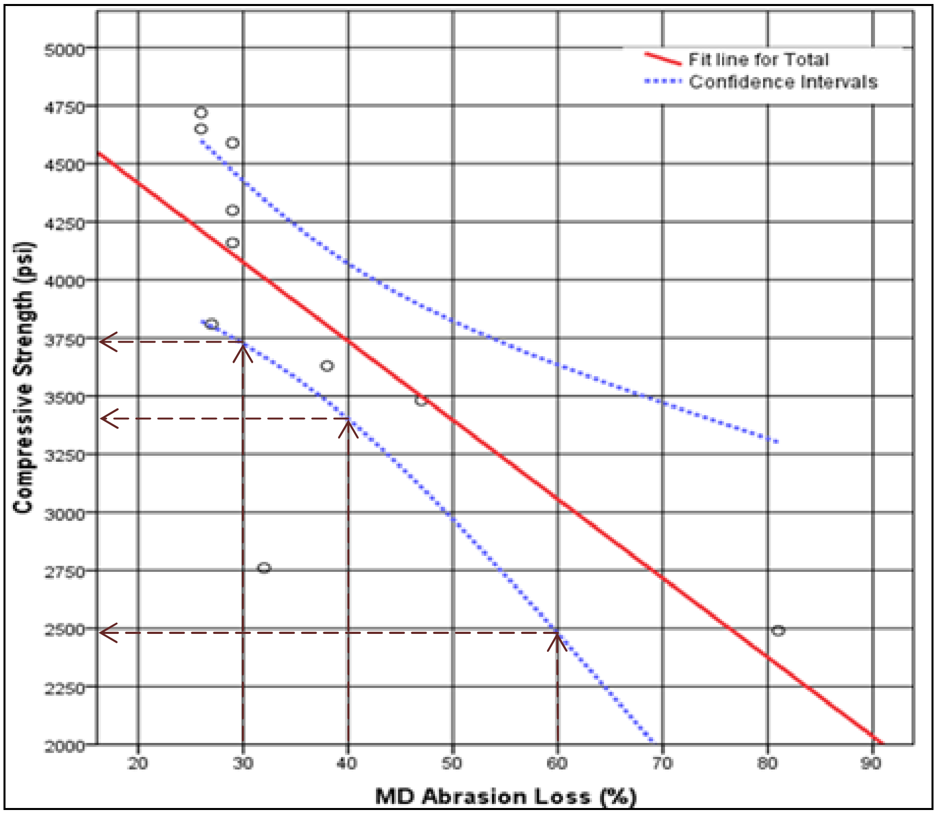


Figure 6-3. Plot of 90% CI for compressive strength vs. Micro-Deval abrasion loss (w/c = 0.6).

Table 6-3. MD Abrasion Loss Prediction of Compressive Strength for Concrete of w/c = 0.6

MD Abrasion Loss Category	90% CI of Predicted Compressive Strength (psi)
MD ≤ 30	Comp ≥ 3740
30 < MD ≤ 40	3385 ≤ Comp < 3740
40 < MD ≤ 60	2490 ≤ Comp < 3385

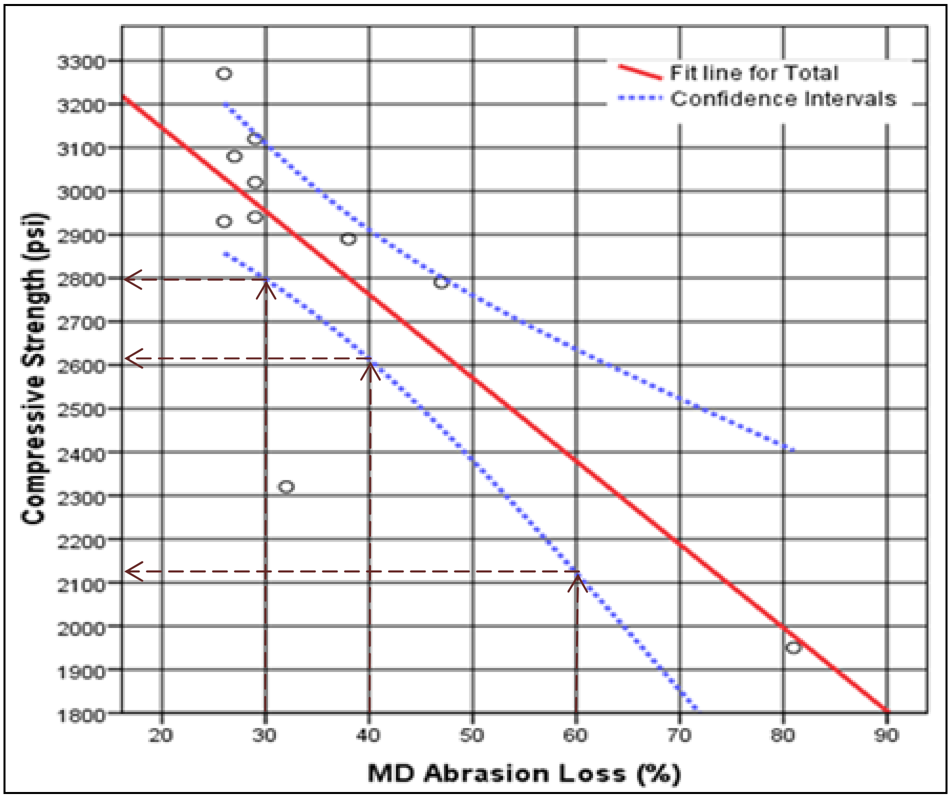


Figure 6-4. Plot of 90% CI for compressive strength vs. Micro-Deval abrasion loss (w/c = 0.7).

Table 6-4. MD Abrasion Loss Prediction of Compressive Strength for Concrete of w/c = 0.7

MD Abrasion Loss Category	90% CI of Predicted Compressive Strength (psi)
MD ≤ 30	Comp ≥ 2800
30 < MD ≤ 40	2620 ≤ Comp < 2800
40 < MD ≤ 60	2130 ≤ Comp < 2620

6.2.3 Back Prediction of Compressive Strength from Soundness Loss

Figure 6-5 shows the plot of compressive strength against soundness loss for concrete with a water/cement ratio of 0.6. The 95% CI is also shown on the plot. Three categories of soundness loss are used to predict the compressive strength. Table 6-5 shows a summary of the predicted compressive strength. A similar analysis was done for concrete with a water/cement ratio of 0.7 as shown in Figure 6-6. The results of this analysis are shown in Table 6-6.

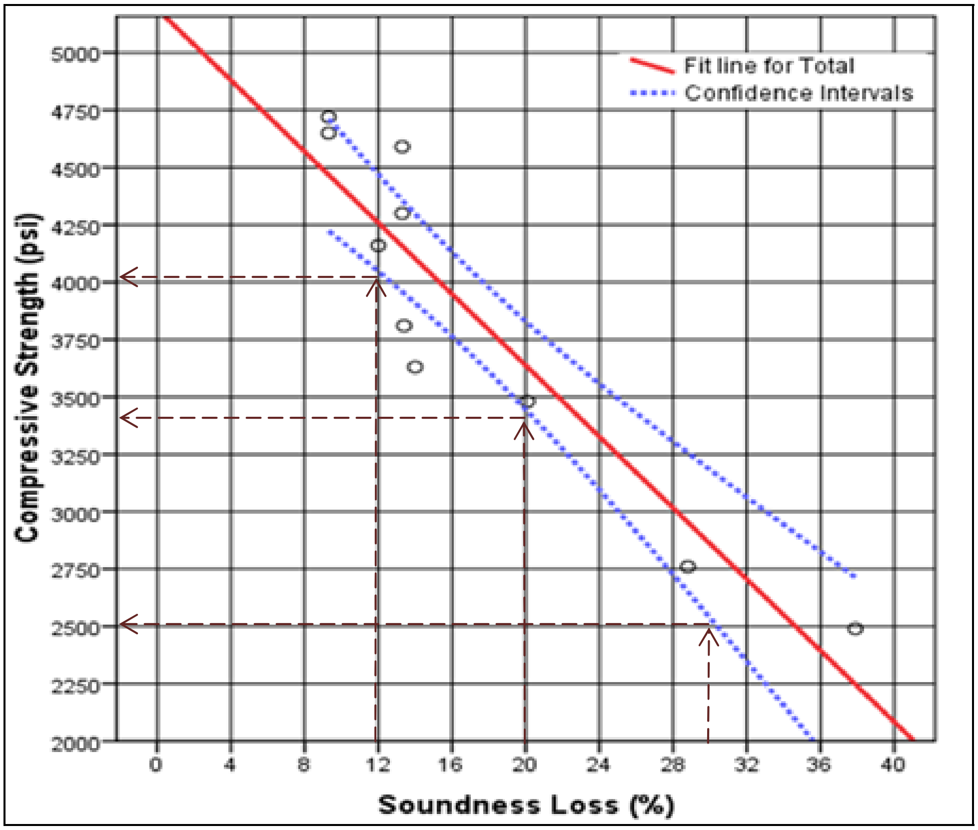


Figure 6-5. Plot of 90% CI for compressive strength vs. soundness loss (w/c = 0.6).

Table 6-5. Soundness Loss Prediction of Compressive Strength for Concrete of w/c = 0.6

Soundness Loss Category	90% CI of Predicted Compressive Strength (psi)
Soundness ≤ 12	Comp ≥ 4050
12 < Soundness ≤ 20	3450 ≤ Comp < 4050
20 < Soundness ≤ 30	2520 ≤ Comp < 3450

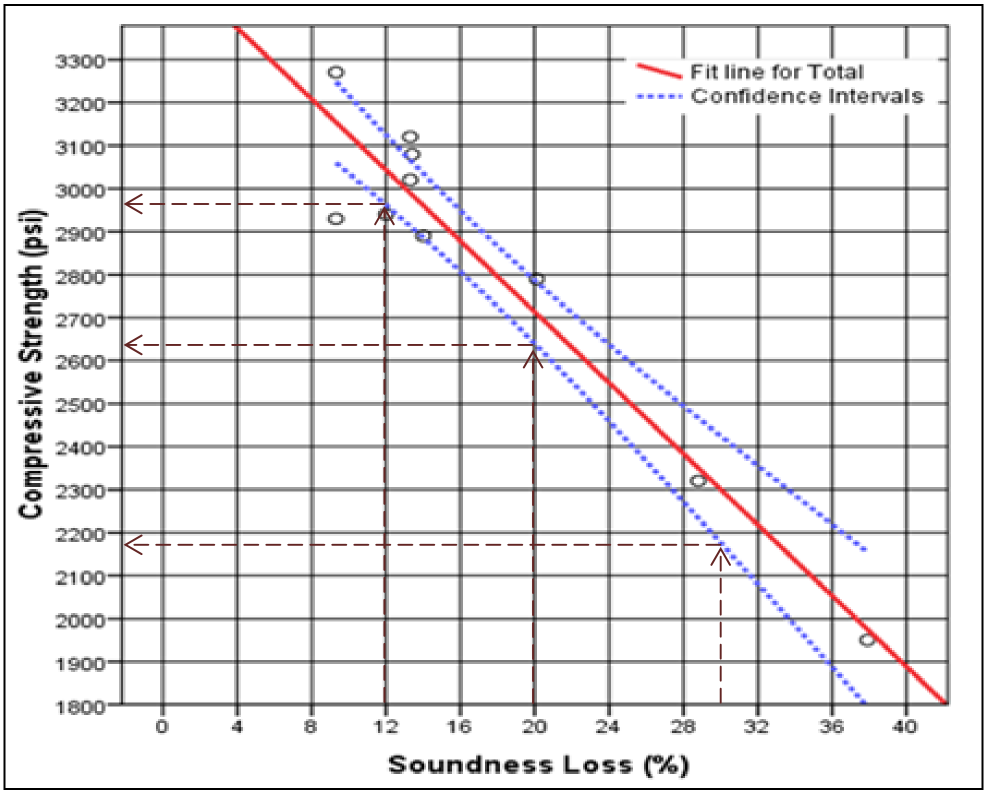


Figure 6-6. Plot of 90% CI for compressive strength vs. soundness loss (w/c = 0.7).

Table 6-6. Soundness Loss Prediction of Compressive Strength for Concrete of w/c = 0.7

Soundness Loss Category	90% CI of Predicted Compressive Strength (psi)
Soundness ≤ 12	Comp ≥ 2960
12< Soundness ≤ 20	2630 ≤ Comp < 2960
20< Soundness ≤ 30	2170 ≤ Comp < 2630

6.2.4 Back Prediction of Percent Passing No. 200 Sieve from LA Abrasion Loss

A regression equation relating the percent passing No. 200 sieve to L.A abrasion loss, which has been presented in Chapter 5, was used to determine the 90% confidence interval (CI) for the predicted percent passing No. 200 sieve from LA abrasion loss. Figure 6-7 shows the plot of the 90% CI for the predicted percent passing No. 200 sieve using the developed regression equation. Table 6-7 shows the predicted percent passing No. 200 for the three categories of the LA abrasion loss.

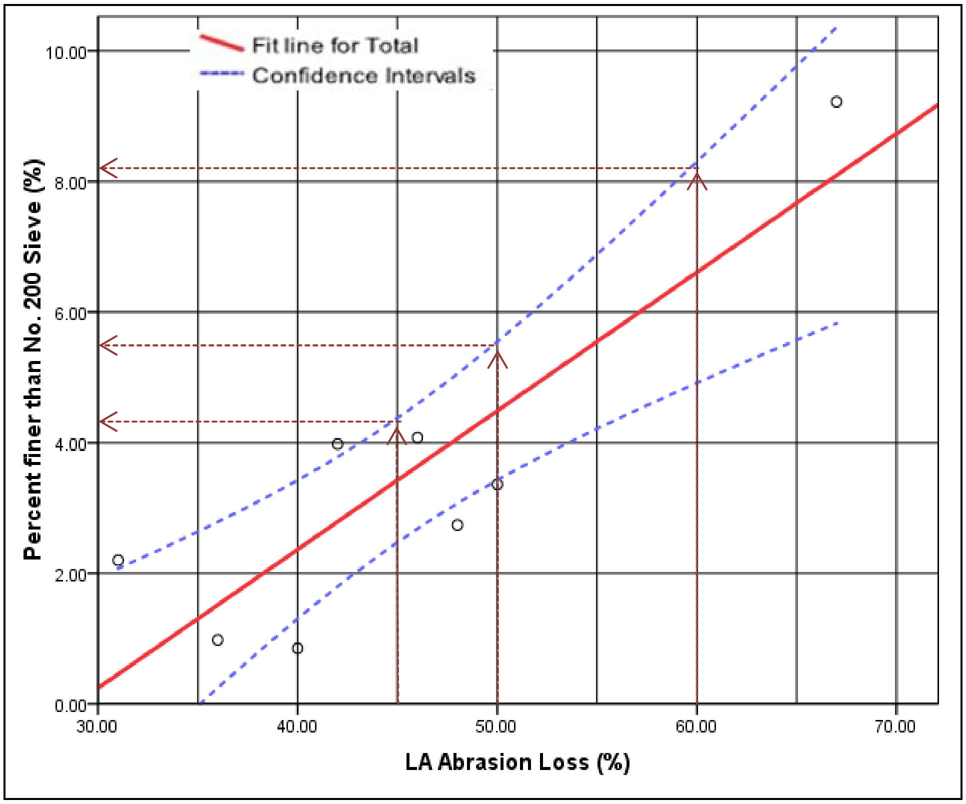


Figure 6-7. Plot of 90% CI for minus 200 vs. Los Angeles abrasion loss.

Table 6-7. LA Abrasion Loss Prediction of Percent Minus 200

LA Abrasion Loss Category	90% CI of Predicted minus 200 (%)
LA ≤ 45	Minus 200 ≤ 4.3
45 < LA ≤ 50	4.3 ≤ Minus 200 ≤ 5.5
50 < LA ≤ 60	5.5 ≤ Minus 200 ≤ 8.2

6.3 Proposed Developmental Tier Aggregate Specifications

Based on the preceding analysis and categorization, the data for concrete with a water/cement ratio of 0.6 were used to develop a possible developmental tiered aggregate specification to be adopted by FDOT. Table 6-8 shows a summary of the predicted compressive strength at 28 days for concrete with a water/cement ratio of 0.6 for the three categories of LA abrasion loss, MD abrasion loss and soundness loss. Three possible hierarchical levels of aggregates based on strength requirements were considered. Category A uses the current status quo in the 2010 FDOT specification, while categories B and C have been added.

Table 6-8. Summary of Categories of Aggregate Properties and Predicted Concrete Compressive Strength (w/c = 0.6)

Aggregate Property	Category	90% CI of Predicted Compressive Strength (psi)
LA abrasion loss	LA ≤ 45	Comp ≥ 3520
	45 < LA ≤ 50	3180 ≤ Comp < 3520
	50 < LA ≤ 60	2300 ≤ Comp < 3180
MD abrasion loss	MD ≤ 30	Comp ≥ 2800
	30 < MD ≤ 40	2620 ≤ Comp < 2800
	40 < MD ≤ 60	2130 ≤ Comp < 2620
Soundness loss	Soundness ≤ 12	Comp ≥ 4050
	12 < Soundness ≤ 20	3450 ≤ Comp < 4050
	20 < Soundness ≤ 30	2520 ≤ Comp < 3450

It must be noted that although the predicted level for minus No. 200 for LA ≤ 45 from Table 6-7 is 4.3%, the existing value of 3.75 was still used in the proposed specification. Also, in view of the inadequate data for varying shell content and also due to the possible harmful effect of high shell content on the durability of concrete, the same maximum value of 1% was kept in all three categories. Finally, in view of the significantly high correlation between LA abrasion loss and MD abrasion loss (Figure 5-4), the MD abrasion loss was not used in the proposed specification. Table 6-9 shows the proposed developmental tiered aggregate specification.

Table 6-9. Proposed Developmental Tiered Aggregate Specification

Category	Requirements
A	LA ≤ 45 Soundness ≤ 12 Minus No. 200 at use ≤ 3.75 Free shell ≤ 1.0
B	LA ≤ 50 Soundness ≤ 20 Minus No. 200 at use ≤ 5.5 Free shell ≤ 1.0
C	LA ≤ 60 Soundness ≤ 30 Minus No. 200 at use ≤ 8.2 Free shell ≤ 1.0

Based on the proposed developmental tiered aggregate specification, the different sources of aggregates used in this study were placed in the various categories with the exception of Modified Miami Oolite and Modified Fort Myers. Table 6-10 shows the categorization of these aggregates in the proposed tiered aggregate specification.

From Table 6-10, it may be seen that Weber South and Brooksville aggregate did not qualify for any of the three proposed categories. Weber South aggregate had a high percentage of shell of 12.1%, while Brooksville had a high LA abrasion loss of 67%, which made them unsuitable for concrete use. It must also be noted that although Fort Myers Aggregate was intended to be a Category A aggregate, the relatively high soundness loss of 13% downgraded it to a Category B aggregate based on the sample collected for this research.

Table 6-10. Categorization of Aggregates in This Study Using the Proposed Developmental Tiered Aggregate Specification

Category	Aggregate
A	Miami Oolite Coral Rock
B	Fort Myers Ocala Inglis Cabbage Grove
C	(None)

6.4 Other Considerations for the Tiered Specification

6.4.1 Gradation Requirements

From Section 3.3.3, it can be noted that most of the gradation requirements for #57 stone were not met by the non-standard aggregates tested, especially for the percent passing #4 and # 8 sieve. Although the aggregate producers were asked to produce aggregate meeting the #57 specifications, the tested aggregates did not meet the gradation specifications. The concrete produced with these aggregates were not adversely affected by their lack of proper gradation. This was due in part to the fact that the Shilstone mix design method, which is based on the optimization of gradation was employed.

Thus, in consideration of the results from this study, it is proposed that for Category B and C aggregates, the requirement on the percents passing #4 and #8 be relaxed by up to 5%. However, proper mix design should be conducted to achieve an optimum mix.

6.4.2 Percent Minus 200 Requirement

The proposed limits for percent minus 200 are based on the "at point of use." There was no data to be analyzed to determine the "at source" values. Based on historical data of the difference between percent passing No. 200 at the plant and at the mine as shown in Appendix E, the calculated mean and standard deviation of the difference was used to predict the at source value. From the data shown in Appendix E, the mean (μ) and the standard deviation (σ) of the difference for the data for mines 87-090, 87-339 and 87-145 were 0.85% and 0.35%, respectively.

The maximum difference between the difference between the "at point of use" limit and the "at source" limit can be approximated to be equal to the mean plus two standard deviations of the difference. This comes out to be: $\mu + 2\sigma = 0.85 + 2(0.35) = 1.55\%$. This value was rounded up to 2%. Thus a value of 2% was subtracted from the "at point of use" limit to obtain the "at source" limit. Consequently, for Category B aggregates, the limit for the "at source" percent minus 200 is proposed to be 3.5%, while that for Category C aggregates is proposed to be 6.2%.

6.5 Possible Applications for the Various Categories of Aggregates

The current FDOT specification already qualifies all aggregate in Category A to be used for both structural and non-structural concrete applications. However, aggregates in the proposed Category B and C would not be acceptable for any FDOT concrete applications under the current specification. The focus of this section is to look at the possibility of using Category B and C aggregates in some structural concrete applications and all non-structural concrete applications. A review of all possible FDOT concrete applications was conducted with focus on non-structural concrete applications. Table 6-11 summaries the current strength and other property requirements for a few selected applications. It must be noted that for all non-structural concrete applications, the current specification on strength as stipulated in Section 347-4.2 of the *2010 FDOT Standard Specification for Road and Bridge Construction* is a minimum 28-day compressive strength of 2500 psi.

Table 6-11. Current FDOT Strength and Other Requirements of Concrete for Selected Applications

Applications	Class of Concrete and Specified Minimum Strength (28 days)	Other Requirements
Non-Structural Concrete Applications:		
Sidewalks	NS, 2500 psi	
Concrete ditch and slope pavement	NS, 2500 psi	
Edgedrain (Draincrete)	NS, 2500 psi	
Concrete gutter	NS, 2500 psi	
Curb elements	NS, 2500 psi	
Traffic separator	NS, 2500 psi	
Structural Concrete Applications:		
Concrete pavement	Class I (pavement), 3000 psi	
Parking lots	Class I (pavement), 3000 psi	
Precast drainage products Culverts Concrete pipes Underdrains Endwalls French drains	Class I (a), Class II (a), or Class III (e) concrete, 3000, 3400, or 5000 psi, respectively	Apply chloride limits of 0.7 lb/yd^3 or 0.4 lb/yd^3 for slightly aggressive environment or moderately/extremely aggressive environment, respectively, to all box culverts for precast drainage systems manufactured at the precast plant (Section 346-3.1(a)). When precast box culverts or precast drainage products require a Class III concrete, the minimum cementitious materials will be 470 lb/yd^3. For concrete pipes, the gradation requirements for concrete aggregates as set forth in Sections 901 and 902 are not applicable (Section 449.2). For precast pipe culverts, the absorption of each concrete specimen should not be more than 9.0% when using Test Method A of ASTM C-497 (Section 410-3.3).
Manholes, inlets, junction boxes	Class II (a) or Class IV, 3,400 or 5,500 psi	Concrete meeting the requirement of ASTM C 478 4000 psi in lieu of Class I and Class II concrete is permitted for precast drainage systems (Section 346-3.1(e)).

Table 6-12 shows the 28-day compressive strength and 5-hour water absorption for the concrete evaluated in this study. From Table 6-12, it may be seen that all the concretes using Category A or B aggregates meet the strength requirements for non-structural concrete applications at the two water/cement ratios and respective cement contents used for this research. Appendix F shows some selected typical mix designs currently used by contractors on FDOT non-structural concrete applications. In Appendix F, it may be seen that currently, the minimum compressive strength of 2500 psi is achieved with a cement content of at least 470 lb/yd3. Thus from the results shown in Table 6-12, it can be concluded that the stipulated minimum compressive strength can be achieved with a cement content of at least 403 lb/yd3. Although there was no aggregate used in this study that could be classified as Class C aggregate, based on the developed relationship between aggregate properties and concrete strength, Category C aggregates are recommended to be used for non-structural applications with a lower strength requirement.

Table 6-12. Strength and 5-hour Absorption Properties of Concrete Evaluated in This Study

Aggregate Category	Aggregate	Compressive Strength (psi) 28 days		5-hour Absorption (%) 28 days	
		w/c = 0.6, Cement Content = 470 lb/yd^3	w/c = 0.7, Cement Content = 403 lb/yd^3	w/c = 0.6, Cement Content = 470 lb/yd^3	w/c = 0.7, Cement Content = 403 lb/yd^3
A	Miami Oolite	4720	3270	8.12	8.92
	Coral Rock	4160	2940	9.03	9.28
B	Fort Myers	4590	3120	8.17	10.06
	Ocala	3480	2790	10.24	10.028
	Inglis	3810	3080	9.25	9.76
	Cabbage Grove	3630	2890	10.54	10.95

Though the mix designs used for this study focused on non-structural concrete applications, the viability of using these aggregates for structural concrete applications was assessed. It has already been established in Section 4.2 that the mechanical behavior of the concrete produced from the aggregates used in this study is similar to that produced from standard aggregates. Thus, the standard relationships already established by ACI codes and other codes are applicable. Concrete made with Class B aggregates can meet the strength requirements for concrete Class I, Class I (pavement), and Class II (a) when compared to the concrete containing water/cement ratio of 0.6. The viability of using these aggregates for concrete pavement applications is analyzed in Section 6.6. This further analysis is conducted on the concrete produced for this study.

For precast drainage products, manholes, inlets and junction boxes, the strength requirement for Class I (a) and Class II (a) is met when producing concrete with water/cement ratio of 0.6. However, for precast pipe concrete, the absorption requirement of 9% is not met by the concrete containing class B aggregates. It must, however, be noted that the maximum water/cement ratio for Class I (a), Class II (a) and Class III concrete is 0.53, 0.53 and 0.44, respectively. Using a lower water/cement ratio would produce higher strength and less absorption than the ones used for this study. From the foregoing considerations, it is recommended Class B aggregates be further evaluated for use in such applications with special emphasis in meeting the water absorption requirements of the concrete.

6.6 Evaluation of the Potential Performance of Concrete Pavement Using Category B Aggregate

In Section 6.2, it may be seen that the strength requirements for concrete pavement and parking lots are met by concrete using Category B aggregate even at a high water/cement ratio of 0.6. The feasibility of using Class B aggregate in concrete pavement application is evaluated in this section. A finite element analysis evaluating the potential performance of concrete containing either Class A or B aggregates was performed.

6.6.1 Finite Element Model Used to Perform Stress Analysis

Concrete from Class A and Class B aggregates and with a water/cement ratio of 0.6 were evaluated to determine their potential performance in a typical concrete pavement in Florida. Their elastic modulus, compressive strength, unit weight and coefficient of thermal expansion were used to model the concrete.

The Finite Element Analysis of Concrete Slabs, Version IV (FEACONS IV) program was used to perform the stress analysis. The FEACONS program was previously developed at the University of Florida for FDOT for the analysis of PCC pavements subjected to load and thermal effects, and has demonstrated to be a fairly effective and reliable tool for this type of analysis. Figure 6-8 shows the finite element model used to perform the stress analysis. Analysis using the FEACONS model was performed to determine stresses in a 10-inch concrete pavement slab if it were loaded by 22-kip axle load at two critical loading positions, namely at the slab corner and at the middle of the slab edge as shown in Figure 6-9. The middle of the slab edge is the most critical loading position in the daytime when the temperature differential in the slab is positive, while the slab corner is the most critical loading position at night when the temperature differential is negative. The following parameters were used to model the concrete pavement:

1) Slab thickness = 10″; slab length = 15′; slab width = 12′
2) Subgrade modulus, k_s = 0.3 kci; edge stiffness, k_e = 30 ksi
3) Joint linear stiffness, k_l = 500 ksi; joint torsion stiffness, k_t = 1000 k-in./in.

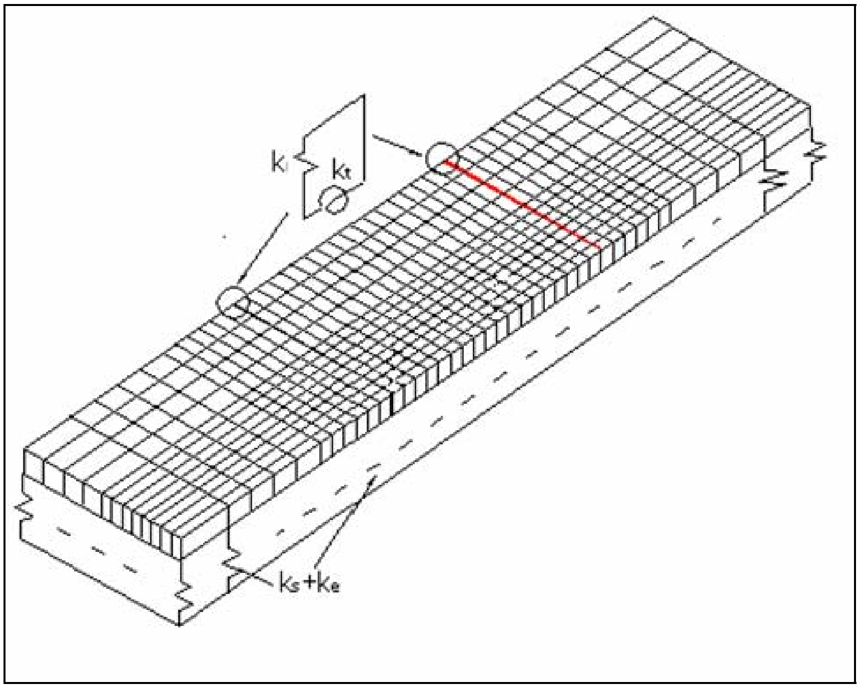


Figure 6-8. Finite element model used in FEACONS IV analysis.

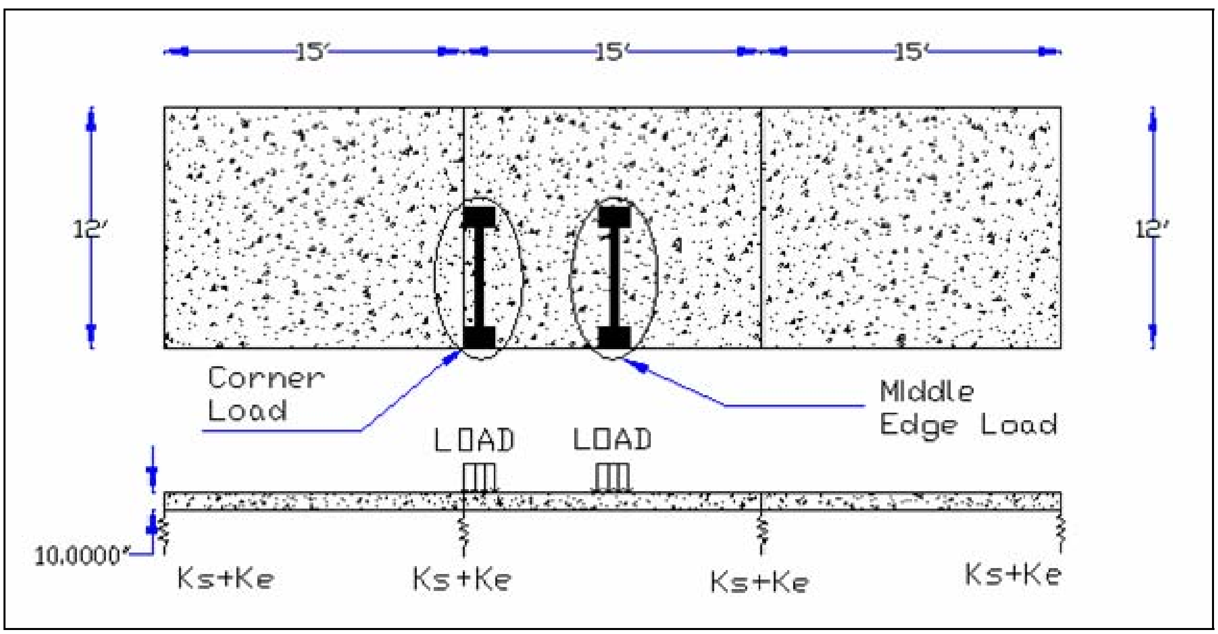


Figure 6-9. 22-kip wheel load at slab corner and middle edge.

6.6.2 Results of Stress Analysis Using FEACONS IV Analysis

Using the stresses calculated by FEACONS IV program and the determined flexural strength, stress-strength ratios were calculated to compare the potential performance of the concrete containing the different categories of aggregates. Tables 6-13 shows the stress/strength ratios at the corner and middle edge of the slab with +20°F, –20°F and +0°F temperature differential. It can be seen that all the stress ratios for the critical positions are less than one. Also, the stress ratios for some of the category B aggregates are comparable to those for the Category A aggregates. In general, it can be seen that it is potentially feasible to use some of the Category B aggregates for concrete pavement and parking lot applications.

6.7 Proposed Developmental Tiered Aggregate Specifications with Possible Applications

Based on the aforementioned evaluation, Table 6-14 presents a proposed developmental tiered aggregate specification and the possible applications. Category A aggregate are aggregates which meet the current FDOT specification and can be used on all FDOT applications.

The proposed applications of Category B aggregates include all non-structural concrete applications, manholes, inlets and junction boxes. Non-structural concrete applications require a minimum compressive strength of 2,500 psi, which can be easily met by the concrete made with Category B aggregates as shown in Table 6-12. Manhole, inlet and junction box applications require a minimum concrete compressive strength of 3,400 psi, which can be met by concrete made with Category B aggregates using a w/c of 0.6, as shown in Table 6-12. It is to be noted that the laboratory concrete mixes which were evaluated in this study used various amounts of water reducing admixtures in order to achieve fixed w/c of 0.6 and 0.7 with corresponding fixed cement contents of 470 and 403 lb/yd^3. In an actual production mixes where water reducing admixtures may not be used, more water would need to be used to produce the same concrete workability, and a higher cement content would need to be used to obtain the same w/c in order to obtain the same concrete strength. The contractor would need to weigh the added cost of water reducing admixture against the cost of additional cement in deciding the concrete mix design to be adopted.

The proposed applications of Category C aggregates include all non-structural concrete applications which require relatively lower strengths. As shown in Table 6-8, the predicted compressive strength of concrete using a Category C aggregate with LA abrasion loss between 50% and 60% and with a w/c of 0.6 is 2300 to 3180 psi. It is thus possible to achieve a compressive strength of 2,500 psi or more using a Category C aggregate. To adopt a concrete mix to be used, the contractor would need to design and test the concrete mix to ensure that the specified strength can be met.

6.8 Summary

A three-tier developmental aggregate specification was developed based on the strength requirements currently specified in the *2010 FDOT Standard Specification on Roads and Bridges*. The possible applications in which these tiered aggregates could be used were also recommended. The possible use of the second-tier aggregate for concrete pavement and parking lot applications was also assessed and found to be feasible.

Table 6-13. Computed Maximum Stresses and Stress-Strength Ratios for Concrete Containing Different Aggregates

Aggregate		Poisson's Ratio	Mean 28-day				Computed Stress (psi)		Stress Ratio	
Category	Source		Water Saturated CTE (10^{-6}/°F)	Compressive Strength (psi)	Modulus of Elasticity (ksi)	Modulus of Rapture (psi)	Corner	Middle Edge	Corner	Middle Edge
+20										
A	Miami Oolite	0.24	7.92	4720	4111	670	423	509	0.63	***0.76***
	Coral Rock	0.25	9.09	4160	3700	650	434	518	0.67	***0.80***
B	Fort Myers	0.27	7.67	4590	3944	655	419	488	0.64	***0.75***
	Inglis	0.22	10.73	3810	3289	570	442	530	0.78	***0.93***
	Cabbage Grove	0.27	9.96	3630	3256	540	435	504	0.81	***0.93***
	Ocala	0.27	7.59	3480	3300	535	397	431	0.74	***0.81***
–20										
A	Miami Oolite	0.24	7.92	4720	4111	670	393	374	***0.59***	0.56
	Coral Rock	0.25	9.09	4160	3700	650	417	399	***0.84***	0.61
B	Fort Myers	0.27	7.67	4590	3944	655	380	3 61	***0.58***	0.55
	Inglis	0.22	10.73	3810	3289	570	433	416	***0.76***	0.73
	Cabbage Grove	0.27	9.96	3630	3256	540	420	403	***0.78***	0.75
	Ocala	0.27	7.59	3480	3300	535	328	311	***0.61***	0.58
0										
A	Miami Oolite	0.24	7.92	4720	4111	670	161	177	0.24	0.26
	Coral Rock	0.25	9.09	4160	3700	650	157	173	0.24	0.27
B	Fort Myers	0.27	7.67	4590	3944	655	161	177	0.25	0.27
	Inglis	0.22	10.73	3810	3289	570	150	165	0.26	0.29
	Cabbage Grove	0.27	9.96	3630	3256	540	153	168	0.28	0.31
	Ocala	0.27	7.59	3480	3300	535	154	169	0.29	0.32

Table 6-14. Proposed Developmental Tiered Aggregate Specification with Applications

Category	Requirements	Proposed Applications	Possible Applications (with further investigation)
A	LA loss ≤ 45% Soundness loss ≤ 12% <u>Minus 200</u> With LA loss < 30 at source = 2.5% With LA loss > 30 at source = 1.75% At point of use ≤ 3.75% Free shell ≤ 1.0%	All FDOT applications	
B	LA ≤ 50% Soundness ≤ 20% <u>Minus 200</u> At source = 3.5% At point of use ≤ 5.5% Free shell ≤ 1.0%	All non-structural concrete applications Manholes, inlet, and junction boxes	Concrete Pavement Parking lots Precast drainage products
C	LA ≤ 60% Soundness ≤ 30% <u>Minus 200</u> At source = 6.2% At point of use ≤ 8.2% Free shell ≤ 1.0%	All non-structural concrete applications	

Note: The percent passing #4 and #8 sieve sizes for Category B and Category C aggregates is proposed to be relaxed by up to 5%.

CHAPTER 7
SUMMARY AND RECOMMENDATIONS

7.1 Summary of Findings

A laboratory testing program was conducted to evaluate the properties of concrete made with non-standard aggregates which do not meet current FDOT aggregate specifications, in order to assess the feasibility of using these aggregates in FDOT concrete applications and to develop tiered aggregate specifications to allow for some of these aggregates to be used. Ten different aggregate sources were selected for use in this study. They included two control standard aggregates which met the 2010 FDOT aggregate specifications and eight aggregates which did not meet at least one of the aggregate specification requirements. The required aggregate properties which were not met included the maximum allowable LA abrasion loss, percent passing No. 200 sieve, and shell content. The concrete which was evaluated had water/cement ratios of 0.6 or 0.7. The main findings of the laboratory study are summarized as follows:

(1) The workability and air content of the concrete mixes using the non-standard aggregates were similar to those using the standard aggregates, though most of the non-standard aggregates had higher percentages of material passing the No. 8, No. 4 and No. 200 sieves. This was attributed partly to use of the Shilstone mix design method which allowed for optimization of the aggregate gradation.

(2) Though the non-standard aggregates had relatively higher water absorption than the standard aggregates, this difference did not affect the control of the fresh concrete properties. This was possibly helped by the use of a 45-minute soak time for the aggregates during concrete mixing.

(3) LA abrasion loss, Micro-Deval abrasion loss, soundness loss and percent passing No. 200 sieve of the aggregates were found to be positively correlated with one another. However, shell content did not correlate well with the other aggregate properties.

(4) Regression equations were developed to relate the compressive strength, splitting tensile strength and modulus of rupture of the concrete using non-standard aggregate to one another. The forms of the regression equation used by ACI were found to work well for these data.

(5) For a fixed water/cement ratio of 0.6 or 0.7, the compressive strength of concrete was found to correlate well with LA abrasion loss, Micro-Deval abrasion loss, and soundness loss. Back prediction models were developed to determine the required values for these aggregate properties to achieve certain required compressive strength of concrete. These back prediction models were used to develop the recommended tiered aggregate specifications based on different required compressive strength of concrete.

7.2 Recommendations

Based on the results from this study, a three-tier developmental aggregate specification as presented in Table 6-14 is recommended for use by FDOT. The first tier (Category A) aggregate has the same requirements as the current FDOT aggregate specification, while the second and third tier (respectively, Category B and Category C) aggregates have somewhat relaxed requirements on LA abrasion loss, soundness loss, percent passing No. 200 sieve and gradation. Category B aggregate is recommended for use in non-structural concrete applications and possibly for concrete pavement and parking lots, while Category C aggregate is recommended only for non-structural concrete applications. It is recommended that further study be conducted to evaluate the suitability of using Category B aggregate for concrete pavement, parking lot and precast concrete pipe applications.

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APPENDIX A
SURVEY OF STATE DOTs TO VERIFY THEIR ADOPTION OF TIERED AGGREGATE SPECIFICATIONS

Request Sent to State DOTs

Short Survey from Florida DOT: States' Use of Tiered Aggregate Specifications

Definition: Defining different construction Aggregate physical property requirements in your Specifications depending on the end use of the Aggregate.

Florida DOT is currently researching the use of Tiered Aggregate Specifications. Some considerations include using a higher L.A. Abrasion or higher Minus #200 sieve contents for non-structural items.

Question: Does your State have a similar specification already in place? If yes, can you please provide URL links or other citations to the information?

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Aggregate Acceptance
How are we doing? *Please let us know if we are meeting your expectations. Select the following link to participate in our 2011 Survey: http://www.dot.state.fl.us/statematerialsoffice/administration/survey/index.shtm*
~~~~~~~~~~~~~~~~~~~~~~~~

Table A-1. Response from State DOTs

Abbreviation of States	Tiered Specification	Specification Link	Comments	Email
AL	Y	http://www.dot.state.al.us/conweb/doc/Specifications/2008%20Standard%20Specifications%20for%20Highway%20Construction.pdf	Y	stricklandc@dot.state.al.us
AK				
AZ				-
AR				
CA	Y	http://www.dot.ca.gov/hq/esc/oe/specifications/std_specs/2010_StdSpecs/2010_StdSpecs.pdf	Y	mfatemi@dot.ca.gov
CO	N	N/A		James.Zufall@dot.state.co.us
CT	N	N/A		Daniel.Guzzo@ct.gov
DE	N	N/A		James.Pappas@state.de.us
DC	N	N/A		wasi.khan@dc.gov
FL	Y	ftp://ftp.dot.state.fl.us/LTS/CO/Specifications/SpecBook/2010Book/901.pdf	Y	john.shoucair@dot.state.fl.us
GA	Y	http://www.dot.ga.gov/doingbusiness/TheSource/Pages/specifications.aspx	Y	jgerman@dot.ga.gov
HI				
ID				-
IL	Y	http://www.dot.il.gov/desenv/spec2012/Div1000.pdf	Y	Sheila.Beshears@illinois.gov
IN	Y	See Work sheet Tab	Y	RWALKER@indot.IN.gov
IA				
KS	JPS	http://www.ksdot.org/burConsMain/specprov/2007SSDefault.asp#1100	Y	RickK@ksdot.org
KY	Y	http://transportation.ky.gov/Construction/Standard%20amd%20Supplemental%20Specifications/800%20Materials%2008.pdf	Y	Amanda.Dees@ky.gov
LA	N	http://www.dotd.la.gov/highways/specifications/documents/2006%20Standard%20Specifications%20for%20Roads%20and%20Bridges%20Manual/14%20-%202006%20-%20Part%20X%20-%20Materials.pdf	Y	Bert.Wintz@LA.GOV
ME	N	N/A	Y	Bruce.Yeaton@maine.gov
MD		http://www.sha.maryland.gov/index.aspx?pageid=42	Y	DSajedi@sha.state.md.us
MA	N	N/A		john.grieco@state.ma.us
MI		-		
MN				
MS	N	N/A		wjrobinson@mdot.state.ms.us

MO	Y	http://www.modot.mo.gov/business/standards_and_specs/Sec1002.pdf See also sections 1003, 1004, 1005, 1006 and 1007	N	William.Stalcup@modot.mo.gov
MT				-
NE	Y	http://www.nebraskatransportation.org/ref-man/specbook-2007.pdf		Mark.Lindemann@nebraska.gov
NV	N	N/A		mgriswold@dot.state.nv.us
NH				-
NJ	Y	http://www.state.nj.us/transportation/eng/specs/2007/Division.shtml	Y	eileen.sheehy@dot.state.nj.us
NM	Y	http://www.nmshtd.state.nm.us/upload/images/Contracts_Unit/2007_Specs_for_Highway_and_Bridge_Construction.pdf	Y	Bryce.Simons@state.nm.us
NY	Y	https://www.nysdot.gov/main/business-center/engineering/specifications/english-spec-repository/espec9-1-11english.pdf	Y	wskerritt@dot.state.ny.us
NC	Y	Sect. 1014-2(B), Sect. 1014-2(D)	Y	ocordle@ncdot.gov
ND				
OH	Y	http://www.dot.state.oh.us/Divisions/ConstructionMgt/OnlineDocs/Specifications/2010CMS/700/703.htm	Y	Jeff.Wigdahl@dot.state.oh.us
OK				
OR	N	N/A		keith.r.johnston@odot.state.or.us
PA	Y	ftp://ftp.dot.state.pa.us/public/bureaus/design/pub408/pdf%20for%20printing%202011%201/703.pdf	Y	TRAMIREZ@pa.gov
RI	N	N/A		mfelag@dot.ri.gov
SC	N	N/A		FletcherMO@dot.state.sc.us
SD	Y	http://www.sddot.com/Operations/specifications/specbook_div3_04.htm	Y	Darin.Hodges@state.sd.us
TN				
TX	Y	ftp://ftp.dot.state.tx.us/pub/txdot-info/cmd/cserve/specs/2004/standard/s421.pdf for standard spec. ftp://ftp.dot.state.tx.us/pub/txdot-info/cmd/cserve/specs/2004/prov/sp421035.pdf for soundness change included in special provision to standard spec.	Y	elizabeth.lukefahr@txdot.gov
UT				-
VT				
VA				
WA				-
WV	N	N/A		David.B.Matics@wv.gov
WI				
WY				

APPENDIX B
AGGREGATE TEST RESULTS

Table B-1. Sieve Analysis and Minus 200 Test Results – Miami Oolite-1, Fort Myers-1, and Inglis-1

Aggregate Source	Miami Oolite-1			Fort Myers-1			Inglis-1		
	Percentage Passing			Percentage Passing			Percentage Passing		
Sieve Size	Sample 1	Sample 2	Sample 3	Sample 1	Sample 2	Sample 3	Sample 1	Sample 2	Sample 3
1½”	100	100	100	100	100	100	100	100	100
1”	100	100	100	100	100	100	96	97	96
½”	37	35	34	29	28	34	49	49	45
#4	3	3	3	3	2	3	12	12	7
#8	2	3	2	2	2	2	9	9	5
Minus 200	1.58	1.81	1.92	0.95	1.07	0.92	3.69	3.89	4.38

Table B-2. Sieve Analysis and Minus 200 Test Results – Cabbage Grove-1, Ocala-1, and Miami Oolite

Aggregate Source	Cabbage Grove-1			Ocala-1			Miami Oolite		
	Percentage Passing			Percentage Passing			Percentage Passing		
Sieve Size	Sample 1	Sample 2	Sample 3	Sample 1	Sample 2	Sample 3	Sample 1	Sample 2	Sample 3
1½”	100	100	100	100	100	100	100	100	100
1”	100	99	99	95	96	96	100	100	100
½”	46	45	40	34	35	39	58	61	59
#4	9	9	7	8	9	9	7	7	7
#8	7	7	6	8	8	8	3	3	3
Minus 200	2.96	2.69	2.55	3.43	3.50	3.74	2.01	2.22	2.37

Table B-3. Sieve Analysis and Minus 200 Test Results – Fort Myers, Modified Miami Oolite, and Modified Fort Myers

Aggregate Source	Fort Myers			Modified Miami Oolite			Modified Fort Myers		
	Percentage Passing			Percentage Passing			Percentage Passing		
Sieve Size	Sample 1	Sample 2	Sample 3	Sample 1	Sample 2	Sample 3	Sample 1	Sample 2	Sample 3
1½”	100	100	100	100	100	100	100	100	100
1”	100	100	100	100	100	100	100	100	100
½”	29	28	34	59	62	60	33	33	38
#4	3	2	3	10	9	10	9	9	9
#8	2	2	2	6	5	6	9	8	9
Minus 200	0.95	1.07	0.92	5.00	5.00	5.00	8.00	8.00	8.00

Table B-4. Sieve Analysis and Minus 200 Test Results – Inglis, Cabbage Grove, and Ocala

Aggregate Source	Inglis			Cabbage Grove			Ocala		
	Percentage Passing			Percentage Passing			Percentage Passing		
Sieve Size	Sample 1	Sample 2	Sample 3	Sample 1	Sample 2	Sample 3	Sample 1	Sample 2	Sample 3
1½”	100	100	100	100	100	100	100	100	100
1”	96	97	96	99	97	98	96	89	95
½”	49	49	95	31	38	37	35	16	41
#4	12	12	7	5	7	7	8	5	8
#8	9	9	5	5	6	6	7	5	7
Minus 200	3.69	3.89	4.38	3.00	3.08	3.99	4.30	4.86	3.09

Table B-5. Sieve Analysis and Minus 200 Test Results – Weber South, Coral Rock, and Brooksville

Aggregate Source	Weber South			Coral Rock			Brooksville		
	Percentage Passing			Percentage Passing			Percentage Passing		
Sieve Size	Sample 1	Sample 2	Sample 3	Sample 1	Sample 2	Sample 3	Sample 1	Sample 2	Sample 3
1½”	100	100		100	100	100	100	100	100
1”	100	100		100	100	100	99	99	100
½”	66	66		44	43	43	72	69	73
#4	8	9		3	3	3	23	21	21
#8	5	6		3	2	2	18	17	17
Minus 200	2.70	2.84		0.83	0.79	0.92	9.29	8.65	9.72

Table B-6. Specific Gravity and Absorption – First Batch

Aggregate Source	Sample No.	Bulk Specific Gravity (Dry)	Bulk Specific Gravity (SSD)	Apparent Specific Gravity	Absorption (Percent)
Miami Oolite-1	1	2.319	2.403	2.530	3.60
	2	2.323	2.408	2.540	3.67
	3	2.317	2.402	2.533	3.68
Fort Myers-1	1	2.305	2.396	2.537	3.98
	2	2.288	2.383	2.526	4.11
	3	2.314	2.404	2.542	3.88
Inglis-1	1	2.358	2.449	2.593	3.84
	2	2.367	2.456	2.599	3.77
	3	2.338	2.430	2.576	3.95
Cabbage Grove-1	1	2.172	2.307	2.510	6.21
	2	2.159	2.294	2.496	6.26
	3	2.142	2.282	2.491	6.55
Ocala-1	1	2.172	2.320	2.550	6.82
	2	2.163	2.316	2.554	7.09
	3	2.178	2.326	2.556	6.78

Table B-7. Specific Gravity and Absorption – Second Batch

Aggregate Source	Sample No.	Bulk Specific Gravity (Dry)	Bulk Specific Gravity (SSD)	Apparent Specific Gravity	Absorption (Percent)
Miami Oolite	1	2.342	2.450	2.624	4.58
	2	2.350	2.455	2.627	4.49
	3	2.348	2.455	2.629	4.55
Fort Myers	1	2.305	2.396	2.537	3.98
	2	2.288	2.383	2.526	4.11
	3	2.314	2.404	2.542	3.88
Inglis	1	2.358	2.449	2.593	3.84
	2	2.367	2.456	2.599	3.77
	3	2.338	2.430	2.576	3.95
Cabbage Grove	1	2.118	2.271	2.500	7.21
	2	2.115	2.265	2.488	7.09
	3	2.103	2.260	2.494	7.45
Ocala	1	2.134	2.295	2.545	7.57
	2	2.107	2.274	2.529	7.93
	3	2.129	2.297	2.558	7.88
Weber South	1	2.172	2.332	2.585	7.35
	2	2.157	2.318	2.570	7.46
	3	2.156	2.314	2.561	7.33
Coral Rock	1	2.230	2.346	2.523	5.21
	2	2.226	2.341	2.514	5.14
	3	2.232	2.347	2.520	5.12
Brooksville	1	1.806	2.069	2.450	14.57
	2	1.839	2.094	2.470	13.89
	3	1.814	2.081	2.473	14.68

Table B-8. Los Angeles Abrasion Loss

Aggregate Source	Loss Angeles Abrasion Loss (%)		
	Sample 1	Sample 2	Sample 3
Miami Oolite-1	34	35	35
Fort Myers-1	35	37	35
Inglis-1	44	43	44
Cabbage Grove-1	47	48	47
Ocala-1	47	47	47
Miami Oolite	29	31	32
Fort Myers	35	37	35
Inglis	41	42	43
Cabbage Grove	50	50	50
Ocala	46	46	47
Weber South	49	48	49
Coral Rock	40	41	40
Brooksville	65	68	66

Table B-9. Micro-Deval Abrasion Loss

Aggregate Source	Miro-Deval Abrasion Loss (%)		
	Sample 1	Sample 2	Sample 3
Miami Oolite-1	30	30	31
Fort Myers-1	29	31	29
Inglis-1	27	26	27
Cabbage Grove-1	33	33	35
Ocala-1	45	45	47
Miami Oolite	27	26	26
Fort Myers	31	30	27
Inglis	27	26	27
Cabbage Grove	38	38	38
Ocala	48	45	49
Weber South	34	34	34
Coral Rock	28	29	30
Brooksville	82	81	82

Table B-10. Sodium Soundness Loss

Aggregate Source	Sodium Soundness Loss (%)		
	Sample 1	Sample 2	Sample 3
Miami Oolite-1	7	8	9
Fort Myers-1	10	12	15
Inglis-1	13	14	-
Cabbage Grove-1	14	18	13
Ocala-1	20	18	22
Miami Oolite	10	9	9
Fort Myers	13	14	13
Inglis	13	14	-
Cabbage Grove	13	14	15
Ocala	20	18	22
Weber South	26	31	29
Coral Rock	13	13	11
Brooksville	37	39	37

Table B-11. Shell Content

Aggregate Source	Shell Content (%)		
	Sample 1	Sample 2	Sample 3
Weber South	14.7	10.8	10.8
Coral Rock	0.5	0.7	0.5

APPENDIX C
PROPERTIES OF CEMENT

Certificate of Analysis
for Sample ID: **1100047690**
Resolution Sample: **NO**

State of Florida
State Materials Office
5007 NE 39th Avenue
Gainesville FL 32609
352/955-6600
Lab ID: DSM001

Project No.: 1909701A101 Contract ID:
Pay Item: N/A Material: 465 V
Material ID on Spec.: 465 IV
Sample Number: N2364 QPL ID:
Design Mix Mix Type:
Plant: 000 Lot: Sublot:
Terminal:
Manufacturer or Producer: FLORIDA ROCK
Aggregate Sample Type: Process:
Sample Status: APPROVED
AC: Compares Favorably
Post Approval Disposition:

Date Sample Taken: 04/19/2011 By A. CAMPS
Logged: 04/19/2011 By RT822AC
Received: 04/19/2011 By RT822AC
Approved: 05/05/2011 By RT822AC

Loggers Remarks: TESTING IS FOR TIERED AGGREGATE STUDY

C114-LOI **ASTM C114 Cement (Portland) Loss on Ignition** Test Status: VALIDATED
Tested: 04/20/2011 By: F32563781-000 Comment:
Validated: 04/20/2011 By KNMOTNF
Technician Qualification Status: N/A

	Rep# 1	Assay Results	Primary Limits	
				PASS
Loss On Ignition	**3.0**	%LOI	<= 3.0	
	11.6650	CruxStartWT	none specified	
	12.6671	CruxSampWT	none specified	
	12.6366	FinalWT	none specified	

C114-INSOL **ASTM C114 Cement Acid Insoluble** Test Status: VALIDATED
Tested: 04/20/2011 By: F32563781-000 Comment:
Validated: 04/21/2011 By KNMOTNF
Technician Qualification Status: N/A

	Rep# 1	Assay Results	Primary Limits	
				PASS
Insoluble residue	**0.57**	%Insoluble	<= 0.75	
	1.0016	SampWT	none specified	
	11.3151	CruxTare	none specified	
	11.3208	CruxAshWt	none specified	

C114-II XRF **ASTM C114 Cement Type II M85** Test Status: VALIDATED
Tested: 04/20/2011 By: M62300077-000 Comment:
Validated: 04/20/2011 By RT820AE
Technician Qualification Status: N/A

	Rep# 1	Assay Results	Primary Limits	
				PASS
Aluminum Oxide	**5.0**	%	<= 6.0	
Ferric Oxide	**4.0**	%	<= 6.0	
Magnesium Oxide	**1.3**	%	<= 6.0	
Sulfur Trioxide	**2.7**	%	<= 3.0	
Tricalcium Aluminate	**6**	%	<= 8	
Tricalcium Silicate	**70**	%		
Total Alkali as Na2O	**0.35**	%	<= 0.60	

C204 **ASTM C204 Fineness of Portland Cement by Air Perm. Apparatus** Test Status: VALIDATED
Tested: 04/20/2011 By: RISHER Comment:
Validated: 04/20/2011 By RT820TR
Technician Qualification Status: N/A

Rep# 1 Assay Results Primary Limits

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				PASS
	Specific surface	408.00 sqm/Kg	>= 260.00 <= 430.(	

C191 **ASTM C191Time of Setting of Hydraulic Cement by Vicat Needle** Test Status: VALIDATED

Tested :05/05/2011 By: CAMPS Comment:
Validated:05/05/2011 By RT822AC
Technician Qualification Status: N/A

	Rep # 1	Assay Results	Primary Limits	
				PASS
	Initial time of setting	100.00 min	>= 45.00	
	Final time of setting	300.00 min	<= 375.00	

C151 **ASTM C151-Autoclave Expansion Portland Cement** Test Status: VALIDATED

Tested :05/05/2011 By: CAMPS Comment:
Validated:05/05/2011 By RT822AC
Technician Qualification Status: N/A

	Rep # 1	Assay Results	Primary Limits	
				PASS
	Change and Percentage of Change	0.04 %	<= 0.80	
		0.0562 Initial	none specified	
		0.0604 Final	none specified	
		0.0042 Change	none specified	

C1437 **ASTM C1437 FLOW** Test Status: VALIDATED

Tested :04/19/2011 By: RISHER Comment:
Validated:04/20/2011 By RT820AE
Technician Qualification Status: N/A

	Rep # 1	Assay Results	Primary Limits	
				PASS
	Amount of Flow	89.00 Min4	none specified	

C109-3DAY **ASTM C109 3-Day Breaks for Compressive Strength of Cement** Test Status: VALIDATED

Tested :04/22/2011 By: ELLIS Comment:
Validated:04/26/2011 By KNMOTSE
Technician Qualification Status: N/A

	Rep # 1	Assay Results	Primary Limits	
				PASS
	Compressive Strength Standard	3,510.0000 PSI	>=1450.00	

C109-7DAY **ASTM C109 7-Day Breaks for Compressive Strength of Cement** Test Status: VALIDATED

Tested :04/26/2011 By: ELLIS Comment:
Validated:04/26/2011 By KNMOTSE
Technician Qualification Status: N/A

	Rep # 1	Assay Results	Primary Limits	
				PASS
	Compressive Strength Standard	4,580.0000 PSI	>=2470.00	

C114TPSUM465 **Toy Poole Sum - Cement w/o Limestone** Test Status: VALIDATED

Tested :04/20/2011 By: Comment:
Validated:04/20/2011 By RT820AE
Technician Qualification Status: N/A

	Rep # 1	Assay Results	Primary Limits	
				PASS
	TPSUM	99 TPSUM	<= 100	

Notepad:

Sample_certificate_V3 bws 7/22/08

APPENDIX D
PROPERTIES OF HARDENED CONCRETE

Table D-1. Results of Compressive Strength Tests for Concrete Containing High Cement Content

Aggregate Source	Sample No.	Compressive Strength (psi)			
		7 days		28 days	
		w/c		w/c	
		0.5	0.6	0.5	0.6
Miami Oolite-1	1	5431	3996	6000	4567
	2	4518	4201	6473	4878
	3	4741	3998	6040	5019
	4	4777	3792	-	4662
Fort Myers-1	1	4479	2728	4604	4210
	2	4508	2911	5566	4176
	3	4560	2452	5479	4216
	4	4495	-	5564	3999
Inglis-1	1	3774	2892	4878	4163
	2	3213	2669	4729	3963
	3	3855	2821	5009	4039
	4	3365	2401	4673	3919
Cabbage Grove-1	1	4214	3462	5315	4344
	2	4135	3466	5281	3746
	3	4296	3492	5076	4338
	4	4602	3447	5311	3687
Ocala-1	1	4394	3570	4995	3390
	2	4679	3726	4949	4170
	3	4570	3499	5111	3970
	4	4567	3454	5001	3765

Table D-2. Results of Compressive Strength Tests for Concrete Containing Low Cement Content

Aggregate Source	Sample No.	Compressive Strength (psi)			
		7 days		28 days	
		w/c		w/c	
		0.6	**0.7**	**0.6**	**0.7**
Miami Oolite	1	3871	2434	4694	3194
	2	3876	2481	4748	3318
	3	3689	2566	4674	3284
	4	3671	-	4760	-
Fort Myers	1	3565	2173	4547	3109
	2	3662	2252	4562	3058
	3	3547	2378	4651	3204
	4	3588	-	4586	3112
Modified Miami Oolite	1	3594	2109	4664	2912
	2	3726	2101	4718	2891
	3	3687	2102	4572	2973
	4	3617	2146	4645	2946
Modified Fort Myers	1	3162	2433	4276	2890
	2	3212	2343	4234	2978
	3	3423	2387	4279	3121
	4	3182	2241	4392	3102
Inglis	1	2908	2104	4169	3043
	2	2970	2106	3986	3100
	3	3081	1919	3169	3004
	4	2908	-	3895	3167
Cabbage Grove	1	2902	2040	3638	2934
	2	2795	2020	3626	2788
	3	2958	2262	3600	2890
	4	2967	2213	3642	2942
Ocala	1	2712	2097	3702	2790
	2	2749	2106	3497	2778
	3	2807	2139	3278	2748
	4	2797	2116	3441	2843
Weber South	1	2119	1726	2794	2328
	2	2265	1661	2747	2287
	3	2196	1584	2806	2346
	4	2094	1653	2685	2323
Coral Rock	1	3252	2096	3938	3029
	2	3068	2119	4176	2890
	3	3184	2243	4400	2866
	4	3099	2213	4135	2983
Brooksville	1	2106	1647	2469	1885
	2	2060	1600	2439	1946
	3	1969	1569	2547	2048
	4	2019	1566	2483	1925

Table D-3. Results of Splitting Strength Tests for Concrete Containing High Cement Content

Aggregate Source	Sample No.	Splitting Tensile Strength (psi)			
		7 days		28 days	
		w/c		w/c	
		0.5	0.6	0.5	0.6
Miami Oolite-1	1	361	469	468	560
	2	387	426	478	478
	3	517	397	533	394
	4	394	334	-	379
Fort Myers-1	1	509	393	544	470
	2	487	409	519	445
	3	494	370	535	458
	4	-	-	-	477
Inglis-1	1	417	345	471	438
	2	451	290	477	476
	3	421	318	484	419
	4	-	-	-	-
Cabbage Grove-1	1	389	393	460	404
	2	463	349	433	392
	3	463	324	459	387
	4	-	351	415	-
Ocala-1	1	473	352	582	458
	2	435	323	579	315
	3	467	301	523	305
	4	425	323	538	390

Table D-4. Results of Splitting Tensile Strength for Concrete Containing Low Cement Content

Aggregate Source	Sample No.	Splitting Tensile Strength (psi)			
		7 days		28 days	
		w/c		w/c	
		0.6	0.7	0.6	0.7
Miami Oolite	1	458	395	454	397
	2	470	367	435	378
	3	391	387	430	405
	4	-	-	-	-
Fort Myers	1	415	301	528	394
	2	399	343	400	438
	3	417	356	393	426
	4	436	-	513	362
Modified Miami Oolite	1	417	239	385	334
	2	432	241	543	364
	3	471	264	415	336
	4	-	-	-	-
Modified Fort Myers	1	398	294	441	337
	2	365	262	411	355
	3	336	314	403	376
	4	403	319	-	365
Inglis	1	342	276	382	311
	2	393	277	397	348
	3	383	273	380	328
	4	-	-	-	-
Cabbage Grove	1	346	285	445	302
	2	354	246	416	290
	3	337	218	417	280
	4	-	-	-	-
Ocala	1	313	265	361	349
	2	340	282	376	320
	3	321	293	364	269
	4	314	-	-	-
Weber South	1	244	246	328	310
	2	281	238	377	315
	3	267	241	373	337
	4	-	-	-	301
Coral Rock	1	335	306	498	359
	2	329	280	476	331
	3	346	266	478	358
	4	346	300	-	388
Brooksville	1	217	182	275	270
	2	234	199	259	258
	3	241	195	305	252
	4	262	209	258	234

Table D-5. Results of Flexural Strength Tests for Concrete Containing High Cement Content

Aggregate Source	Sample No.	Flexural Strength (psi)			
		7 days		28 days	
		w/c		w/c	
		0.5	0.6	0.5	0.6
Miami Oolite-1	1	709	578	845	706
	2	763	509	780	729
	3	677	591	770	669
	4	734	568	708	729
	5	682	596	765	665
Fort Myers-1	1	681	641	693	712
	2	699	611	773	682
	3	674	647	752	686
	4	700	552	727	661
	5	725	617	779	711
Inglis-1	1	554	596	628	564
	2	531	466	571	529
	3	531	479	555	605
	4	566	473	598	612
	5	536	511	610	550
Cabbage Grove-1	1	658	540	656	595
	2	629	471	659	555
	3	600	507	630	593
	4	632	514	632	565
	5	-	-	-	572
Ocala-1	1	585	558	622	578
	2	567	573	674	698
	3	550	519	624	604
	4	625	566	718	608
	5	617	580	704	593

Table D-6. Results of Flexural Strength Tests for Concrete Containing Low Cement Content

Aggregate Source	Sample No.	Flexural Strength (psi)			
		7 days		28 days	
		w/c		w/c	
		0.6	0.7	0.6	0.7
Miami Oolite	1	567	485	649	572
	2	584	484	651	593
	3	593	474	683	522
	4	642	469	659	523
	5	604	445	719	530
Fort Myers	1	581	444	716	529
	2	547	463	668	541
	3	599	421	633	595
	4	559	469	623	506
	5	575	420	642	528
Modified Miami Oolite	1	552	530	670	547
	2	590	419	688	491
	3	583	442	680	557
	4	587	448	665	494
	5	530	382	666	517
Modified Fort Myers	1	560	487	614	555
	2	542	467	630	570
	3	567	457	599	534
	4	568	449	680	547
	5	540	430	590	576
Inglis	1	583	425	610	485
	2	561	404	540	492
	3	505	383	549	482
	4	575	394	577	490
	5	584	401	577	453
Cabbage Grove	1	480	380	479	466
	2	477	421	519	464
	3	444	424	585	484
	4	437	391	505	433
	5	428	414	576	493
Ocala	1	480	380	479	466
	2	477	421	519	464
	3	444	424	585	484
	4	437	391	505	433
	5	428	414	576	493
Weber South	1	454	332	516	434
	2	465	350	508	477
	3	434	374	497	500
	4	451	329	508	465
	5	-	-	514	-
Coral Rock	1	556	473	633	493
	2	511	361	656	475
	3	537	416	645	507
	4	535	431	645	537
	5	541	-	664	509
Brooksville	1	340	291	401	367
	2	357	301	446	362
	3	367	311	404	371
	4	-	-	424	337
	5	-	-	380	351

Table D-7. Results of Modulus of Elasticity and Poisson's Ratio Tests for Concrete Containing High Cement Content

Aggregate Source	Sample No.	Modulus of Elasticity and Poisson's Ratio							
		7 days				28 days			
		w/c				w/c			
		0.5		0.6		0.5		0.6	
		MOE ($x10^6$)	ν	MOE ($x10^6$)	ν	MOE (x106)	ν	MOE ($x10^6$)	ν
Miami Oolite-1	1	4.22	0.25	3.95	0.27	4.65	0.20	4.03	0.27
	2	3.78	0.23	3.65	0.24	4.70	0.27	3.75	0.27
	3	3.85	0.26	3.75	0.29	4.60	0.28	4.00	0.28
Fort Myers-1	1	4.02	0.27	3.85	0.26	4.23	0.27	3.90	0.26
	2	4.20	0.29	3.60	0.27	4.20	0.25	3.85	0.29
	3	4.22	0.27	3.37	0.25	4.33	0.31	4.04	0.30
Inglis-1	1	3.02	0.29	2.65	0.28	3.50	0.30	3.13	0.25
	2	3.13	0.28	2.82	0.27	3.33	0.27	3.25	0.29
	3	2.92	0.26	2.93	0.28	3.43	0.27	3.28	0.28
Cabbage Grove-1	1	3.43	0.21	2.82	0.23	3.75	0.29	3.52	0.32
	2	3.63	0.23	2.88	0.22	3.95	0.27	3.48	0.29
	3	3.42	0.24	2.92	0.25	3.82	0.30	3.58	0.31
Ocala-1	1	3.85	0.30	3.35	0.30	4.05	0.30	3.70	0.30
	2	3.65	0.30	3.32	0.28	4.03	0.32	3.58	0.32
	3	3.72	0.30	3.22	0.30	4.25	0.30	3.82	0.32

Table D-8. Results of Modulus of Elasticity and Poisson's Ratio Tests for Concrete Containing Low Cement Content

Aggregate Source	Sample No.	Modulus of Elasticity and Poisson's Ratio							
		7 days				28 days			
		w/c				w/c			
		0.6		0.7		0.6		0.7	
		MOE ($x10^6$)	ν	MOE ($x10^6$)	ν	MOE (x106)	ν	MOE ($x10^6$)	ν
Miami Oolite	1	3.71	0.25	3.18	0.28	4.33	0.23	3.57	0.22
	2	3.75	0.21	3.20	0.25	4.02	0.25	3.83	0.27
	3	3.68	0.25	3.43	0.23	3.98	0.23	3.80	0.27
Fort Myers	1	3.60	0.25	3.12	0.23	4.15	0.26	3.05	0.25
	2	3.60	0.24	2.93	0.20	3.70	0.28	3.25	0.27
	3	3.65	0.28	3.07	0.20	3.98	0.28	3.20	0.26
Modified Miami Oolite	1	3.62	0.25	3.05	0.28	4.08	0.27	3.12	0.26
	2	3.38	0.22	2.88	0.23	4.08	0.25	3.50	0.25
	3	3.55	0.25	3.03	0.21	3.86	0.23	3.45	0.28
Modified Fort Myers	1	3.58	0.23	3.10	0.22	3.57	0.20	3.55	0.22
	2	2.92	0.21	2.82	0.22	3.77	0.26	2.60	0.26
	3	3.20	0.22	-	-	3.85	0.31	3.18	0.23
Inglis	1	3.27	0.22	2.87	0.16	3.35	0.20	2.98	0.24
	2	3.03	0.20	2.77	0.23	3.30	0.22	2.90	0.23
	3	3.08	0.25	2.97	0.22	3.22	0.24	2.87	0.23
Cabbage Grove	1	3.14	0.22	2.62	0.22	3.17	0.27	2.68	0.26
	2	2.68	0.21	2.75	0.23	3.33	0.28	2.84	0.24
	3	2.84	0.24	2.73	0.18	3.27	0.26	2.81	0.22
Ocala	1	2.87	0.20	2.77	0.23	3.50	0.27	3.13	0.30
	2	2.65	0.24	2.72	0.21	3.10	0.29	3.01	0.22
	3	2.98	0.21	2.83	0.21	3.30	0.24	3.33	0.32
Weber South	1	2.72	0.18	2.47	0.23	2.97	0.22	3.00	0.21
	2	2.56	0.18	2.56	0.22	3.03	0.22	2.65	0.22
	3	2.65	0.16	2.35	0.20	3.11	0.22	2.58	0.22
Coral Rock	1	3.25	0.23	2.93	0.18	3.72	0.27	3.13	0.23
	2	3.30	0.22	2.87	0.17	3.72	0.25	3.22	0.22
	3	3.30	0.20	3.01	0.21	3.67	0.23	3.40	0.20
Brooksville	1	2.42	0.20	2.07	0.21	2.62	0.25	2.22	0.19
	2	2.28	0.19	1.96	0.23	2.62	0.24	2.31	0.26
	3	2.27	0.19	2.08	0.21	2.66	0.26	2.28	0.22

Table D-9. Results of Compressive Strength Tests Conducted after Loading to 40% of Failure Load and Unloading for Concrete Containing High Cement Content

Aggregate Source	Sample No.	Compressive Strength (psi)			
		7 days		28 days	
		w/c		w/c	
		0.5	0.6	0.5	0.6
Miami Oolite-1	1	5146	4307	5964	4586
	2	4820	3975	6156	4953
	3	4787	4282	5761	4295
Fort Myers-1	1	4289	3795	5840	5064
	2	4911	3763	6003	4769
	3	5024	3612	5882	5012
Inglis-1	1	3816	2726	5061	4244
	2	3426	3103	4928	3973
	3	3301	2682	4654	3818
Cabbage Grove-1	1	4121	2879	5262	4392
	2	4794	3120	5395	4014
	3	4421	2812	5156	4146
Ocala-1	1	4732	3607	5046	3847
	2	4643	3565	4935	3720
	3	4911	3817	5029	4294

Table D-10. Results of Compressive Strength Tests Conducted after Loading to 40% of Failure Load and Unloading for Concrete Containing Low Cement Content

Aggregate Source	Sample No.	Compressive Strength (psi)			
		7 days		28 days	
		w/c		w/c	
		0.6	0.7	0.6	0.7
Miami Oolite	1	3677	2508	4852	3356
	2	3704	2558	4853	3422
	3	3664	2442	4808	3509
Fort Myers	1	3529	2344	4757	3130
	2	3608	2394	4673	3116
	3	3649	2347	4740	3076
Modified Miami Oolite	1	3502	2073	4778	2870
	2	3560	2077	4748	2814
	3	-	2158	4632	2852
Modified Fort Myers	1	3142	2463	4367	3421
	2	3330	2325	4335	3107
	3	3277	-	4415	3138
Inglis	1	3011	2085	3990	3035
	2	2994	2108	3956	2760
	3	3107	2226	3998	2786
Cabbage Grove	1	3235	2138	3420	2800
	2	3149	2346	3699	2743
	3	2883	2184	3709	-
Ocala	1	2795	2153	3519	2781
	2	2585	2876	3374	2707
	3	2629	1986	3539	2960
Weber South	1	2088	1629	2891	2376
	2	2112	1661	3094	2177
	3	2201	1739	2999	2288
Coral Rock	1	3245	2158	4106	2907
	2	3339	2157	4316	2826
	3	3173	2256	4146	3044
Brooksville	1	2038	1530	2457	2022
	2	1907	1453	2463	1989
	3	2072	1578	2402	2026

Table D-11. Results of Absorption and Volume of Voids for Concrete Containing High Cement Content

Aggregate Source	Sample No.	Absorption after 5hr boil		Volume of voids	
		28 days		28 days	
		w/c		w/c	
		0.5	0.6	0.5	0.6
Miami Oolite-1	1	7.02	8.75	15.21	18.36
	2	7.67	8.55	16.48	18.09
	3	7.69	8.27	16.57	17.53
Fort Myers-1	1	9.29	19.54	9.60	20.08
	2	8.87	18.76	10.25	21.16
	3	8.32	17.91	12.14	25.24
Inglis-1	1	10.73	21.95	11.03	22.68
	2	10.52	21.67	11.33	22.97
	3	9.82	20.52	11.42	23.17
Cabbage Grove-1	1	10.16	20.67	12.42	24.29
	2	10.29	20.81	12.15	23.97
	3	10.22	20.81	12.23	24.12
Ocala-1	1	9.32	19.45	10.35	21.24
	2	9.52	19.72	10.76	21.86
	3	9.99	20.44	10.59	21.60

Table D-12. Results of Absorption and Volume of Voids for Concrete Containing Low Cement Content

Aggregate Source	Sample No.	Absorption after 5hr boil		Volume of voids	
		28 days		28 days	
		w/c		w/c	
		0.6	0.7	0.6	0.7
Miami Oolite	1	8.11	8.97	17.16	18.53
	2	8.16	9.15	17.31	18.88
	3	8.20	8.64	17.33	18.02
Fort Myers	1	8.55	10.09	17.90	20.52
	2	8.05	9.65	17.12	19.84
	3	7.91	10.43	16.93	21.06
Modified Miami Oolite	1	8.85	9.56	18.49	19.40
	2	9.29	9.34	19.17	18.99
	3	9.39	8.73	19.32	18.22
Modified Fort Myers	1	8.96	8.38	18.43	17.55
	2	8.37	8.68	17.57	18.03
	3	8.34	9.10	17.51	18.73
Inglis	1	8.83	10.06	18.43	20.51
	2	9.20	9.64	19.07	19.84
	3	9.71	9.58	19.81	19.77
Cabbage Grove	1	10.80	10.35	21.39	20.74
	2	10.90	11.31	21.50	22.24
	3	9.92	11.18	19.92	22.06
Ocala	1	10.23	10.59	20.60	21.05
	2	10.62	10.35	21.09	20.83
	3	9.87	9.91	20.06	20.13
Weber South	1	10.31	10.94	21.06	21.80
	2	10.96	10.51	21.92	22.56
	3	10.99	10.81	22.17	21.46
Coral Rock	1	8.90	9.28	18.60	18.85
	2	9.14	8.79	18.97	18.35
	3	9.06	9.77	18.71	19.68
Brooksville	1	13.13	13.69	24.74	25.73
	2	13.38	13.91	24.96	26.06
	3	13.64	13.13	25.57	25.15

Table D-13. Result of Coefficient of Thermal Expansion for Concrete Containing High Cement Content

Aggregate Source	Sample No.	Coefficient of Thermal Expansion ($x10^{-6}$)	
		28 days	
		w/c	
		0.5	0.6
Miami Oolite-1	1	6.80	8.22
	2	6.29	7.81
	3	6.50	7.18
Fort Myers-1	1	7.45	8.30
	2	8.37	8.55
	3	-	-
Inglis-1	1	8.16	9.72
	2	8.57	-
	3	8.92	-
Cabbage Grove-1	1	9.10	9.51
	2	9.09	9.88
	3	-	9.60
Ocala-1	1	6.36	7.48
	2	6.22	7.32
	3	6.70	6.61

Table D-14. Result of Coefficient of Thermal Expansion for Concrete Containing Low Cement Content

Aggregate Source	Sample No.	Coefficient of Thermal Expansion (x10^{-6}) 28 days w/c 0.6	0.7
Miami Oolite	1	7.92	9.65
	2	-	9.54
	3	-	9.99
Fort Myers	1	7.67	6.96
	2	-	8.88
	3	-	-
Modified Miami Oolite	1	8.98	9.43
	2	5.58	8.66
	3	8.91	8.86
Modified Fort Myers	1	8.81	9.27
	2	9.11	9.20
	3	-	-
Inglis	1	10.86	11.21
	2	10.60	10.94
	3	-	-
Cabbage Grove	1	9.96	10.14
	2	-	10.25
	3	-	-
Ocala	1	7.52	8.58
	2	7.66	7.75
	3	-	-
Weber South	1	10.23	10.22
	2	8.98	9.83
	3	-	-
Coral Rock	1	8.63	9.66
	2	9.56	9.82
	3	-	-
Brooksville	1	8.45	9.69
	2	9.19	11.13
	3	-	-

APPENDIX E
PAST RECORDS OF MINUS 200 LEVELS AT THE MINE AND PLANT

Period	Mine No.	Mine			Plant No.	Plant			Difference in mean values
		Mean (μ)	Standard deviation (σ)	Number of samples (n)		Mean (μ)	Standard deviation (σ)	Number of samples (n)	Plant (mean)-Mine (mean)
7/17/90-1/7/91	87-090	0.86	0.266	17	75-221	1.82	0.52	26	0.96
7/16/90-1/7/91	87-090	0.86	0.266	17	79-072	1.21	0.53	25	0.35
7/19/90-1/7/91	87-090	0.86	0.266	17	79-148	1.31	0.63	26	0.45
12/10/90-1/9/91	87-090	0.80		1	75-298	1.78	0.68	5	0.98
7/10/90-1/7/91	87-090	0.85	0.261	18	86-076	1.89	0.84	26	1.04
7/11/90-1/7/91	87-090	0.85	0.261	18	86-129	1.89	1.03	26	1.04
7/11/90-1/7/91	87-090	0.85	0.261	18	86-190	1.82	0.65	26	0.97
7/16/90-1/10/91	87-090	0.84	0.261	18	72-108	1.75	0.66	23	0.91
7/20/90-1/10/91	87-090	0.84	0.255	19	72-337	1.25	0.51	24	0.41
7/20/90-8/13/90	87-090	1.33	0.601	2	72-054	1.64	0.40	4	0.31
7/10/90-1/8/91	87-339	0.56	0.078	21	87-347	1.29	0.53	24	0.73
7/11/90-1/7/91	87-145	0.64	0.109	6	86-271	1.76	0.60	26	1.12
7/17/90-12/6/90	87-145	0.63	0.118	5	75-298	1.65	0.75	21	1.02
7/17/90-1/10/91	87-145	0.64	0.109	6	72-336	1.71	0.50	23	1.07
8/21/90-9/17/90 10/11/90-11/2/90	87-145	0.71	0.092	2	72-054	2.33	0.47	9	1.62
7/19/90-1/7/91	87-089	1.17	0.277	27	75-201	1.76	0.44	26	0.59
Mean									***0.85***
Standard Deviation									***0.35***

APPENDIX F
SELECTED TYPICAL MIX DESIGNS CURRENTLY USED FOR NON-STRUCTURAL CONCRETE APPLICATIONS

Selected Typical Mix Designs Currently Used for Non-Structural Concrete Applications

Aggregate Type	Grad SG	Pit Number	Cement Qty	Fly Ash Qty	Total	Coarse Aggregate Qty	Fine Aggregate Qty	Water Qty	% Air Content	W/C	Admixtures
CRUSHED LIMESTONE	57	2.5 12-260	408	100	508	1600	1412	258	4	0.51	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.45 08-005	410	100	510	1775	1211	280	4	0.55	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.45 03-017	410	100	510	1775	1211	280	4	0.55	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.48 03-017	408	100	508	1650	1395	269	5	0.53	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.45 12-260	408	100	508	1600	1412	258	4	0.51	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.46 08-012	408	100	508	1700	1252	275	3	0.54	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.46 08-012	408	100	508	1670	1306	266.6	6	0.52	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.46 08-012	408	100	508	1700	1252	275	3	0.54	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.46 08-012	408	100	508	1670	1306	266.6	6	0.52	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.45 01-305	410	100	510	1775	1211	280	4	0.55	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.45 08-005	408	100	508	1630	1395	269	5	0.53	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.43 08-005	408	100	508	1630	1395	269	5	0.53	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.45 01-305	408	100	508	1643	1395	269	5	0.53	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.45 86-062	508	0	508	1760	1234	279	3	0.55	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.46 87-090	410	100	510	1716	1320	258.2	6	0.51	AASHTO M-194 TYPE D

Aggregate Type	Grad SG	Pit Number	Cement Qty	Fly Ash Qty	Total	Coarse Aggregate Qty	Fine Aggregate Qty	Water Qty	% Air Content	W/C	Admixtures
CRUSHED LIMESTONE	57	2.46 87-090	410	100	510	1716	1320	258.2	6	0.51	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.46 87-090	508	0	508	1715	1340	266.6	4	0.52	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.46 87-090	508	0	508	1715	1340	266.6	4	0.52	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.45 01-305	408	100	508	1643	1395	269	5	0.53	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.48 01-305	408	100	508	1650	1395	269	5	0.53	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.46 03-340	380	97	477	1641	1375	262	7.5	0.55	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.45 12-260	370	100	470	1600	1425	258	4	0.55	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.44 12-008	408	100	508	1636	1395	269	5	0.53	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.44 12-008	408	100	508	1636	1395	269	5	0.53	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.44 12-008	408	100	508	1623	1395	269	5	0.53	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.5 08-004	380	90	470	1780	1287	258	4.7	0.55	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.45 MX-411	408	100	508	1643	1395	269	5	0.53	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.45 08-005	408	100	508	1643	1395	269	5	0.53	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.46 03-017	470	0	470	1700	1325	258	4	0.55	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.46 03-017	370	100	470	1700	1285	258	6	0.55	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.46 08-012	508	0	508	1700	1308	267	2	0.53	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.46 08-012	508	0	508	1700	1308	267	2	0.53	AASHTO M-194 TYPE D

Aggregate Type	Grad SG	Pit Number	Cement Qty	Fly Ash Qty	Total	Coarse Aggregate Qty	Fine Aggregate Qty	Water Qty	% Air Content	W/C	Admixtures
CRUSHED LIMESTONE	57	2.43 12-008	380	90	470	1500	1311	258	3.5	0.55	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.45 12-260	370	100	470	1600	1425	258	4	0.55	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.48 86-140	470	0	470	1760	1316	258.2	3	0.55	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.5 08-004	370	130	500	1750	1270	258	5	0.52	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.46 87-090	410	100	510	1716	1320	258.2	6	0.51	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.46 08-012	408	100	508	1700	1252	275	3	0.54	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.46 87-090	402	100	502	1644	1400	270.7	0.4	0.54	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.44 87-297	410	100	510	1688	1297	258.2	6	0.51	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.45 12-008	382	96	478	1690	1341	264	6	0.55	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.46 87-145	470	0	470	1745	1316	258.2	3	0.55	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.41 87-145	470	0	470	1725	1300	258.2	3	0.55	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.44 08-012	400	108	508	1688	1334	264.1	4.5	0.52	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.45 08-005	410	100	510	1775	1211	280	4	0.55	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.44 12-008	408	100	508	1636	1395	269	5	0.53	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.5 08-004	380	90	470	1780	1287	258	4.7	0.55	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.45 12-008	239	239	478	1690	1332	264.1	1.2	0.55	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.43 05-455	370	100	470	1650	1349	258.2	5	0.55	AASHTO M-194 TYPE D
CRUSHED LIMESTONE	57	2.43 87-145	192	288	480	1650	1418	237.5	6	0.49	AASHTO M-194 TYPE D

APPENDIX G
SAMPLE QUESTIONNAIRE SENT TO AGGREGATE PRODUCERS

Name of Company

……………………………………………………………

How long have you been operating the mine?

....

What is the life expectancy for this mine in years?

....

Do you supply any aggregate from this mine for FDOT projects?

Yes or No

If yes, approximately what percentage do you supply for FDOT projects?

....

Please fill out the tables below:

FDOT-Approved Coarse Aggregates

Size of Aggregate Produced	Unit price ($/ton)	Unit cost of operation if available ($/ton)	Total amount produced (ton)	Known Application where they are used
2011-projected				
2010				
2009				
2008				
2007				
2006				

FDOT-Non-Approved Coarse Aggregates

Size of Aggregate Produced	Unit price ($/ton)	Unit cost of operation if available ($/ton)	Total amount produced(ton)	Known Application where they are used	Area where FDOT Specifications Not Met?
2011-projected					
2010					
2009					
2008					
2007					
2006					