STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

STATE MATERIALS OFFICE
STRUCTURAL MATERIALS LABORATORY

5007 NE 39th Avenue
Gainesville, Florida 32609
(352) 955-6600

REPORT OF INVESTIGATION OF 78 STONE FOR THE USE AS COARSE AGGREGATE IN PRESTRESSED CONCRETE

Author:	*Christopher C. Ferraro*
Approved:	*Michael Bergin*
Report Number:	*FL/DOT/SMO/05-487*
Date:	*August 26, 2005*

CONTENTS

BACKGROUND

Presently, the Florida Department of Transportation (FDOT) Standard Specification for Road and Bridge Construction does not make any provisions for the use of 78 stone as coarse aggregate stone in concrete structural components. Per the request of several precast concrete providers, the FDOT performed a limited study which was intended to investigate the viability of using 78 stone as coarse aggregate in prestressed concrete components. This report presents the results a comparison study of Portland Cement concrete materials made with the 67 and 78 stone as coarse aggregate.

In June of 2004, The FDOT sent a team of certified technicians to the Durastress Precast Concrete batch plant in Leesburg, Florida to perform testing. The testing regimen consisted of the onsite testing of the plastic properties of the concrete; Additionally, specimens were created for physical, strength and durability testing which was performed at the FDOT State Materials Office. The objective of this analysis was to determine the feasibility of using 78 stone for coarse aggregate in prestressed concrete for qualification of the material for use on FDOT structures.

EXPERIMENTAL PROGRAM

The intent of this experiment was to compare the plastic properties for each of the concretes via the use of applicable tests.

The petrographic testing program consisted of the following:

- Standard Test Method Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete (ASTM C457-90)
- Standard Practice for the Petrographic Examination of Concrete (ASTM C856-95)

The plastic property testing program consisted of the following:

- Standard Test Method for Slump of Hydraulic Cement Concrete (ASTM C143/C143M-03)
- Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method (ASTMC173/C173M-01e1)

The physical testing program consisted of the following:

- Standard Test Method for Length Change of Hardened Hydraulic-Cement, Mortar, and Concrete (ASTM C157-03)
- Standard Test Method for Density, Absorption, and Voids in Hardened Concrete(ASTM C642-97)

The Strength Testing Regimen consisted of the following:

- Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens (ASTM C-39-03)
- Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression (ASTM C469-02)
- Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens (ASTM C496-02)
- Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading) (ASTM C78-02)

- Standard Test Method for Comparing Concretes on the Basis of the Bond Developed with Reinforcing Steel (ASTM C234-91a) (Withdrawn 2000)
- Standard Test Method for Creep of Concrete in Compression (ASTM C512-02)

The Durability Testing Regimen consisted of the following:

- Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration (ASTM C1202-97)
- Florida Method of Test for Concrete Resistivity as an Electrical Indicator of its Permeability (FM 5-578)
- An Accelerated Laboratory Method for Corrosion Testing of Reinforced Concrete Using Impressed Current (FM 5-522)

MIXTURE DESIGN

The mixtures used for each group of experimental specimens were virtually the same with the exception of the maximum size of the coarse aggregate. Both mixes were designed to meet the requirements of an FDOT Class VI mix. The control mix used 67 stone for coarse aggregate where the experimental group used 78 stone for its coarse aggregate. Table 1 provides a list of batch quantities used to cast the concrete for this study per the batch tickets certified by Dura-Stress Inc.

Table 1. Batch Quantities for Each Concrete Mix

Control Mix (67 Stone)			Experimental Mix (78 Stone)		
Ingredient	**Type**	**Quantity**	**Ingredient**	**Type**	**Quantity**
Sand	Pit No. 36-491	4440 lb	Sand	Pit No. 36-491	4655
67 Granite	GA 553	9220 lb	78 Granite	GA 553	9000
Flyash		855 lb	Flyash		850
Cement		3840 lb	Cement		3855
Water		1130 lb*	Water		1110*
Air Entrainer	Dara 1000	15 oz	Air Entrainer	Dara 1000	15 oz
Water Reducer	WDRA 60	115 oz	Water Reducer	WDRA 60	175 oz
Superplasticizer	ADVA 540	230 oz	Superplasticizer	ADVA 540	230 oz

*Does not account for water added to the mixture as a result of aggregate moisture
Coarse aggregate moisture was 3.2% Fine aggregate moisture was 3.4%

RESULTS

Petrographic Analysis

A portion of this comparison study required the construction of structural testing of full-scale type III beams. The structural testing was performed at the FDOT Structural Research Center. [1] Upon completion of the structural testing several core samples were removed and sent to a laboratory for petrographic analysis. The petrographic analysis was complete in accordance with Standard Test Method Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete ASTM C457-90 and the Standard Practice for the Petrographic Examination of Concrete ASTM C856-95. The findings from the petrographic analysis are as follows:

- The coarse aggregate within both sets of samples consisted of granite whereas, the fine aggregate consisted of quartz.
- The maximum aggregate size for the samples containing 67 ad 78 stone was ¾" and ½" respectively.
- The water-to-cementitious materials ratio (w/cm) for both sample sets was estimated to be in the range of 0.42 – 0.47.
- The total cementitious material content, including cement and fly ash of both sample sets was estimated to be 850 lb/yd^3.
- The total air content was for both specimen groups was measured to be within the range of 1.1 to 1.5%.

The petrographic testing results confirmed the aggregate size, aggregate type, and air content of the hardened concrete samples was within the expected results per mix design and batch ticket certification. However, the petrographic testing estimated the cementitious content was lower and water-cementitious ratio of the material was higher than the design. The 2004 FDOT Standard Specifications for Road and Bridge Construction 346-3.4 Table 3 specifies the maximum water-cementitious ratio for Class VI concrete is 0.37 [5]. The petrographic testing results indicate that the concrete used for this experiment failed to meet FDOT specifications.

In the event the concrete has a lower cement content and higher water-cement ratio than the original design, the properties of hardened concrete product could be significantly altered per design specifications. The effect of increasing the water-cementitious ratio will result in a concrete product which has an inferior relative strength properties and durability characteristics [2,3]. Additionally, the effect of using lower cement content will similarly result in a concrete product with inferior relative strength properties and durability characteristics [2,3].

Since the water-cementitious ratio and cement content of both mixes are of equal value, the relative difference in properties should be minimal. However, there are some material properties that can vary due to aggregate size differences even though the mixes have the same water cement ratio and cement content.

Plastic Properties

Slump Test

The slump test was designed to provide a technique to monitor the consistency of concrete in its plastic state. The standard test method for the slump of plastic concrete per ASTM C 143-03 was used to evaluate the consistency of both concretes used in this study. A portion of the structural testing required the construction of full-scale type III beams. The structural testing was performed by the FDOT Structural Research Center. [1] Each full-scale beam was cast in 2 lifts and a slump test was performed from a sample of concrete used for each lift. Table 2 contains a summary of the test data and results.

Table 2. Slump Test Results

Sample #	Lift A	Lift B
67 Stone	6.50 in	5.50 in
78 Stone	7.75 in	6.25 in

The results provided by Table 2 indicate that the concrete batched using 78 stone has a slightly greater slump than the concrete using the 67 stone. Although the slump test itself has some variability [2], the greater slump experienced by the mix with 78 stone is most likely a result of concrete mixture design. Despite the fact that the concrete mixes are almost identical with the exception of the coarse aggregate type, Table 1, indicates the concrete made with the 78 stone was batched with approximately 50% more water reducing admixture than the concrete made with 67 stone.

Air Content

The Air Content test was developed to determine the air content of plastic concrete. The standard test method for air content of freshly mixed concrete by the volumetric method is ASTMC173/C173M-01e1. Table 3, contains a summary of the test data and results obtained from air content testing. Air content was measured for each lift.

Table 3. Density Measurements and Calculations

Sample	Lift A	Lift B
Control (67 Stone)	2.2 %	3.0 %
Experimental (78 Stone)	2.6 %	2.0 %

Physical Properties

Length Change

The standard test method for length change of hardened concrete as per ASTM C157-03 was used to obtain the length change or shrinkage measurements of both concrete materials. Three 3"x 3" x 11.25" concrete samples wee cast from each concrete mixture which comprised each sample set. Figure 1 is a graphical representation of the average length change of sample set vs. time of the control group using 67 stone, and the experimental group using 78 stone. The data in the graph indicates that both specimen groups experience length change at similar rates. Additionally, the average measured length change recorded at 273 days (the last reading), is virtually the same for both sample sets. Although there are some slight differences of length change within the data it is appropriate to suggest that the difference of length change between the samples is negligible.

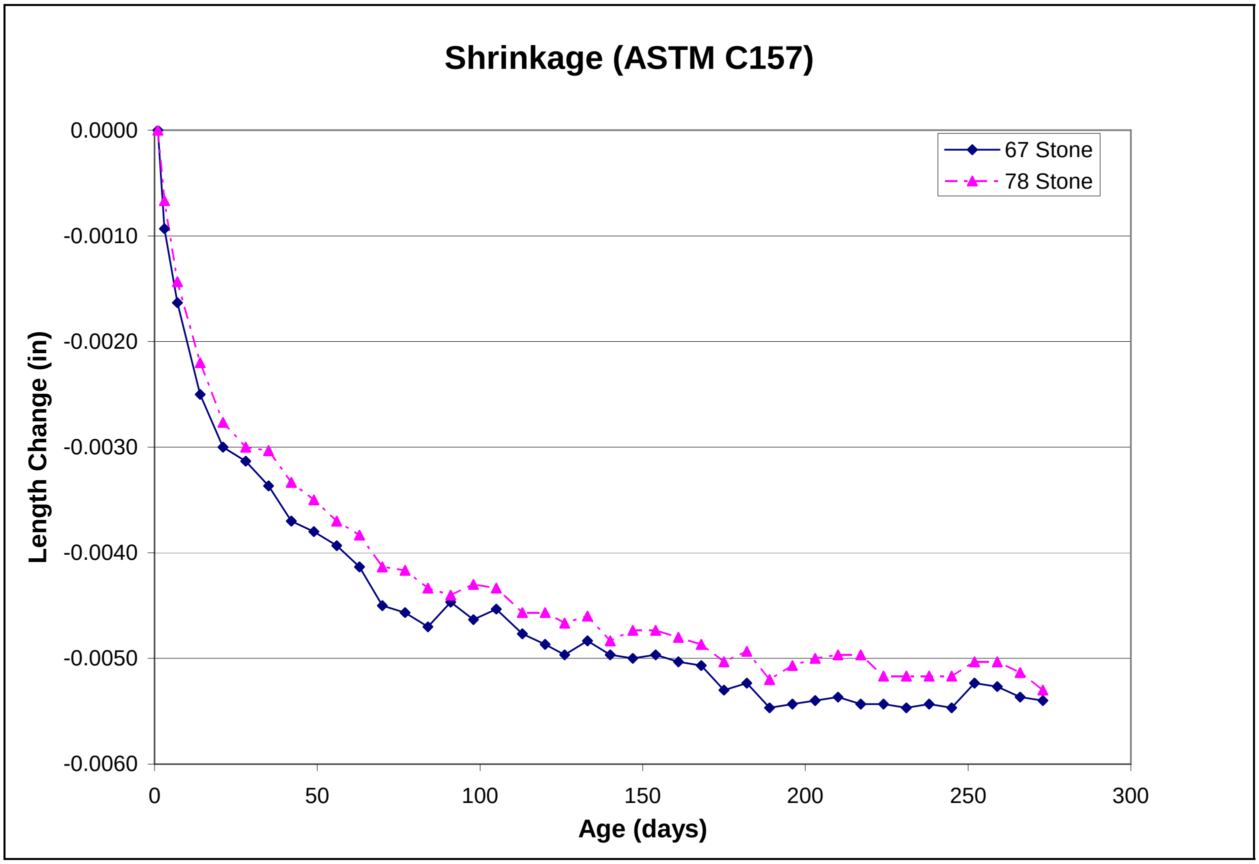

Figure 1.Length Change Measurements

Density, Absorption and Voids

Studies have shown that concrete compressive strength, tensile strength, modulus of rupture, are a function of the relative pore space of concrete. For each strength property, the strength values decrease with increasing relative porosity thus, the more pore space contained within a concrete sample, the less relative strength it will have. [3]

The test procedure used for quantification of relative pore space in each the concrete samples was the Standard Test Method for Density, Absorption, and Voids in Hardened Concrete (ASTM C642-97). Tables 4 and 5 are summaries of results of the absorption, density and voids testing.

Table 4. Absorption Results

Control (67 Stone)			**Experimental (78 Stone)**		
	Absorption			**Absorption**	
Sample #	Immersion %	Boiling %	Sample #	Immersion %	Boiling %
A	6.0	6.4	A	6.0	6.4
B	6.1	6.5	B	6.0	6.4
C	6.2	6.6	C	6.0	6.3
Average	**6.1**	**6.5**	**Average**	**6.0**	**6.4**

The results provided by Table 4, indicate the absorption for concrete samples from each mix are virtually identical. ASTM C 642-97 does not provide a precision statement for absorption calculations due to the lack of sufficient data available [4]. Therefore, it is acceptable to consider the absorption percentages of each sample set to be of equal value.

Table 5 is a summary of the data used to obtain the density and volume of voids for both concretes used in the study. A statistical comparison of the calculations of the density and volume of voids was performed for both experimental and groups. The statistical results, in conjunction with precision statement provided by ASTMC 642-97, reveal that similarly to the absorption calculations, the density and volume of voids for each concrete group are of equal value.

Table 5. Density and Volume of Voids Results

		Bulk Densities			**Apparent Density (lb/ft3)**	**Volume of Voids %**
Material Type	Sample #	Dry (lb/ft^3)	Immersion (lb/ft3)	Boiling (lb/ft3)		
Control 67 Stone	A	142.4	151.0	151.5	166.8	14.6
	B	142.3	151.1	151.5	166.9	14.7
	C	141.7	150.5	151.0	166.6	15.0
	Average	**142.1**	**150.9**	**151.4**	**166.8**	**14.8**
Experimental 78 Stone	A	142.0	150.5	151.1	166.2	14.5
	B	141.7	150.2	150.8	165.8	14.5
	C	142.2	150.7	151.2	166.2	14.4
	Average	**142.0**	**150.5**	**151.0**	**166.0**	**14.5**

Strength Properties

Compressive Strength

The compressive strength of concrete is the primary physical property and one that is frequently used for the design calculation of structures. Compressive strength is often used as an index other strength properties of concrete such as, flexural strength, tensile strength, torsional strength, and shear strength. Traditionally, the compressive strength testing has been the most widely used method of test for quality assurance in concrete materials. The standard test method for the compressive strength of cylindrical concrete specimens (ASTM C-39-03) was used to obtain the compressive strength of each of the experimental groups. Table 6 presents the data and results from the compressive strength testing. The columns denoted as A, B, and, C within Table 6, are individual specimen strength values and the final column presents an average of the values in columns A,B, and, C. Figure 2 is a graphical representation of the compressive strength data vs. age. Each data point represents the average compressive strength for a set of three samples. The error bars indicate the 95% confidence interval range of compressive strength for each three sample set.

Table 6. Compressive Strength Testing Results

Compressive Strength (psi)				
	Control (67 Stone)			
Age	A	B	C	**Average**
3 Day	4710	4912	4713	**4780**
7 Day	5614	5627	5429	**5560**
14 Day	6796	6842	6988	**6880**
28 Day	8216	7617	8057	**7960**
56 Day	9477	9553	10299	**9780**
91 Day	9860	10410	10260	**10180**
182 Day	11120	11140	10870	**11040**
273 Day	11280	10660	10920	**10950**
	Experimental (78 Stone)			
Age	A	B	C	**Average**
3 Day	5133	5287	5181	**5200**
7 Day	6206	6530	6325	**6350**
14 Day	7518	7417	8233	**7720**
28 Day	8484	8689	8669	**8610**
56 Day	9529	9748	9942	**9740**
91 Day	10800	10600	10990	**10800**
182 Day	11010	11910	11760	**11560**
273 Day	11430	11220	11260	**11300**

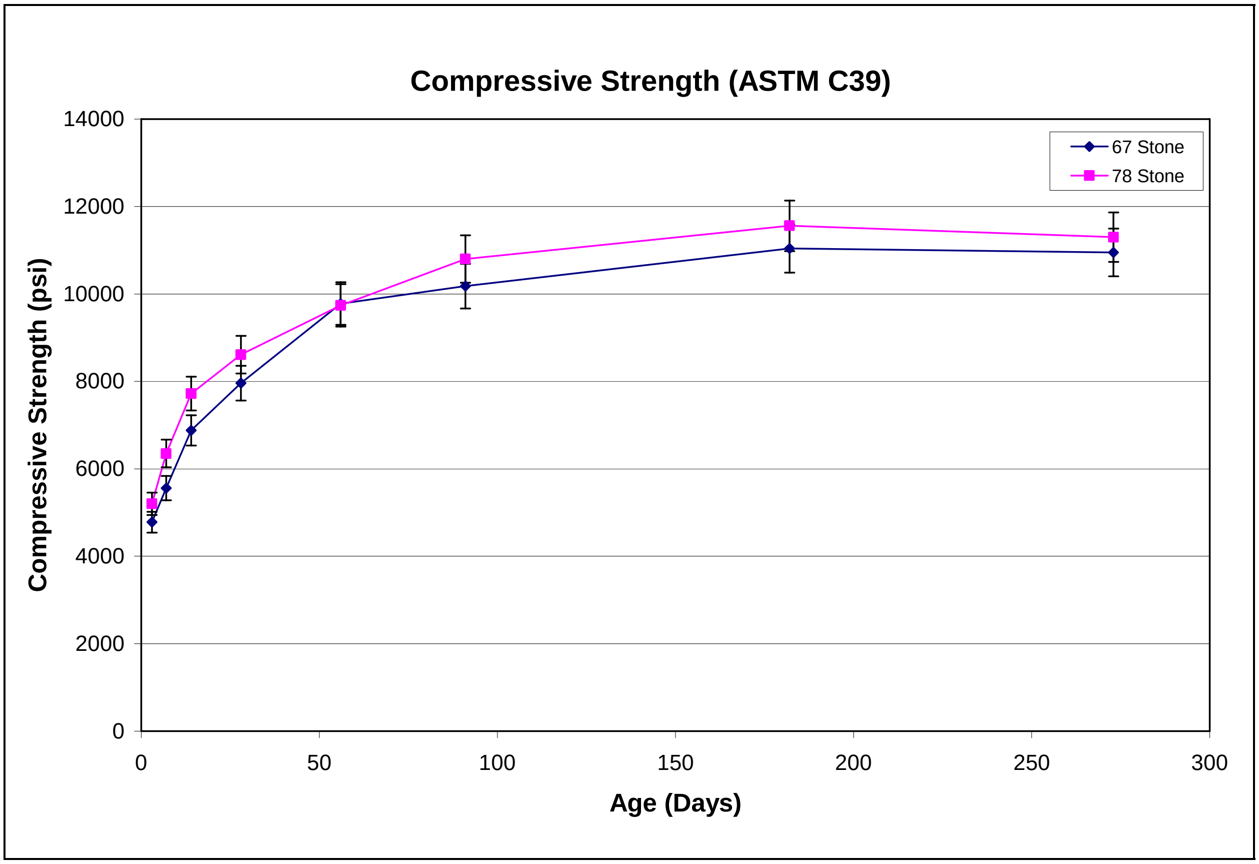

Figure 2. Compressive strength testing results

The results from the compressive strength indicate that the concrete bated with the 78 stone had slightly higher ultimate compressive strength compared to the concrete batched with 67 stone. The result is to be expected due to the water-cement ratio used in the mix design and the aggregate size difference between the two experimental groups. At low water-cement ratios, crushed stone will result in higher concrete strengths due to the better mechanical bond experienced between smaller sized aggregate particles and hydrated cement paste. This effect disappears as the water-cement ratio increases [2].

The average 28 day compressive strength test results shown in Table 6 for the concretes made with 67 and 78 stone are 7960 psi and 8610 psi, respectively. The 2004 FDOT Standard Specifications for Road and Bridge Construction 346-3.1 Table 2 specifies the minimum 28-day compressive strength for Class VI concrete is 8500 psi [5]. Accordingly, the concrete used for this experiment cast with the 67 stone failed to meet FDOT 346-3.1 specifications, whereas the concrete with the 78 stone passed.

The FDOT Materials Manual concrete production facilities guideline, Section 9.2.5(6) requires that for minimum required strengths over 5,000 psi, an over design of the minimum strength plus an additional 1400 psi is must be achieved to satisfy the required over design for concrete plants which the FDOT cannot determine a standard deviation for the plant based on historical data [6]. Both concrete experimental provided for this experiment did not meet minimum FDOT compressive strength requirements. Revisiting the results from the petrographic analysis, it is most likely that the cement content and water-cementitious ratio are the controlling factors for the inadequate concrete strength.

Modulus of Elasticity

The modulus of elasticity of concrete is a particularly important property, as it is essential to the design of concrete structures. Many design codes consider the modulus of elasticity of concrete to be a direct function of its compressive strength [7,8]. However, the materials science of concrete has revealed that concrete is a nonlinear inelastic material [2,3] thus, concrete strain is not a linear function of its compressive strength. The equations used to calculate Modulus of Elasticity directly from compressive strength do not always yield conservative results. Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression (ASTM C469-02) was used to determine Modulus of Elasticity for the concrete used in this study.

Table 7. Modulus of Elasticity Testing Results

67 Stone				**78 Stone**		
	Compressive Strength @ 14-days (psi)	**Modulus of Elasticity @ 14-days (psi)**			**Compressive Strength @ 14-days (psi)**	**Modulus of Elasticity @ 14-days (psi)**
A	7230			A	7110	
B	7100	4235020		B	7110	4413838
C	7280	4556808		C	7290	4459085
Average	**7203**	**4396000**		**Average**	**7170**	**4436000**
	Compressive Strength @ 28-days (psi)	**Modulus of Elasticity @ 28-days (psi)**			**Compressive Strength @ 28-days (psi)**	**Modulus of Elasticity @ 28-days (psi)**
A	8020			A	7950	
B	8560	4655649		B	8310	4474626
C	7890	4768105		C	8660	4692771
Average	**8157**	**4712000**		**Average**	**8307**	**4584000**
	Compressive Strength @ 56-days (psi)	**Modulus of Elasticity @ 56-days (psi)**			**Compressive Strength @ 56-days (psi)**	**Modulus of Elasticity @ 56-days (psi)**
A	9490			A	9060	
B	9150	4834859		B	9570	5016408
C	8970	4900273		C	9450	4947372
Average	**9203**	**4868000**		**Average**	**9360**	**4982000**

Table 7 presents the data and results from the modulus of elasticity testing. The rows denoted as A, B, and, C within Table 7, are individual specimen modulus values and the final row presents an average of the values in rows A, B, and, C

.

The compressive strengths results shown in Table 7, were obtained as part of the modulus of elasticity testing and are typically lower than pure compressive strength values due to the fact that the testing procedure prescribed by ASTM C469-02 requires the cyclic loading of the sample. Cyclic loading of concrete materials is known to reduce ultimate strength as a result of the initiation of damage to the microstructure, which can take place at loads as low as 40% of ultimate strength [2]. It is to be expected that the compressive strength values presented in Table 7, are lower than the values presented in Table 6.

The results from the modulus of elasticity indicate that the concrete produced with the 78 stone had slightly higher modulus of elasticity compared to the concrete batched with 67 stone. This result is consistent with values obtained in the compressive strength testing. As previously stated, low water-cementitious materials ratios, and crushed aggregate will result in higher concrete strengths due to the increased mechanical bond experienced between smaller sized aggregate particles and hydrated cement paste [2]. Although Mindess et al, do not specifically state that modulus of elasticity is directly effected by this phenomenon, it is the general trend that concretes that exhibit higher compressive strength also exhibit higher modulus of elasticity.

Tensile Strength

The tensile strength of concrete is a particularly important strength parameter due to the fact the localized tensile loading of concrete structures is the most common cause of cracking of structural concrete due to loading. The standard test method for the splitting tensile strength of cylindrical concrete specimens as per ASTM C496-02 was used to obtain the tensile strength each specimen group. The splitting tensile strength tests were performed at 28 days. The rows denoted as A, B, and, C within Table 8, are individual specimen tensile strength values and the final column presents an average of the values presented in rows A,B, and, C.

Table 8. Splitting Tensile Testing Results

	Specimen	**Splitting Tensile Strength (psi)**	**Average Tensile Strength (psi)**
67 Stone	A	715	**732**
	B	730	
	C	750	
78 Stone	A	700	**737**
	B	765	
	C	745	

The results provided by Table 8, indicate the splitting tensile strength for concrete samples from each mix are virtually identical. ASTM C 496-02 statement of bias states there is a coefficient of variation of 5% within a given batch. [9]. The coefficient of variation for the sample set used for this experiment is less than 1%, therefore the statistical analysis validates the data obtained from the experiment. Since the difference of the average values of splitting tensile strength are less than the coefficient of variance for each experimental group, it is acceptable to consider splitting tensile strength of each sample set to be of equal value.

Flexural Strength

Due to the fact that flexural strength is often the controlling strength parameter in concrete pavements, the strength of concrete for pavements and roadways is typically specified by its flexural strength [9]. The flexural strength of concrete is commonly represented by its Modulus of Rupture, as designated in ASTM C 78-02. Modulus of Rupture testing was performed when the concrete specimen reached an age of 28 days. The rows denoted as A, B, C, and D, within Table 9, are individual specimen modulus values and the final column presents an average of the values presented in rows A,B,C, and, D.

Table 9. Flexural Strength Testing Results

	Specimen	**Modulus of Rupture (psi)**	**Average Modulus (psi)**
67 Stone	A	885	**871**
	B	845	
	C	895	
	D	860	
78 Stone	A	865	**891**
	B	910	
	C	890	
	D	900	

The results provided by Table 9, indicate the modulus of rupture for concrete samples from each mix are virtually identical. ASTM C 78-02 statement of bias states there is a coefficient of variation of 5.7% within a given batch. [11]. The coefficient of variation for the sample set used for this experiment is less than 1%, therefore the statistical analysis validates the data obtained from the experiment. Since the average values of modulus of rupture are less than the coefficient of variation for each experimental group, it is acceptable to consider the modulus of rupture of each sample set to be of equal value.

Bond Strength

Structural concrete is primarily used with steel reinforcement therefore, the bond between the two materials is of significant importance [3]. The bond strength of concrete is obtained by measuring the stress required to pull-out embedded steel reinforcement from concrete specimens. The standard Test Method for Comparing Concretes on the Basis of the Bond Developed with Reinforcing Steel (ASTM C234-91a) was used to obtain the bond strength of each concrete specimen group. ASTM has officially withdrawn the C234 standard as of 2000, however, the method of test was chosen for this testing regimen due to its applicability to this comparison study. As per the ASTM 234-91a, the bond strength testing of each experimental group was performed when the concrete specimen reached an age of 28 days. The rows denoted as A, B, and C, within Table 10, are individual specimen bond strength values and the final row presents an average of the values presented in rows A,B, and, C.

Table 10. Bond Strength Testing Results

Vertically Cast Bar		
	67 Stone	**78 Stone**
	Bond Strength (psi)	Bond Strength (psi)
A	1899	1623
B	1493	1659
C	1624	1782
Average	**1672**	**1688**

Horizontally Cast Bar (Top Portion)		
	67 Stone	**78 Stone**
	Bond Strength (psi)	**Bond Strength (psi)**
A	1759	1745
B	1788	1752
C	1773	1779
Average	**1773**	**1758**

Horizontally Cast Bar (Lower Portion)		
	67 Stone	**78 Stone**
	Bond Strength (psi)	**Bond Strength(psi)**
A	1746	1743
B	1752	1777
C	1772	1748
Average	**1757**	**1756**

The results provided by Table 10, indicate the bond strength for concrete samples from each mix are virtually identical. ASTM C 234-91 does not provide a precision statement for bond strength calculations due to the lack of sufficient data available [12]. The coefficient of variation calculated for the each sample set is larger than the differences in the average values of bond strength obtained from each testing arrangement. Therefore, it is statistically valid to regard the bond strength of each sample set to be of equal value.

Creep in Compression

The creep of a material is defined as the gradual increase in the strain of a material under constant stress [13]. There are several factors that influence the effects the creep of concrete materials, some of which include [2]:

- Water-cement ratio
- Curing Conditions
- Temperature
- Moisture
- Cementitious Composition

Research has indicated that the role of aggregates in creep is that they act as a resistant to reduce the potential deformations of the hardened cement paste. Therefore, the aggregate content and the modulus of elasticity are the most important factors influencing the creep of concrete materials. Aggregate size, grading, and surface texture have little influence on creep of concrete [2].

Upon revisiting the batch quantities for each experimental group listed in Table 1, the coarse and fine aggregate are of the same type and quantities for both sample sets. Although the exact quantities of coarse and fine aggregate are differ by approximately 2% between the specimen groups, the sum total of aggregate in each mix is the same. Theoretically, the creep of each experimental group should be the same.

Standard Test Method for Creep of Concrete in Compression (ASTM C512-02) was used to determine the Creep for each of the experimental groups used in this study. ASTM C512-02 requires the specimens be loaded to 40% of ultimate strength at 28 days. As per the compressive strength obtained via ASTM C39-03, it was determined that the concrete created with the 67 stone and 78 stone loaded to pressures of 3260psi and 3330psi respectively.

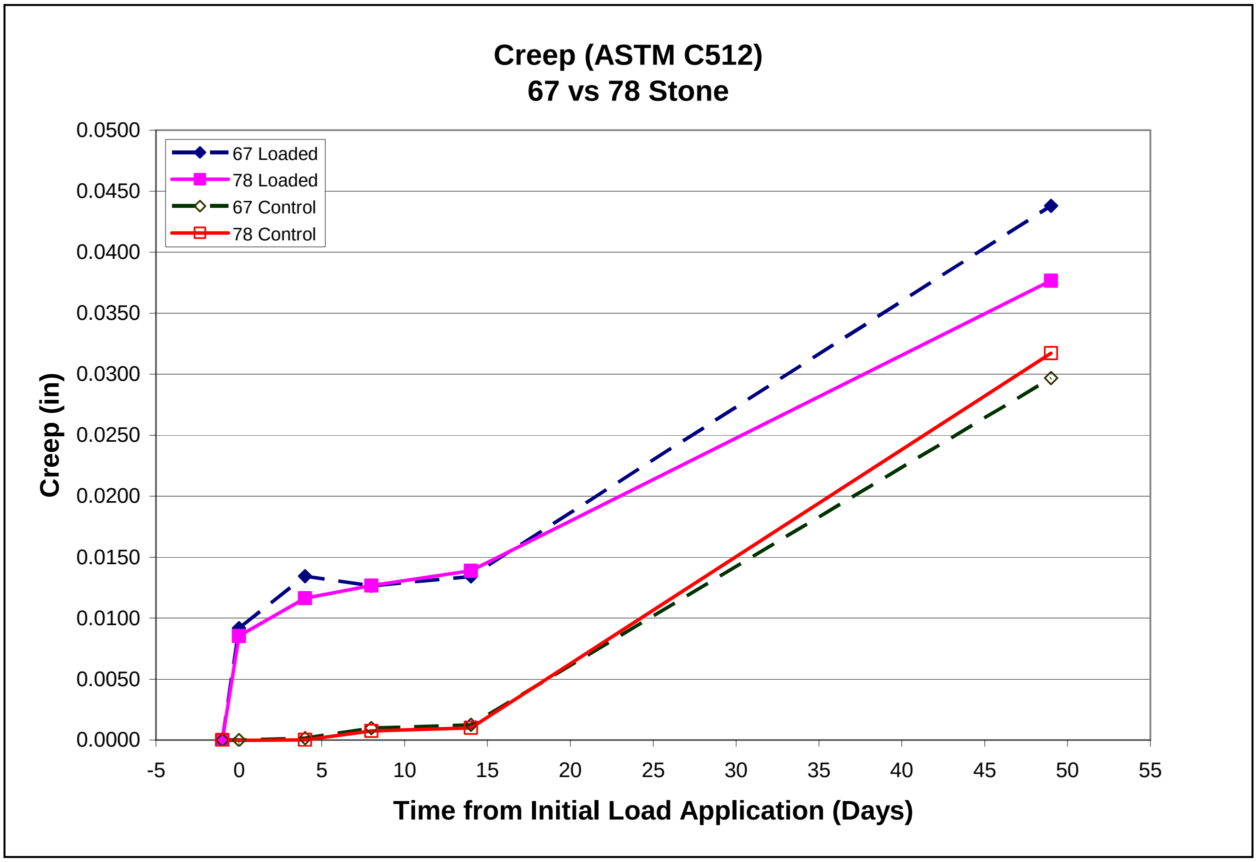

Figure 3. Creep in Compression Testing Results.

Figure 3 provides a graphical representation of the creep in compression vs. age. ASTM C 512-02 specifies that creep measurements be taken at intervals for 90 days. However, due to the fact that the epoxy between the cylinders failed and, as a result, the experimental setup became unstable, the experiment was terminated at 49 days. Although the data for this experiment is technically incomplete, it does provide useful results which reveal some slight differences between the experimental groups.

The results provided by Figure 3, indicates the concrete sample set utilizing 67 stone experienced approximately 16% more creep than the concrete made with 78 stone. The statement of bias provided by ASTM 512-02 states that the single batch coefficient of variation has been determined to be 4%. [13]. Therefore, the results from this experiment have determined that there is a statistically significant difference between the creep of each experimental group. The concrete created with the 78 stone displayed a superior performance when subjected to creep loads when compared with the concrete created using the 67 stone.

Durability Properties

Chloride Ion Penetration

The Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration (ASTM C1202-97) was used to compare the concrete for each experimental group's resistance to chloride ion penetration. The chloride ion penetration test is an electrical conductance of concrete samples intended to provide an indication of their resistance to chloride ion penetration. The test method involves the application of a 60V electrical potential is applied across a concrete specimen. After six hours of testing the potential is removed and the total charge in coulombs is obtained.

Table 11. Chloride Ion Penetration Testing Results

	67 Stone	**78 Stone**
	Charged Passed (coulombs)	**Charged Passed (coulombs)**
A	4553	4333
B	4324	3814
C	4359	3735
Average	**4438.5**	**3960.7**

The results provided by Table 11, indicates the concrete sample set utilizing 67 stone experienced approximately 12% more charge passed than the concrete made with 78 stone. The statement of bias provided by ASTM C1202-97 states that the results of two properly conducted tests from the same concrete should not differ by more than 42%. [15]. The coefficient of variation for the sample set created with 67 stone is approximately 2.8% where the coefficient of variation for the sample set created with 78 stone is approximately 8%. Thus the statistical analysis performed confirms the validity of the data obtained from the chloride ion penetration testing.

Table 1 in ASTM C1202-97 defines a qualitative value of chloride ion penetrability within a concrete sample based on charge passed in coulombs [15]. It states that a concrete that allows 4000 or more coulombs to pass to have a high chloride ion penetrability and, a concrete which that allows 2000-4000 coulombs to pass, to have a moderate chloride ion penetrability. The average values for charge passed for the concrete using 67 stone and 78 stone are 4438.5 and 3960.7 respectively (Table 11). Although, the concrete experience relatively similar chloride ion penetration values, According to ASTM 1202-97, Table 1, the concrete created with 67 stone has a high chloride ion penetrability where the concrete with 78 stone has a moderate chloride ion penetrability.

Accelerated Corrosion Using Impressed Current

The corrosion of steel reinforcement is one of the major components influencing the long-term performance of concrete structures [16]. The impressed current technique is used for the study of concrete materials with the presence of corrosion in reinforcing steel. The impressed current technique has gained much of its favor for use in research mainly because it can allow for the completion of durability testing of reinforced concrete in a greatly reduced time interval. The Florida Department of Transportation has standardized the method of test for impressed current testing for the use in the investigation of corrosion susceptibility of concrete materials, protective coatings, rebar coatings and rebar claddings [17].

A typical sample used for evaluation in impressed current testing can be seen in figure 4 is basically a modified cylinder sample with a #4 rebar embedded within. The casting procedure of the concrete specimens is similar to the casting of cylindrical samples with the exception of the suspension of rebar within the center of the sample.

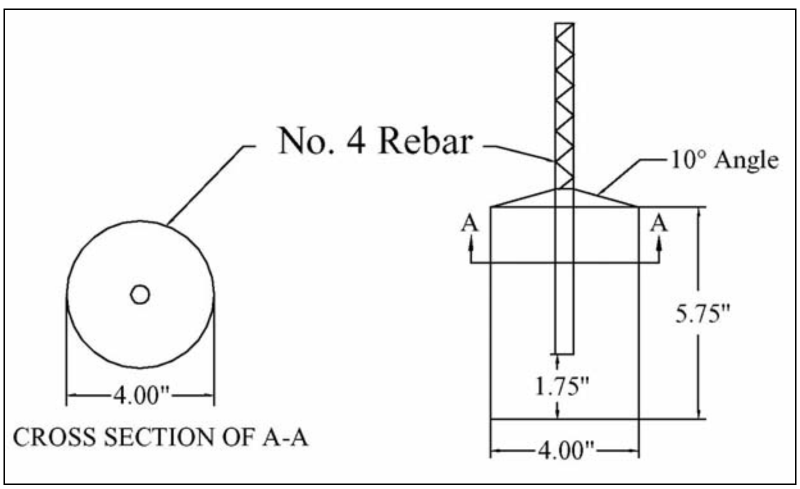

Figure 4 –Schematic of a typical sample used for impressed current testing

After removal from its mold, the concrete is moist cured for 28 days. After moist curing, the sample is partially submersed in a 5% Sodium Chloride (NaCl) solution for an additional 28 days to facilitate the initiation of corrosion activity. It is not uncommon for impressed current samples used for research to have NaCl added directly to the concrete mixture for the facilitation of corrosion activity [18].

The impressed current test involves the use of an electrical source for the provision of current to the sample. The exposed portion of the rebar is connected to a 6V DC power supply. Each sample is connected to an individual shunt so current to each sample can be measured. The specimen is partially submersed in a 5% Sodium Chloride (NaCl) solution to provide a conductive solution. Figures 5 shows a schematic impressed current system used for the sample testing in this experiment.

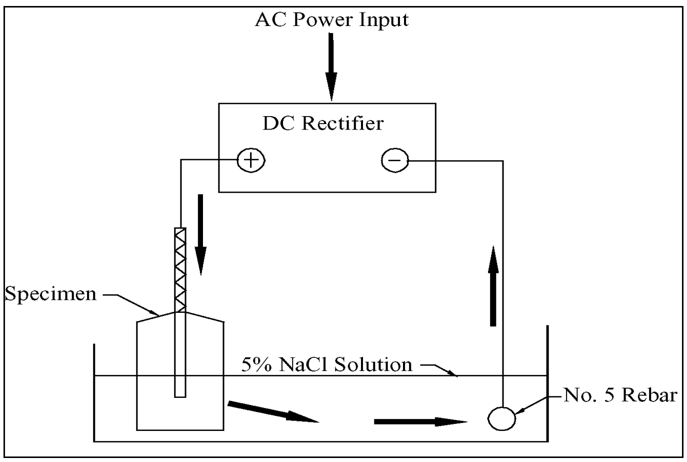

Figure 5 –Schematic of a test configuration used for impressed current testing

Upon the initiation of a current to the system, the sample current, and Voltage drop are continuously monitored until failure. Failure is defined by one of two means:

- The appearance of a visible crack
- A sharp current increase is detected indicating that a non-visible crack has formed.

The appearance of the visible crack is usually accompanied by the sharp current increase. Figure 6 is a photograph of typical samples used for impressed current testing. The specimen in the foreground of the photo has visible rust staining and a visible crack, thus is a failed specimen as per the definition of failure. It is common practice to continue to test for several days after failure has been established to ensure that the sample has failed in the typical manner.

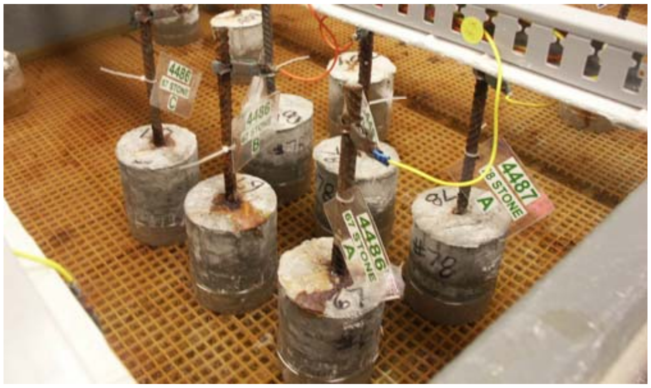

Figure 6 –Photograph of impressed current apparatus and typical samples

Table 12 is a summary of the data obtained from the impressed current testing for both concretes used in the study. The rows denoted as A, B, and C, within Table 12, are individual specimen impressed current values and the final row presents an average of the values presented in rows A, B, and, C.

Table 12. Impressed Current Testing Results

	67 Stone		**78 Stone**	
	Time to Failure (Days)	**Resistance (Ohms)**	**Time to Failure (Days)**	**Resistance (Ohms)**
A	81	1634	144	3171
B	106	1721	255	5091
C	95	1556	209	4683
Average	**94.0**	**1637**	**202.7**	**4315**

The results provided by Table 12, indicates the concrete sample set utilizing 78 had an average time of failure approximately 115% longer than the concrete with 67 stone. Additionally, the average resistance at the time of failure of the sample set containing 78 stone is approximately 163% higher than the concrete with 67 stone.

The results indicate that the concrete batched with the 78 stone performed in a substantially superior manner than the concrete batched with 67 stone with regard to the impressed current test. However, there is currently no research available which has provided substantial changes in the durability characteristics of concrete materials based on coarse aggregate substitution. While it is known that aggregate size and gradation do have influences on the durability characteristics of concrete, the data obtained via the impressed current testing has considerably different values. Such a phenomenon is not typically exhibited by concrete materials which have almost equal physical and strength properties as per the testing results provided herein. Thus, the data provided by the impressed current testing regimen may be fallible.

Surface Resistivity Testing

The surface resistivity of concrete materials is commonly used as an indicator of the ion penetrability. The FDOT developed a method of test for the Concrete Resistivity as an Electrical Indicator of its Permeability [19]. The test is applicable for the use in this experiment as a method of comparison of permeably characteristics of the concrete in each specimen group.

Surface resistivity testing has gained much of its favor for use in research for the reason that it is a nondestructive test that can be performed in a relatively quick manner. Since the test is nondestructive, it also allows researchers to continuously monitor the same samples over time. The Florida Department of Transportation has created a standard method of test for surface resistivity testing which is mainly used to indicate the concretes ability to resist penetration or electron transfer [19].

The test procedure requires use of a resistivity meter and a wenner array probe to be used to obtain the electrical resistance of a saturated concrete surface. Typically three 4” x 8” concrete cylinders are used to create a sample set. Measurements are taken at 4 locations on each sample specimen, as the specimen is turned 90° between tests in order to obtain an average resistivity for the sample.

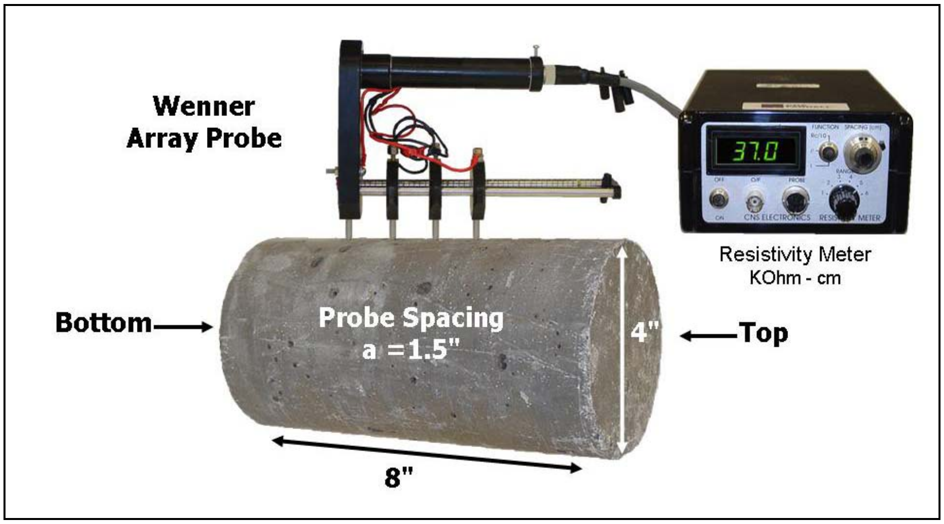

Figure 7. Schematic of configuration used for surface resistivity testing

The test is limited to the testing of concrete samples which have not been cut or cored as the concentration of conductive ions in such samples may alter the test results.

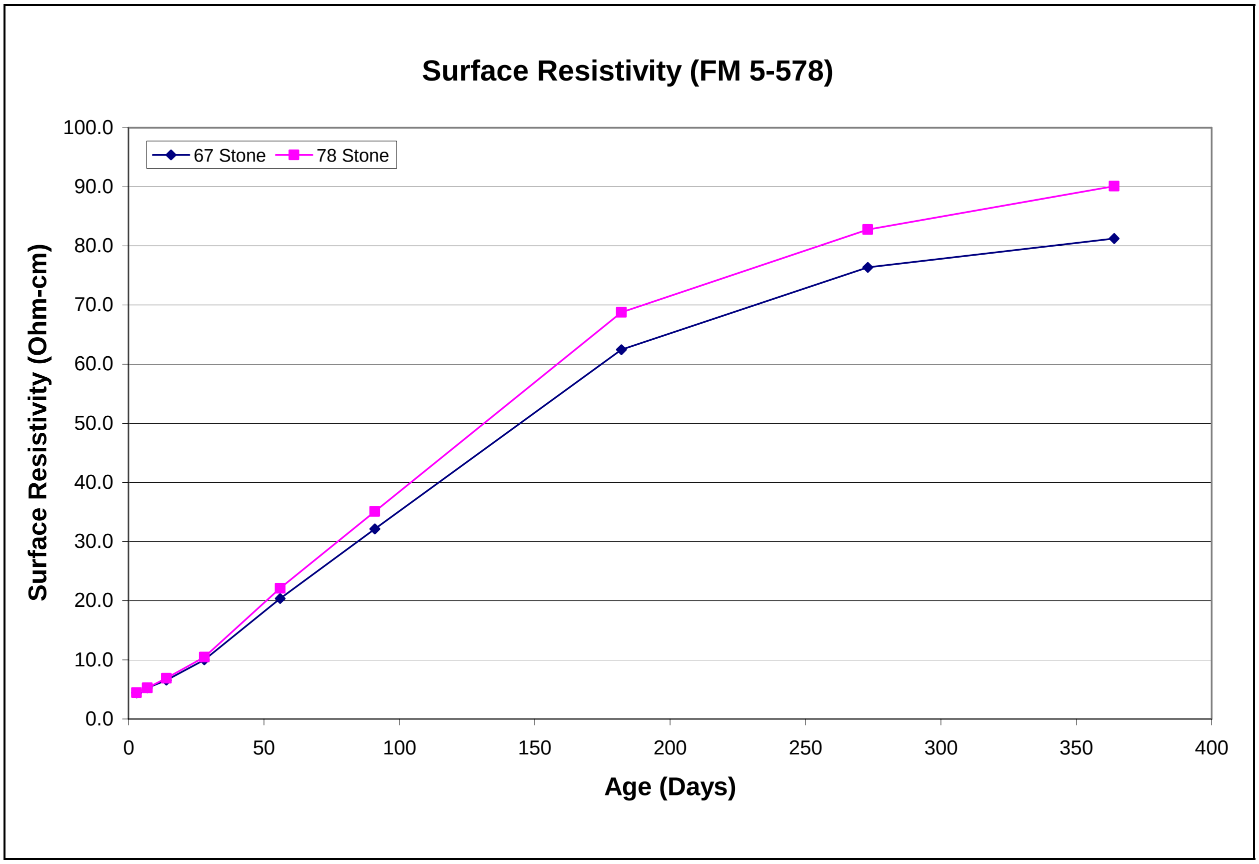

Figure 8. Surface resistivity testing results

Figure 8 provides a graphical representation of the surface resistivity data vs. age. Each data point represents the average surface resistivity for a set of three samples. The results indicate that the surface resistivity of both concrete used for this experiment behaved in a similar fashion. The concrete experimental group created with the 78 stone showed slightly higher surface resistivity than the concrete created with 67 stone. According to Table 2 in FM 5-578, both concrete exhibit a “very low” propensity for chloride ion penetrability based on surface resistivity.

CONCLUSIONS

The comparison study of concretes created with 67 and 78 stone successfully quantified the material characteristics of the concretes. Research has shown that differences in aggregate size shape and grading can influence the physical and strength, and durability characteristics of concrete [20]. The testing regimen has confirmed that substituting coarse aggregates of the same type with different maximum sizes did slightly alter the strength and durability properties of the resultant concrete while the physical properties of the concrete were not significantly altered.

The experiment revealed substituting 78 stone for 67 stone, while holding all other batch quantities constant enhanced the following characteristics of the concrete:

- Compressive strength
- Modulus of elasticity
- Resistance to creep in compression
- Resistance to chloride penetration
- Resistance to corrosion of steel

The study revealed that substituting 78 stone for 67 stone did not effect the following properties of the concrete:

- Length change or shrinkage
- Density, absorption and void space
- Tensile strength
- Flexural strength
- Bond strength
- Surface resistivity

The use of 78 stone as coarse aggregate in concrete with structural applications will not adversely affect the performance of the concrete material when used in accordance with the FDOT Standard Specification for Road and Bridge Construction.

RECOMMENDATIONS

The FDOT Standard Specification for Road and Bridge Construction should temporary make provisions for the use of 78 stone as coarse aggregate in structural applications, until a more complete study can be performed.

ACKNOWLEDGEMENTS

The authors wish to thank the following people for their efforts and input into this report: Phillip Armand, Sandra Bober, Richard Delorenzo, Dan Haldi, Charlotte Kasper, Guy Padgett, Sue Rose, Terry Thomas, Beth Tuller.

REFERENCES

1. Ansley, M (September, 2004) "Test of Type III Beams for Comparison of Aggregate Types, Testing done at the Request of the State Materials Office" Florida Department of Transportation
2. Mindess, S., Young, J. F., & Darwin, D. (2003). *Concrete* (2nd ed.). Upper Saddle River, New Jersey: Prentice Hall.
3. Neville A,M. (1996) *Properties of Concrete*, 4th Ed., Wiley, New York
4. ASTM (2005), *Standard Test Method for Density, Absorption, and Voids in Hardened Concrete*, C642-97, West Conshohocken, Pennsylvania: American Society for Testing and Materials
5. Florida Department of Transportation, *Standard Specifications for Road and Bridge Construction*, Florida Department of Transportation, Tallahassee 2004
6. Florida Department of Transportation, *Materials Manual* Florida Department of Transportation, Tallahassee 2004
7. American Concrete Institute, Committee 318 (2002), *Building Code Requirements for Structural Concrete*, American Concrete Institute, Detroit

8. Kosmatka, S.H., and Panarece, W.C., (1994) *The Design and Control of Concrete Mixtures* (4th Printing)., Skokie, Illinois: Portland Cement Association
9. ASTM (2005), *Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens*, C496-02, West Conshohocken, Pennsylvania: American Society for Testing and Materials
10. Akroyd, T.N.W., (1962). Concrete Properties and Manufacture. New York: Pergamon Press.
11. ASTM (2005), *Standard Test Method for Flexural Strength of Concrete, (Using Simple Beam with Third-Point Loading)* (ASTM C78-02) West Conshohocken, Pennsylvania: American Society for Testing and Materials
12. ASTM (1991) *Standard Test Method for Comparing Concretes on the Basis of Bond Developed with Reinforcing Steel* (ASTM C234-91a) West Conshohocken, Pennsylvania: American Society for Testing and Materials
13. ASTM (2005), *Standard Test Method for Creep of Concrete in Compression* (ASTM C512-02) West Conshohocken, Pennsylvania: American Society for Testing and Materials
14. Merritt, F.S., Loftin, M.K., and Ricketts J.T., (1995) *Standard Handbook for Civil Engineers*, 4th Ed., McGraw-Hill, New York
15. ASTM (2005), *Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration,* (ASTM C1202-97) West Conshohocken, Pennsylvania: American Society for Testing and Materials
16. Izquierdo, D., Andrade, C., Arteaga, .A, Tanner, P., (2003) "Economical Optimization of Appraising Concrete Degradation Processes" Proceedings of Life-Cycle Performance of Deteriorating Structures, IABMAS/ASCE Lausanne, Switzerland, pp. 37-48.
17. FDOT (2000) *Florida Method of Test for an Accelerated Laboratory Method for Corrosion Testing of Reinforced Concrete Using Impressed Current* (FM 5-522) Florida Department of Transportation
18. Maaddawy, T. A., & Soudki, K.A. (2003) Effectiveness of Impressed Current Technique to Simulate Corrosion of Steel Reinforcement in Concrete. ASCE Journal of Materials in Civil Engineering, 15(1), 41-47.
19. FDOT (2004) *Florida Method of Test for an Concrete Resistivity as an Electrical Indicator of its Permeability* (FM 5-578) Florida Department of Transportation
20. Mehta. K.P., Monteiro. P.J.M, (1993). *Concrete - Microstructure, Properties, and Materials,* New York: McGraw-Hill

APPENDIX A

PETROGRAPHIC ANALYSIS

PETROGRAPHIC ANALYSIS REPORT

PROJECT:
Mockup Beam Concrete Evaluation

REPORTED TO:
Florida Department of Transportation
State Materials Office/Research Park
5007 NE 39th Avenue
Gainesville, FL 32609
Attention: Ms. Charlotte Kasper
Phone: (352) 955-6691
Fax: (352) 955-6689

Petrographic Lab No.: 807-I-56107

Date: August 8, 2005

BACKGROUND

This report presents the results of petrographic analysis of two (2) concrete core samples, submitted by Ms. Charlotte Kasper of Florida Department of Transportation. Reportedly, the samples were retrieved from mockup structural beams, which were constructed on June 10, 2004. The core samples were labeled as follows:

Sample No.	Sample Designation
#1	#67 Stone/N-Top 3
#2	#78 Stone/N-Top 3

The objective of our analysis was to determine the constituent proportions of the samples and compare them with the mix design data.

SUMMARY OF FINDINGS

The findings of our petrographic analysis are summarized below:

1. The coarse aggregates in the analyzed samples primarily consisted of granite. Whereas, the fine aggregates in the analyzed samples primarily consisted of quartz and granite.

2. The nominal maximum size of the coarse aggregates in Sample #1 was ¾ in. Whereas the nominal maximum size of the coarse aggregates in Sample #2 was ½ in.

3. The water-to-cementitious materials ratio (w/cm) in the bulk portion of the analyzed samples was estimated to be in the range of 0.42 to 0.47.

4. The total cementitious material content, including cement and fly ash, of the analyzed samples was estimated to be 850 lb/yd^3.

5. The total air content of the analyzed samples was measured to be in the range of 1.1 to 1.5%.

6. Cementitious material was reasonably hydrated in all the analyzed samples. Unhydrated cementitious particles were estimated to be about 15 to 20%.

7. Evidence of deleterious reactions such as alkali-silica reaction (ASR) and ettringite formation was not observed in the samples.

CONCLUSIONS

The estimated w/cm of the analyzed samples was higher the designed w/cm of 0.35. The w/cm in the bulk portion of the analyzed samples was estimated to be in the range of 0.42 to 0.47.

The estimated total cementitious material content of the analyzed samples was less than the designed total cementitious material content of 945 lb/yd^3. The total cementitious material content of the analyzed samples was estimated to be 850 lb/yd^3.

The total air content of the analyzed samples was measured to be in the range of 1.1 to 1.5%, which was within the designed range of 1 to 5%.

TEST PROCEDURES

Petrographic analysis

The petrographic analysis was performed in general accordance with ASTM C856-95. The analysis included a blue-dyed thin section using a polarized light microscope. Depth of carbonation was determined using a 0.15% phenolphthalein solution. Water-to-cement ratio was estimated based upon the appearance of a finely lapped sample surface, cement reaction to needle scratching, absorption of a water drop, and examination of a thin section under a polarized light microscope.

Air content testing

Air content testing was performed in general accordance with ASTM C457-90, Procedure A—Linear Traverse Method. A thick polished section cut from the concrete core was examined using a stereo microscope at a magnification of 125x.

REMARKS

The test samples will be retained for a period of 30 days from the date of this report. Unless further instructions are received by that time, the samples will be discarded.

PETROGRAPHIC ANALYSIS DATA SHEET

Petrographic Lab Report No.: 807-I-56107

Sample I.D: #1
#67 Stone/N-Top 3

A. General Observations

1. Sample Dimensions: The sample was a 4-in. diameter concrete core, about 6 ½-in. long. One polished section was examined under a stereo microscope, and one thin section with blue epoxy impregnation was studied under a polarized light microscope.

2. Surface Conditions: Smooth.

3. Reinforcement: None observed.

4. General Conditions: The concrete sample appeared to be in stable condition. No visible cracking was observed. The sample was rich in paste and lean in aggregates.

B. Aggregate

1. Coarse : The coarse aggregates were sub-angular to angular and primarily consisted of granite. The nominal maximum size of the aggregates was about ¾ in. The coarse aggregates were generally sound.

2. Fine: The fine aggregates were sub-angular to angular and primarily consisted of quartz and granite.

C. Cementitious Paste

1.	Paste Content:	39.2%.
2.	Air Content:	1.5% total; 1.5% entrained, 0.0% entrapped.
3.	Carbonation:	Negligible.
4.	Pozzolan Presence:	Fly ash particles were observed.
5.	Paste/Aggregate Bonding:	Good.
6.	Paste Color:	Gray.
7.	Paste Hardness:	Moderate.
8.	Secondary Deposits:	None observed.
9.	Water-to-Cementitious Materials Ratio (W/CM):	Average w/cm in the bulk portion of the sample was estimated to be in the range of 0.42 to 0.47.
10.	Paste Quality:	The cement paste was generally sound; cementitious material was reasonably hydrated, with about 15 to 20% unhydrated cementitious particles.
11.	Microcracks:	Microcracks were observed.

PETROGRAPHIC ANALYSIS DATA SHEET

Petrographic Lab Report No.: 807-I-56107

Sample I.D: #2
#78 Stone/N-Top 3

A. General Observations

1. Sample Dimensions: The sample was a 4-in. diameter concrete core, about 6 ½-in. long. One polished section was examined under a stereo microscope, and one thin section with blue epoxy impregnation was studied under a polarized light microscope.

2. Surface Conditions: Smooth.

3. Reinforcement: None observed.

4. General Conditions: The concrete sample appeared to be in stable condition. No visible cracking was observed. The sample was rich in paste and lean in aggregates.

B. Aggregate

1. Coarse : The coarse aggregates were sub-angular to angular and primarily consisted of granite. The nominal maximum size of the aggregates was about ½ in. The coarse aggregates were generally sound.

2. Fine: The fine aggregates were sub-angular to angular and primarily consisted of quartz and granite.

C. Cementitious Paste

1.	Paste Content:	39.1%.
2.	Air Content:	1.1% total; 1.1% entrained, 0.0% entrapped.
3.	Carbonation:	Negligible.
4.	Pozzolan Presence:	Fly ash particles were observed.
5.	Paste/Aggregate Bonding:	Good.
6.	Paste Color:	Gray.
7.	Paste Hardness:	Moderate.
8.	Secondary Deposits:	None observed.
9.	Water-to-Cementitious Materials Ratio (W/CM):	Average w/cm in the bulk portion of the sample was estimated to be in the range of 0.42 to 0.47.
10.	Paste Quality:	The cement paste was generally sound; cementitious material was reasonably hydrated, with about 15 to 20% unhydrated cementitious particles.
11.	Microcracks:	Microcracks were observed.

AIR VOID SYSTEM ANALYSIS REPORT

PROJECT:
Mockup Beam Concrete Evaluation

REPORTED TO:
Florida Department of Transportation
State Materials Office/Research Park
5007 NE 39th Avenue
Gainesville, FL 32609
Attention: Ms. Charlotte Kasper
Phone: (352) 955-6691
Fax: (352) 955-6689

Petrographic Lab No.: 807-I-56107

Date: August 8, 2005

Sample I.D.:	#1 #67 Stone/N-Top 3
Sample Data:	
Sample Description:	Concrete Core 4-in. Diameter and 6 ½-in. Long
Test Data:	
Air Void Content (%)	1.5
Entrained (%)	1.5
Entrapped (%)	0.0
Air Voids/inch	2.47
Average Void Length (in.)	0.006
Specific Surface (in^2/in^3)	673.6
Spacing Factor	0.014
Paste Content (%)	39.2
Magnification	125x
Test Date	08/08/05

Conformance: The air void system in the analyzed sample was inadequate to resist freezing and thawing damage. However, freezing and thawing is generally not a concern in the climatic conditions of Florida.

Remarks:

1. The analysis was performed in general accordance with ASTM C-457, Procedure A—Linear Traverse Method.
2. The test sample will be retained for 30 days from the date of this report. After 30 days, the sample will be discarded unless other instructions are received.

AIR VOID SYSTEM ANALYSIS REPORT

PROJECT:
Mockup Beam Concrete Evaluation

REPORTED TO:
Florida Department of Transportation
State Materials Office/Research Park
5007 NE 39th Avenue
Gainesville, FL 32609
Attention: Ms. Charlotte Kasper
Phone: (352) 955-6691
Fax: (352) 955-6689

Petrographic Lab No.: 807-I-56107 **Date:** August 8, 2005

Sample I.D.: #2 #78 Stone/N-Top 3

Sample Data:

Sample Description:	Concrete Core 4-in. Diameter and 6 ½-in. Long

Test Data:

Air Void Content (%)	1.1
Entrained (%)	1.1
Entrapped (%)	0.0
Air Voids/inch	1.98
Average Void Length (in.)	0.005
Specific Surface (in^2/in^3)	745.4
Spacing Factor	0.015
Paste Content (%)	39.1
Magnification	125x
Test Date	08/08/05

Conformance: The air void system in the analyzed sample was inadequate to resist freezing and thawing damage. However, freezing and thawing is generally not a concern in the climatic conditions of Florida.

Remarks:

1. The analysis was performed in general accordance with ASTM C-457, Procedure A—Linear Traverse Method.
2. The test sample will be retained for 30 days from the date of this report. After 30 days, the sample will be discarded unless other instructions are received.

APPENDIX B

STATISTICAL ANALYSIS

Statistical Analysis- Absorption

CORE #	ABSORPTIONS	
	IMMERSION, %	BOILING, %
67-A	6.0	6.4
67-B	6.1	6.5
67-C	6.2	6.6
Avg. - 67 A-B-C	**6.1**	**6.5**
Std Dev. - 67 A-B-C	**0.09**	**0.09**
Coef of Var. - 67 A-B-C	**1.418%**	**1.452%**
78-A	6.0	6.4
78-B	6.0	6.4
78-C	6.0	6.3
Avg. - 67 A-B-C	**6.1**	**6.5**
Std Dev. - 67 A-B-C	**0.04**	**0.03**
Coef of Var. - 67 A-B-C	**0.586%**	**0.494%**

Statistical Analysis- Density and Voids

Core #	Bulk Densities			Apparent Density (lb/ft^3)	Volume of Voids, %
	Dry (lb/ft^3)	Immersion (lb/ft^3)	Boiling (lb/ft^3)		
67-A	142.40	151.02	151.54	166.84	14.65
67-B	142.34	151.07	151.52	166.90	14.72
67-C	141.68	150.50	151.03	166.64	14.98
Avg. - 67 A-B-C	**142.1**	**150.9**	**151.4**	**166.8**	**14.8**
Std Dev. - 67 A-B-C	**0.40**	**0.31**	**0.29**	**0.14**	**0.17**
Coef of Var. - 67 A-B-C	**0.281%**	**0.209%**	**0.193%**	**0.084%**	**1.171%**
78-A	142.02	150.49	151.09	166.16	14.53
78-B	141.70	150.23	150.76	165.75	14.51
78-C	142.19	150.65	151.19	166.16	14.43
Avg. - 67 A-B-C	**142.1**	**150.9**	**151.4**	**166.8**	**14.8**
Std Dev. - 67 A-B-C	**0.25**	**0.21**	**0.23**	**0.24**	**0.05**
Coe of Var. - 67 A-B-C	**0.174%**	**0.142%**	**0.150%**	**0.142%**	**0.366%**

Statistical Analysis Tensile Strength					
	Sample #	**Strength (psi)**	**Average (psi)**	**Std Dev. (psi)**	**Coe of Var.**
67 Stone	**A**	715			
	B	730	**732**	**17.6**	**2.40%**
	C	750			
78 Stone	**A**	700			
	B	765	**737**	**33.3**	**4.52%**
	C	745			

Statistical Analysis Modulus of Rupture					
	Sample #	**Mod of Rupture (psi)**	**Average (psi)**	**Std Dev. (psi)**	**Coe of Var.**
67 Stone	A	885			
	B	845	**871**	**22.9**	**2.62%**
	C	895			
	D	860			
78 Stone	A	865			
	B	910	**891**	**19.3**	**2.17%**
	C	890			
	D	900			

Statistical Analysis Bond Strength

Vertically Cast Bar

# 67 Stone				# 78 Stone		
Sample #	**Total Load (lb)**	**Bond Strength (psi)**			**Total Load (lb)**	**Bond Strength (psi)**
A	22369	1899		A	19117	1623
B	17589	1493		B	19540	1659
C	19136	1624		C	20991	1782
Avg. - 67 A-B-C		**1672.2**				**1687.8**
Std Dev. - 67 A-B-C		**207.05**				**83.44**
Coef of Var. - 67 A-B-C		**12.382%**				**4.943%**

Horizontally Cast Bar, Top Portion

# 67 Stone				# 78 Stone		
Sample #	**Total Load (lb)**	**Bond Strength (psi)**			**Total Load (lb)**	**Bond Strength (psi)**
A	20717	1759		a	20556	1745
B	21067	1788		b	20634	1752
C	20889	1773		c	20952	1779
Avg. - 67 A-B-C		**1773.4**				**1758.4**
Std Dev. - 67 A-B-C		**14.86**				**17.81**
Coef of Var. - 67 A-B-C		**0.838%**				**1.013%**

Horizontally Cast Bar, Bottom Portion

# 67 Stone				# 78 Stone		
Sample #	**Total Load (lb)**	**Bond Strength (psi)**			**Total Load (lb)**	**Bond Strength (psi)**
A	20570	1746		a	20528	1743
B	20639	1752		b	20934	1777
C	20878	1772		c	20594	1748
Avg. - 67 A-B-C		**1756.8**				**1756.0**
Std Dev. - 67 A-B-C		**13.72**				**18.49**
Coef of Var. - 67 A-B-C		**0.781%**				**1.053%**

Statistical Analysis Chloride Ion Penetration			
# 67 Stone		# 78 Stone	
Sample #	**Charged Passed (coulombs)**	**Sample #**	**Charged Passed (coulombs)**
A	4553	A	4333
B	4324	B	3814
C	4359	C	3735
Avg. - 67 A-B-C	**4412**	**Avg. - 78 A-B-C**	**3961**
Std Dev. - 67 A-B-C	**123**	**Std Dev. - 78 A-B-C**	**325**
Coef of Var. - 67 A-B-C	**2.80%**	**Coef of Var. - 78 A-B-C**	**8.20%**