

## *Final Report*

# **Contract No. BDB09**

## **Investigating the Statewide Variability and Long-Term Strength Deformation Characteristics of RAP and RAP-Soil Mixtures**



*Florida Institute of Technology*

*High Tech with a Human Touch*

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<p>Reclaimed Asphalt Pavement (RAP) usage is limited because tests on RAP and RAP-Soil blends indicates that creep may pose a concern and the statewide variability of RAP has not been addressed. RAP was blended with A-3 sands and the results showed that the addition of 20% A-3 improved the gradation, density, and bearing strength of RAP. Conventional highway materials testing were performed.</p> <p>The one-dimensional creep behavior of RAP and RAP-soil blends was evaluated. Axial strains were analyzed using Singh and Mitchell's approach. Using the resulting strain rate equations, settlements versus time predicted correctly characterized the material behavior. The soil in the RAP-soil blends effectively lowers the asphalt content. Blends containing less than 3.5% asphalt produced substantially lower strain and were not influenced by the applied stress level. The backfill settlements increased exponentially with increasing wall height. The exponential growth was faster at higher RAP contents. Most settlement was predicted to occur during construction.</p> <p>Horizontal and vertical creep and pullout characteristics of reinforcement embedded in 100% RAP, 50/50% RAP-sand blends and 100% A-3 were evaluated. The 100% RAP consistently produced the lowest pullout capacity of the steel reinforcing strips and the greatest reinforcing strip displacement of the three materials during pullout testing. The 50/50% blend yielded the greatest pullout capacity, however this blend yielded a greater strip creep displacement than the A-3. The 100% RAP yielded the largest vertical creep displacement followed by the 50/50% blend and A-3. For alternating RAP and sand layers, placing the sand as the top layer substantially reduced the amount of vertical creep. As a result of this work, it is recommended that RAP and RAP-sand blends not be used in MSE wall applications.</p> <p>The statewide variability testing program was performed on milled or crushed RAP. The asphalt content of RAP can vary from 3.5% to 11.0%. Typically milled RAP varies between 5.5% and 8.0% asphalt while crushed RAP varies from 4.5% to 7.0% asphalt due to blending. RAP's moisture density behavior is non-Proctor like. No RAP materials tested in this study met the minimum FDOT base course unsoaked LBR value of 43. RAP should not be used as a base or subbase course material under highway pavements. Asphalt content is correlated to the engineering behavior of RAP. For every 1% increase in asphalt content, there is a decrease of 3.8 lb/ft<sup>3</sup> in compacted density and a decrease of 4.5 in the LBR value. RAP materials with high asphalt content should not be used in earthwork applications where bearing strength is required.</p>					
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# **Executive Summary**

## **Investigating the Statewide Variability and Long Term Strength Deformation Characteristics of RAP and RAP-Soil Mixtures**

by

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Reclaimed Asphalt Pavement (RAP) stockpiles in Florida have grown because stringent asphalt pavement SUPERPAVE specifications prevent re-using high percentages of RAP as aggregate in Hot Mix Asphalt (HMA) production. Research has shown that RAP can be used as a general highway fill, as a base course on roadway shoulders, and as a sub-base beneath rigid pavements even though it has a relatively low bearing value. Environmentally, it meets all current EPA heavy metals criteria. In several field applications it displayed excellent drainage characteristics and was compacted under adverse moisture conditions using conventional equipment (Cosentino, and Kalajian 2001, and Cosentino et al., 2003).

Despite the positive research results, RAP usage is still limited because of two major concerns. First, testing of RAP and RAP-Soil blends indicates that the overall stiffness and strength behavior can be improved; however, creep may pose a concern if these materials were to be used in highway applications such as, base courses in flexible pavements or behind mechanically stabilized earth (MSE) walls. While secondly, the research to date has been based on RAP from a single plant in Melbourne Florida, therefore, the statewide variability of RAP has not been addressed.

The creep or long-term deformation behavior of RAP and RAP-soil blends was evaluated using a thorough laboratory testing program. RAP and RAP-blended with a control A-3 soil, containing 80%, 60%, 40%, and 20% RAP, were used. The RAP, RAP-soil blends and A-3 soil were evaluated to determine: grain-size distribution, dry rodded unit weight, moisture density, Limerock Bearing Ratio (LBR), asphalt content, and viscosity. Creep testing was performed using a one-dimensional oedometric apparatus with samples subjected to three stress levels well below bearing failure. The axial strain data was analyzed using Singh and Mitchell's constitutive model for determining the strain rate as a function of time. Strain rate equations were formulated using parametric values from the experimental data. These equations were further used to predict a settlement for selected time intervals in RAP and RAP-soil mixtures, used as backfill behind MSE walls. LBR data collected from tests conducted before and after creep testing was used to evaluate the strength increase due to creep deformations in RAP and RAP-soil blends.

The results from engineering properties testing demonstrated that the addition of up to 20% A-3 soil improved the gradation, density, and bearing strength of RAP. Adding higher amounts of soil did not further enhance the engineering properties of RAP.

The laboratory creep testing proved that the one-dimensional oedometer provided a reliable method for evaluation of the long-term deformation of RAP and RAP-soil blends. The applied constitutive model correctly characterized the behavior of RAP and RAP-soil blends at the stress levels used in this study. The soil in the RAP-soil blends effectively lowers the asphalt content of the mixtures. The RAP-soil blends containing less than 3.5% asphalt produced substantially lower strain and were not significantly influenced by the applied stress level. The backfill settlements increased exponentially with increasing wall height. The exponential growth was faster at higher RAP contents. Most settlement was predicted to occur during construction.

To investigate potential use of RAP as backfill behind MSE walls a thorough laboratory program was developed that included both conventional and large scale tests. First, the horizontal creep and pullout characteristics of soil reinforcement embedded in three materials 100% RAP, 50/50% RAP-sand blend and 100% sand were tested, and second, the vertical creep characteristics of these three backfill materials were evaluated. Poorly graded sand, typically used as MSE wall backfill, was blended with RAP and used as a control.

For the large scale testing, a series of pullout tests were conducted in an 8 x 6 x 24 ft. test pit at the FDOT State Materials Office (SMO) in Gainesville, Florida. Steel soil reinforcing strips used in MSE walls were pulled while embedded in the backfill to determine the maximum pullout capacity and the horizontal creep characteristics of each material. During pullout testing, the three materials were subject to vertical stresses of 6, 12 and 18 psi to simulate varying overburden depths. Vertical creep testing was also performed on the three materials plus an alternate RAP-sand layered system. The vertical surface displacement of the material was measured when subjected to the same vertical stresses of 6, 12 and 18 psi. To help validate the large scale tests, three vertical creep tests were performed on these material combinations using the one-dimensional oedometric apparatus developed at the Florida Institute of Technology. Data from these tests were then compared to data from two large scale tests at the FDOT SMO test pit.

The 100% RAP consistently produced the lowest pullout capacity of the steel reinforcing strips and the greatest reinforcing strip displacement of the three materials during pullout testing. The 50/50% blend yielded the greatest pullout capacity, however this blend yielded a greater strip creep displacement than the sand control material. The 100% RAP yielded the largest vertical creep displacement followed by the 50/50% blend and sand control material. When alternating RAP and sand layers were used, placing the sand as the top layer substantially reduced the amount of vertical creep displacement. As a result of this work, it is recommended that RAP and RAP-sand blends not be used in MSE wall applications.

To address the statewide variability a thorough testing program was developed that included sampling from 50 plants across Florida and then conducting tests to determine gradation, asphalt content, dry rodded unit weight, as well as moisture-density and LBR behaviors. Results from these tests were analyzed and compared to determine if location within Florida might cause variability, or if variability was really a concern.

RAP was categorized as either milled or crushed, with milled RAP being defined as material obtained directly from a field rehabilitation process, and crushed RAP being defined as a blend of milled RAP with materials available at the plants. Based on this testing, the asphalt content of RAP can vary widely, from 3.5% to 11.0%. Typical milled RAP has between 5.5% and 8.0% asphalt. Typical crushed RAP has between 4.5% and 7.0% asphalt. The reduction in asphalt content from milled RAP to crushed RAP is due to the blending of RAP materials with reclamation base materials. RAP's moisture density behavior is non-Proctor like and does not

produce a clear peak density and associated optimum moisture content. It is similar to a course graded soil as improving the gradation for the well-graded RAP produces higher compacted densities. Typical crushed or milled RAP materials are well-graded and exhibit gradation characteristics close to the maximum density model.

No RAP materials tested in this study met the minimum FDOT base course unsoaked LBR value of 43. RAP should not be used as a base or subbase course material under highway pavements.

Asphalt content is correlated to the engineering behavior of RAP. In general, for every 1% increase in asphalt content, there is a decrease of 3.8 lb/ft<sup>3</sup> in compacted density and a decrease of 4.5 in the LBR value. RAP materials with high asphalt content should not be used in earthwork applications where bearing strength is required.

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# 1 Introduction

## 1.1 Problem Statement

As Florida's population and associated development experiences dramatic increases, the highway construction industry faces a critical shortage of conventional fill. Agencies throughout the state have expressed interest in reusing a wide variety of waste materials. One abundant waste material, Reclaimed Asphalt Pavement (RAP) is produced when existing asphalt concrete is removed from the road by either surface milling or full-depth reclamation. Currently the Florida Department of Transportation (FDOT) permits the use of RAP in non-structural base course and subgrade stabilization applications, as stipulated in Sections 283 and 914 of the FDOT *Standard Specifications for Road and Bridge Construction 2004*, herein referred to as FDOT Standard Specifications.

There are two major concerns associated with the engineering behavior of RAP; 1) its long term deformation under constant load or creep and 2) its statewide variability. Laboratory testing on creep indicates it is excessive and needs further research (Cleary, 2005) while the research to date has been conducted from a limited number of RAP sources (Cosentino et al, 2001, Cosentino and Kalajian 2003).

RAP is not permitted to be used as backfill behind Mechanically Stabilized Earth (MSE) walls as specified in FDOT Memorandum MB 1-00 (Malerk and Xanders, 2000) which was updated both in 2005 (Prasad and Malerk, 2005) and 2007 (Horhota, Jones and Putcha, 2007). The reason for this limitation is the potential creep of the reinforcing straps embedded in the RAP and the memorandum states that research on the use of RAP as backfill is recommended. Creep of RAP such as excessive deflections, rutting potential or significant permanent strain has been reported in the literature, e.g. Sayed et al (1993), Garg and Thompson (1996), Bennert et al (2000).

The Federal Highway Administration (FHWA) RAP User Guidelines (1998), as well as several other literature sources, refer to variability as one of the unresolved issues for RAP use, influencing its performance, especially when processed and unprocessed materials are mixed. Research to date, conducted on limited sources, demonstrated differences not only between processed and unprocessed RAP, but also between RAP-produced from different down-sizing processes (Cosentino et al, 2001). In summary, the variability may result in unexpected changes in RAP's strength-deformation behavior.

### 1.1.1 Historical Uses of RAP

In the early 1970s pavement recycling became of interest as a result of severe inflation due to decrease supplies from the Oil Producing and Exporting Countries (OPEC). In 1976, FHWA initiated a project to demonstrate the technical viability of asphalt recycling as a rehabilitation technique (Sullivan, 1996). This effort

resulted in materials, mix design, and construction guidelines for implementing an asphalt recycling project. States and paving contractors began making extensive use of RAP and over the years a number of applications were developed including: addition to hot-mix asphalt (HMA); aggregate in cold-mix asphalt; granular or stabilized base and subbase course; fill or embankment material. FHWA reported that presently all 50 states are using RAP.

Using RAP as recycle material was developed through a combination of environmental, economical and technological factors. Recycling eliminates the disposal and the concurrent hauling and transportation costs, provides a source of readily available aggregate as a substitute for limited natural resources, and finally takes advantage of the technological advancements brought about by inexpensive material processing techniques. In 2000, FHWA reported that in the United States 33 million metric tons of RAP were used in highway construction applications out of 41 million metric tons RAP produced (Eighmy, 2002), amounting to some 80% reuse.

Florida began a successful recycling program in 1979 (Smith, 1996). RAP was utilized predominantly in HMA production and was added to mixes placed on FDOT projects. RAP was also incorporated into new pavements on city, county, and residential projects. The earliest projects allowed for HMA mixes containing as high as 50% RAP. After 1990 the RAP proportion in mixes steadily declined to approximately 35% and after the adoption of Superpave (Superior Performing Asphalt Pavement) mixes in 1998, RAP was limited to 15% whenever changing of the virgin binder grade was not intended. In 2002, FHWA reported that in Florida an average of about 20% RAP was added to Superpave mixes and 30% was added to Marshall mixes. Since the states annual asphalt production is approximately 3 million tons; assuming 25% overall reuse yields about 750,000 tons of RAP is reused in pavements. The Recycled Materials Resource Center (RMRC) estimated that the annual RAP production in Florida surpassed 1 million tons (Eighmy, 2003). The difference between RAP produced and utilized as an HMA component proves that there is a demand for expansion on RAP utilization in Florida. Possible uses include earthwork applications such as roadway base courses, embankments or fills.

### **1.1.2 FDOT Research on RAP**

There has been several research initiatives focused on characterizing and limiting the time-dependent deformations of RAP. One approach involved mixing RAP with soil, thus reducing the viscosity and improving the gradation of the mixture. Cosentino and Kalajian (2003) developed testing methodology and demonstrated that creep deformations decrease with addition of soil. Cleary (2005) obtained significantly reduced creep after testing RAP and clean sand mixtures in one-dimensional confined compression at three stress levels.

## **1.2 Objective**

In order to advance the knowledge of RAP the objectives of this project are:

1. To assess the statewide variability of RAP to determine the applicability of specifications and research conclusions throughout Florida;
2. To evaluate and develop specifications on the long-term behavior of RAP and RAP-Soil mixtures.

### **1.3 Approach**

In order to meet the objective a 24-month laboratory and field-testing program was conducted. The tasks proposed are briefly explained below.

**Task 1 Statewide Variability Sampling** – Representative RAP samples were obtained from numerous locations throughout the state. FDOT district geotechnical engineers were contacted and their assistance was requested in obtaining samples. Sufficient quantities of samples were obtained to enable a thorough testing program to be completed. All sampling protocols followed the Manual of Florida Sampling and Testing methods (FM) and the American Society for Testing and materials (ASTM) procedures.

**Task 2 Statewide Variability Testing** – The testing program consisted of classification, moisture-density and Limerock Bearing Ratio (LBR) strength-deformation testing. Sufficient tests were conducted on each sample to determine the variability within each stockpile along with the variability from within the state.

**Task 3 Statewide Variability Analysis** – Results from the testing were compared and statewide statistical variations were developed based on grain size, compaction and LBR data. The affects that this variability would have on the engineering use of RAP has been documented based on the current specifications and conclusions from the previous research.

**Task 4 RAP-Soil Mixing** – The research conducted to date indicates that RAP density, bearing strength and stiffness improved with the addition of silty-sand size materials (i.e. materials graded between the number 40 and 200 sieves). The long-term or creep behavior of similar mixes has been evaluated. Materials selected for mixing were soils that can easily be blended with RAP in the field.

**Task 5 Laboratory Creep Testing** – Numerous lab tests have been conducted on RAP and RAP-Soil mixtures to evaluate creep characteristics. Models of the creep behavior were developed and will be presented in this final report.

**Task 6 Creep and Pullout Testing for MSE Simulation** – RAP and RAP-Soil mixtures were subjected to creep and reinforcement pullout tests;



performed at the FDOT State Materials Office (SMO) in Gainesville, Florida, to determine their long-term behavior when used as backfill behind MSE walls. RAP, RAP-soil mixes and an A-3 sand control were used for the testing.

**Task 7 Long Term Pavement Performance Evaluation(s)** – Pavement design parameters will be characterized through a series of plate-bearing tests also conducted in the indoor FDOT SMO test pit. RAP, RAP-soil mixes and an A-3 sand control were also used for this testing.

## 2 Background and Theory

### 2.1 Aggregates in Florida

In 2005, Florida was the 2<sup>nd</sup> largest producer of crushed stone, annually producing 123 million tons and due to a statewide shortage it had the nation's highest value at \$944 million. When crushed stone is combined with sand and gravel, the total aggregates produced was 159 million tons, with a value of \$1.20 billion, making Florida the third largest producer of aggregates (USGS 2006). These excessive material demands are a result of Florida's new construction from rapid population growth. In 2004, approximately 83% of our aggregates came from mines within the state, nearly 10% was imported from other states and countries and 7% came from recycled construction wastes. Nearly 17% of the demand is currently met from sources other than the traditional mines and this percentage has increased rapidly during the last decade. The Florida road-building and construction industries are expected to consume 143 million tons of crushed stone in 2007. If projections hold, construction of new structures may require 86 million tons of crushed stone, almost half of which will be used to meet housing needs of our rapidly expanding population. Forty-two million tons of rock will be needed for construction of roads, bridges, runways, and other infrastructure (Lampl 2007).

In Florida, coarse aggregate and crushed stone is generally produced from limestone, which is mined from naturally occurring deposits found in 22 counties as shown in Figure 2.1.

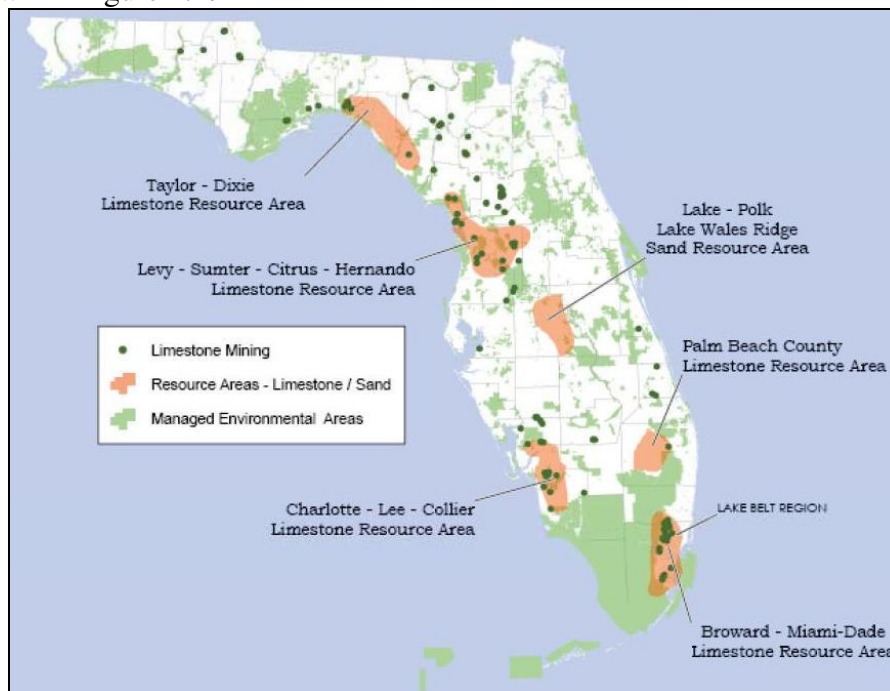


Figure 2.1 Limestone and sand resource areas (Lampl 2007)

## **2.2 Current Earthwork Applications for RAP**

Economically, Florida RAP is most valuable when reused in the production of new hot mix. RAP earthwork applications are currently described in several FDOT specifications and memorandums. Sections 283 and 914 of FDOT Standard Specifications (2004) govern the earthwork applications of RAP in Florida highway construction as base course and subgrade stabilization respectively. According to Section 283 the use of RAP as base course is restricted to paved shoulders, bike paths, or other non-traffic applications. Section 283 specifies the asphalt content of RAP to be greater than 4%, whenever the material does not originate from FDOT projects and its source is unknown. Section 914 regulates RAP for subgrade stabilization use provided; a) that a minimum LBR of 40 can be achieved, b) the material does not cause excessive deformations, and c) it improves the bearing capacity of the stabilized material.

In 2000, 2005 and 2007, the above described applications were supplemented by FDOT Memorandums (Malerk and Xanders, 2000), (Prasad and Malerk, 2005) (Horhota, Jones and Putcha, 2007). Statements in these memos prohibit RAPs' use below the groundwater table, in the top 6 inches of highway slopes and shoulders to be vegetated and behind MSE walls. Two methods of incorporating RAP into the embankment are permitted: 1) as a mixture of RAP and soil with ratio from 1:2 to 2:5, and 2) in alternating equal layers with soil. For the layered method a soil of minimum LBR of 40 was recommended, where for the mixtures no further requirements were imposed. However, due to the possibility of creep of the metallic reinforcing strips, neither RAP nor RAP-Soil mixes are permitted for use as backfill behind MSE walls.

In contrast to the limited RAP usage in Florida other than HMA, many states permit RAP for use in a variety of earthwork applications such as: stabilized base courses, unbound aggregate base and subbase, shoulder aggregates, and opengraded drainage courses. The results from a 1994 questionnaire sent to all state highway agencies indicated that 16 states have used RAP as an unbound aggregate base or subbase (Collins and Ciesielski, 1994).

## **2.3 Field Studies on RAP Earthwork Applications**

The RAP literature review focus was on research with gradation, compaction and bearing strength information, as essential parameters monitored for field construction quality control during earthwork operations.

### **2.3.1 RAP Shoulder Base Usage**

Untreated reclaimed asphalt pavement was used for construction of paved shoulders on State Road No. 500 in Holopaw, Florida (Sayed et al, 1993). The material was obtained by milling existing pavement. The shoulders were

constructed using a 4-inch RAP base, overlaid with 1½ inch of type S-1 asphalt concrete and a friction course. Both laboratory and field tests were conducted.

The laboratory analysis included gradation, compaction, strength and physical properties. The RAP grain-size distribution indicated that the material was well graded with 97% passing the 1½ inch sieve and 4% fines. The maximum dry unit weight was comparable to other granular materials, with a maximum modified Proctor compaction density of 122.9 lb/ft<sup>3</sup> obtained at 6.2% optimum moisture content. The LBR values were noticeably lower than those for conventional base course material – in the range of 25 to 30 for soaked samples, and 29 to 38 for unsoaked samples. The asphalt content ranged from 5 to 7%.

The field investigations included density and LBR testing, as well as surface deflections. The maximum in situ unit weight for RAP was 129.6 lb/ft<sup>3</sup> at 6.6% moisture content. The maximum LBR value from in situ testing was 54 for RAP, compared to approximately 70 for a control section constructed with limerock base material. Surface deflections of the RAP section were measured and compared with the limerock control section using the Dynaflect nondestructive testing equipment. Deflections in the RAP exceeded deflections in the limerock by 20 to 40%. The RAP section was visually inspected after construction indicating that the paved shoulders performed well. The results suggested that RAP base was slightly weaker than limerock base, but RAP could be comparable to limerock in shoulder base applications. It was also concluded that due to the deflections continued monitoring of the long-term pavement performance is necessary.

### **2.3.2 RAP in Pavement Base**

The research findings for using RAP as a pavement base at the Lincoln Avenue reconstruction project in Urbana, Illinois, were summarized by Garg and Thompson (1996). RAP was produced by crushing Illinois Department of Transportation (IDOT) Class I reclaimed asphalt concrete. A 1,200 ft. two-lane demonstration section was constructed with a 12-inch stabilized subgrade layer, an 8-inch compacted RAP base course overlain by a 3-inch asphalt concrete surface course. A control section containing the same subgrade and asphalt surface course with the 8-inch crushed stone aggregate (CA-6, with a 1½ inch top size) base replacing the RAP was built.

Laboratory testing included gradation, moisture-density, and triaxial shear. Sieve analysis results indicated that RAP was coarse grained poorly graded with 100% passing the 1½ inch sieve and 4% fines. The percent passing both the #4 and #200 sieves approached the lower gradation tolerances of IDOT base construction specifications. The standard Proctor test yielded a dry unit weight of 125.2 lb/ft<sup>3</sup> at 7.2% optimum moisture content compared to 141.5 lb/ft<sup>3</sup> at 5.8 % optimum moisture for the crushed stone control base. The triaxial testing program included rapid shear tests and repeated loading tests conducted to determine the shear strength parameters, resilient modulus and rutting potential of RAP. At the 15 psi confining stress RAP displayed a cohesion of 19 lb/ft<sup>2</sup> with a friction angle of 45°.

versus cohesion of 25 lb/ft<sup>2</sup> with a friction angle of 44° for crushed stone use as the research control. RAP yielded higher resilient moduli than the crushed stone. Permanent deformation testing proved that RAP had higher rutting potential than crushed stone.

The field evaluation consisted of; 1) Dynamic Cone Penetrometer (DCP) tests in the base materials prior to paving at 50 ft. intervals along the project length, 2) Falling Weight Deflectometer (FWD) testing using the 9 kips load in the outer wheel path, and 3) pavement distress monitoring.

An LBR obtained from DCP bearing ratio correlations was estimated at 188 for crushed stone, but 81 for RAP, thus RAP would not meet FDOT's required LBR of 100 for aggregate base. FWD results showed about 13% higher surface deflections in RAP than the crushed base. One positive note occurred when a 19 month pavement evaluation on the RAP section produced only minor rut depths. The LBR values were determined assuming the bearing strengths from the correlations were for standard California Bearing Ratio (CBR) testing at 0.1-inch deflection. The LBR was then obtained by multiplying the CBR value by 1000 psi and then dividing it by 800 psi.

Based on the field and laboratory data it was concluded that RAP is comparable to crushed stone and can be successfully used as conventional pavement base material. It was suggested, however, that the RAP source and processing are important factors for its behavior. It was recommended to further investigate RAP shear strength and rutting potential.

### **2.3.3 RAP as Base and Subbase Material**

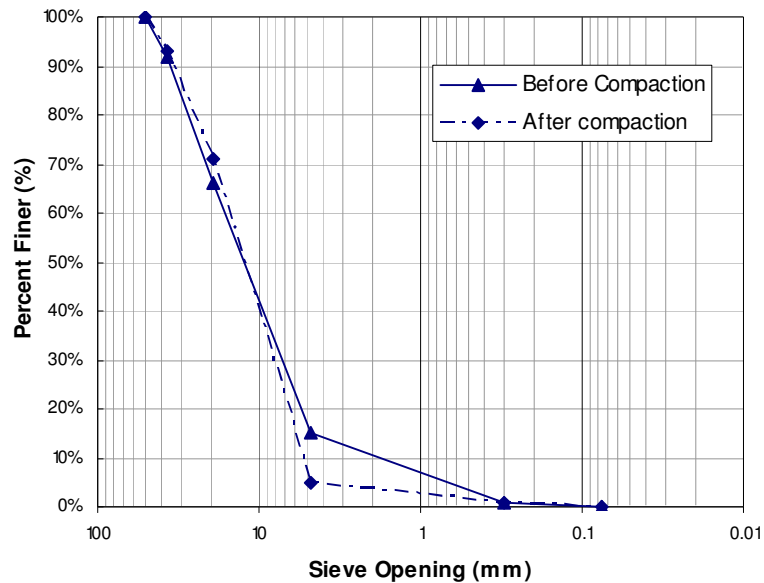
A widening project on US Route 1 in New Jersey was implemented to evaluate RAP performance as base material (Maher et al, 1997). The test section consisted of a 12 ft. wide outside shoulder plus one 11 ft. wide slow lane. The entire project length was constructed with three subsections for the base course; a) the conventional base aggregate, b) a 100% RAP and c) a blend of 75% RAP and 25% aggregate respectively. The shoulder and the slow lane had identical design profiles consisting of a 2 inch asphalt concrete surface course; underlain by a 7-inch asphalt stabilized base course, then a 6 inch base course, and finally an 8-inch subbase.

Standard laboratory tests were performed to characterize RAP according to its moisture-density properties, degradation, and resilient modulus. Sieve analysis results indicated that RAP was well graded with 93% passing 1½ inch sieve and 0% fines.

A comparison between the gradation curves before and after compaction, (Figure 2.2), indicated slight grain size degradation occurring between 1½ inch and ½ inch sieves, which based on visual observations, was attributed to debonding of the larger diameter agglomerated particles. The maximum dry unit weight of RAP determined from Standard Proctor compaction test was 113 lb/ft<sup>3</sup> at 5.5% optimum

moisture content. The resilient modulus testing yielded RAP values between 65 and 80% higher than those for typical base aggregate.

Field investigations were conducted using a Seismic Pavement Analyzer, a device similar to FWD. The thickness and elastic moduli of all courses were evaluated. The elastic modulus values for RAP and conventional aggregate were comparable, while the blend of 75% RAP showed slightly higher results. From this study it was concluded that RAP was a viable alternative to conventional aggregate in base and subbase applications. The investigators recommended that research needs to be continued to determine the permanent deformations characteristics of RAP.



**Figure 2.2 Gradation of RAP before and after compaction (Maher et al 1997)**

Based on the conclusions from the three earthwork field studies, RAP has the potential to replace conventional base material. In these field studies, RAP displayed a lower bearing strength than base aggregate and was prone to long-term deformations. To improve both its strength and creep RAP was mixed in varying percentages with conventional aggregate.

## 2.4 RAP-Soil Mixtures

### 2.4.1 Field Studies

#### 2.4.1.1 Mix of RAP and Aggregate in Pavement Base

The granular base course performance of RAP when mixed with aggregate was evaluated, on State Highway 395 in California (Bejarano et al, 2003). The rehabilitation process required that the distressed asphalt concrete layer be

pulverized and mixed with a portion of the existing underlying base material to create a new thicker base course.

A laboratory testing program was conducted to characterize the pulverized material and compare it with conventional base aggregate. Sieve analyses data indicated that the pulverized material was well graded with 98% passing 1½ inch sieve and about 2% fines, thus meeting the base course material gradation requirements. Compaction tests\* yielded a maximum dry unit weight of 145.6 lb/ft<sup>3</sup> at optimum moisture content of 5.5% for the pulverized material, a value slightly higher than the dry unit weight for conventional aggregate. Triaxial test results lead researchers to conclude that the pulverized material had similar shear strength to the conventional aggregate, possessed zero cohesion, and a friction angle between 51.2 and 57.5° depending on the relative compaction.

The field testing program included DCP and FWD tests before and after construction. From the DCP results the investigators estimated that the base bearing strength of the pulverized base improved approximately 100% compared to the deteriorated old pavement. FWD data indicated some decrease in the surface deflections after construction. Based both on laboratory data and field testing it was concluded that the pulverized base course material was stronger than a conventional aggregate base, possessed a reduced potential for rutting and lower deflections, and overall exhibited a better performance.

#### **2.4.1.2 Using RAP in Pavement Subgrade**

Laboratory and field tests were performed in Kuwait to assess the improvement of subgrade soil properties with the addition of RAP (Aljassar et al, 2005). The original road bed soil was classified as A-1-b according to the American Association of State Highway and Transportation Officials (AASHTO) Soil Classification and was compacted to a maximum unit weight of 120.4 lb/ft<sup>3</sup> at a CBR bearing strength of 21. The milled RAP added was well graded with high percentage of coarse particles and very little fines. Compaction, gradation, and bearing strength testing were performed on the various mixes of RAP and aggregate.

The 22% RAP mix produced a gradation close to the local base course aggregate gradation and was selected for further field testing. This mix yielded a maximum unit weight of 127.5 lb/ft<sup>3</sup> and CBR of 88, which is equivalent to a 6% increase in density and a 320% increase in bearing strength. The mix was used as a subgrade for a rural road and Dynaflect testing results fell well within the specification requirements. The study showed that addition of RAP improved significantly the properties of subgrade soil in terms of maximum unit weight and

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\* The California Department of Transportation specifications require a higher compaction effort for moisture-density testing than FDOT.



CBR and had the potential to incur considerable cost savings in pavement construction projects.

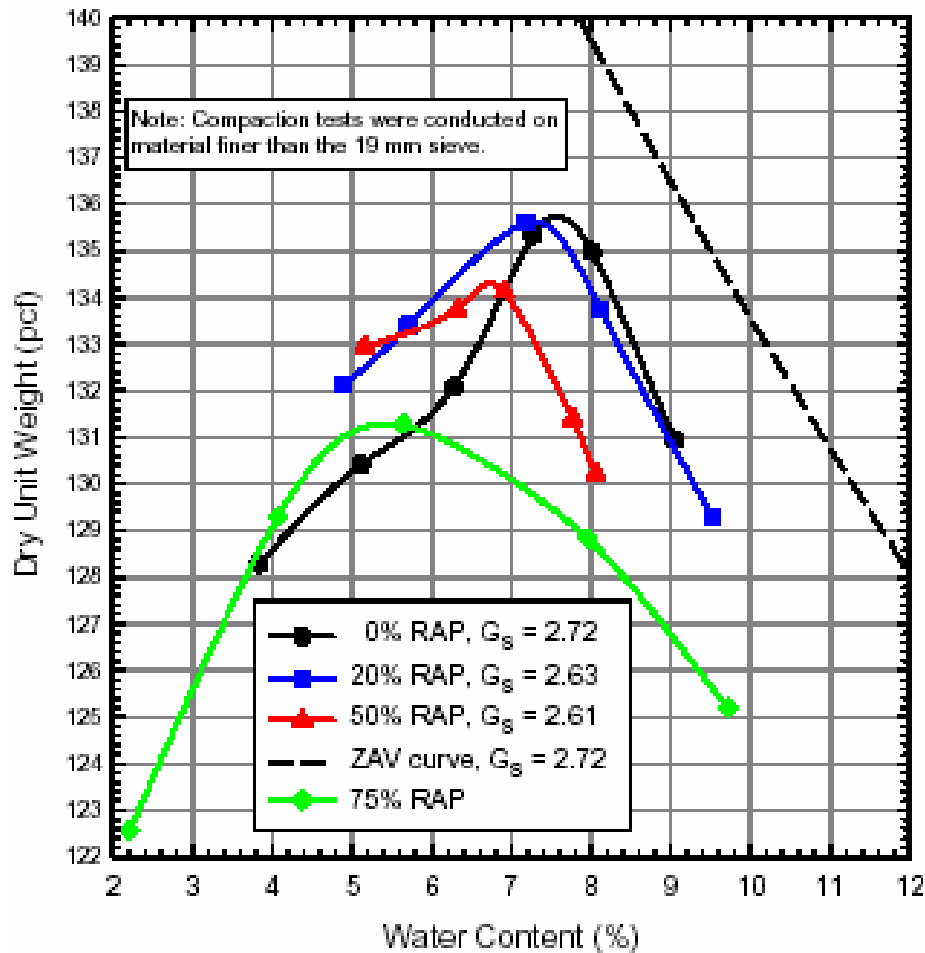
## **2.4.2 Laboratory Studies**

### **2.4.2.1 Engineering Characteristics Evaluation of RAP/Aggregate Blends**

The changes in engineering properties of granular soils from various sources in Montana were evaluated after blending them with RAP (Mokwa and Peebles, 2005). The research focused on primary engineering properties including compaction, gradation, strength, stiffness, permeability, and resistance to degradation. Milled RAP was mechanically mixed at percentages of 20%, 50% and 75% by weight with four aggregates – three of them mechanically processed materials meeting the crushed base course specifications, and the fourth being a natural gravel material.

The grain-size distributions for the unblended materials showed that they were well-graded. RAP alone displayed a well graded curve with 96% passing 1½ inch sieve and 1% fines. For all four aggregates tested, the addition of RAP to the virgin materials resulted in an increase in the amount of particles passing the larger-opening sieves, and a decrease in the percentage of particles passing the smaller-opening sieves. The smaller particles percentage reduction was explained with adhesiveness due to viscosity of the asphalt particles, and the milling process used to produce the RAP material.

The modified Proctor compaction tests on the four materials indicate that adding RAP causes the compaction curves to decrease and shift to the left. The curves for one of the aggregates used, pit run gravel, are presented in Figure 2.3. The curves in the figure displayed a relatively small decrease of the maximum unit weight of 3.3% at 75% RAP content, still suggesting that the blends were comparable to the virgin aggregate. It was believed that the decrease was caused by the lower specific gravity of RAP. For the same pit run gravel the gradation changes due to compaction were analyzed and found to be minimal. The particles between the #4 and #16 sieve were slightly affected with changes of less 5%. These changes were attributed to the presence of water, the viscosity of RAP, and the dynamic impact of the compaction equipment.



**Figure 2.3 Moisture-density curves for blends of pit run gravel with RAP (Mokwa and Peebles 2005)**

Results from large direct shear tests conducted on blends of crushed base course aggregate and RAP indicated that at a given normal stress, the soil shear strength decreased as the quantity of RAP in a sample increased. The effect was more pronounced as the normal stress increased. Overall the shear strength decrease was considered insufficient for the exclusion of RAP as a blending alternative.

Permeability tests demonstrated that with higher RAP percentage the permeability of blends increased, which was attributed to the uniform gradation of the material occurring upon addition of RAP. In general, the conclusion from this study was that blending RAP with conventional base aggregates results in only minor changes to the engineering properties of the original material. It was recommended, however, that future studies on the long-term field performance of blends are necessary and depending on the change of engineering properties

inflicted by RAP addition it should be considered establishing limits on the maximum amount of RAP allowed in the blend.

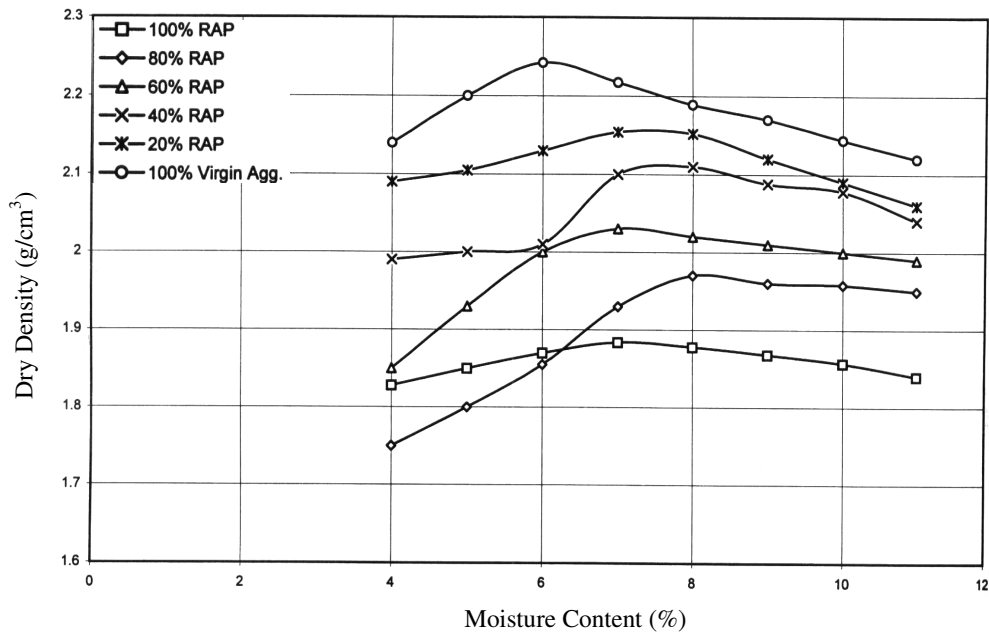
### 2.4.2.2 Evaluation of RAP in Bases and Subbases

Laboratory evaluation of RAP and RAP-virgin aggregate mixtures, used as both road base and subbase, was conducted in the Sultanate of Oman (Taha et al, 1999). Gradation, compaction, and bearing strength tests were performed on RAP/aggregate blends of 100/0%, 80/20%, 60/40%, 40/60%, 20/80%, and 0/100%.

The virgin aggregate was a mix of well graded sand and gravelly sand with little or no fines. RAP was obtained through milling and contained 5.5% asphalt content. According to the sieve analysis RAP was classified as well graded with 100% of the particles passing 1½ inch sieve and 0.5% fines.

Moisture-density testing for the six materials resulted in a series of relatively flat curves, shown in Figure 2.4, with 100% RAP positioned at the bottom and achieving a maximum unit weight of 117.3 lb/ft<sup>3</sup> at 7.3% optimum moisture content for the modified Proctor procedure. The 100% RAP also resulted in the lowest bearing strength value, producing a CBR of 11.

Based on the testing performed, all RAP-aggregate blends, with the exception of 100% RAP and 80% RAP, were considered suitable for road subbase construction. For base construction, however, only mixes containing 10% RAP or less were recommended.



**Figure 2.4 Moisture-density curves for various RAP-aggregate blends (Taha et al 1999)**

### **2.4.2.3 Highway Characteristics of RAP-Soil Mixtures**

Gomez (2003) investigated the strength and drainage characteristics of RAP mixed with a dredge spoil classified as fine sand with trace clay. Laboratory tests were conducted to determine the grain-size distribution, moisture-density relationship, permeability, LBR, stiffness, shear strength, and resilient modulus of 100% RAP and of RAP/soil mixtures containing 80% and 60% RAP.

The RAP used in this study was hammermill-crushed with an asphalt content of 5.2%, and was obtained from the surface of a previously constructed field research site at the APAC Melbourne facility. Based on the gradation curve the RAP was classified as well graded coarse material with an AASHTO A-1-a classification and contained 98% passing 1/2 inch sieve and 0.4% fines. The modified Proctor compaction testing produced the highest dry density for the 80% mix of 121.7 lb/ft<sup>3</sup> at 6% water content, while 100% RAP achieved a maximum dry density of 117.8 lb/ft<sup>3</sup> at 8% optimum water content. The 80% mix also yielded the highest LBR value of 84, while 100% RAP had an LBR of 40. Triaxial compression tests at 5 psi confining pressure resulted in a friction angle of 44 degrees and cohesion of 4.9 psi for 100% RAP. Permeability decreased by two orders of magnitude from 10<sup>-4</sup> to 10<sup>-6</sup> with as percent dredge spoil increased from zero to 40.

The conclusions from this study suggested that although permeability of RAP decreased by mixing it with the dredge spoil, the strength properties of the mix improved. RAP was considered acceptable for subgrade construction, but did not meet the strength requirements for highway base applications.

### **2.4.3 Variability of RAP and RAP-Soil Mixtures**

A summary of the engineering properties of RAP and RAP-soil mixtures is presented in Table 2.1. It shows gradation, soil classification, moisture-density, LBR, shear strength parameters, and asphalt content data. The results indicate that the engineering properties of RAP and RAP-soil mixtures fall within conventional geotechnical and highway engineering ranges and/or categories, however, they do show some variability.

Generally RAP is well graded with less than 5% fines. The Unified Soils Classification System (USCS) classification varies over a relatively large range from well graded sand (SW) to poorly graded gravel (GP) while it consistently classifies as A-1-a according to AASHTO. The maximum modified Proctor dry unit weight is between 112 and 120 lb/ft<sup>3</sup> at moisture contents in the range of 5 to 7%, while the standard Proctor density values range from 104 to 125 lb/ft<sup>3</sup> and 5.5 to 8.5% for optimum moisture. The LBR typically varies from 11 to 43; however, a value of 81 was reported by Garg and Thompson (1996). This higher value was most likely the result of much stronger aggregates typical of this region. The LBR values from work in Florida ranged from 16 to 43. In general, values above 10, if based on soaked tests and 12 if based on unsoaked tests are acceptable for use as subgrade soils, while values greater than 40, if soaked and 43 if unsoaked are

acceptable for subbase applications, and the angle of internal friction exceeds  $44^\circ$ . The asphalt content values fall between 5 and 7%.

This literature indicates that RAP could provide an economical and structurally sound alternative to conventional soils in earthwork applications; however, creep requires additional research. Studies on creep have focused on cohesive soils instead of granular soils, and only one study focusing on creep in RAP was identified.

**Table 2.1 Summary of RAP engineering properties from literature review**

Study	Author	Year	RAP Application	Processing	Gradation		Soil Classification		Compaction			LBR Equivalent <sup>4</sup>	Shear Strength Parameters c(lb/ft <sup>2</sup> );φ(°)	Asphalt Content
					Passing 1.5 in sieve	Passing #200 sieve	USCS	AASHTO	Opt w	Max γ <sub>d</sub> Mod Pr, pcf	Max γ <sub>d</sub> St Pr, pcf			
<b>100% RAP</b>														
SR 500, Holopaw, FL	Sayed et al	1993	Shoulder base	Milled	97%	4%	GW/SW	A-1-a	6.2%	122.9		25 to 38		5.6%
Lincoln Avenue, IL	Garg & Thompson	1996	Base course	Crushed	100%	4%	GW/GP	A-1-a	7.2%		125.2	81	c=19;φ=45	
US Route 1, NJ	Maher et al	1997	Base & Subbase	Milled	93%	0%	GW	A-1-a	5.5%		113	11 to 21		
SH 395, CA	Bejarano et al	2003	Base	Pulverized	98%	2%	GW	A-1-a	5.5%	145.6 <sup>1</sup>			c=0;φ=58	
Florida Tech Research	Montemayor	1998	Base & Subbase	Milled	100%	0.5%	GW/SW	A-1-a	7 / 8.5%	112	104	16 to 43		6.7%
Florida Tech Research	Gomez	2003	Base & Subbase	Crushed, used on a testing site	98%	0%	GW	A-1-a	8.0%	117.8		40	c=5;φ=44	5.2%
Florida Tech Research	Cleary	2005	Backfill	Crushed	100%	1%	GW/SW	A-1-a	7.0%	116.8	114.3			5.2%
<b>RAP-Soil Mixtures</b>														
Kuwait <sup>2</sup>	Aljassar	2005	Subgrade	Milled	100%	6.5%	GW	A-1-a	8.5%	127.5		110		
University of Montana	Mokwa & Peebles	2005	Base & Subbase	Milled	96%	1%	GW	A-1-a	5.4%	131.3 <sup>3</sup>				
Oman	Taha et al	1999	Base & Subbase	Milled	100%	0.5%	GW	A-1-a	7.3%	117.3		14		
<sup>1</sup> Compaction tests were conducted following CTM 216, which requires higher compaction energy than Proctor Modified Test (AASHTO T-180).														
<sup>2</sup> All testing data is for a mix of 22% RAP and 78% A-1-b soil														
<sup>3</sup> Max unit weight for a mixture of 75% RAP with 25% pit run gravel.														
<sup>4</sup> LBR equivalent determined by using ratio: LBR/CBR =1.25														

## 2.5 Creep Behavior of Soils

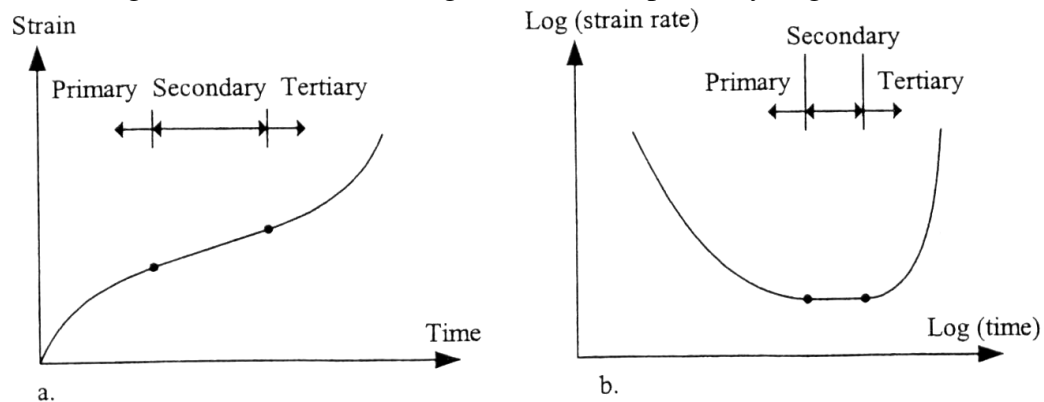
A review of relevant literature on soil creep behavior follows, including definition and stages of creep, and specifics of using oedometer apparatus for creep testing.

Soil strength is typically described as shear strength. As compressive stresses are applied to soils they produce shear stresses as the particles roll and slide past one another. As the applied shear stress approaches the soils shear strength, a phenomenon called creep may occur. A creep process is characterized by increasing strain at constant stress (Coduto, 1994).

Augustesen et al (2004) compiled a concise review of soils' time-dependent behavior. Observations from both triaxial compression and one-dimensional oedometer tests were evaluated considering different time-related effects including creep, relaxation and rate dependency. The characteristic behavior of cohesionless and cohesive materials was analyzed separately. With regard to creep, 1) the authors defined the phenomenon, 2) discussed its different stages, 3) treated the problem of reference time, and 4) analyzed behavioral specifics of cohesive and cohesionless soils. Since the testing methodology adopted for the current study incorporates oedometer-based testing, the one-dimensional test observations reported specifically for the case of granular soils were considered of particular importance to the theoretical background of the study.

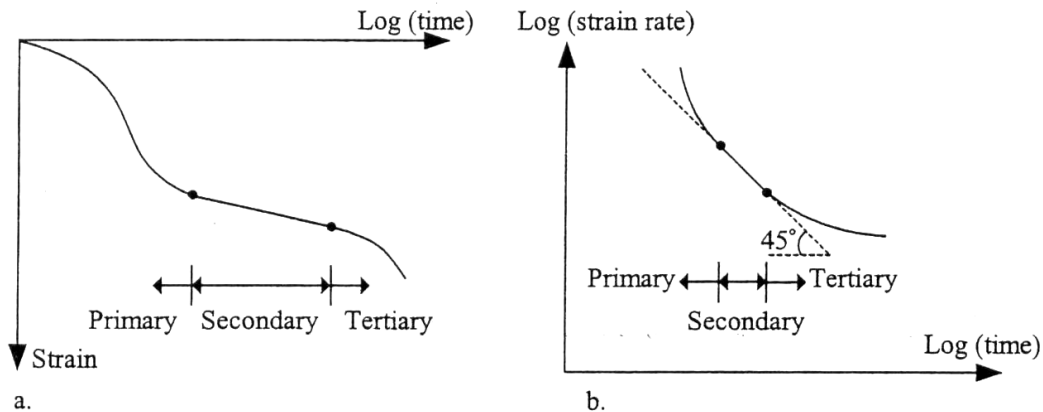
### 2.5.1 Definition and Stages of Creep

A comparison between triaxial creep and oedometric creep revealed that there are differences in the process stages between the two setups. The process under triaxial loading was divided into primary, secondary and tertiary creep, identified as decreasing, constant and increasing strain rate, respectively, Figure 2.5.



**Figure 2.5 Creep stages for triaxial testing: (a) strain versus time and (b) log strain rate versus log time (Augustesen et al 2004)**





**Figure 2.6 Primary, secondary, and tertiary compression for oedometer testing: (a) strain versus log time and (b) log strain rate versus log time, (Augustesen et al 2004)**

Augustesen et al (2004) also termed the stages typical for oedometer apparatus as primary, secondary, and tertiary compression (Figure 2.6). The primary stage was attributed to void reduction or excess pore pressure dissipation and was regarded as identical to the primary consolidation. The secondary stage, where the relationship between strain and logarithm of time was linear, was equated to secondary consolidation, or pure creep, and was considered to result from deformations in the soil skeleton. The tertiary stage was also considered as pure creep that occurs subsequent to secondary compression, but with a nonlinear relationship between logarithm of time and strain.

The significant behavioral difference of soil subjected to oedometer and triaxial creep, was that in oedometer testing the strain rate decreased with time throughout all three compression stages.

### 2.5.2 Reference Time

Reference time is defined as the end of primary consolidation (EOP), the definition of which influences the magnitude of creep. Two hypotheses are presented in the literature. The first assumes that the secondary compression, or creep, occurs only after primary consolidation ends; while the second implies that creep occurs during the whole consolidation process. In granular soils there is little distinction between the first and second hypothesis. After establishing EOP, a logarithmic function can be used to model the vertical strain versus time test data.

### 2.5.3 Observations from One-Dimensional Tests on Granular Soils

Two important aspects of the oedometer testing of granular soils were described by Augustesen et al (2004): stress dependency and strain-time behavior. The stress dependency behavior of granular materials was grouped into:

- 1) behavior at low confining stresses with deformations produced by soil skeleton rearrangement due to sliding and
- 2) rolling of the particles and behavior at high

confining stresses with deformations caused by continuous fracturing, crushing and deformation of the grains.

Studies performed at both low and high stresses proved that the compression of sand is not instantaneous, but continuous over a long period of time at an ever-decreasing rate. The stress dividing the two behavior modes was dependent on the nature of the material and was termed “critical pressure”. For example, a sample of calcareous sand produced a critical pressure at 116 psi, while siliceous sand produced a critical pressure at 726 psi. The critical pressure concept was validated by analyzing grain-size distributions before and after tests at different pressures. It was determined that under high confining pressures the time-deformation effects can be correlated to grain crushing.

The strain-time relationship for both low and high confining stress modes resulted in a linear function when plotted as strain versus logarithm of time. Some materials tested under high confining stress exhibited high initial strain rates, suggesting that a power relation between strain and time may give a better trend; such behavior, however, was observed only for a limited time and with specific soils.

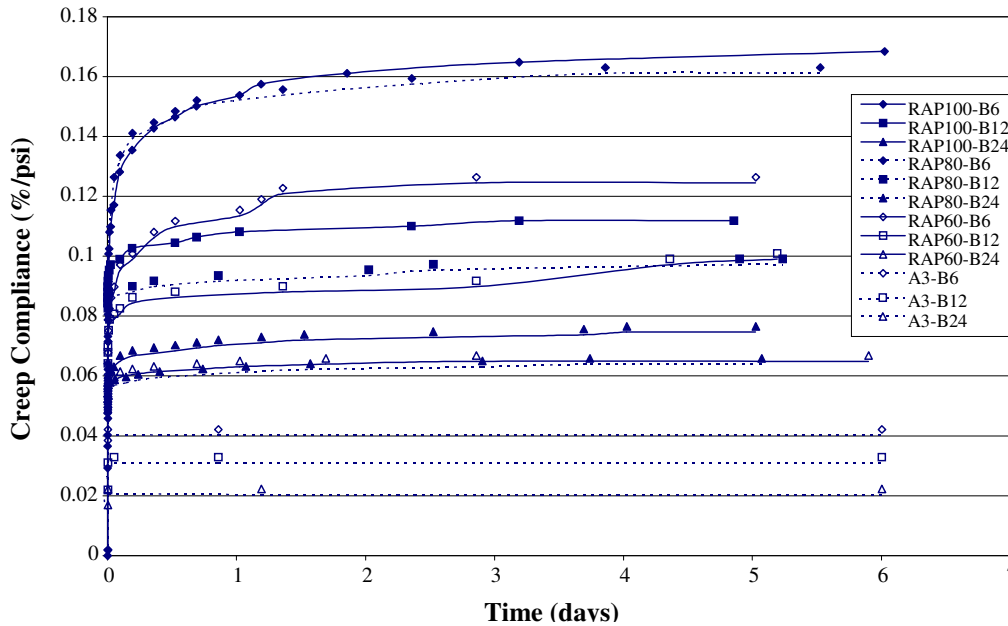
#### **2.5.4 Creep Analysis of RAP, Sand and RAP-Sand Mixtures**

Creep typically occurs in highly plastic cohesive soils such as clays and silts. Recent studies have determined that non-cohesive materials such as RAP have the ability to creep significantly due to the asphalt surrounding the aggregates. Creep can be characterized by the vertical deflection of the material caused by a constant normal stress applied to the material.

Research conducted by Cleary (2005) and Dikova (2006) identified the creep potential of RAP and RAP-sand mixtures. Cleary’s study utilized the creep compliance of Maxwell and Voigt (Huang, 2004) models and calculation of settlement based on the rate process theory from Singh and Mitchell (1968), which was also implemented in Dikova’s (2006) study.

Cleary (2005) used two consolidometers and one pneumatic loading device (PLD) to measure the creep of RAP and RAP-sand mixtures. The samples were compacted using standard Proctor compaction and modified Proctor compaction tests. RAP proportions of 100, 80, 60 and 0% were mixed with an AASHTO classified A-3 sand. Vertical stresses of 6, 12 and 24 psi were applied to the samples to create overburden pressures. The materials were evaluated under constant stress for approximately 6 to 12 days.

Figure 2.7 presents creep compliance, or time dependent strain divided by constant stress, versus time curves for 100% RAP compacted using the modified Proctor. It is observed that as the amount of RAP in the mixture decreases, the amount of creep also decreases.



**Figure 2.7 Creep compliance versus time for 100% RAP using modified Proctor compaction (Cleary 2005)**

Cleary (2005) also calculated settlement based on strain rate. Using the secant method, strain rates were calculated from the slope of cumulative axial strain versus time plots by selecting specific time increments. The strain rates were then plotted versus time, both on a logarithmic scale, and a trend-line was used to develop an equation for the linear relationship. Using logarithmic relationships between strain rate and time, when plotted, created a linear function that was independent of creep stress intensity. The increase in stress only affected the linear function by shifting the line upwards and to the right as seen in Figure 2.8. This equation was then used to calculate long-term settlement. Fifty year settlement calculations based on strain rate for 15 and 30 ft. walls using 100, 80 and 60% RAP are presented in Figure 2.9.

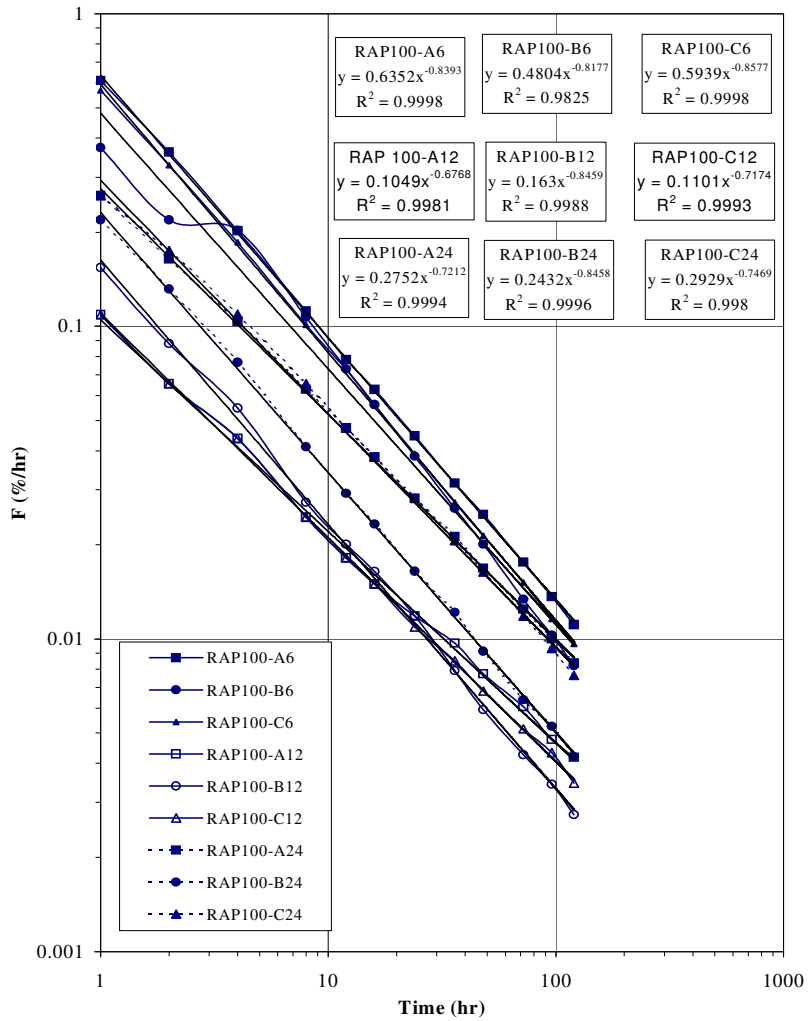
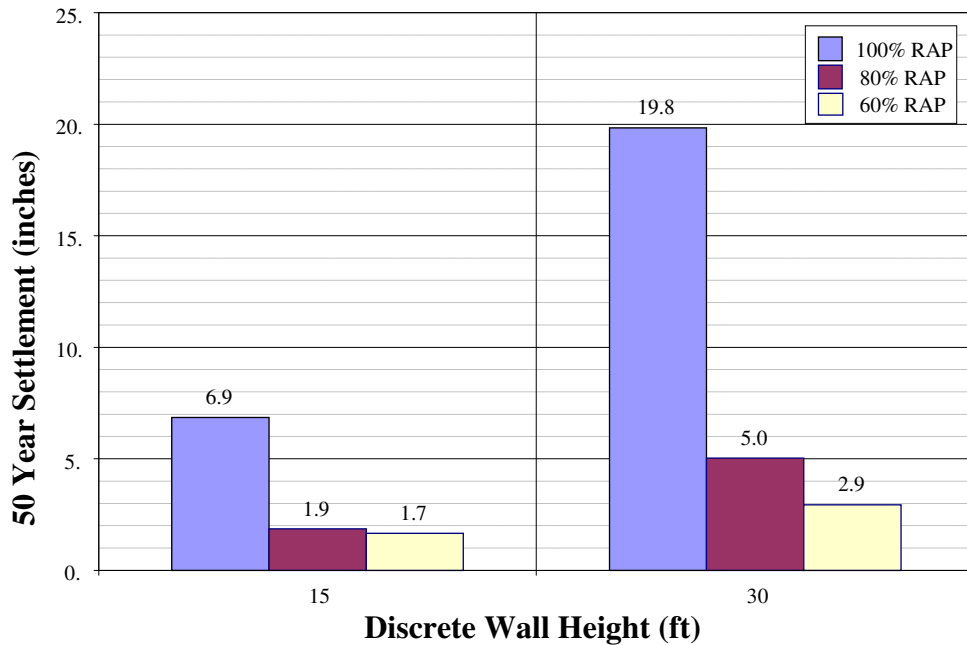


Figure 2.8 Log strain versus log time relationship (Cleary 2005)



**Figure 2.9 50 year settlements for discrete wall heights based on standard Proctor compaction (Cleary 2005)**

The greatest improvement of creep occurred in the 80% RAP mixture. The 60% mixture showed improvement; however, not compared to the difference between the 100 and 80% mixtures. Fifty year settlement calculations similar to Figure 2.9 were also developed for samples compacted with modified Proctor; however Cleary (2005) concluded that relative compaction and laboratory equipment may have affected the results for each sample, therefore creating inconsistent patterns in data.

Dikova's (2006) study continued on the recommendations of Cleary's (2005) study by incorporating techniques to maintain consistency in relative compaction and material testing. This was accomplished by producing samples with relative compaction values within 2% of one another for samples loaded at one stress level. Dikova also eliminated the use of various styles of consolidometers, consequently creating a series of identical PLDs.

This series of tests incorporated a wide range of RAP and sand mixtures including 100, 80, 60, 40, 20 and 0% RAP. AASHTO classified A-3 sand similar to the material used by Cleary (2005) was mixed with the RAP. The vertical stresses of 6, 12 and 18 psi were applied to the entire top surface of the samples with the use of a load plate.

## 2.6 Rate Process Theory

### 2.6.1 Development of Rate Process Theory

Singh and Mitchell (1968) developed a simple phenomenological equation, describing the creep relationships between strain and time, as well as strain rate and time. The authors stated that this relationship was applicable to a reasonable range of stresses; covers a variety of soil types; accounts for both linear and nonlinear relationships between strain and logarithm of time; and includes easily determinable parameters.

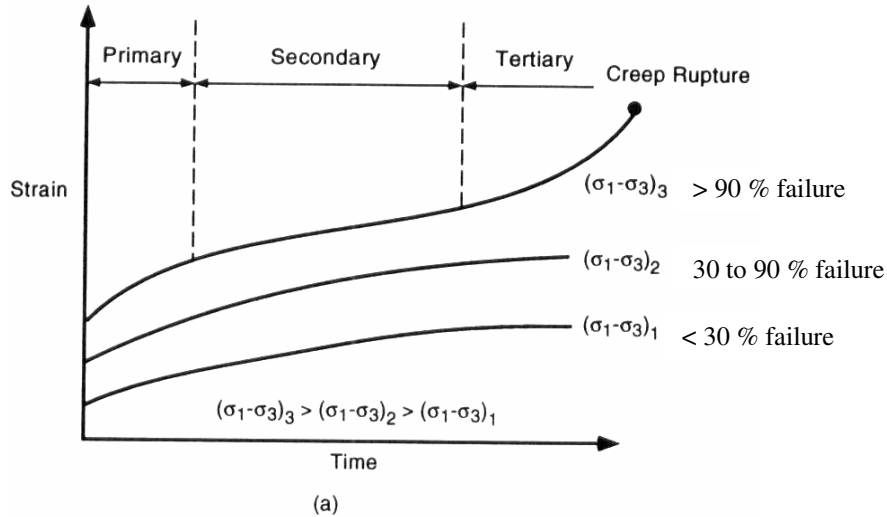
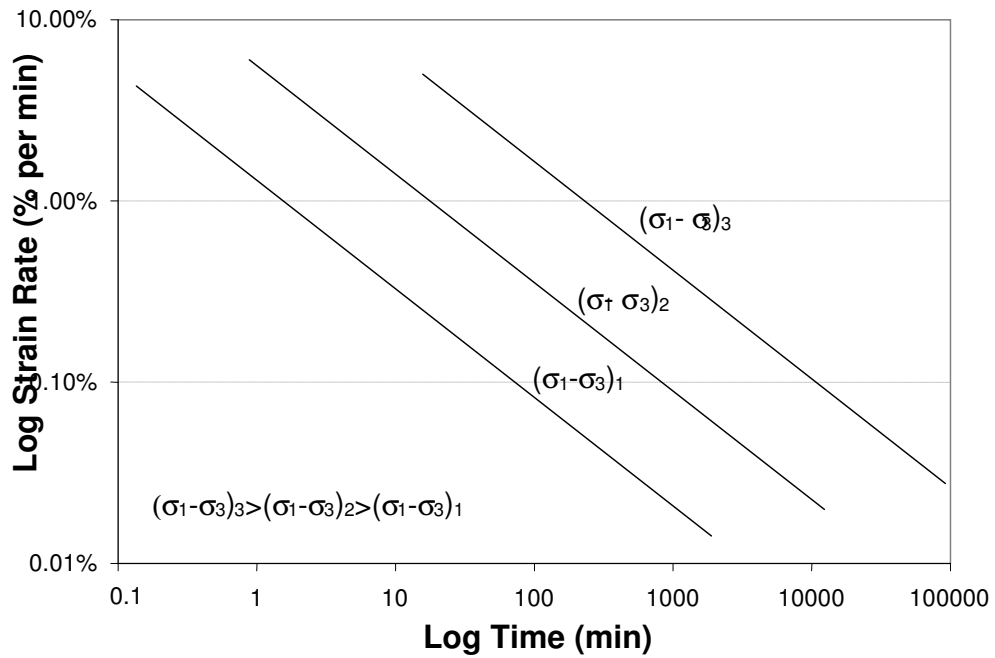


Figure 2.10 Creep under constant stress (Mitchell 1993)

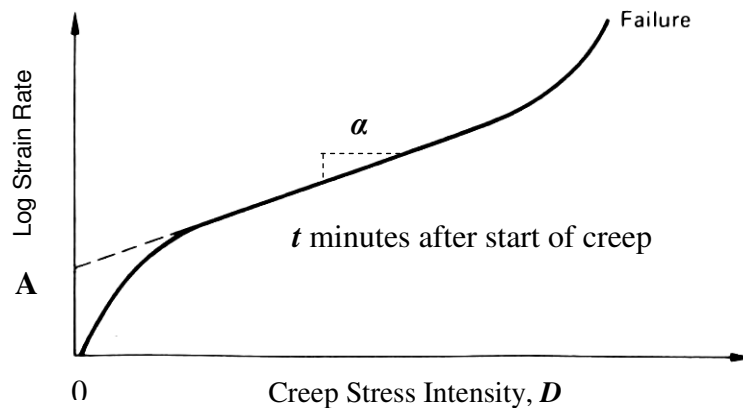
The general creep behavior of soils under a constant deviator stress ( $\sigma_1 - \sigma_3$ ) as per Mitchell (1993) is presented in Figure 2.10. The stress level was defined as the creep stress divided by the strength determined in a conventional strength test. The stress level affects the shape of the strain-time curves. At relatively low deviator stresses, from 0% to 30% of the soil strength according to Singh and Mitchell (1968), the creep deflections were small and prone to cease after some time period. This behavior is displayed by curve  $(\sigma_1 - \sigma_3)_1$  in Figure 2.10. Higher stress levels (i.e., up to 90% of the strength) resulted in increasing creep and prolonged or infinite creep movements as displayed by curve  $(\sigma_1 - \sigma_3)_2$ . The  $(\sigma_1 - \sigma_3)_3$  curve was obtained from stresses approaching the soil strength and sufficient to produce creep rupture (Figure 2.10).

Despite the differences in the shape of the strain versus time curves displayed on Figure 2.10, Mitchell (1993) discovered that if the same data was plotted on log of strain rate versus log of time graph, the resulting function was always linear, as shown in Figure 2.11. The line slopes proved to be independent of the creep stress intensity and the only effect from stress increase was a shift of the lines upwards and to the right.



**Figure 2.11 Linear relationship between log strain rate and log time**

A generalized representation of the relationship between creep stress intensity and creep rate at some given time after the stress application is presented in Figure 2.12. Singh and Mitchell (1968) associated the form of this relationship with a strain-rate variation dependent on a hyperbolic sine function of stress, as it could be predicted by the rate process theory. It was also found for a variety of soils and conditions, that within the midrange of stresses, a nearly linear function was formed between the log of strain rate and stress (Figure 2.12).



**Figure 2.12 Influence of creep stress intensity on the creep rate (Singh and Mitchell 1968)**

The concept of creep as a thermally activated rate process is discussed in detail by Mitchell, Campanella, and Singh (1968) and a thorough review of the concept is outside the scope of this research. Essentially, the rate process theory is based on the hypothesis that for material particles to move from one equilibrium position to another they need to overcome an energy barrier. The displacement of particles takes place when sufficient activation energy is acquired.

The relationship between strain rate and time as shown on a log-log plot like Figure 2.11 can be expressed by Equation 2.1 (Singh and Mitchell, 1968). The same data plotted in a semi-log form as shown in Figure 2.12 could be expressed by:

$$\ln \frac{\dot{\epsilon}}{\dot{\epsilon}(t_1, D)} = -m \ln \left( \frac{t_1}{t} \right) \quad \text{Equation 2.1}$$

where:  $\dot{\epsilon}$  - strain rate at any time  $t$ ;

$D$  - deviator stress  $(\sigma_1 - \sigma_3)$ ;

$t$  - any time;

$t_1$  - unit time;

$m$  - slope of logarithm strain versus logarithm time straight line.

$$\ln \left( \frac{\dot{\epsilon}}{\dot{\epsilon}(t, D_0)} \right) = \alpha D \quad \text{Equation 2.2}$$

where:  $\dot{\epsilon}_{(t, D_0)}$  - fictitious value of strain rate after start of creep at  $D=0$ ;

$\alpha$  - slope of the linear portion of the logarithm strain versus deviator-stress plot.

Eliminating  $\dot{\epsilon}$  from Equation 2.1 and Equation 2.2, and assuming the case of  $D=0$  produces a simple three-parameter relationship that adequately characterizes the creep rate of a variety of soils:

$$\dot{\epsilon} = A e^{\alpha D} \left( \frac{t_1}{t} \right)^m \quad \text{Equation 2.3}$$

where:  $A = \dot{\epsilon}_{(t_1, D_0)}$  - strain rate obtained by projecting the straight line portion of the relationship between log strain rate and deviator stress at unit time to a value of  $D=0$ .

For the development of Equation 2.3 by Singh and Mitchell (1968) the stress intensity  $D$  was taken as the deviator stress  $(\sigma_1 - \sigma_3)$  from triaxial creep test, but in a later study Mitchell (1993) stated that stress level could also be used, as would be the case with oedometer setup. In order to establish the parameters  $A$ ,  $\alpha$  and  $m$ , a



minimum of two creep tests are needed. Testing identical specimens at different creep stress intensities, a plot of log strain rate versus log time would yield the value of  $m$ , and a plot of log strain rate versus stress for different time intervals would produce  $\alpha$  and  $A$  from the slope and intercept, respectively. Equation 2.3 will be used further in this study for data processing and estimating the predicted strain rate of RAP and RAP-soil mixtures for selected long-term time intervals.

### **2.6.2 Other Research Based on Rate Process Theory**

Feda (1989) used experimental data from four types of soil to analyze distortional and volumetric creep on the basis of rate process theory. According to Feda, the physical processes characterizing the stages of creep were structural hardening through primary creep, structural softening through tertiary creep, and their balance resulting in secondary creep. Structural hardening was associated with volumetric creep, while structural softening was associated with distortional creep, and both were interpreted as an increase or decrease of the number of structural bonds per unit area. This researcher emphasized that due to these structural processes, secondary creep is a very short-lived or even absent stage of permanent deformation. In conclusion Feda (1989) stated that the geometric idea involved in the rate process theory correctly describes creep, however solely when obtained from uniaxial deformation test, and not for the conditions of multiaxial testing.

### **2.6.3 Application of Rate Process Theory for Particulate Materials**

Abyaneh et al. (2001) investigated the time-dependent creep behavior of particulate materials using the rate process approach examining the possible reasons for strain rate decrease with time. They dismissed the possibility of deformations occurring in the body of the particles and studied the processes taking place in the zones of interparticle contact adopting the assumption that the axial creep strain rate could be related to the rate of movement of one layer of particles over another in the contact zone by applying the rate process theory.

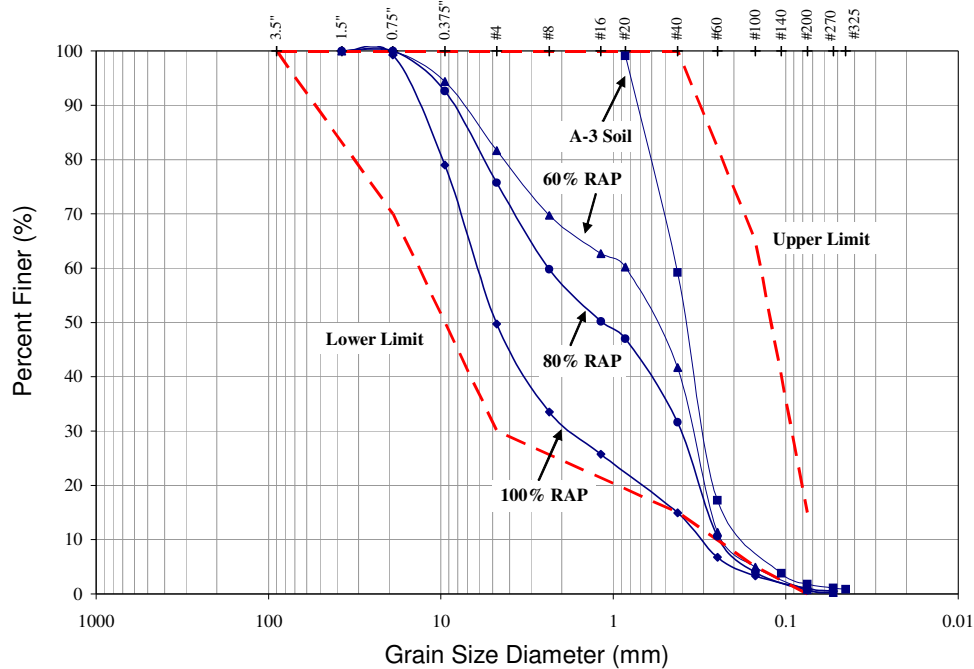
Creep tests were conducted on cylindrical plastic wrapped samples of lime-pulverized-fuel-ash mixtures, loaded axially in an oedometer at constant stress. The applied stress was calculated as a percentage of the failure load determined from unconfined compression test. Decrease in the strain rate with time was observed. Abyaneh et al. (2001) assumed that the decrease was mainly due to the dissipation of excess temperature from the interparticle contact zones heated during the loading stage to an above-ambient temperature. The researchers attempted to apply Newton's law of cooling to a typical particle contact and combined it with the creep strain rate equation. After comparing the obtained equation to their experimental results, the researchers concluded that it satisfactory modeled creep behavior only during the very early stages of a creep test. Furthermore, it was hypothesized that at extended times other time-dependent mechanisms, such as progressive increase in the volume of the contact zones due to interparticle merging, had a prevalent effect.

## **2.7 Evaluation of Creep in RAP**

Cleary (2005) conducted thesis research entitled “Long-Term Behavior of RAP-Soil Mixtures for Use as Backfill behind MSE Walls”. This work was used to investigate the creep behavior of 100% RAP and RAP/soil mixtures with proportions 80/20 and 60/40. The results were used to determine if the materials could be used as backfill behind mechanically stabilized earth walls. The soil used was poorly graded sand classified as A-3 according to AASHTO. It was chosen for its availability and it is accepted for used as MSE wall backfill by FDOT. The fundamental approach and testing methodology of Cleary’s (2005) work was evaluated and modified for the testing described in the current study.

### **2.7.1 Geotechnical Properties**

For the selection of RAP-Soil mix proportions Cleary (2005) used information obtained from dry rodded unit weight testing, starting with 100% RAP and incrementally adding A-3 sand with the intention to identify the mix that produced the highest unit weight. The resultant density versus RAP-Soil mix percentage curve showed that the peak unit weight of 103.5 lb/ft<sup>3</sup> was obtained at RAP content of 80%. To target a broader data range, Cleary (2005) decided to include 100% RAP and mixtures of 80 and 60% RAP. The RAP used was obtained from the APAC Melbourne facility and processed by hammermill crushing. With addition of sand the gradation associated with the 80% and 60% mixes became finer. According to AASHTO, RAP and the 80% mix classified as A-1-a, while the 60% mix classified as A-1-b. All four curves, with the exception of the lowest portion of the 100% RAP curve, produced data well within the specified limits for gradation of MSE backfill, as presented in Figure 2.13.

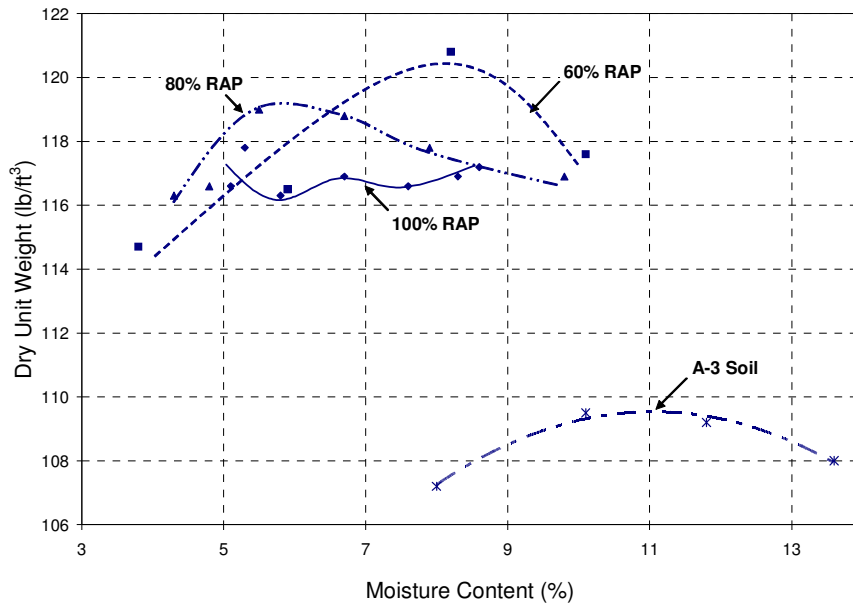


**Figure 2.13 Comparison of grain-size distributions of the RAP mixtures and A-3 soil with MSE backfill gradation criteria (Cleary 2005)**

The moisture-density testing produced four moderately flat curves (Figure 2.14). The 100% RAP displayed a wavy pattern with an unpronounced peak, indicating that the material's density was not sensitive to changes in moisture content. The highest dry unit weight was obtained from the 60% mix.

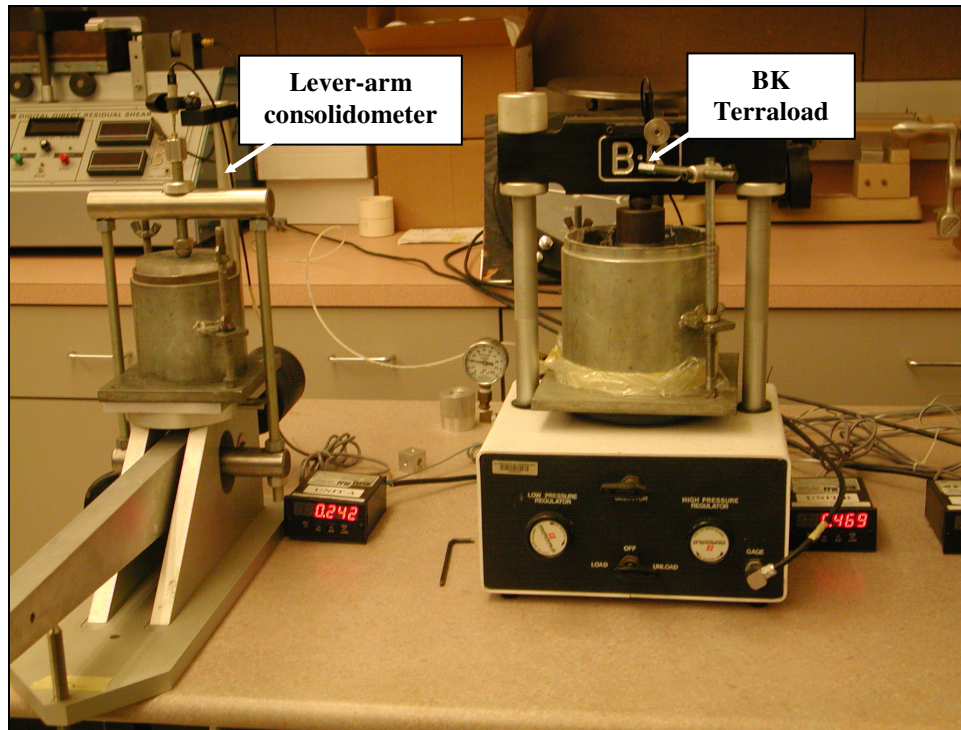
### 2.7.2 Creep Testing

Cleary (2005) adapted three testing units for creep tests, loading three samples of the same mixture simultaneously. Two of the units were lever-arm consolidation loading frames modified to accommodate 4-inch Proctor molds. The third unit was a pneumatically activated Brainard-Kilman Terraload (BK Terraload) consolidation unit using a 6-inch Proctor mold. Both types of units are shown in Figure 2.15. The consolidometer units were labeled Units A and C and the pneumatic unit was labeled Unit B. Rigid loading plates were placed on the sample surface to evenly distribute the load and simulate one-dimensional deformation.



**Figure 2.14 Moisture-density curves for 100%, 80%, 60% RAP-soil mixtures and A-3 soil (Cleary 2005)**

Three stress levels; 6 psi, 12 psi and 24 psi, were applied in succession on the same sample. These stress levels were estimated from evaluation of typical MSE walls 10 to 30 ft. high backfilled with a 115 lb/ft<sup>3</sup> dry unit weight material. Each stress level was held constant for approximately 5 days. The data acquisition (DAQ) system for each unit consisted of a potentiometer to measure displacement, and a digital readout. A computer with a serial port and a LabView<sup>®</sup> software module was used to collect and store data in a text file from each unit. The initial reading was recorded at one second and all subsequent time increments were doubled up to four hours, after which data was collected every four hours until test completion.

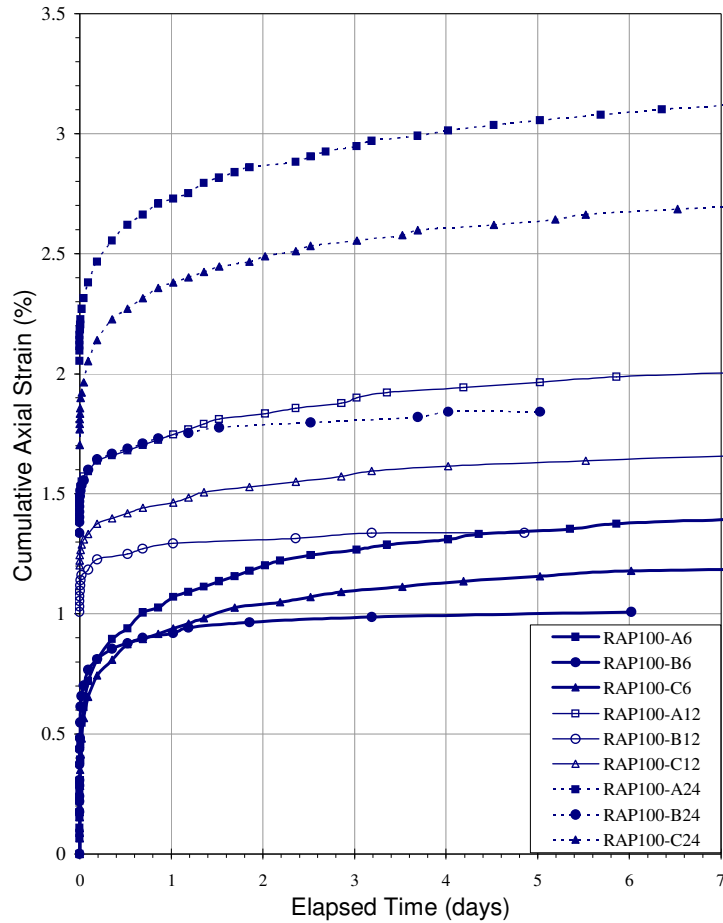


**Figure 2.15 Photo of the two types of creep testing units (Cleary 2005)**

### **2.7.3 Data Analysis**

Sample deflections for each stress level were converted to cumulative axial strain and then plotted versus time. The curves obtained from testing 100% RAP from each of the three units are shown in Figure 2.16. Axial strain increased rapidly during the first day, but the slopes decreased with time. For all units the increase in stress level from 6 to 12 to 24 psi produced larger deformations.

The cumulative axial strain (Figure 2.16) shows inconsistencies between the three units. Results from Unit B were expected to deviate from the others, because of the larger 6-inch diameter samples. Units A and C, however, both used identical 4-inch diameter samples and similar lever-arm consolidometer. The cumulative strains from Unit A are consistently higher than those from Unit C, while the cumulative strains from B level off at earlier times than either A or C. Based on this data it was concluded that the variations were most likely equipment related (Cleary, 2005).



**Figure 2.16 Cumulative axial strain of 100% RAP as a function of time from units A, B, C at stress levels of 6, 12 and 24 psi (Cleary 2005)**

### 2.7.4 Estimated Settlements of Typical MSE Walls

In order to estimate a long-term settlement in a RAP-soil fill, Cleary (2005) calculated the strain rates at 12 preselected times up to just over 100 hours for each mix. Based on the relationships developed by Singh and Mitchell (1968), the logarithm of strain rate was plotted versus logarithm of time. All data was modeled with the general equation:

$$F = at^{-b} \quad \text{Equation 2.4}$$

Where: F is the strain rate (%/hr)  
a is the constant associated with a strain rate of 1%/hr  
t is the desired time (hrs)  
b is the exponent or the slope of the log-log plot.

Review of Cleary's (2005) data indicates regression coefficients on the log-log plots for the 100% RAP and the two RAP-Soil mixes were nearly 1.000. It was therefore concluded that the Singh and Mitchell (1968) model could be applied to RAP and RAP-Soil mixes. Using Equation 2.4, the 50-year settlements were estimated for fill behind both a hypothetical 15- or 30-ft. high wall with mixtures of 100%, 80% and 60% RAP. The numbers generated exclude the immediate settlement and therefore are based on the creep movements. The data was averaged for the three units with one statistical outlier associated with Unit B at 100% RAP, was excluded. The standard deviations were used to determine minimum and maximum settlements for both wall heights and the results are summarized in Table 2.2.

**Table 2.2 Minimum and maximum 50 year settlement predictions for RAP and RAP-A-3 sand blends (Cleary 2005)**

RAP Percentage	15 ft. wall settlement (inches)		30 ft. wall settlement (inches)	
	Minimum	Maximum	Minimum	Maximum
<b>100</b>	5.9	8.8	9.4	30.3
<b>80</b>	1.0	2.3	3.1	6.5
<b>60</b>	0.8	2.5	2.3	3.4

The results indicate that the estimated creep settlements would decrease significantly with the addition of 20 or 40% A-3 sand. They also indicate that 100% RAP creep settlements would not be tolerable for either wall height, and that the creep movements associated with the 80% and 60% blends would be excessive.

### **2.7.5 Cleary (2005) Conclusions and Recommendations**

Cleary (2005) concluded that the creep testing and analysis methodology provided a reliable basis for long-term testing of RAP behavior. The 100% RAP was not recommended for use as fill behind walls due to the excessive settlements predicted from the creep testing results. The greatest relative improvement of creep behavior was observed for 80% RAP mix, which deformed 70% less than the 100% RAP.

Cleary (2005) recommended improving the testing methodology by using a new sample for each load increment, and by using a seating load before applying the actual stress levels. The applied pressure increments calculated for fills of up to 30 ft. height were considered excessive for the current construction practices and resulted in a reassessment of the applied pressures during future testing. Finally, Cleary (2005) suggested that tolerances for sample compaction control should be applied.

## **2.8 Mechanically Stabilized Earth (MSE) Walls**

MSE walls technology, pioneered by Henri Vidal in the early 1960s, has been in use for over 30 years. As the technology developed, the system evolved to include various materials and wall designs. These walls have become increasingly popular in metropolitan areas where there are space limitations, especially along transportation corridors.

MSE walls have many advantages over gravity or cantilever cast-in-place concrete walls, including faster and more efficient construction plus a more economical system which includes the use of pre-cast concrete panels or modular blocks. In some cases existing onsite backfill may be used in construction. These walls also require less site preparation and less space in front of the structure for construction operations. Their structural flexibility allows significantly higher total and differential settlement compared to cast-in-place concrete walls (Elias et al. 2001).

Many materials such as concrete wall facing panels, soil reinforcement and granular backfill are combined to form MSE wall. Wall facing panels are used to hold the soil in position at the face of the wall. Soil reinforcement may be either extensible, polymeric reinforcements, or inextensible, metallic reinforcement. The wall/reinforcing connection is where the connection between the reinforcing strips and the wall facing is made. A geotextile filter fabric is placed behind the face panels to cover the joints between face panels. This prevents the soil from being eroded through the joints and allows excess water to flow out from behind the wall. Select backfill is fill that meets gradation, corrosion, unit weight, internal friction angle and any other requirements of the specifications.

### **2.8.1 Soil Reinforcement**

Soil reinforcement has two benefits when used as a part of MSE walls; it increases the soil's tensile strength, and develops shear resistance from the friction between the soil-reinforcement interaction boundaries (Das 2004). Reinforcement may consist of galvanized metal strips, geotextiles or geogrids. These materials fall into one of the two categories; extensible or inextensible reinforcement.

Extensible reinforcement includes non-biodegradable fabrics and geogrids. These materials exhibit creep under constant stress. Non-biodegradable fabrics or geotextiles are typically made from petroleum products or fiberglass. Geogrids are high modulus polymer reinforcement materials that have openings, or apertures, and allow the surrounding soil to interlock.

Inextensible reinforcements such as steel wire mesh or metallic strips, which do not exhibit creep under stress, are galvanized to reduce the rate of corrosion. Ribbed galvanized steel reinforcing strips with dimensions of 2 inch wide by 0.16 inch thick are commonly used inextensible soil reinforcements. The embedment length of the reinforcement should be at least 70% of the total face panel height or 8 ft., whichever is greater.



## 2.8.2 MSE Wall Backfill

Most MSE walls are designed with free draining granular soil for use as backfill. Section 548 of the FDOT Standard Specifications for Road and Bridge Construction (2008) require that all backfill used in the retaining wall is free draining and cannot have more than 2.0% by weight of organic material. The material must be non-plastic and the liquid limit must be less than 15. If reinforcement is to be used in the construction of the MSE wall, the soil should meet the gradation limits shown in Table 2.3.

**Table 2.3 Gradation limits for soil used in MSE walls with reinforcement (FDOT 2008)**

Sieve Size		Percent Passing
3 1/2 inches	90 mm	100
3/4 inches	19.0 mm	70 - 100
No. 4	4.75 mm	30 - 100
No. 40	425 mm	15 - 100
No. 100	150 mm	0 - 65
No. 200	75 mm	0 - 12

## 2.8.3 Ultimate Pullout Resistance of MSE Soil Reinforcement

According to Christopher et al. (1990), three criteria should be evaluated when designing soil reinforcement systems for long-term pullout performance; a) pullout capacity, b) allowable displacement and c) long-term displacement. To determine pullout capacity, one or a combination of two soil – reinforcement interactions provide pullout resistance of the soil reinforcement. First, interface friction occurs when stresses are transferred between the soil and reinforcement. Second, passive resistance develops when soil is pushed into the reinforcement ribs during pullout. The interaction of friction and passive resistance can be determined for different soil reinforcements and backfills by the friction factor equation:

$$F^* = F_q \cdot \alpha_\beta + K \cdot \mu^* \cdot \alpha_f \quad \text{Equation 2.5}$$

where:  $F_q$  = the embedment, or surcharge, bearing capacity factor

$\alpha_\beta$  = the structural geometric factor for passive resistance

$K$  = the ratio of normal to effective vertical stress; influenced by the reinforcement geometry

$\mu^*$  = apparent friction coefficient for reinforcement

$\alpha_f$  = structural geometric factor for frictional resistance

The pullout capacity design parameters  $F_q$ ,  $\alpha_\beta$ ,  $K$  and  $\alpha_f$  are all equal to 1 for inextensible ribbed strips; therefore  $F^* = \mu^*$  (Christopher 1990).

The ultimate strip pullout resistance is used to determine  $F^*$ . This force is evaluated by the following standardized expression presented by both Christopher et al. (1990) and Elias et al. (2001):

$$P_r = F^* \cdot \alpha \cdot \sigma_v' \cdot L_e \cdot C \quad \text{Equation 2.6}$$

where:  $P_r$  = ultimate pullout resistance per unit width of reinforcement  
 $F^*$  = pullout resistance factor or friction-bearing factor  
 $\alpha$  = correction factor which accounts for softening effects for extensible reinforcements ( $\alpha = 1$  in metallic reinforcement)  
 $\sigma_v'$  = effective vertical stress at the soil reinforcement interface  
 $L_e$  = length of reinforcement in the resisting zone behind the failure surface  
 $C$  = effective unit perimeter of the reinforcement ( $C = 2$  for shear resistance developed on the top and bottom of the strip)

$F^*$ ,  $\alpha$ , and  $C$  are parameters analyzed for the specified soil and reinforcement in order to calculate  $P_r$ . The pullout resistance factor  $F^*$  can be measured experimentally from laboratory or field pullout tests with the backfill that is to be used in the field (Christopher et al. 1990 and Elias et al. 2001).  $F^*$  can be determined as the force per strip by re-writing Equation 2.6 as:

$$F^* = \frac{\left( \frac{P_0}{W} \right)}{(\alpha \cdot \sigma_v' \cdot L_e \cdot C)} \quad \text{Equation 2.7}$$

where:  $W$  is the reinforcement width (2 inches)  
 $\alpha = 1$ ,  $\sigma_v'$  is the vertical effective stress used during testing  
 $L_e$  = embedded reinforced length  
 $C = 2$  for metallic reinforcement.

In the absence of pullout tests, semi-empirical relationships can be used conservatively to determine  $F^*$  (Elias et al. 2001). The relationships produce a minimum design envelope for  $F^*$  versus depth for a particular backfill according to the following:

$$F^* = \tan \rho = 1.2 + \log C_u \quad \text{at top of the structure } \leq 2.0 \text{ maximum} \quad \text{Equation 2.8}$$

$$F^* = \tan \Phi \quad \text{at depth of 20 ft. and below} \quad \text{Equation 2.9}$$

where:  $C_u$  is the uniformity coefficient  
 $\rho$  is the interface friction angle mobilized along the reinforcement  
 $\phi$  is the soil friction angle.

## 2.9 Reclaimed Asphalt Pavement

Previous laboratory creep tests performed by Cleary (2005) and Dikova (2006) examined many proportions of RAP-sand mixtures. Cleary (2005) used 100% RAP, 80% RAP to 20% sand, 60% RAP to 40% sand and 100% A-3 sand. Dikova (2006) had mixtures ranging from 100% RAP to 100% A-3 sand where the mixture proportions of RAP and sand would change by 20% as more A-3 was added and RAP removed.

A summary of gradation parameters from grain size distribution analysis on RAP conducted by Gomez (2003), Cleary (2005) and Dikova (2006) are compared to Rathje et al (2006) in Table 2.4.

The RAP from Rathje et al (2006) is not from the APAC Melbourne facility while the others are from various stockpiles over several years from this one location. In general, there is a difference in the gradation associated with the Rathje et al (2006) material which results in a coarser material.

**Table 2.4 Gradation parameters from sieve analysis of 100% RAP**

	<b>100% RAP Gomez (2003)</b>	<b>100% RAP Cleary (2005)</b>	<b>100% RAP Dikova (2006)</b>	<b>100% RAP Rathje et al (2006)</b>
D <sub>10</sub> (mm)	0.27	0.31	0.54	1.20
D <sub>30</sub> (mm)	0.65	1.80	2.50	4.20
D <sub>60</sub> (mm)	4.70	6.00	7.00	9.60
Cu	17.4	19.4	13.0	1.5
Cc	0.3	1.7	1.7	8.0
<b>USCS Classification</b>	<b>SP</b>	<b>GW/SW</b>	<b>GW</b>	<b>GW</b>

### **2.9.1 Pullout Testing on Inextensible Reinforcement**

The following section summarizes large scale pullout tests performed on inextensible, ribbed steel soil reinforcement embedded in RAP and sand. Studies from the Center for Transportation Research at the University of Texas at Austin (Rathje et al. 2006), FDOT State Materials Office by Lai and Moore (1990) and The Reinforced Earth Company (RECO) (2004) will be discussed and compared. The pullout tests in these studies were performed using similar testing methodologies and identical steel soil reinforcement strips. The results from the test yielded field measurements of friction factors,  $F^*$  for a material as a result of varying vertical stresses. The  $F^*$  values were plotted and compared to the design envelope for MSE wall construction backfill material.

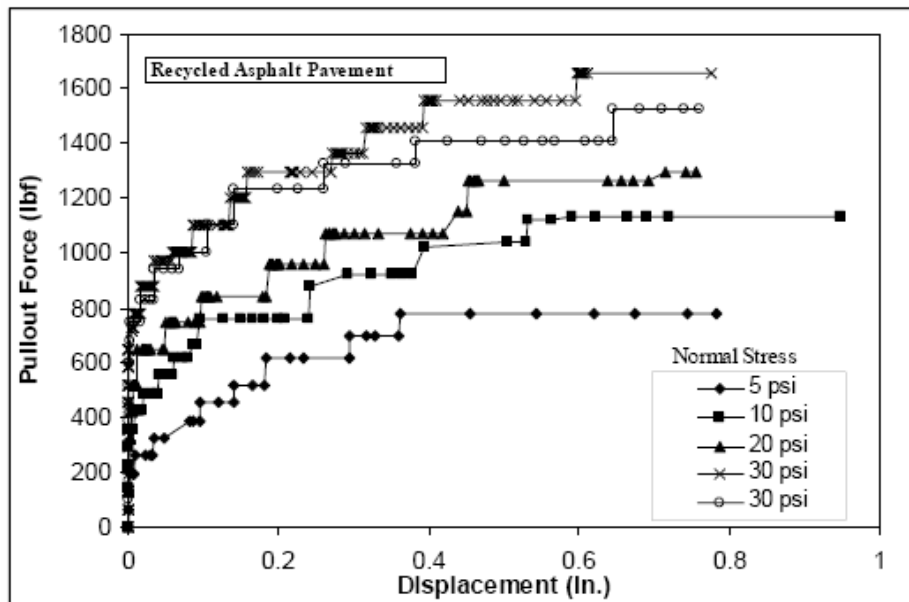
### **2.9.2 Evaluation of Pullout Resistance for Ottawa Sand and RAP**

Rathje et al. (2006) investigated the pullout resistance of ribbed galvanized steel soil reinforcement in Ottawa Sand, RAP, crushed concrete (CC) and conventional fill material (CFM). Only the pullout tests for the Ottawa Sand and RAP will be presented. The Ottawa Sand was classified as poorly graded sand (SP) by the USCS with all particles between the No. 100 and No. 16 sieves and had a target density of 103 pcf at a moisture content of approximately 3.0%. The RAP backfill had an average density of 118.2 pcf at a moisture content of approximately 3.0% and was classified as well-graded gravel (GW) by USCS.

A large shear box (20 inches by 20 inches by 10.5 inches tall) was used to pull the 18 inch reinforcing strips out of the backfill. The strips were pulled by a pneumatic piston and a proving ring measured the pullout force. The reinforcing

strip was embedded at mid-height of the backfill and an air bladder assembly resting on top of the material applied a normal stress. Normal pressures of 5, 10, 20 and 30 psi were applied to the RAP backfill while 5, 11 and 17 psi were applied to the Ottawa Sand and reinforcing strip in the shear box. The normal stresses applied to the material consolidated the backfill and vertical strains of 1 to 2% were observed in RAP. After the material was prepared and loaded under normal pressure, the reinforcement was horizontally pulled out of the backfill with incremental increases in the pullout force. Failure was defined at a displacement of 0.75 inch as recommended by Elias et al. (2001). The pullout force was increased in increments of 60 to 100 lbs for the tests. Force increments were only increased when the displacement from the previous force had stopped.

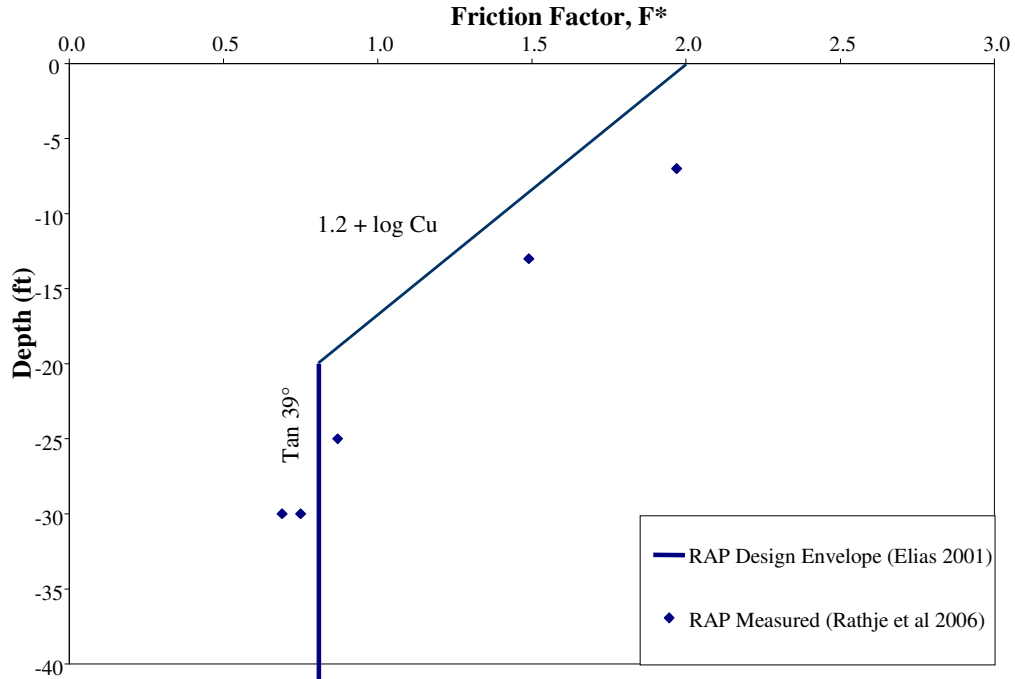
Figure 2.17 displays data between the pullout tension forces on the strip versus the strip displacement at various normal confining stresses for RAP. Strip pullout loads were increased in increments of 60 to 100 lbs after the strip stopped displacing under the previous load. As the normal stress increased the ultimate pullout force also increased. At each normal stress, strip displacement in the RAP increased at a pullout force equal to approximately 30 to 50% of the ultimate pullout force.



**Figure 2.17 Results from five pullout tests on ribbed metallic strip in RAP (Rathje et al 2006)**

Based on these results, the interaction between the reinforcement and the RAP produced large creep displacements even at lower vertical confining stresses. Although not shown in the figure, RAP yielded a higher ultimate pullout resistance than the Ottawa Sand at all comparable overburden pressures.

Figure 2.18 displays the back calculated  $F^*$  data along with the relationship developed by Rathje et al. (2006) for the RAP backfill. Five pullout tests with increasing overburden pressures were performed. The ultimate pullout force from each pullout test was used to calculate  $F^*$ . Lower confining pressures produced slightly larger  $F^*$  values than the design envelope based on Equation 5 and Equation 6; however, at larger depths the measured  $F^*$  values were slightly lower than the design envelope values. According to Rathje et al. (2006) the lower friction angles can be attributed to creep deformations of RAP during the pullout tests.



**Figure 2.18 Measured versus semi-empirical friction factors for RAP (Rathje et al 2006)**

Ottawa sand was used to verify the test results. A study performed by Law Engineering and Environmental Services, Inc. (1998) conducted pullout testing on Ottawa sand using a larger pullout box than the box used by Rathje et al (2006). To confirm the validity of the results from the pullout tests, Rathje et al (2006) conducted pullout tests on Ottawa sand at the same confining pressures used by Law Engineering and Environmental Services, Inc. Figure 2.19 presents the pullout resistance factors versus the equivalent overburden depth for Ottawa Sand. The calculated  $F^*$  from the pullout test performed by Rathje et al. (2006) were very similar to Law Engineering and Environmental Services, Inc. (1998) and were greater than those predicted by semi-empirical Equation 5 and Equation 6 from Christopher et al. (1990).

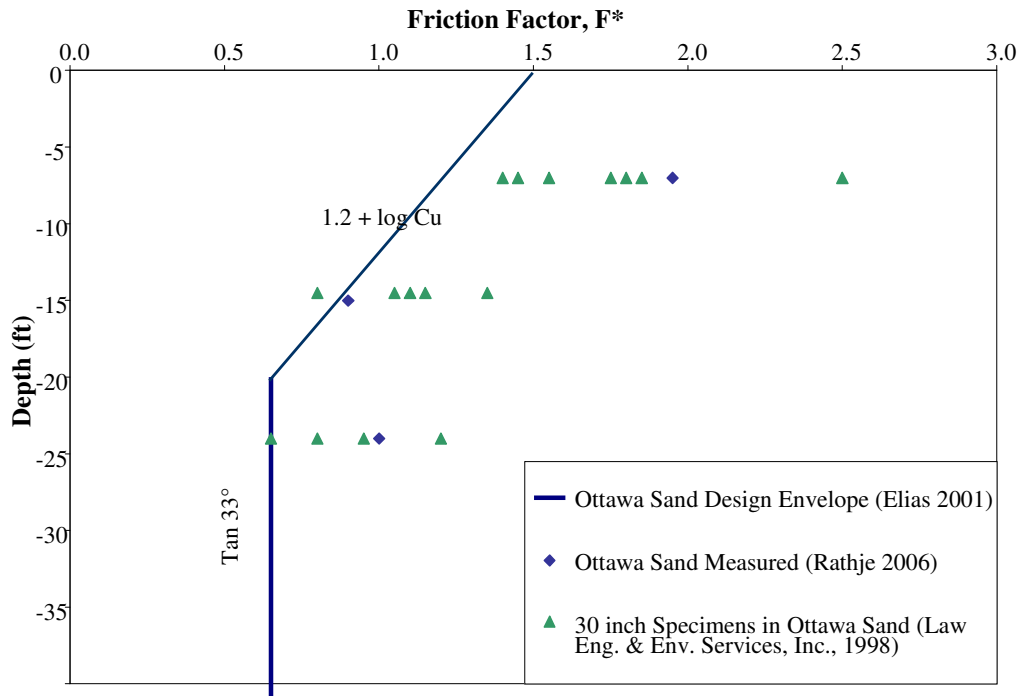


Figure 2.19 Measured friction factors for Ottawa Sand compared to semi-empirical equations (Rathje et al 2006)

### 2.9.2.1 Pullout Testing of Inextensible Reinforcement in A-3 Sand

Lai and Moore (1990) evaluated four 300 ft. long by approximately 25 ft. high instrumented MSE walls built by the FDOT during construction. Two VSL Inc. (steel mesh) walls, constructed in Pinellas Park, FL, and two Reinforced Earth Company (RECO) (steel strips), constructed in Fort Lauderdale, FL, were evaluated.

Laboratory pullout testing was conducted on the VSL and the RECO strips to identify backfill properties and reinforcing pullout resistance with and without vibration. Only the laboratory pullout tests results conducted without vibration on the RECO galvanized steel strips will be presented due to inconsistencies with the data. Backfill for the RECO strip pullout tests had the following properties; classified as A-3 by AASHTO and SP by USCS; maximum modified Proctor of 104 pcf at 10% moisture, a LBR of 19; and a maximum friction angle of 36.9 degrees.

Pullout testing was conducted on RECO steel galvanized strips inside a 6 ft. deep by 8 ft. wide by 24 ft. long test pit located at the FDOT State Materials Office in Gainesville, FL. A 1.67 ft. deep by 8 ft. wide by 3 ft. long box was constructed inside the test pit. The backfill was compacted to approximately 100% modified Proctor maximum density inside the box and the metallic reinforcing strips were embedded at mid-depth (10 inch) in the compacted backfill. Materials Testing

System (MTS) hydraulic loading equipment was used to apply the normal stress to the backfill. This system was capable of applying a 20,000 lb vertical load. A turnbuckle was used to create a pullout tension force on the strip and a 20 kip BLH tension load cell was used to measure the applied force. Dial gauges mounted in the front and back of the strip were used to measure strip displacements during pulling.

Five pullout tests were conducted with the RECO strips. The simulated overburden height was increased for each test with values of 4.0, 6.8, 9.6, 12.4, and 15.2 ft. The strips were pulled until a displacement of approximately 0.25 inch was achieved or when the ultimate pullout load was reached. Figure 2.20 displays the pullout test results.

Lai and Moore (1990) reported that the pullout force increases as the equivalent overburden is increased. The initial displacement of the strip decreases at greater overburden heights, as observed by the initial slope of each curve. Apparent coefficient of friction,  $\mu^*$  values were determined and plotted versus depth in Figure 2.21. The plot shows that  $\mu^*$  values from the pullout tests meet the semi-empirical design criteria for pullout resistance of steel ribbed reinforcement.

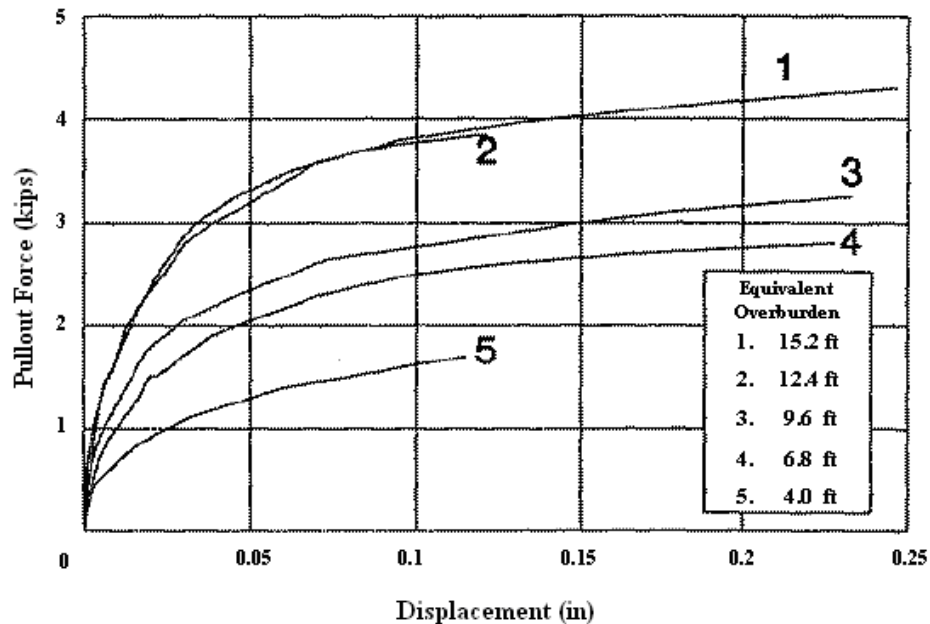
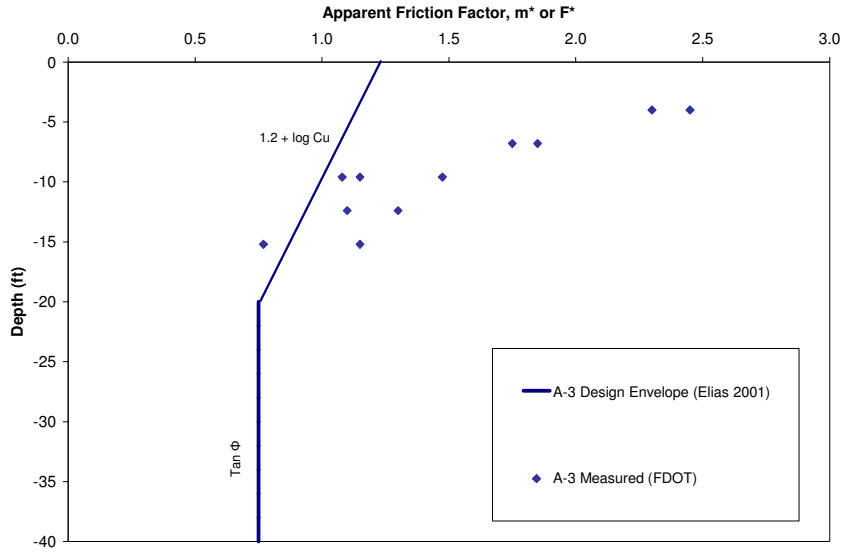


Figure 2.20 Pullout test results for RECO strips (Lai and Moore 1990)



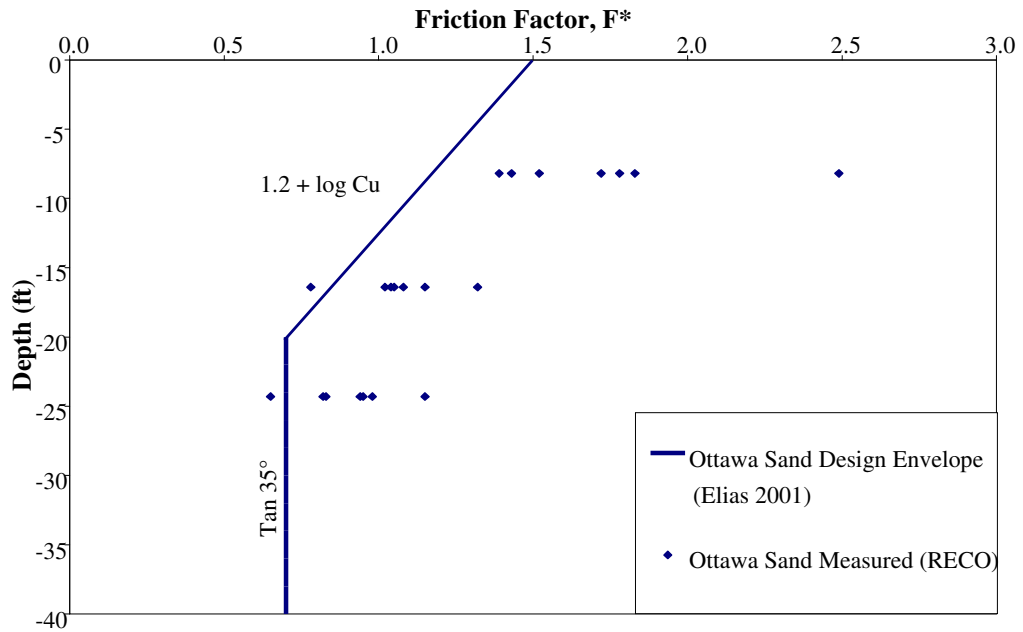
**Figure 2.21 Apparent coefficient of friction versus depth (Lai and Moore 1990)**

### **2.9.2.2 Pullout Testing of Inextensible Reinforcement in Ottawa Sand**

A series of pullout tests were performed by the Reinforced Earth Company to establish the coefficient of friction of high adherence (HA) ladders and steel ribbed strips. Only the pullout results for the ribbed steel reinforcement embedded in Ottawa sand will be discussed as their results most closely relate to this research. The strips were pulled from a 3.0 ft. wide by 3.0 ft. long by 2 ft. deep box embedded at mid height in Ottawa sand. Overburden pressures of 5.8, 11.6 and 17.4 psi, which represented overburden heights of 8.2, 16.4 and 24.3 ft respectively, were applied to the Ottawa sand.

The reinforcing strips were pullout until a total displacement of 0.5 inch was achieved and the pullout resistance force was recorded.  $F^*$  was back calculated from the data for each pullout test. Figure 2.22 presents the  $F^*$  versus the overburden height for the strips embedded in Ottawa sand. In general, the majority of results produced friction factors that would be acceptable





**Figure 2.22 Coefficient of friction,  $F^*$  versus overburden height for reinforcing strips embedded in Ottawa Sand (RECO)**

### 2.9.2.3 Summary of Pullout Test Result

Figure 2.23 presents a summary of the pullout test results performed on the backfill materials mentioned in the previous sections. The Ottawa sands and AASHTO classified A-3 sands used in the studies have similar characteristics; therefore, the results can be compared to one another based on the friction factors calculated from each overburden height. The friction factors for the pullout tests performed by Rathje (2001) on RAP are also presented. In general, it was concluded that friction factors obtained from RAP would not meet the semi-empirical guidelines proposed by Elias et al (2001).

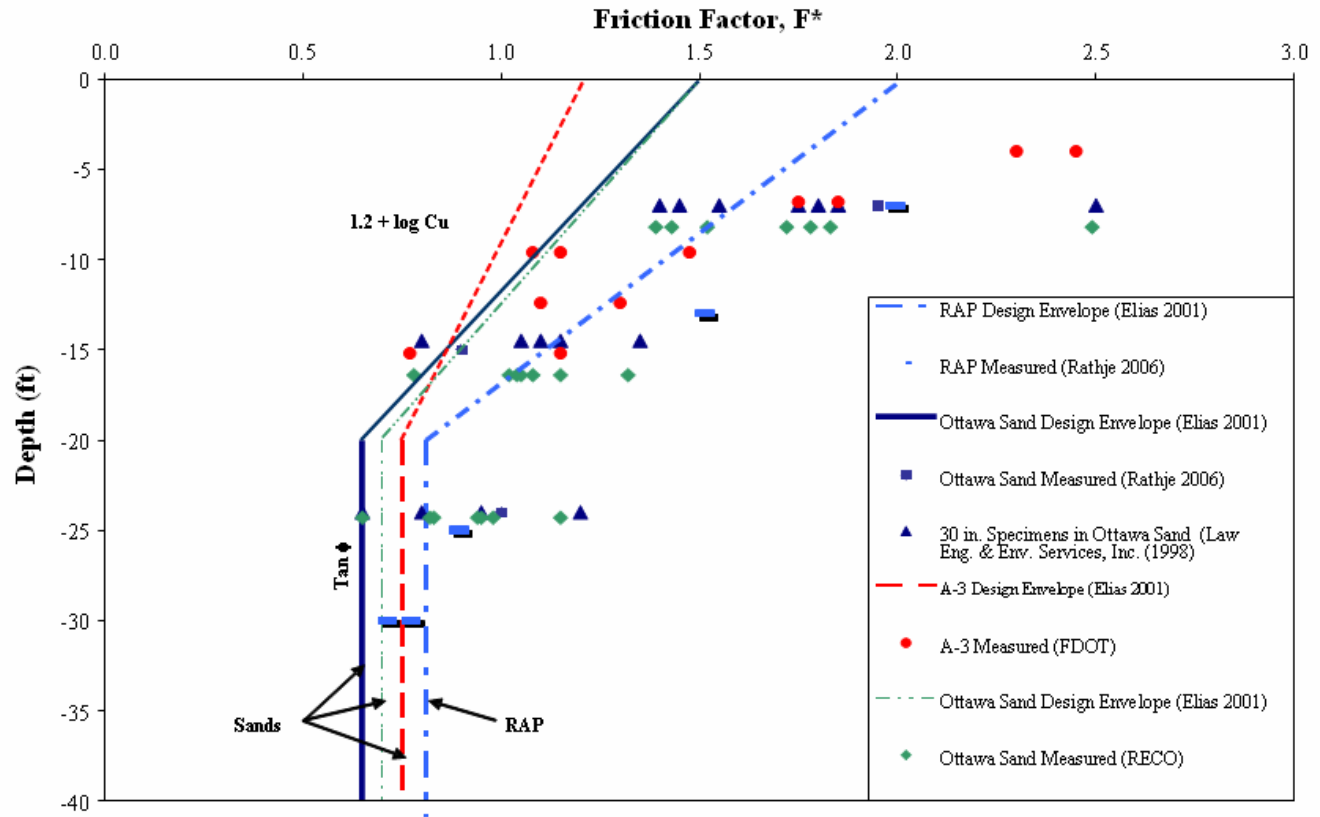


Figure 2.23 Summary of friction factors from pullout tests

### 3 Methodology and Test Procedures

The testing program consisted of three components, an extensive laboratory creep testing program, which led to a large scale creep evaluation and finally a separate state-wide evaluation of the variability of RAP's basic engineering properties. The fundamental engineering properties of all materials used during the study were evaluated.

#### 3.1 Fundamental Engineering Evaluation

FDOT Manual of Florida Sampling and Testing Methods (FM) (1994) and ASTM standards were used to determine the fundamental engineering properties of all RAP, A-3 soil and RAP-soil mixtures evaluated. A summary of the applied laboratory testing procedures is presented in Table 3.1. After samples were obtained, they were stored and quartered to produce representative test samples. These representative samples were then subjected to grain size, dry rodded unit weight, moisture-density, asphalt content and LBR testing. The asphalt viscosity and the gradation of the extracted aggregates were determined on select samples.

**Table 3.1 Summary of applied laboratory tests and procedures**

Laboratory Test	Procedure	Description
Grain Size Analysis	FM 1-T 027 / AASHTO T27	Sieve Analysis of Fine and Coarse Aggregate
Sampling	FM 1-T 002	Sampling Coarse and Fine Aggregate
Quartering	FM 1-T 248	Reducing Field Samples of Aggregate to Testing Size
Dry Rodded Unit Weight	FM 1-T 019	Standard Test Method for Unit Weight and Voids in Aggregate
Moisture Density	FM 5-521 [FM 1-T 180] / AASHTO T 180-74, Method D	Moisture Density Relations of Soils Using 10-lb. Rammer and 18-inch Drop
LBR	FM 5-515	Limerock Bearing Ratio
Asphalt Content	FM 5-563	Quantitative Determination of Asphalt Content from Asphalt Paving Mixtures by the Ignition Method
Viscosity	FM 1-T 202 / AASHTO T202	Absolute Viscosity of Asphalts
Gradation of Extracted Aggregate	FM 1-T 027 / AASHTO T27	Sieve Analysis of Fine and Coarse Aggregate

### **3.1.1 Grain-size Analysis**

The grain-size distributions of RAP, A-3 and RAP-soil mixtures were determined according to FM 1-T 027, "Sieve Analysis of Fine and Coarse Aggregate" by dry sieving. For the RAP and RAP-soil mixtures U.S. standard sieve sizes of either 2.0 or 1.5 inch, 1 inch, 0.75 inch, 0.5 inch, 0.375 inch, 0.25 inch, #4, #8, #10, #16, #20, #30, #40, #50, #60, #100, #140, and #200 agitated by a mechanical apparatus were used. The large number of sieves allowed finer evaluation of material grain sizes.

A total weight of about 10 lbs of the air-dried RAP was tested for each mixture. It was split into 3 or 4 batches of smaller mass to enable multiple sets of data to be obtained from each sample. The individual batches weighed approximately 3 lbs each and were used to limit the quantity of material on a given sieve according to Section 7.3 of FM 1-T 027. The amounts retained on each sieve for the individual batches were summed to produce the total retained for the entire 10-lb sample.

The sieve analysis of the A-3 soil followed the same procedure with an arrangement of sieves comprising 0.25 inch, #4, #8, #10, #16, #20, #30, #40, #50, #100, and #200. The individual batches consisted of 1 to 2 lbs mass of material.

### **3.1.2 Dry Rodded Unit Weight**

The dry rodded unit weight test on RAP, A-3 and RAP-soil mixtures was performed following FM 1-T 019. The procedure involved rodding of loose aggregate with 25 blows per layers as a measure of known volume is filled with the aggregate in three layers. For the rodding a straight 24 inch long, 5/8 inch diameter steel rod with a hemispherical tip was used. The measure was a compaction mold with 6 inch diameter and 9 inch height, resulting in capacity of approximately  $1/7 \text{ ft}^3$ .

The test was conducted starting with 100% RAP. Calculated quantities of A-3 soil were premixed with the initial 100% RAP sample, each material alteration was followed by rodding, unit weight calculation and remixing of the sample. Thus all RAP-soil mixtures with proportions varying from 100% RAP to 40% RAP using 5% increments were tested and the relationship between RAP content and dry rodded unit weight was plotted.

While performing the test it was observed that it was very sensitive to operational errors. Therefore, particular care was taken to ensure that the entire test was completed by a single operator and conducted with consistent methodology. A minimum of two repetitions per increment were performed and the result was considered acceptable, if they differed by less than  $2.5 \text{ lb/ft}^3$ .

### **3.1.3 Moisture-Density**

The modified Proctor compaction test was conducted on RAP, A-3 and RAP-soil mixtures according to FM 5-521, “Moisture Density Relations of Soils Using 10-lb. Rammer and an 18-inch Drop.” Material was quartered and size modified. Portions of size-modified RAP and RAP-soil mixtures were placed into bins and thoroughly mixed with water to achieve target moisture contents of 4%, 6%, 7%, 8%, and 10%. The target moisture contents used for A-3 were 8%, 9%, 10%, 11%, and 12%. The bins were sealed and the material was allowed to hydrate overnight.

Immediately before compacting, the material was remixed. Samples were compacted in cylindrical 6-inch diameter, 4.59-inch height molds in 5 equal layers with 56 blows per layer. The compactive effort exerted during modified Proctor test was 56,000 ft-lb/ft<sup>3</sup>. Compaction was performed using a mechanical compactor model M100, manufactured by Ploog Engineering Company, Inc. A comparison test with manual compaction technique showed that the M100 mechanical compactor achieved an insignificant difference of 0.6% from the manually performed process.

### **3.1.4 Limerock Bearing Ratio**

The strength of RAP, A-3 and RAP-soil mixtures was determined using LBR testing according to FM 5-515. LBR testing was conducted on one set of samples not subjected to creep and on a second set of samples following creep testing.

Samples were not soaked in water prior to testing because of two reasons: first, neither RAP nor A-3 has shown any tendency to swelling or bulking, and second according to FDOT Memorandum (Malerk and Sadler, 2005) RAP is not permitted below the water table in earthwork applications. No surcharge weights were applied, since the minimum possible LBR value at optimum moisture content was of interest.

LBR testing was performed on the samples not subjected to creep but produced from moisture-density test. LBR was also obtained after creep testing with the intentions of evaluating LBR changes with and without creep. Samples were subjected to LBR testing once either 6, 12 or 18 psi creep testing was complete.

### **3.1.5 Asphalt Content and Viscosity**

Representative RAP samples were placed in sealed bins and shipped to FDOT State Materials Office in Gainesville, Florida. The asphalt content and viscosity testing was performed by FDOT Bituminous Laboratory. The asphalt content was determined following FM 5-563, “Quantitative Determination of Asphalt Content from Asphalt Paving Mixtures by the Ignition Method”, typically used for establishing the asphalt content in pavement mixtures. The material is placed into oven and the asphalt is burned by ignition at 538°C (1000°F). The asphalt content

is calculated from the initial weight of the sample, the final weight of the remaining aggregate, a temperature compensation factor for the measurement system, and a calibration factor to account for the aggregate weight loss due to ignition. The asphalt content is expressed as both a weight loss in grams, and as a weight percentage of the initial sample weight.

Viscosity was estimated in poises according to FM 1-T 202, “Absolute Viscosity of Asphalts” with vacuum capillary viscometer, where the time is measured for a fixed volume of liquid asphalt to be drawn up through a capillary tube by means of vacuum, under closely controlled conditions of vacuum and temperature. The viscosity is calculated by multiplying flow time in seconds by the viscometer calibration factor.

### **3.1.6 Extracted Aggregate**

In conjunction with the asphalt content test, a sieve analysis was conducted on the extracted aggregate. Extracted aggregate is the blend of aggregate particle remaining when the asphalt content test removes and measure the bituminous asphalt binder from RAP. Tests were also conducted according to FM 1-T 027, “Sieve Analysis of Fine and Coarse Aggregate” by dry sieving. U.S. standard sieve sizes of 0.75 inch, 0.5 inch, 0.375 inch, #4, #8, #16, #30, #50, #100, and #200 were used. A total weight of 3.0-4.5 lbs of extracted aggregate was tested for each sample. Due to available resources, only one trial was conducted on each sample. Where available, these test results were supplemented and compared to gradation data provided by the source facility for the same stockpile.

### **3.1.7 Degradation of Post-Compacted Samples**

For all samples subjected to Proctor compaction, the material was dried again and a second sieve analysis was conducted to compare the particle degradation characteristics under laboratory compaction conditions.

## **3.2 Laboratory Creep Testing Program**

RAP, A-3 soil, and RAP-soil mixtures at various proportions were evaluated initially by performing engineering properties testing and later by subjecting them to creep. Grain-size analysis, dry rodded unit weight, moisture density, LBR, asphalt content, and viscosity tests were performed to characterize the engineering properties of the materials. The long-term deformation behavior was investigated by testing the materials using uniaxial compression in one-dimensional oedometer setup at load increments of 6, 12 and 18 psi.

Since a recommended creep testing procedure for granular materials does not exist, the time-dependent response methodology used in this study was selected following literature search findings and previous research experience. Mitchell (1993) stated that most studies of time-dependent deformation in soils have been

based on triaxial compression or oedometer testing, and developed his constitutive model for analytical creep description for the case of uniaxial compression. Augustesen et al (2004) presented a review of various observations on time-dependent soil testing, separately analyzing the behavior of sand during one-dimensional oedometer creep tests. Cleary's work (2005) on RAP-soil mixtures, whose results are used as a basis for comparison with the current study, was completed with oedometer setup. Therefore, the creep testing methodology for this thesis research was based on oedometric loading. The load increments calculation and test procedure details are explained later in this chapter.

Creep is a process characterized by strain increase during constant stress. To model the creep behavior of RAP and RAP-soil mixtures there were several aspects to be considered. The testing apparatus had to simulate a deformational behavior compliant with the analytical models to be used for data processing. The load increments had to represent the actual stress conditions for earthwork applications of RAP. The data collection procedure and the testing duration had to accumulate sufficient amount of data to conduct long-term behavior analysis.

The current study adopted Cleary's creep testing methodology with certain improvements oriented towards providing consistency in testing setup and sample preparation. Stress increments were recalculated to reflect realistic in-situ fill heights. Each pressure was applied on a newly prepared sample and thus the actual deformation resulting from a particular stress application was measured. It was believed that the data collected in this manner would have closer correspondence to Singh and Mitchell's rate process model for creep estimation and the data analysis would produce more reliable results.

### **3.2.1 Determination of RAP-Soil Mixtures**

The purpose of mixing RAP with soil is to obtain a material of improved gradation with increased strength and reduced deformation. A proper gradation, defined by good representation of various grain size fractions, generally produces material of higher density and hence, of increased shear strength and bearing capacity (Barksdale, 1991).

RAP exhibits a deficiency of particles smaller than the # 4 sieve because it is typically a coarse grained well graded material. Garg and Thompson (1996) observed that RAP sieve analysis resulted in percentage passing #4 to #200 sieves close to the lower tolerance of gradation requirements. Maher et al (1997) concluded that these critical percentages decreased further after compaction due to bonding of smaller particles to the asphalt binder. The RAP grain-size distribution curve obtained by Cleary (2005) plotted below the specified lower gradation criteria for MSE backfill in the range between #40 and #200 sieves (FDOT Standard Specifications, Section 548). In general, the reported gradation data indicates that the amount of fines in RAP rarely exceeds 4% and typically lies

between 0 to 1%, where fines are defined as the amount of material passing the # 200 sieve.

According to Talbot's equation, for materials with a maximum aggregate size of 1 ½ inches, typical of RAP, the fines content should be greater than 6%. Percentages above 6% would ensure that both maximum density and shear strength are achieved, and should also result in increased LBR values (Barksdale, 1991).

A-3 soil is a good choice for mixing with RAP, since the major portion of its gradation is between the # 30 and # 200 sieves and would fill-in gaps when mixed with RAP. A-3 soils are readily available throughout Florida and are one of the materials approved by FDOT for earthwork applications such as embankment construction and backfilling (FDOT Standard Specifications, Sections 120, 125). Cleary's research (2005) on RAP-soil mixtures proved that A-3 soils were easy to work with and its addition improved the properties of RAP. Therefore, an A-3 soil was selected to produce the mixtures for the current study.

In addition to 100% RAP and 100% A-3 as control material, blends of RAP and soil were tested in proportions of RAP/soil as follows: 80/20, 60/40, 40/60 and 20/80. These mixtures will be further referred to as 100%, 80%, 60%, 40% and 20% RAP, and A-3 soil respectively. By testing RAP mixtures using 20% increments it was ensured that the entire range from 100% to 0% RAP was enveloped with regard to the comprehensiveness of data analysis. In addition, Cleary (2005) tested 100%, 80% and 60% RAP mixtures and including identical proportions would allow for data comparison.

### **3.2.2 Material Sampling**

The RAP and A-3 soil used for this study were obtained from stockpiles located at the APAC-Southeast Inc. asphalt plant in Melbourne, Florida. The original source of this RAP was a severely deteriorated alligator cracked parking lot pavement. The material was not processed by a post-milling processes.

The materials were sampled following FM 1-T 002, "Sampling Coarse and Fine Aggregate". The RAP stockpile was conically shaped relatively large and was estimated to contain approximately 4,000 tons of milled RAP. RAP samples were collected after removing the outside face of the pile with a front-end loader in order to break down the formed surface crust and to avoid collection of segregated layers. A total of 30 sampling bags weighing about 50 lbs each were collected from three different elevations and multiple locations throughout the pile to ensure collection of representative samples. Also, 20 bags of A-3 soil were collected. The total quantity of material sampled was determined based on an evaluation of the quantities required for the planned laboratory tests.

### **3.2.3 Material Preparation**

The field material sampled was collected during humid conditions and was noticeably moist upon delivery to the laboratory. Field material was spread in a thin



layer and air dried at room temperature of 75 °F for 5 to 7 days. Air drying was preferred over oven drying due to the effects that drying temperatures would have on the asphalt in RAP. RAPs' aged asphalt binder may soften at elevated temperatures and produce alterations in its gradation, which would introduce errors during testing. After large agglomerated pieces were manually fragmented, RAP was divided into batches to minimize the variations in gradation before further testing. The reduction into test size samples followed the quartering method of FM 1-T 248, "Reducing Field Samples of Aggregate to Testing Size."

Some of the planned tests, such as Modified Proctor Compaction and LBR, required size modifications to the original material gradation according to the procedure outlined in Section 3.2 of FM 5-521 and FM 5-515. Therefore, the designated RAP batches were processed following the sample preparation procedure. Field material was separated through a stack consisting of three sieves: 2 inch; ¾ inch and #4. All particles retained on the 2-inch sieve were discarded. The material passing the 2 inch sieve and retained on the ¾ inch sieve was weighed and replaced by an equal amount of material passing ¾ inch sieve and retained on the #4 sieve.

The A-3 soil was air dried by spreading in a thin layer at room temperature. After verifying that the soil moisture content approached zero, any clumped material was manually fragmented and the material was thoroughly mixed.

### **3.2.4 Selection of Load Increments**

The selection of realistic load increments was considered critical for the validity of creep testing results. Cleary (2005) used application pressures of 6, 12 and 24 psi, and concluded that they were excessive compared to the stresses at the midheight of backfills behind typical MSE walls.

The load increments used for creep testing during the current study were selected based on three factors. The first factor was determined from bearing capacity calculations, for the case of general shear failure of MSE walls with a vertical face and a horizontal backslope. Bearing calculations were conducted for 100% RAP following the FHWA requirements presented by Elias and Christopher, (2001). The 100% RAP shear strength parameters needed in the calculations were adopted from Gomez (2003). According to this design approach, which in addition to the backfill weight adds a 2 psi surcharge, a bearing pressure of 24 psi would exist at the base of a 20 ft. high MSE wall, which equates to 12 psi at the wall midheight.

The second factor taken into account for the stress increments determination was the practice adopted by FDOT for verifying the shear strength properties of sandy materials used as backfill behind MSE walls (personal communication, September 2nd, 2005). Information received from FDOT indicates that direct shear test is performed with normal stresses of 7, 14, and 21 psi, which would correspond to the vertical pressures at midheight of the backfill. The used normal stresses

represent wall heights between 5 and 16 ft. considered typical for the majority of MSE structures in Florida.

The pressure of 12 psi obtained from bearing capacity computations was accepted as one of the loading increments to be used during creep testing. In order to spread out the range of collected data and to envelop a variety of wall heights, also 6 psi and 18 psi were accepted as stress increments, bracketing 12 psi through equal intervals to facilitate data presentation and interpretation. Finally, the selected 6, 12 and 18 psi increments would allow for data comparison with Cleary's (2005) testing implemented at load levels of 6, 12 and 24 psi.

### **3.2.5 Laboratory Equipment**

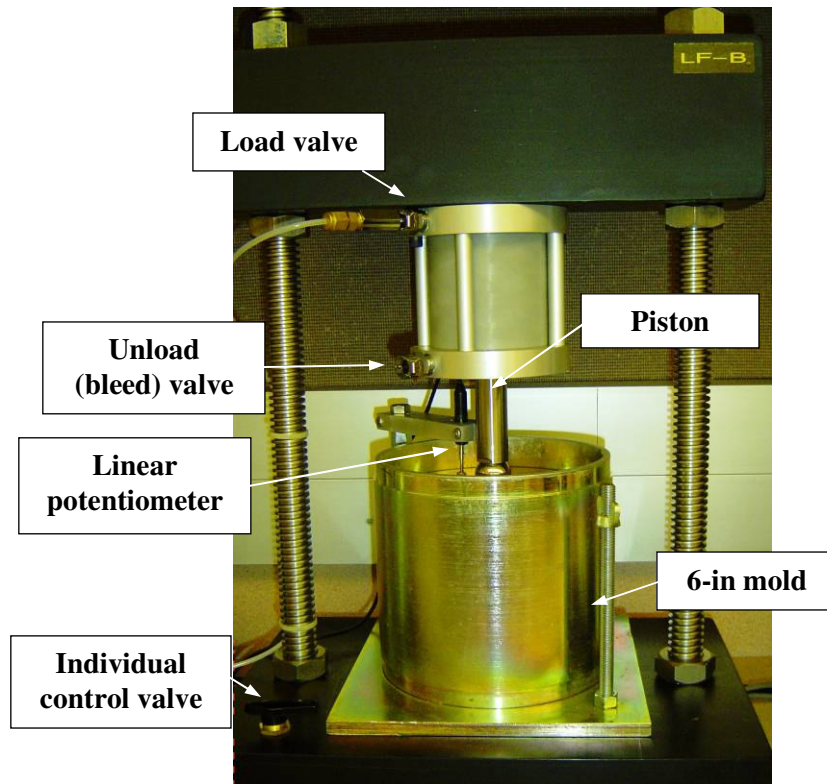
Cleary (2005) conducted RAP creep testing by loading samples in two different oedometer type apparatuses – one was based on a classic lever-arm consolidation frame and the other was a pneumatic loading device BK Terraload. Throughout the testing process substantial inconsistencies between the data generated by the two types apparatuses were reported by Cleary. The BK Terraload unit yielded consistently lower deformations than the lever-arm units. The two identical lever-arm consolidation units, however, also gave data discrepancies between each other, which were attributed mainly to differences in the relative compaction of the samples.

After analyzing Cleary's approach, it was decided that in order to obtain valid results, identical units should be utilized and a set of three replicas per material should be loaded at each pressure level. Since the BK Terraload unit was considered easier to control, it was selected as a basic model for the construction of six oedometer-based loading devices presented in Figure 3.1.



**Figure 3.1 Set of PLD's, pneumatic loading device**

The PLD was designed to apply loads instantaneously and to maintain the loads indefinitely on the sample regardless of the rate of deformation. Each PLD was constructed to hold a 6-inch mold below a loading piston and was connected to a constant air supply. The air pressure acting on the piston was adjusted from a precision air regulator connected to either of two pressure gauges: a low range pressure gauge for settings ranging between 0 to 50 psi and a high range for settings up to 100 psi. Each PLD had two quick connect valves positioned on the piston - a load valve to apply pressure and an unload valve to bleed the pressure. There was an individual control valve on each PLD and a main control valve for the entire set of testing units to switch the supply pressure on and off. The pressure was transferred to the sample through a one-point contact using a steel ball below the piston. An aluminum plate  $\frac{1}{2}$ -inch thick and with diameter slightly smaller than the inner diameter of the mold was placed on the sample surface to distribute the load as shown on Figure 3.2. Displacement was measured with a linear potentiometer attached with a steel arm to the mold base.



**Figure 3.2 PLD components**

The PLDs were calibrated utilizing load cells placed between the piston and the frame base on each unit. The calibration indicated that the load applied on a sample correlates with the reading on the pressure gauge when multiplied by a factor of four. Thus, a reading on the pressure gauge of 24 psi actually resulted in application pressure of 6 psi on the sample.

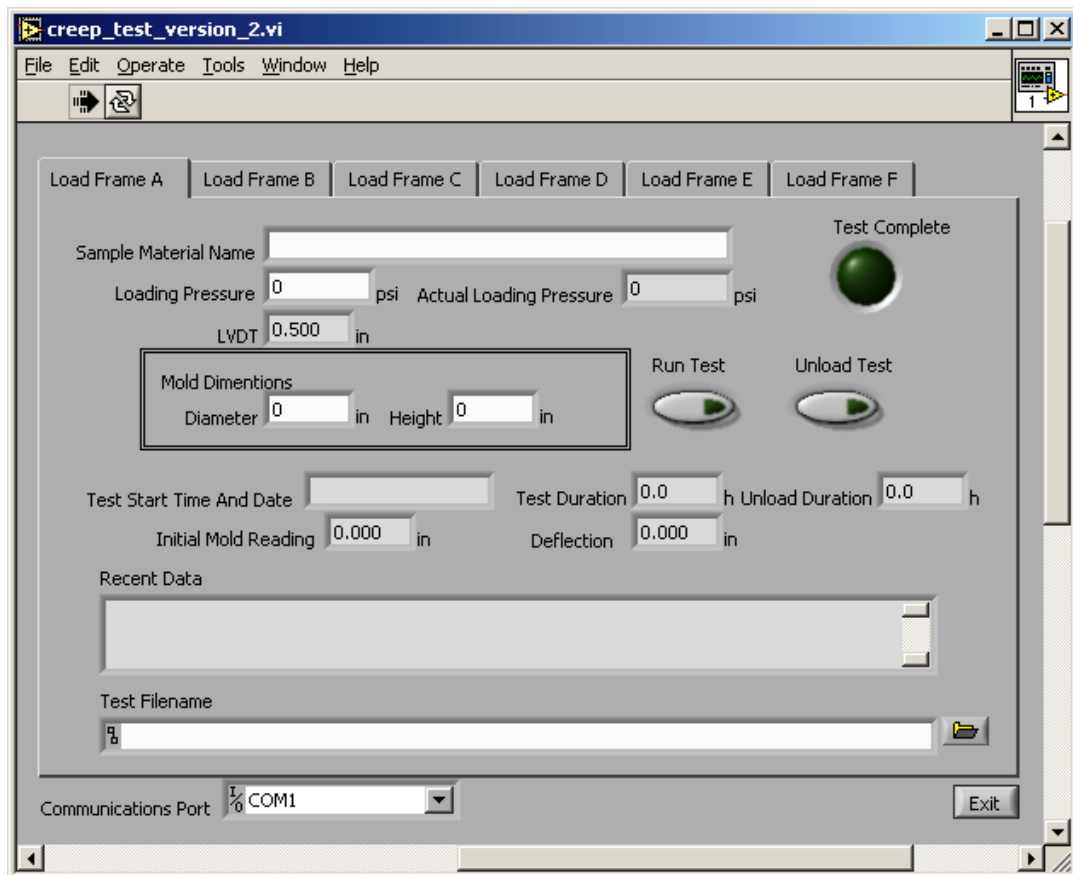
### **3.2.6 Data Acquisition System**

For the purposes of continuous data collection of the deformation as a function of time, a data acquisition system (DAQ) was developed. A custom circuit was built to implement the data collection from the linear potentiometer on each of the six pneumatic units. The circuit used a single PIC microprocessor to read the analog signal from each potentiometer and send the data serially the serial port of a PC.

A LabView<sup>®</sup> software module was utilized to collect, store and display deflection and time data from each of the six PLDs. The program was created to obtain data at specific time intervals. For the first two minutes from the beginning of the test, when large and rapid deformation was expected to occur, readings were recorded every second. After the first two minutes elapsed, the program collected

data by progressively doubling the previous time intervals. Thus, the sequence of data recording became 2, 4, 8, 16 minutes, etc. for the first four hours of the test. After four hours data was continuously recorded at four-hour intervals until unloading. The collected data was recorded as a.csv file and stored on the hard drive. The module was designed to simultaneously collect and store data for all six PLDs.

The user interface for the program module is presented in Figure 3.3. The module contained six displays, designated for the six PLDs. Each display showed description of the tested material and sample, as well as the time-deformation data for a particular loading unit. After positioning of the sample on the PLD and adjusting the desired pressure level on the regulator, it was necessary to assign a file name and storing location for the respective.csv file, start the program, and load the sample. Each loading unit was built with an individual load valve, which in combination with the specific software module display and data control enabled each sample to be independently tested.



**Figure 3.3 LabView® software module interface**

### **3.2.7 Sample Preparation**

Creep samples were prepared by compaction using the M100 mechanical compactor. Two bins containing two different mixtures were set up prior to compaction. The material in the bins was quartered, size-modified, and mixed at the optimum moisture content for the particular mix as determined by moisture-density test. The materials were allowed to hydrate overnight. Immediately before compaction the material was thoroughly remixed and moisture contents samples were collected. Three samples of each material were mechanically compacted with modified Proctor method following FM 5-521. Molds 6 inches in diameter and 6 inches in height were used with a spacer disc to obtain the required standard volume of 0.075 ft<sup>3</sup>. The spacer disc was separated from the material with a thin plastic sheet to prevent any adhesion between the material particles and spacer disc surface. Upon completion of compaction, the spacer disc was removed and the samples were inverted.

Following the compaction the unit weight of each sample was calculated and the relative compaction was established based on the maximum unit weight obtained from moisture-density test. The relative compaction was intended to fall within the range 95-100%. The three replicas were considered acceptable if the difference between the maximum and minimum of the three relative compactions did not exceed 2%. If this requirement was not fulfilled, the sample with a unit weight varying the most from the maximum unit weight as obtained from moisture-density test was discarded and a new sample was prepared. The procedure was repeated until the 2% limit on relative compaction was achieved.

### **3.2.8 Lab Creep Testing Procedure**

Each sample prepared as described in the preceding paragraphs was covered with plastic wrap and transferred to a PLD. After positioning the mold, the wrap was removed and a ½-inch thick load distributing plate and ball were placed on the surface of the sample. The unit frame was inspected with a level to verify that the cross-head beam was horizontal. A potentiometer was mounted on a steel arm attached to the mold. After verifying that the main control valve and the individual control valve were turned off, the air supply was connected to the quick connect load valve. A minimum pressure equivalent to 1 psi was adjusted on the regulator, the main control and individual valves were opened, and the sample was preloaded for 10 seconds. The preloading aimed to reduce any surface irregularities resulting from the sample preparation. Deformation data from the preloading was not recorded. After the preloading was implemented, all valves were closed and the pressure on the regulator was readjusted in accordance with the testing plan to one of the loading increments – 6, 12 or 18 psi.

After adjusting the pressure on the PLD's and opening the LabView<sup>®</sup> software module, all potentiometers were inspected for proper functioning. Data collection files were created and the necessary information for sample identification

was input. The program was started, followed by opening of the master valve first and subsequently - of the individual valves. The LabView<sup>®</sup> displayed data was checked for periodical collection.

### **3.3 Large Scale Creep Testing Methodology**

#### **3.3.1 Introduction**

Tests were conducted on RAP and RAP-sand mixtures to simulate large scale testing for use in MSE wall applications. Results from the large scale tests were compared to laboratory tests where possible. The testing program includes a sequence of horizontal pullout tests of the galvanized steel strips and vertical creep testing of 100% RAP, 50-50% RAP-sand mixtures, and A-3 sand. The horizontal pullout testing sequence involved two different tests for each material, a maximum pullout to determine an ultimate load and the incremental pullout test. Trial tests were conducted in the FDOT testing pit to evaluate the testing equipment and procedures following a suggested procedure by Lai and Moore (1990).

Laboratory creep testing was conducted for comparison to results reported by Cleary (2005) and Dikova (2006). Large scale pit vertical creep tests conducted in the FDOT test pit facility utilized the entire pit volume and creep was measured over a number of days. The selection of the load increments were based on analysis of creep of 100% RAP and RAP-sand mixtures performed by Cleary (2005) and Dikova (2006). Vertical confining pressures of 6, 12 and 18 psi were used in all laboratory field tests in this study.

#### **3.3.2 Testing Program**

The testing program conducted on RAP and RAP-sand mixtures is presented in Table 3.2. Maximum pullout testing was conducted on one strip in each material at every vertical confining stress producing the nine tests shown in the table. The incremental pullout test was conducted on two strips of each material at every vertical confining stress yielding a total of 18 tests (Table 3.2). Six tests were performed on each material to evaluate vertical creep by applying vertical stresses of 12 and 18 psi, producing a total of 12 tests (Table 3.2). Vertical creep was also evaluated for each sample with applied vertical stresses of 6, 12 and 18 psi in the laboratory. Two tests were performed for each sample producing six creep tests (Table 3.2).

**Table 3.2 Summary of testing program**

Testing Program			Number of Tests		
			Confining Stress (psi)		
Location	Test	Material	6	12	18
FDOT State Materials Office Pit	Pullout-Maximum*	100% RAP	1	1	1
		50/50% RAP-Sand Mixture	1	1	1
		100% A-3 Sand	1	1	1
	Pullout- Incremental*	100% RAP	2	2	2
		50/50% RAP-Sand Mixture	2	2	2
		100% A-3 Sand	2	2	2
	Vertical Creep (Large Scale)	100% RAP-100% A-3 Sand Layered	--	2	2
		50/50% RAP-Sand Mixture	--	2	2
		100% A-3 Sand	--	2	2
Florida Institute of Technology Laboratory	Vertical Creep	100% RAP	3	3	3
		50/50% RAP-Sand Mixture	3	3	3
		100% A-3 Sand	3	3	3

\* A 45 minute vertical creep test was performed prior to pullout testing.

### 3.3.3 Material Properties

#### 3.3.3.1 Selection of RAP-Soil Mixtures

Proportions of 100% RAP, 50-50% RAP-sand mixture, and 100% A-3 sand were used for comparison in this study. These proportions were selected based on the results reported by Cleary (2005) and Dikova (2006). Neither researcher tested a 50-50% RAP-soil mixture. However, it was concluded from their testing, that this proportion would be more practical to mix and use in field earthwork applications and would yield a useful engineering material. The A-3 sand is readily available in Florida and is a FDOT approved backfill material (FDOT Standard Specifications, Sections 120, 125, 548) for embankment, pipe, and MSE walls, and is selected as a control soil in each test of this study.

#### 3.3.3.2 Sampling and Origin

The A-3 sand and RAP for this study was provided by VE Whitehurst & Sons, Inc. located in Gainesville, FL. The sand originated from a formation referred to as Starvation Hill near Gainesville, Florida. A-3 sand was taken from uncovered stockpiles at VE Whitehurst & Sons. The RAP and sand materials used in the pit were delivered by dump truck to the FDOT testing facility. The milled RAP material used in this study was taken from uncovered stockpiles located in the VE Whitehurst & Sons facility. Approximately 14 cubic yards of RAP and 32 cubic yards of A-3 sand were used for this study.

Samples used to determine engineering properties were obtained in five gallon plastic buckets and cloth bags weighing approximately 60 lbs. each. Ten samples of RAP and six samples of sand were acquired for testing. All materials



were sampled according to ASTM D75 “Standard Practice for Sampling Aggregates”.

### **3.4 Statewide Variability**

RAP materials available in Florida come from a variety of sources and processing methods. In order to analyze the statewide variability of RAP materials, representative samples will be collected for fundamental engineering testing to evaluate critical variables. Laboratory test data will be compared to FDOT field data to measure the applicability of sample test results. Simple statistical methods will be implemented to analyze results and develop conclusions.

#### **3.4.1 Material Sampling**

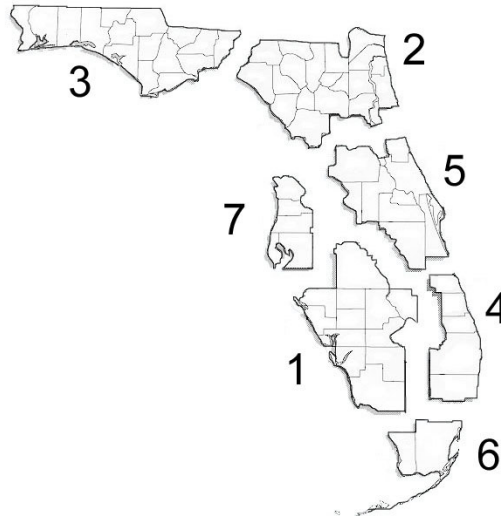
Material sampling procedures were developed at three levels: source location, site and test sample. The source location of each sample was selected based on a representative distribution of active FDOT-approved asphalt plants, which are the largest centralized facilities that stockpile and recycle RAP materials in Florida. The distribution of source facilities was developed to address geographic supply differences. Site sampling procedures were utilized to ensure the small sample removed from each stockpile provided a representative sample to conduct laboratory testing. Test sample procedures involved further division of each sample in a smaller representative size for each test trial. In conjunction with site sampling, a verbal survey was conducted at each facility to gather information regarding typical material characteristics, processing methods, level of production, and examples of RAP reuse.

##### **3.4.1.1 Geographic Distribution**

Distinct differences exist between Florida’s geographic regions regarding population, economics, material resources, and demand for new construction. Selection of samples considered these factors and how they relate to statewide material variability and potential earthwork applications. Florida’s population growth, urban development and roadway infrastructure is not concentrated in a single region; therefore opportunities for reuse of RAP exist in all regions. Population growth and new construction is occurring throughout the state, but is more pronounced in several regions. Higher population growth and density requires additional and larger roadways, leading to concentrated demands for large quantities of earthwork materials in the urban and suburban counties.

The FDOT districts will be used as geographic regions for the sampling distribution. FDOT is subdivided into seven multi-county districts for administrative purposes, shown in Figure 3.4. Florida’s Turnpike Enterprise acts as an eighth district to administer toll-roads under state jurisdiction, which exist within the boundaries of five of the other districts. All roadway data from the

Florida Turnpike Authority has been assigned to the corresponding geographic District where it is located, although it is not administered by the corresponding District. FDOT Districts 4 & 6 and 1 & 7 have been combined due to their proximity and similar development and geologic characteristics, representing the southeast and southwest regions of the state.



**Figure 3.4 Map of FDOT district boundaries by county**

Four factors will be used to analyze large-scale regional differences that may influence RAP reuse and develop a representative distribution for sampling RAP materials across the state. These factors are existing development density, recent construction growth trends, recent FDOT construction, as well as additional considerations. Although the original intent was to collect 100 samples, a total of 104 samples were collected from 50 asphalt plants. The relative importance of each factor was used to weight the sampling distribution based on factors that were perceived to be more representative to variables influencing RAP production and reuse applications. Table 3.3 shows the distribution and number of samples for each factor.

**Table 3.3 Sampling distribution factors**

<b>Distribution Factors</b>	<b>Number of Samples</b>	<b>Proportion of Sampling</b>
Existing Development Density	20	19%
Recent Construction Growth Trends	34	33%
Recent FDOT Construction Projects	35	34%
Additional Considerations	15	14%
<b>Total Samples</b>	<b>104</b>	<b>100%</b>

**3.4.1.1.1 Existing Development Density**

Urban areas, with a higher population and roadway density may have higher demands for earthwork materials, because existing material supplies require long transport distances to quarries from outside the urban area. This can create an opportunity for reuse of RAP in earthwork applications. The development density is estimated by analyzing two variables, the population density and average traffic density. Population density is calculated by combining the 2005 Population and Land Area from the U.S. Census data for all counties included in a district. FDOT roadway mileage and traffic count data was utilized to estimate average traffic density. For all highways under state jurisdiction, referred to as the State Highway System (SHS), FDOT compiles Mileage Reports of the number of centerline mile, lane miles and daily vehicle-miles traveled (DVMT). Centerline lines measures the length of a highway. A Lane Mile is a measure of the size of a roadway and is the centerline length multiplied by the number of lanes. DVMT is a measure of the total traffic on a road, and is the product of the average daily traffic count and the length of the road. If the DVMT is divided by the lane miles, the result is the daily average vehicles per lane, estimating the overall traffic density.

Table 3.4 lists the variables for each District relating to population and traffic density, as well as the distribution of samples based on this factor. Many urbanized areas are within regions with large portions of rural undeveloped land, causing the average population density for a specific city many be much higher than the District-wide average. Higher average population densities in Districts 1+7, 4+6, and 5 highlight the influence of the major urban areas on these Districts. These same Districts also show high values for overall traffic density, showing that high traffic volumes demands are highly related to population density. A total of 20 samples are allocated based on this factor.

**Table 3.4 District density variables and ranking**

FDOT District	1 + 7	2	3	4 + 6	5	State Total
Geographic Location	Southwest	Northeast	Northwest	Southeast	East Central	
Major Cities	Tampa, Lakeland, St. Petersburg, Sarasota, Ft. Myers	Jacksonville, Gainesville	Tallahassee, Pensacola, Panama City	Miami, Ft. Lauderdale, West Palm Beach	Orlando, Daytona Beach, Melbourne	
Average Population Density (2005) (population / sq. mi.)	354	154	115	775	413	
Population Density Rank	3	4	5	1	2	
Traffic Density (2006) (DVT per Lane Mile)	7.5	5.4	4.3	9.8	8.2	
Traffic Density Rank	3	4	5	1	2	
Number of Samples	4	2	2	7	5	20

**3.4.1.1.2 Recent Construction Growth Trends**

RAP primarily originates from milling and reclamation of existing state highways, which carry the largest proportion of the state’s roadway traffic. A significant portion of RAP is produced from city and county roads as well as from redevelopment of smaller commercial or residential projects with parking lots. Roadway resurfacing can produce RAP in all areas, but reuse of RAP may become more practical in areas of higher construction growth.

Recent trends in construction growth can be estimated using population growth, roadway growth and traffic growth variables. Population growth is calculated by combining the 2005 and 2000 Population the U.S. Census data for all counties included in a district. FDOT Mileage Reports from 2000 and 2006 was utilized to estimate average annual growth for lane miles and DVMT. Table 3.5 lists the variables for each District relating to growth rate of population, highway lane miles, and traffic density. Population and roadway data was compiled from different time sets; therefore the average annual growth rate in calculated for comparison. Districts 1+7 and 5 consistently show high growth rates for population, lane miles and traffic density, highlighting the influence of the fast-growing urban areas in these Districts. The other three Districts showed moderate values for growth rate. District 3 was unique that despite having the lowest population density and population growth rate, the traffic density growth rate was moderately high. A total of 33 samples are allocated based on this factor.

**Table 3.5 District growth rate variables**

FDOT District	1 + 7	2	3	4 + 6	5	State Total
Geographic Location	Southwest	Northeast	Northwest	Southeast	East Central	
Population (2005)	5,257,348	1,821,186	1,304,580	6,008,156	3,398,594	17,789,864
Population (2000)	4,651,066	1,663,067	1,222,492	5,519,526	2,926,227	15,982,378
Average Annual Growth	2.2%	1.7%	1.2%	1.6%	2.6%	2.0%
Rank	2	3	5	4	1	
Lane Miles (2006)	10,268	8,062	6,548	9,008	7,727	41,614
Lane Miles (2000)	9,670	7,749	6,377	8,711	7,223	39,730
Average Annual Growth	1.9%	0.6%	0.4%	1.1%	1.1%	1.0%
Rank	1	4	5	3	2	
DVMT per Lane Mile (2006)	7.5	5.4	4.3	9.8	8.2	
DVMT per Lane Mile (2000)	6.5	4.9	3.9	8.9	7.1	
Average Annual Growth	2.4%	1.8%	1.9%	1.5%	2.4%	
Rank	1	4	3	5	2	
Number of Samples	10	5	4	6	8	33

**3.4.1.1.3 Recent FDOT Construction Projects**

To supplement the sampling due to recent trends in overall growth rate, an analysis of recent FDOT projects was done to more precisely estimate the current production of RAP throughout the state. By compiling data from all FDOT bid item contracts for the years 2004 to 2006, the quantity of RAP milled from state highways was estimated for each District. The contract quantity is estimated based on the area of milling and pavement thickness, calculating the bank volume of RAP materials for each project and District. An average unit weight of 120 pcf was applied to calculate the mass. A summary of the total quantity of RAP produced by milling on state highways is presented in Table 3.6. A total of 35 samples were assigned based on this factor.

**Table 3.6 Summary of RAP contract quantity analysis**

FDOT District	1 + 7	2	3	4 + 6	5	State Total
Geographic Location	Southwest	Northeast	Northwest	Southeast	East Central	
Quantity Produced (2004) (tons)	91,216	46,623	235,518	395,069	380,863	1,149,289
Quantity Produced (2005) (tons)	117,811	35,766	8,101	222,432	166,061	550,172
Quantity Produced (2006) (tons)	181,645	150,681	227,959	416,378	173,637	1,150,299
Total Quantity Produced (2004-2006) (tons)	390,672	233,070	471,578	1,033,879	720,561	2,849,760
Proportion of Total	14%	8%	17%	36%	25%	
Rank	4	5	3	1	2	
Number of Samples	5	3	6	11	10	35

The limitations of this analysis method is that it does not consider RAP materials milled as a part of a Lump Sum or Design Build contract without bid item quantities for milling area or thickness. Based on the three years analyzed, the amount of Lump Sum contracts can vary from 15% to 20%, suggesting the contract quantity analysis may under-represent the actual total quantity of RAP produced by milling. Furthermore, in the years studied Districts 1 through 4 utilized Lump Sum contracts at a higher rate than Districts 5 through 7, representing 15% to 40% of the total value of bids. This suggests that Districts 1 through 4 may be underrepresented in the contract analysis; in particular District 2, which had approximately 40% Lump Sum contracts, as measured by total value of bids. This and other limitations will be considered and accommodated for in the additional consideration category.

#### **3.4.1.1.4 Additional Considerations**

Although a comprehensive approach was taken to provide a sampling distribution that is representative to Florida's regional population, highway mileage, growth and construction characteristics, additional samples have been allocated for regional aggregate geology and sample availability considerations.

As discussed in Chapter 2, crushed stone aggregates vary considerably in different regions of the state. Limestone aggregate originates from three primary regions, southeast (Miami-Dade to Palm Beach County), southwest coast (Collier, Lee, and Sarasota Counties), and Big Bend (Hernando to Taylor County). The high quality limestone aggregate from Miami's Lake Belt and Palm Beach County is mined and transported by rail and truck across the entire Florida peninsula, providing the vast majority of aggregate supplies to Districts 4 and 6, with a significant influence on Districts 1, 5 and 7. The crushed stone aggregates from along the southwest coast contain a substantial portion of marine fossils, causing some typical RAP in District 1 to have more unique aggregates. The limestone aggregates from the Big Bend region are characterized by having a higher density than other areas (McCaulley, 1990). Aggregates from this region are generally shipped to markets in Districts 1, 2, 5, and 7. In addition, significant portions of crushed granite is imported from Alabama and Georgia into Districts 2 and 3, producing higher density and higher strength aggregate than typical of the rest of the state. To supplement market needs, granite is also imported from Nova Scotia, Mexico, and the Bahamas through Ports in Tampa Bay and Jacksonville, producing a greater influence on variability in Districts 1, 2, 5, and 7.

To supplement the sampling identified by the other distribution factors, Districts 1+7, 2, 3 and 5 were allocated 16 additional samples in the statewide distribution. It was assumed the high population density and level of roadway infrastructure in District 4+6 already provided sufficient samples to accommodate for the relatively consistent aggregate supplies in this region. District 3 was allocated 7 additional samples due to the geographic separation of it from the rest of the state, under the assumption that smaller mines in that region and imported rock would produce significantly different RAP materials than any other region of the state. The population density and growth factors did not adequately address the potential future growth and development in this region, which could place additional demands on material supplies. District 1+7 was allocated 5 additional samples due to the unique geological characteristics of aggregates mined along the southwest coast. In addition, the influence of aggregates imported from other regions and overseas at the Port of Tampa suggests that RAP materials in the Tampa Bay area may produce high variability.

District 2 was allocated 4 additional samples due to the combined influence of multiple Florida, domestic and overseas aggregate sources as well as the potential for future aggregate demands. During the sampling collection of this study, the to

availability of a large variety of samples at several asphalt plants in District 2 causing the desired number of samples to be reduced by 2. Additional samples from plants located in the northern counties of District 5 were utilized to compensate for these samples. These plants conduct a significant amount of their work in District 2 as well as 5, and it was determined they would also provide a representation of the unique geologic properties in north central Florida. Overall, District 5 is a large and growing region that is influenced by aggregates from many areas, but the allocation of samples due to population density, recent growth, and continued construction in District 5 provided a sufficient number of samples to represent this variability.

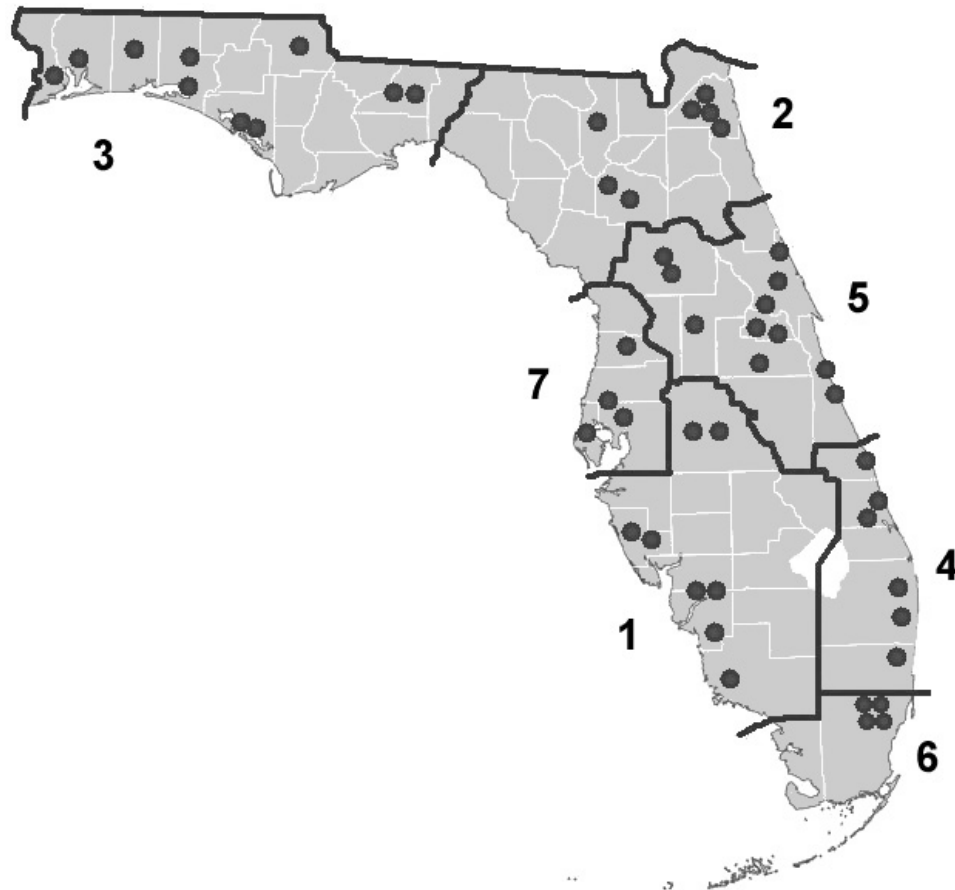
### 3.4.1.1.5 Sampling Plan

In summary, the four factors considered for producing a geographic sample distribution representative of Florida’s RAP material resources were combined to develop a region-based Sampling Plan to identify asphalt plants for sample collection. Table 3.7 presents a summary of the overall Sampling Plan based on each District and distribution factor. In selecting asphalt plants for sampling, a wide spread of sites throughout each District was chosen to provide representative samples from most urban areas throughout the state. Figure 3.5 shows the location of all asphalt plants sampled in this study and the FDOT District boundaries.

**Table 3.7 Summary of geographic sampling distribution**

FDOT District	1 + 7	2	3	4 + 6	5	Number of Samples	Proportion of Sampling
Geographic Location	SW	NE	NW	SE	East Central		
Existing Development Density	4	2	2	7	5	20	19%
Recent Construction Growth Trends	10	5	4	6	8	33	32%
Recent FDOT Construction Projects	5	3	6	11	10	35	34%
Additional Considerations	5	2	7	0	2	16	15%
<b>Total Samples</b>	<b>24</b>	<b>12</b>	<b>19</b>	<b>24</b>	<b>25</b>	<b>104</b>	
<b>Proportion of Total</b>	<b>23%</b>	<b>12%</b>	<b>18%</b>	<b>23%</b>	<b>24%</b>		
<b>Final Number of Asphalt Plants Sampled</b>	<b>12</b>	<b>7</b>	<b>10</b>	<b>10</b>	<b>11</b>	<b>50</b>	





**Figure 3.5 Map of asphalt plants sampled and FDOT district boundaries**

### **3.4.1.2 Site Sampling**

#### ***3.4.1.2.1 Processing Method***

During site visits for this study, three types of RAP stockpiles were identified: milled RAP, crushed RAP, and waste RAP. Milled RAP materials are unprocessed and generally contain larger particles created by the milling process. Milled RAP materials show a higher potential for reuse in earthwork applications due to its relatively lower reuse cost and the existence of large stockpiles throughout Florida. Although some milled RAP stockpiles can be directly recycled into HMA mixes, a majority of these stockpiles are processed to produce crushed RAP. Crushed RAP materials are processed for reuse in HMA mixes by screening and crushing, if the RAP contains sufficient asphalt and aggregate properties for recycling. Crushed RAP materials show little promise for reuse in earthwork applications due to the cost of processing and high demand for new and economic asphalt pavement. A milled or crushed RAP stockpile may originate from a single roadway project, and

is separated from other piles to maintain consistency and higher quality. At plants with a smaller site area or limited RAP supplies, a milled or crushed RAP stockpile may include a blend of RAP materials from multiple projects. Waste RAP stockpiles are accumulations of several leftover and scrap RAP materials, including RAP from small milling projects, excess fresh mix, unusable fresh mix from plant start-up and shut-down processes, and small portions of base rock or soil accumulated during milling operations. Significant material variability is visually observed in waste RAP stockpiles due to the mixture of materials from a variety of sources.

To satisfy the task objectives of statewide RAP sampling, it was concluded the sampling of waste RAP stockpiles would not produce representative results applicable to earthwork reuse applications, due to the high in-pile variability. Furthermore, waste RAP is typically processed to produce crushed RAP. Many plants have shown that if appropriate and controlled procedures are used to process these materials into crushed RAP, a homogenous product can be produced with reduced variability. Therefore, the focus of variability sampling will be limited to milled RAP and crushed RAP in order to assess the variability of these two products in accordance with their potential reuse applications.

To fulfill objectives of collecting samples representing differences in typical RAP processing methods, it was decided that one post-milling processed sample (crushed RAP) and one un-processing raw milled sample (milled RAP) should be collected from each asphalt plant. Although not all plants had stockpiles of both crushed and milled RAP suitable for sampling, many plants had more than the minimum, allowing for multiple samples from the same site. Overall, a total of 50 plants were sampled, collecting 55 milled RAP samples and 49 crushed RAP samples.

#### ***3.4.1.2.2 Stockpile Sampling***

Based on the higher reuse potential for un-processed milled RAP materials, more testing will be done on milled RAP than crushed RAP samples. Basic tests will be conducted on all samples. Additional tests will be conducted on most milled RAP samples. Where multiple milled RAP stockpiles were sampled at the same facility, only one sample was subjected to the additional testing.

In order to provide material for the tests proposed, sampling of each crushed RAP stockpile should be 50 lbs (one bag or bucket) and each milled RAP stockpile should be 200 - 250 lbs (four to five bags or buckets). The materials used for this study were obtained from different stockpiles located at asphalt plants throughout Florida. The materials were sampled following FM 1-T 002, "Sampling Coarse and Fine Aggregate". Large representative RAP piles were selected and estimates of the quantity of material per pile were recorded. RAP samples were collected after removing the outside face of the pile with a shovel. Where available, a front-end loader was used to break down the hardened surface and blend part of the pile

for our sampling. To ensure that a representative sample of the entire stockpile was collected, material was taken from at least three different elevations or multiple horizontal locations around each pile.

### 3.4.1.3 Test Samples

The sampled field material was collected in generally warm and humid weather and some samples were significantly moist during collection and delivery to the laboratory. All RAP materials were spread in a thin layer and air dried at room temperature of 75°F for 2 to 5 days. Air drying was preferred over oven drying due to the asphalt content of RAP and the large quantity of material required for testing. In addition, the aged asphalt binder was expected to soften at elevated temperatures and cause test results to be inconsistent, introducing errors into the overall sample comparison. After air drying RAP samples were thoroughly blended and any large clumped particles stuck together due to moisture were manually broken down. RAP was then divided into batches to minimize variations in gradation before further testing. The reduction into test size samples followed the quartering method of FM 1-T 248, “Reducing Field Samples of Aggregate to Testing Size.”

### 3.4.2 Laboratory Testing

FDOT Manual of Florida Sampling and Testing Methods (1994) and ASTM standards were used to determine the engineering properties of RAP samples. A summary of the applied laboratory testing procedures is presented in Table 3.8 and Table 3.9. A majority of tests were conducted at the Florida Institute of Technology’s Applied Research Laboratory (FIT-ARL) in Melbourne, FL. Asphalt Content and Extracted Aggregate Gradation tests were conducted by the FDOT State Materials Office (FDOT-SMO) in Gainesville, FL.

**Table 3.8 Basic RAP tests and procedures**

Laboratory Test	Standard Procedures	Testing Location
Sieve Analysis (Un-modified Sample)	FM 1-T027 / AASHTO T27	FIT-ARL
Dry Rodded Unit Weight	FM 1-T019 / AASHTO M-19	FIT-ARL
Asphalt Content	FM 5-563	FDOT-SMO
Sieve Analysis (Extracted Aggregate)	FM 1-T027 / AASHTO T27	FDOT-SMO

**Table 3.9 Additional RAP tests and procedures**

<b>Laboratory Test</b>	<b>Standard Procedures</b>	<b>Testing Location</b>
Sieve Analysis (Post-Compacted Sample)	FM 1-T027 / AASHTO T27	FIT-ARL
Moisture Density Relations, Modified Proctor Compaction	FM 1-T180 / AASHTO T180, Method D	FIT-ARL
Limerock Bearing Ratio (LBR)	FM 5-515	FIT-ARL
Particle Shape Index	Not Applicable	FIT-ARL

### **3.4.2.1 Asphalt Content**

RAP samples were placed in sealed bins and shipped to the Bituminous Laboratory at the FDOT SMO in Gainesville, Florida. The asphalt content was determined following FM 5-563, “Quantitative Determination of Asphalt Content from Asphalt Paving Mixtures by the Ignition Method”, typically used for establishing the asphalt content in pavement mixtures. The material is placed into oven and the asphalt is burned by ignition at 538°C (1000°F). The asphalt content is calculated from the initial weight of the sample, the final weight of the remaining aggregate, a temperature compensation factor for the measurement system, and a calibration factor to account for the aggregate weight loss due to ignition. The asphalt content is expressed as both a weight loss in grams, and as a weight percentage of the initial sample weight. Where available, these test results were supplemented and compared to asphalt content data provided by the source facility for the same stockpile.

### **3.4.2.2 Sieve Analysis**

The gradation of all un-modified RAP samples was determined at FIT-ARL according to FM 1-T 027, “Sieve Analysis of Fine and Coarse Aggregate” by dry sieving and agitation by a mechanical apparatus. U.S. standard sieve sizes of 2.0 inch, 1.0 inch, 0.75 inch, 0.5 inch, 0.375 inch, 0.25 inch, #4, #8, #10, #16, #20, #30, #40, #50, #60, #100, #140, and #200 were used. The large number of sieves was utilized to limit the amount of material retain on the critical sieve between 0.25 inch and #40 sieves. Additional sieves were also used in order to compare results to the same sieve sizes used for the extracted gradation tests. A total weight of about 10 lbs air-dried RAP material was tested for each sample, split into a smaller batches. The individual batches weighing approximately 3 lbs each were used to limit the quantity of material on a given sieve according to Section 7.3 of FM 1-T 027. A minimum of two trials were conducted on each sample. If the difference in percent passing between these two trials exceeded 5%, a third or fourth trial was

conducted to develop a representative average. The percent difference between the initial mass and the sum of the mass retained was checked and controlled to a maximum of 0.30%.

RAP gradations will be compared to theoretical maximum density models for coarse-grained materials, known as Fuller’s model and the Talbot equation. The Fuller’s model is an allometric curve, or curve based on relative exponential growth. The generic form is shown in Equation 3.1. Where “i” represents a given sieve size,  $d_i$  is the diameter for the given sieve,  $P_i$  is the percent finer than diameter  $d_i$ ,  $D_{max}$  is the maximum size of particles in the given gradation, and  $n$  is the gradation coefficient or shape factor. (Asphalt Institute, 2001).

$$P_i = \left( \frac{d_i}{D_{max}} \right)^n \quad \text{Equation 3.1}$$

The  $D_{max}$  variable determines the location of the origin of the curve at 100% passing. The shape factor  $n$  is critical to determine the location of the curve relative to the amount of fines at the #200 sieve ( $d = 0.075$  mm). Talbot’s equation is a specific form of Fuller’s model, where the shape factor ranges from 0.33 to 0.50, with an optimum value of 0.45. Theory states a coarse-grained material with a gradation curve at or near the Talbot equation curve would produce an ideal well-graded mix with a good representation of all particle sizes starting at  $D_{max}$ , including an optimum content of fines. An aggregate or soil gradation curve at or near the Talbot equation curve would theoretically enable the material to be compacted to its maximum density and highest shear strength, also resulting in higher bearing strength (Barksdale, 1991). The correlation coefficient between each sample gradation curve and the Talbot equation curve will be used to measure the accuracy of fit.

A gradation plot for any granular material can be fit to a Fuller’s model equation, with two variables,  $n$  and  $D_{max}$ , defining the curve. Sánchez-Leal (2007) utilized this method, using statistical software to calculate the unique pair of  $n$  and  $D$  values for each sample that produced a curve best fit to each aggregate sample gradation, therefore maximizing the correlation coefficient. The  $D$  variable was calculated independent of the actual maximum ( $D_{max}$ ) or nominal maximum particle size ( $D_{nom}$ ). This variable is noted as the optimal maximum particle size,  $D_{opt}$ . The correlation coefficient between each sample gradation curve and the fitted Fuller’s model will be used to measure the accuracy of a given Fuller’s model curve to represent the sample gradation. Although Sánchez-Leal (2007) compared gradations of aggregate blends for use in HMA mixes, the same analytical procedure can be applied to other materials, including soils and RAP.

#### ***3.4.2.2.1 Extracted Aggregate***

In conjunction with the ignition furnace asphalt content test, following FM 5-563, a sieve analysis was conducted on the extracted aggregate. Extracted aggregate is the blend of aggregate particle remaining when the asphalt content test removes and measure the bituminous asphalt binder from RAP. This measures the size distribution of the solid particles held within the RAP composite. Tests were also conducted according to FM 1-T 027, “Sieve Analysis of Fine and Coarse Aggregate” by dry sieving. U.S. standard sieve sizes of 0.75 inch, 0.5 inch, 0.375 inch, #4, #8, #16, #30, #50, #100, and #200 were used. A total weight of 3.0 to 4.5 lbs of extracted aggregate was tested for each sample. Due to available resources, only one trial was conducted on each sample. Where available, these test results were supplemented and compared to gradation data provided by the source facility for the same stockpile.

#### **3.4.2.3 Dry Rodded Unit Weight**

The dry rodded unit weight test was utilized as a measure of bulk density of loose aggregate or soil. The test was performed on both crushed and milled RAP samples following FM 1-T 019 and AASHTO M-19, “Bulk Density and Voids in Aggregate.” The procedure specifies the material be tamped with a rod for 25 blows for three layers. A straight 24 inch long steel rod was used, with a diameter of 5/8 inch and a hemispherical tip. The measure used was a modified Proctor compaction mold with 6 inch diameter and 9 inch height, resulting in capacity of 0.147 ft<sup>3</sup>, or approximately 1/7 ft<sup>3</sup>.

High variability was observed in some initial test results. Analysis of these samples indicated that test results are very sensitive to operational errors caused by poor mixing of the sample material, leading to test samples including a non-representative proportion of fines or coarse particles. Therefore, particular care was taken to ensure that every sample was mixed thoroughly before each test was conducted. In addition, a minimum of five trials were performed for each sample. Results were considered acceptable if the range between the minimum and maximum results was 0.45 lb or less, producing a maximum range in unit weight of 3.0 lb/ft<sup>3</sup>. If five trials did not meet this criterion, additional trials were conducted until five trials fell met this threshold. Samples with a larger number of trials will produce a higher range of values, the variability of each sample will be measured using the standard error, or the standard deviation divided by the square root of the number of trials. This value eliminates the influence of multiple trials, allowing for an unbiased comparison of material variability. The standard deviation cannot exceed the range; therefore the maximum range restriction of 0.45 lb will cause the standard error for all samples to remain below 0.2 lb. In summary, although the dry rodded unit weight test method can produce highly variable results, procedures were implemented to control the quality and assure precise results to compare samples.

### 3.4.2.4 Particle Shape Index

The particle index test was utilized as a simple method to measure the angularity and shape properties of the compacted RAP samples. The particle index test was based on the idea that void ratio is affected by the combined influence of shape, angularity, and surface texture of aggregate particles Janoo (2004). Tests were conducted only on select milled RAP samples that were also subjected to the moisture-density and LBR tests. The particle index was determined by measuring the bulk density of loose aggregate in different fractions and under different compaction conditions. The procedure specifies the material be poured into a measure and tamped with a rod for three layers, like the dry rodded unit weight test. A straight 24 inch long steel rod was used, with a diameter of 5/8 inch and a hemispherical tip. The measure used was a modified Proctor compaction mold with 6 inch diameter and 7 inch height, resulting in capacity of approximately 0.115 ft<sup>3</sup>. The weight of the soil in the mold is then measured.

The particle index ( $I_a$ ) was calculated following the modified Michigan Test Method, using the formula shown in Equation 3.2. This method was established by the Michigan Transportation Commission (1983) and used by Janoo (2001) and Janoo (2004). For comparison, the particle index formula from ASTM D3398 is shown in Equation 3.3. Both formulas are calculated from the voids content after a compactive effort of 10 or 50 blows per layer ( $V_{10}$  or  $V_{50}$ ). The equation for voids content is shown in Equation 3.4, where  $W_{10}$  is the average weight of aggregates in the mold,  $G_{SB}$  is the bulk dry specific gravity, and  $V$  is the volume of the mold (0.115 ft<sup>3</sup>). Since no specific gravity tests were conducted on samples in this study, a  $G_{SB}$  value of 2.2 was used for all RAP samples.

$$I_a = 0.983 \times V_{10} - 30 \quad \text{Equation 3.2}$$

$$I_a = 1.25 \times V_{10} - 0.25 \times V_{50} - 32 \quad \text{Equation 3.3}$$

$$V_{10} = \left[ 1 - \left( \frac{W_{10}}{G_{SB} \times V} \right) \right] \times 100 \quad \text{Equation 3.4}$$

The modified Michigan Test Method differs from the ASTM method by calculating the particle index from the density after 10 drops per layer, instead of the density after both 10 and 50 drops per layer. The Michigan Transportation Commission (1983) report concluded the particle index can be more quickly determined and may be subject to less variation if based directly on the density obtained by 10 compactive blows rather than by adding the additional tests at 50 compactive blows for the ASTM equation.

The modified Michigan Test Method also differs from ASTM D3398 in that the particle index is calculated for two size fractions, coarse and fine. The Michigan Transportation Commission (1983) report concludes that testing aggregate with the total mixture of coarse particles will produce the same relative

values as testing each individual fraction required in the ASTM method. The coarse fraction included all material passing the 3/4 inch sieve (9.5 mm) and retained on the #4 sieve (4.75 mm). The fine fraction included all material passing the #4 sieve (4.75 mm) and retained on the #30 sieve (0.60 mm). All material retained on the 3/4 inch sieve and passing the #30 sieve was discarded.

Approximately 80 lbs of material was required for the test, with minimum 20 lbs of each size fraction required. The sample particle index was calculated based on a weighted average of the two size fraction, based on the percent retained and passing the #4 sieve in the sieve analysis. Five trials were conducted on each sample. Previous studies using the modified Michigan Test Method are shown in Table 3.10, comparing the typical particle index values for crushed and natural aggregates.

**Table 3.10 Typical values of particle index**

Reference	Location	Type of Material	Particle Index	
			Average	Range
Janoo 2001	Vermont	100% Crushed Aggregate	8	N/A
		50% Crushed Aggregate	6	N/A
		Natural Rounded Aggregate (0% Crushed)	3	N/A
Michigan Transportation Commission 1983	Michigan	100% Crushed Aggregate	11	9 - 14
		Natural Rounded Aggregate (0% Crushed)	5	3 - 7

### 3.4.2.5 Moisture Density

The modified Proctor compaction test was conducted on most milled RAP samples according to FM 1-T180, “Moisture Density Relations of Soils Using 10-lb. Rammer and an 18-inch Drop.” RAP samples were size-modified to remove all particles retained on the 3/4 sieve (9.5 mm). These particles were replaced with an equal weight of particles passing the 3/4 sieve (9.5 mm) and retained on the #4 sieve (4.75 mm). The proportion of particles passing the #4 sieve (4.75 mm) did not change. The RAP material was loaded in bins and thoroughly mixed with water to achieve target moisture contents. Target moisture contents were standardized for all samples to be able to directly compare compaction characteristics at intervals of 4%, 6%, and 8%. Additional tests were conducted at interval of 2%, 3%, 5%, 9% or 10% if the initial intervals were not conclusive. An air-dry sample, at or near 0% moisture content was also compacted. The bins were sealed and the material was allowed to hydrate for a minimum 12 hours. Before compaction, the material was mixed again to ensure a balanced distribution of



particle sizes and moisture content. Samples were compacted in cylindrical molds with a 6 inch diameter and 4.59 inch height for five equal layers with 56 blows per layer. The compactive effort exerted during modified Proctor test was 56,000 ft-lb/ft<sup>3</sup>. Compaction was performed using a mechanical compactor model M100, manufactured by Ploog Engineering Company, Inc. A comparison to the manual hammer technique showed the mechanical compactor achieved an insignificant difference of 0.6% from the manually performed process.

#### **3.4.2.6 Bearing Strength (Limerock Bearing Ratio)**

The strength of RAP, A-3 and RAP-soil mixtures was determined using LBR test designated as FM 5-515. LBR testing was performed on the samples produced from moisture-density tests after compacting and weighing the molds with material. Samples were not soaked in water prior to testing because RAP has not shown any tendency to swell or bulk. Also, according to FDOT Memorandum (Malerk and Sadler, 2005), RAP is not permitted to be placed below the water table in earthwork applications. No surcharge weights were applied, since the minimum possible LBR value at optimum moisture content was of interest.

#### **3.4.3 FDOT Project Data**

To supplement laboratory testing, project data from several FDOT sources will be compiled for comparison. Laboratory test procedures evaluate a combination of variables to analyze the applicability of RAP samples in earthwork applications. FDOT project data will be used to analyze the relationship between RAP samples and field data from projects using RAP.

Samples of RAP collected for laboratory testing represent the condition of RAP at an asphalt plant, before and after processing for recycling into new HMA. FDOT project data will measure RAP properties in two other conditions, the in-situ condition before reclamation and a post-processing reuse application. The in-situ condition of RAP will be evaluated in composition reports from existing pavements on state highways. The reuse application evaluated will be base course under shoulders.

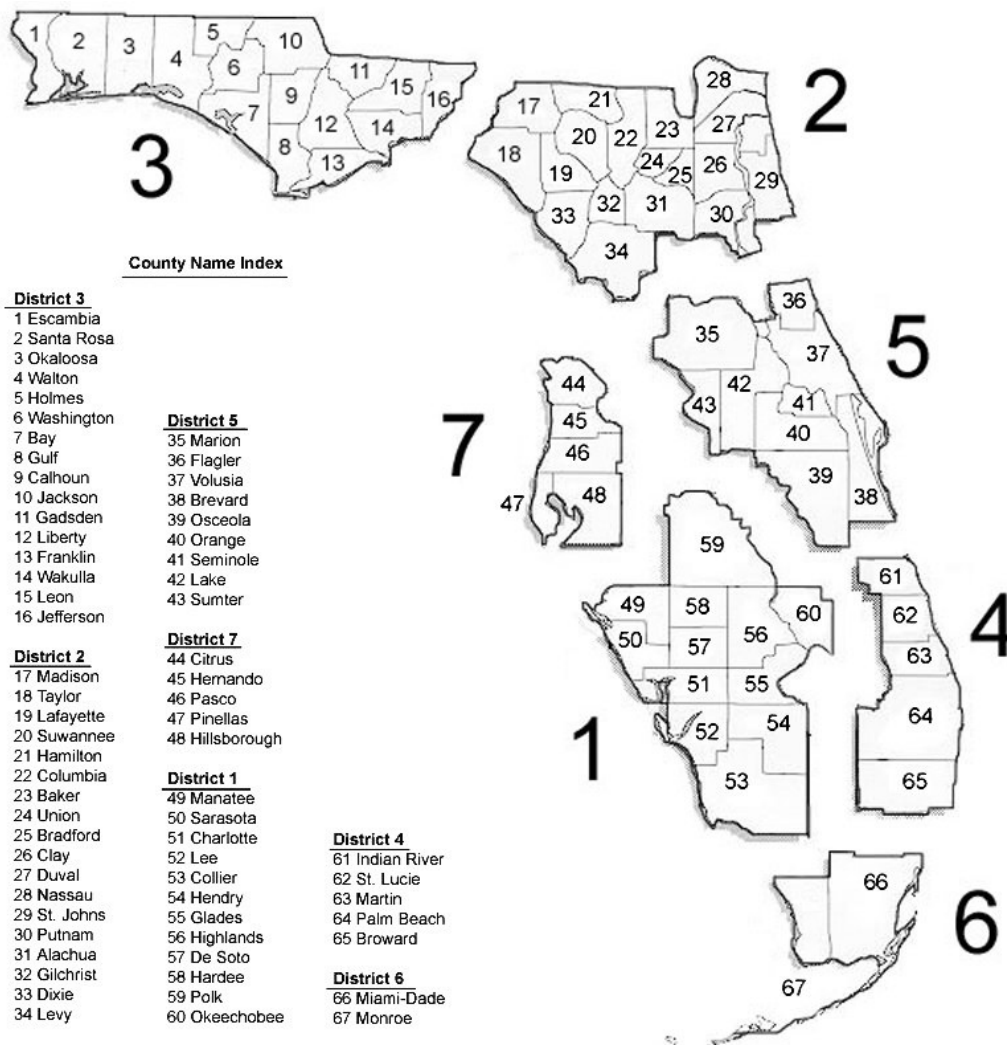
##### **3.4.3.1 Existing Pavement Composition**

Pavements in existing state highways are tested before they are removed and resurfaced to determine the proportion of RAP from that source can be recycled in new HMA. Pavement composition reports are conducted by FDOT District staff and compiled by the Central Asphalt Laboratory at FDOT SMO. Reports are compiled from projects throughout the state and include projects dated from 2007 to 2000, providing a large source of data to develop regional comparisons of existing asphalt pavements. A major source of RAP is milling and resurfacing on state highways, therefore the materials tested in these reports represent a large portion of candidate RAP materials in Florida. In addition, it is a reasonable assumption that any regional characteristics observed in the composition of state

highway pavements should manifest in other pavement sources of RAP. While laboratory asphalt content tests will evaluate the composition of collected RAP samples, pavement composition reports will be used to evaluate the variability in asphalt content in a much larger statewide sample population.

Project information will be used to identify, categorize and analyze the test results for the pavement each road segment. Identification information recorded includes state road number, federal road number (if applicable), financial project number (FPN), federal aid project number (FAP), county, beginning and end mile posts, beginning and end locations, number of lanes, and proposed depth of milling. State road numbers will be used to identify and categorize road segments based on type. Counties will be used to categorize road segments by region. Mileage and number of lanes will be used to calculate the number of lane miles, estimating the size of each road segment. Road segments administered by Florida's Turnpike Enterprise will be categorized by the county and District located in.

Reports are conducted at variable frequencies based on the roadway width, length, and number of intersections; the average sampling frequency ranges from up to two for every 300 ft. to only one per project. Pavement composition samples, or cores, are dug through all surface and structural pavement courses. Each pavement layer is identified and the thickness is recorded. Given the entire core is then tested for asphalt content and extracted aggregate gradation, the properties of each core represent the combined properties of all the layers on the existing surface. Likewise, all layers are combined during reclamation; RAP will also include constituent parts from each layer in the existing pavement. Therefore, the properties of each core will approximate the properties of RAP reclaimed from that pavement. Results recorded for each core include layer type, layer thickness, aggregate gradation and asphalt content. Typically, all cores are measured for layer type and thickness, while fewer cores are tested for asphalt content and aggregate gradation. This study will only analyze the asphalt content results for each project.



**Figure 3.6 Map of FDOT districts and county names**

For reference, the county name and FDOT District location for each county are shown in Figure 3.6. Pavement composition results were available from projects in 62 of Florida's 67 counties. FDOT District 3, representing the largely rural region of northwest Florida, had four counties with no projects available; Santa Rosa, Calhoun, Liberty and Franklin. Additionally, Hernando County, in District 7, also had no project data available for analysis. In summary, 496 projects were analyzed, representing 2,673 centerline miles and 8,508 lane miles of highway. This represents the pavement of approximately 20% of Florida's existing state roads, as measured by the number of lane miles.

**Table 3.11 Projects analyzed for pavement composition**

FDOT District	Pavement Composition Projects			Total State Road Lane Miles (2006)	Proportion of State Road Lane Miles Represented
	Number of Projects	Total Centerline Miles	Total Lane Miles		
1	56	310	1,008	5,944	17%
2	172	937	2,997	8,062	37%
3	56	258	812	6,548	12%
4	66	231	881	6,084	14%
5	101	640	1,998	7,727	26%
6	22	214	522	2,925	18%
7	23	85	291	4,324	7%
<b>Total</b>	<b>496</b>	<b>2,673</b>	<b>8,508</b>	<b>41,614</b>	<b>20%</b>

Table 3.11 shows a summary of projects analyzed for pavement composition by FDOT District. Pavement composition reports contained a large number of projects from District 2, representing 37% of lane miles within that region. Notably, District 7, including the urbanized region north of Tampa Bay, was underrepresented, with only 7% of total lane miles analyzed in the pavement composition reports. See Appendix C-1 for a complete list of projects analyzed, categorized by county and District.

### 3.4.3.2 RAP Base

Base course under shoulders is one application where RAP is currently approved for use by FDOT under Section 283 of FDOT Standard Specification (FDOT, 2007; FDOT, 2004). To approve the use of RAP in shoulder base course construction, modified Proctor compaction tests are conducted on field samples following FM 1-T180. Construction contractors will submit base course density reports to FDOT District staff for approval; these reports are then compiled by the State Materials Office (SMO). Reports were compiled from projects using RAP in shoulder base dating back to 2004. The use of RAP as shoulder base is not common throughout the state; projects were primarily located in Districts 2, 3, and 5. In addition, a small number of projects were also located in Districts 4 and 7. In total, data was collected from 27 projects, representing 101 centerline miles of highway. Although data was not available from throughout the state, the RAP tested in these specific projects are field samples subjected to similar compaction conditions tested as the laboratory samples, therefore allowing direct comparisons. In addition, these projects are located in several Districts across the state and may indicate regional differences.

Identification information recorded for each sample includes highway number, financial project number (FPN), sample number, sample ID code, sample type (verification or quality control), as well as station and offset location. Results recorded for each sample include maximum density, optimum moisture content, and percent passing the #200, #4, and 3.5 inch sieves. Only 12% sample results indicated a value for the percent passing the #200 sieve. Despite the lack of test data, it is assumed that samples still met the specification at #200. Density values were measured to the nearest 1 lb/ft<sup>3</sup> and moisture content values were measured to the nearest 1%.

## 4 Presentation and Discussion of Results

### 4.1 Basic Engineering Properties for Lab Creep Evaluation

#### 4.1.1 Content and Viscosity

Asphalt content is a property of RAP influenced by the properties of the old pavement from which it was reclaimed. The type of mix used in the old pavement, the quality of the old pavement, the number of times it has been resurfaced, all influence the asphalt content of RAP. In addition, the presence of granular material, soils or debris in the RAP stockpiles, as well as the intermixing of material from different sources also affect the asphalt percentage.

The asphalt content by weight of RAP was determined by the ignition method (FM 5-563). Two samples, approximately 1500 g each, were tested and the average asphalt content was  $5.8 \pm 0.08\%$ . Based on this laboratory asphalt content for the 100% RAP, estimates of the asphalt percentage contained in the RAP-soil mixtures were determined and are presented in Table 4.1.

**Table 4.1 Relative asphalt content of RAP and RAP-soil mixtures**

Mixture	100% RAP	80% RAP	60% RAP	40% RAP	20% RAP	A-3 Soil
Relative Asphalt Content	5.8%	4.6%	3.5%	2.3%	1.2%	0%

The asphalt content determined for 100% RAP was within the typical range of 4.5 to 6% for most wearing surface mixes as specified by FHWA (1998). The minimal difference between the tested asphalt content and the upper limit of the typical range could be attributed to the fact that the RAP used in this study was directly from milling. An analysis of the numbers reported by various researchers showed that milled RAP generally contains a higher asphalt percentage than the crushed RAP, and some milled RAP possesses asphalt contents as high as 7%. Montemayor (1998), for instance, reported 5.8% for crushed RAP and 6.7% for milled RAP. Testing RAP from different sources produced asphalt contents between 5 and 5.2% for crushed RAP and between 5.5 and 5.9% for milled RAP. It was reasonable to believe that the processing techniques associated with crushing increased the disintegration or crumbling of the age hardened and brittle binder. The additional fines created through crushing were partially lost to dust during the handling of the crushed material and consequently the asphalt content was reduced.

According to FHWA (1998), the absolute viscosity values at 140°F for asphalt cement recovered from RAP may range from 4,000 to 25,000 poises. This

increased viscosity of the asphalt cement is caused by changes in the chemical composition of the asphalt, termed age hardening. Asphalt cement is a combination of viscous asphaltenes and less viscous maltenes. The oxidation processes with atmospheric oxygen during the service life of asphalt cause transformation of maltenes to asphaltenes, leading to progressive hardening and increased viscosity (FHWA, 1998). The recovered viscosity of the RAP in this investigation yielded a result of approximately 24,000 poises, which was towards the upper limit of the range given by FHWA, thus indicating a highly aged material.

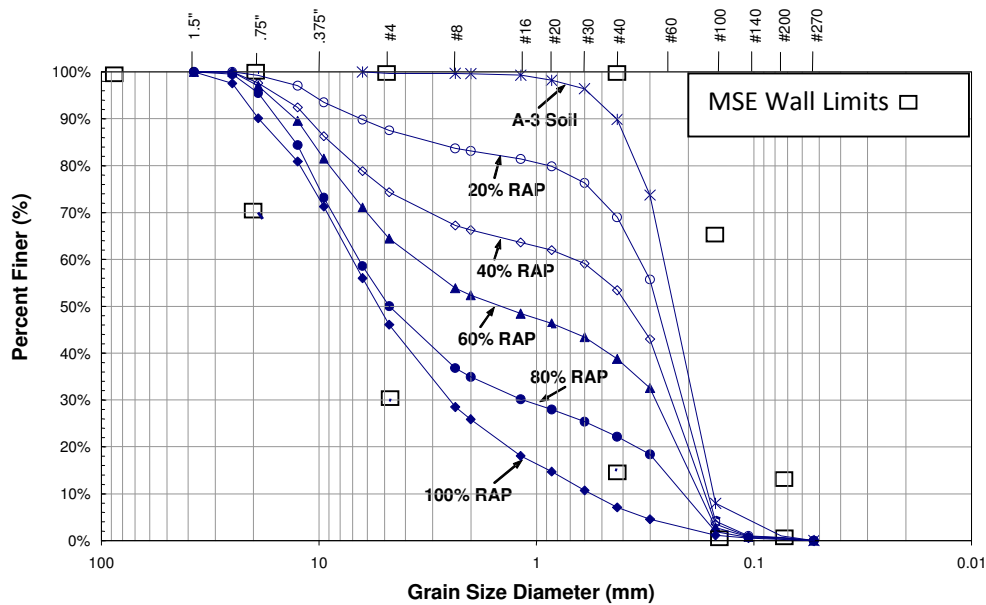
#### 4.1.2 Grain Size Distribution

Sieve analyses were conducted on RAP, RAP-soil mixtures, A-3 soil, and on aggregate extracted from RAP by the ignition method. The extracted aggregate, (EA), was tested before and after creep to determine if any degradation had occurred during the creep testing. The results are presented in Figure 4.2 with a summary of the gradation properties given in Table 4.2. The grain-size distribution of 100% RAP was that of a well graded coarse material with 100% passing 1½ inch sieve and 0.3% fines. According to AASHTO it was classified as A-1-a. These gradation parameters agreed with the values from the literature summarized in Table 2.1.

**Table 4.2 Summary of gradation parameters**

	100%	80%	60%	40%	20%	A-3 Soil	EA before creep	EA after creep
Percent passing #4	46.0%	50.0%	64.4%	74.4%	87.6%	99.7%	65.6%	72.5%
Percent passing #10	25.9%	34.9%	52.4%	66.3%	83.1%	99.6%	42.6%	52.2%
Percent passing #40	7.1%	22.2%	38.7%	53.4%	69.0%	89.9%	25.8%	33.7%
Percent passing #200	0.3%	0.4%	0.6%	0.6%	0.7%	1.0%	3.0%	4.4%
D <sub>10</sub> (mm)	0.54	0.21	0.18	0.17	0.17	0.155	0.19	0.17
D <sub>30</sub> (mm)	2.50	1.18	0.28	0.24	0.21	0.2	0.60	0.33
D <sub>60</sub> (mm)	7.00	6.50	3.50	0.64	0.32	0.26	4.00	3.00
Cu	13.0	31.0	19.4	3.8	1.9	1.7	21.1	17.6
Cc	1.7	1.0	0.1	0.5	0.8	1.0	0.5	0.2
<i>USCS Classification</i>	<i>GW</i>	<i>SW</i>	<i>SP</i>	<i>SP</i>	<i>SP</i>	<i>SP</i>	<i>SP</i>	<i>SP</i>
<i>AASHTO Classification</i>	<i>A-1-a</i>	<i>A-1-a</i>	<i>A-1-b</i>	<i>A-3</i>	<i>A-3</i>	<i>A-3</i>	<i>A-1-a</i>	<i>A-1-b</i>

The 100% RAP gradation satisfied the requirements for maximum particle size and content of fines of Section 120 from the Standard Specification for embankment construction. When compared with the gradation requirements from Section 548 of the Standard Specification for backfill material behind MSE walls, however, the 100% RAP curve partially fell below the lower gradation limit within the range between sieves #10 and #200, Figure 4.1. This gradation curve resembled the grain-size distribution of 100% RAP obtained by Cleary (2005) (See Figure 2.5) and confirmed the observation made by other researchers (Garg and Thompson, 1996) that RAP generally lacks smaller size fractions.

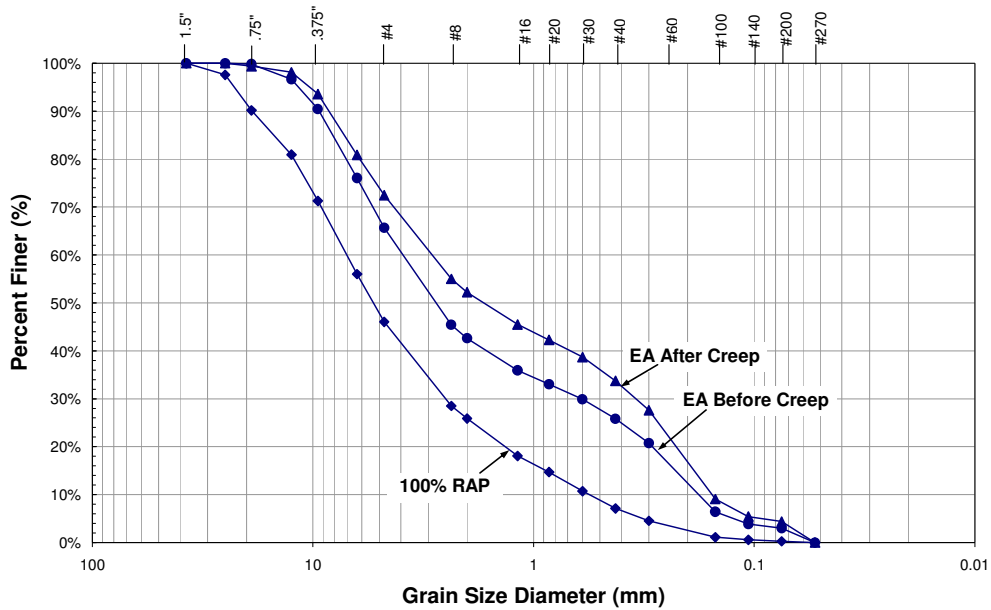


**Figure 4.1 Grain-size distributions of RAP, RAP-soil mixtures, and A-3 soil**

The A-3 soil is uniformly graded with particle sizes predominantly between sieves #10 and #100. The addition of A-3 to RAP significantly modified the grain-size distribution of the resulting blends. The grain-size distribution of these blends made up the deficiency of fractions between the #10 and 100 sieves and repositioned the curves to within the MSE wall gradation limits (FDOT Standard Specification Section 548 2008). The percentage of material passing the #4 sieve increased from 46% for 100% RAP to 87.6% for the 20% mixture. The gradation changed from well graded gravel for 100% RAP to well graded sand for the 80% mixture and to poorly graded sand for the remaining mixtures. The AASHTO classification remained A-1-a for the 80% mixture, but changed to A-1-b for the 60% mixture and to A-3 for the mixtures containing higher proportions of soil.



Since the A-3 soil was clean sand with little or no fines, the percentage of material passing #200 sieve did not increase significantly for the mixtures, maintaining a value of about 1%.



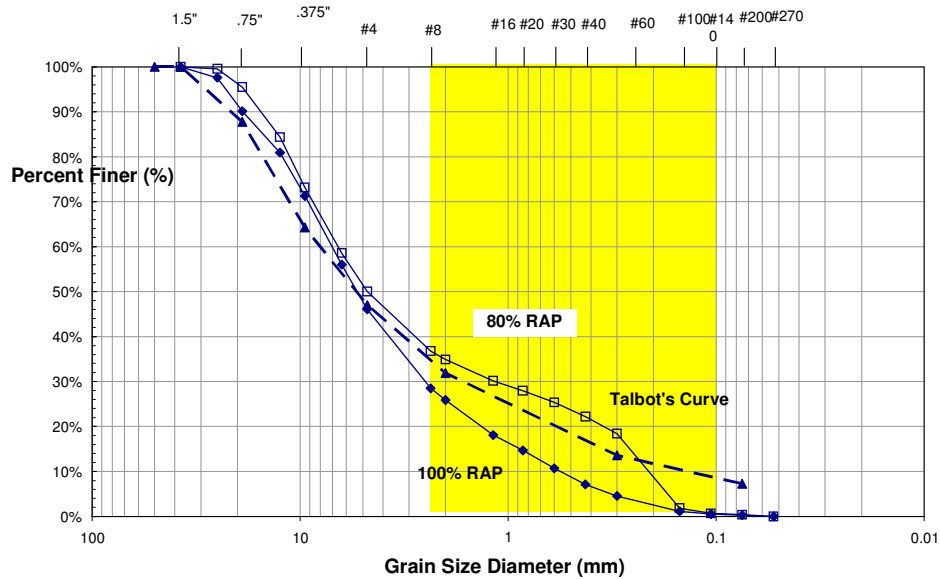
**Figure 4.2 Extracted aggregate (EA) gradation before and after creep as compared to 100% RAP**

Figure 4.2 depicts the change in the gradations between 100% RAP and one EA sample obtained from RAP after the asphalt cement was extracted. The before creep EA curve represents the gradation before compacting the sample and subjecting it to creep and therefore should reflect the changes due to asphalt from extraction only. The after creep EA curve demonstrates the changes due to compaction and creep testing. Most of the modification to the gradation took place before sample preparation and creep testing. The before creep EA displayed a significantly different grain-size distribution from RAP, with the characteristics of a poorly graded fine material. The percentage of material passing #40 sieve increased from 7% in 100% RAP to 26% in EA. This gradation change could be attributed to the asphalt cement removal and to the applied ignition extraction method. Originally the asphalt binder in RAP produced agglomerations of particles, which in its absence decomposed into smaller fractions. In addition, the asphalt coating increased the particle size, while the asphalt binder caused finer particles to adhere to the surface of larger ones thus increasing their diameter. After extraction such particles disintegrated. According to Sondag et al, (2002) the reduction of grain size before creep could also be partially attributed to the asphalt extraction ignition method, whose high temperatures lead to mass loss or aggregate breakdown.

In Figure 4.3, the 100% RAP grain-size distribution curve was plotted next to a Talbot curve (Barksdale, 1991). Talbot's gradation produces a soil of higher density which is well graded. Talbot's equation is as follows:

$$P = \left( \frac{d}{D} \right)^n 100 \quad \text{Equation 4.1}$$

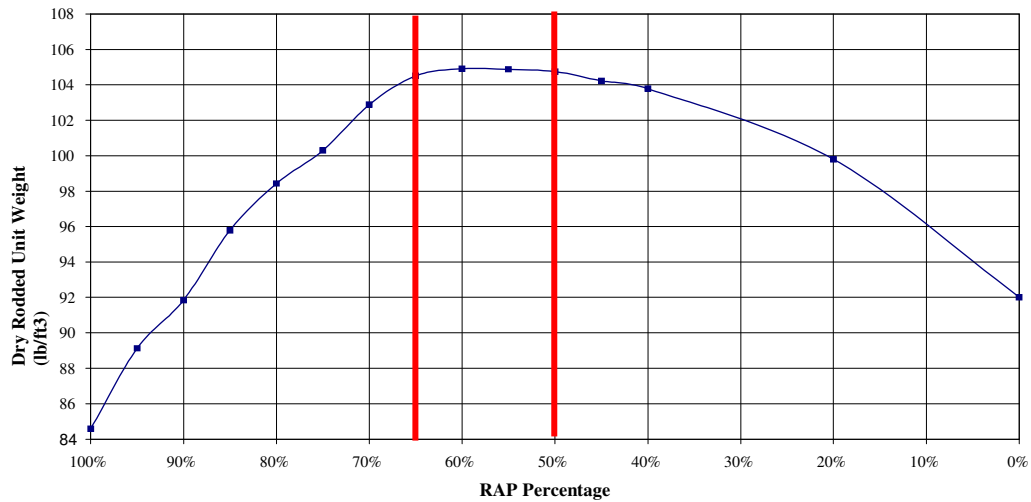
where P is the percent passing sieve size "d", d is the sieve size opening expressed in millimeters for the associated percent passing (P), D is the maximum particle size in millimeters, and n is an empirical gradation exponent with a typical value of 0.45. Depending upon the maximum particle size various Talbot curves can be generated.



**Figure 4.3 Comparison of 100% RAP gradation curve and Talbot's Curve**

The 100 and 80% curves in Figure 4.3 show that the 100% RAP has a smaller percent passing the sieves below the #8 while the 80% RAP contains enough A-3 sand to allow it to closely follow the Talbot curve. Hence 100% RAP deficiency of smaller particles prevents it from being well graded. This characteristic of the RAP had been observed by Mokwa and Peebles (2005) as well.

In conclusion, based on the evaluation of the Figure 4.1 gradation curves the 100% and 80% curves are the closest to Talbot's optimum gradation, with the 80% curve producing a well graded blend. Adding A-3 soil in excess of 20% increased the distance of the curves from the optimum grain-size distribution.



**Figure 4.4 Variation in dry rodded unit weight of RAP-soil mixtures**

### 4.1.3 Dry Rodded Unit Weight

Dry rodded unit weight testing was used as a less material consuming approach to evaluate the variations in unit weight of RAP blends. The results from this test served as an indicator of the optimum blend for subsequent laboratory testing.

The test was conducted starting with a 100% RAP and adding 5% increments of sand to obtain the desired RAP-soil blends. The unit weights were plotted against the RAP content in Figure 4.4. In the figure, as well as in the following discussion the A-3 soil was designated as 0% RAP mixture for simplicity of the data presentation.

The dry rodded unit weight reached a peak at 60%. A maximum dry rodded unit weight of 104.9 lb/ft<sup>3</sup> was achieved by the mixtures containing between 55 and 60% RAP. This value represents a 24% increase in the unit weight of RAP and a 14% increase in the unit weight of the A-3. The maximum unit weight compares closely with Cleary's (2005) maximum of 103.5 lb/ft<sup>3</sup> obtained from a mixture of crushed RAP with A-3 soil at 85% RAP content. Cleary's (2005) dry rodded unit weight for 100% RAP of 95.2 lb/ft<sup>3</sup> was somewhat higher than the value of 84.6 lb/ft<sup>3</sup> obtained during this research. Testing conducted by MacGregor et al (1999) on 100% RAP resulted in a closer value of 93 lb/ft<sup>3</sup>. It is likely that the variations in the unit weight numbers are generated by differences in the gradation, aggregate type, and the processing means of RAP.

The 100% RAP yielded a lower dry rodded unit weight than the A-3 soil. The coarser gradation of RAP and the presence of particle agglomerations created a higher volume of voids within the material. In addition, the weight of RAP was reduced by the presence of asphalt cement having a specific gravity of

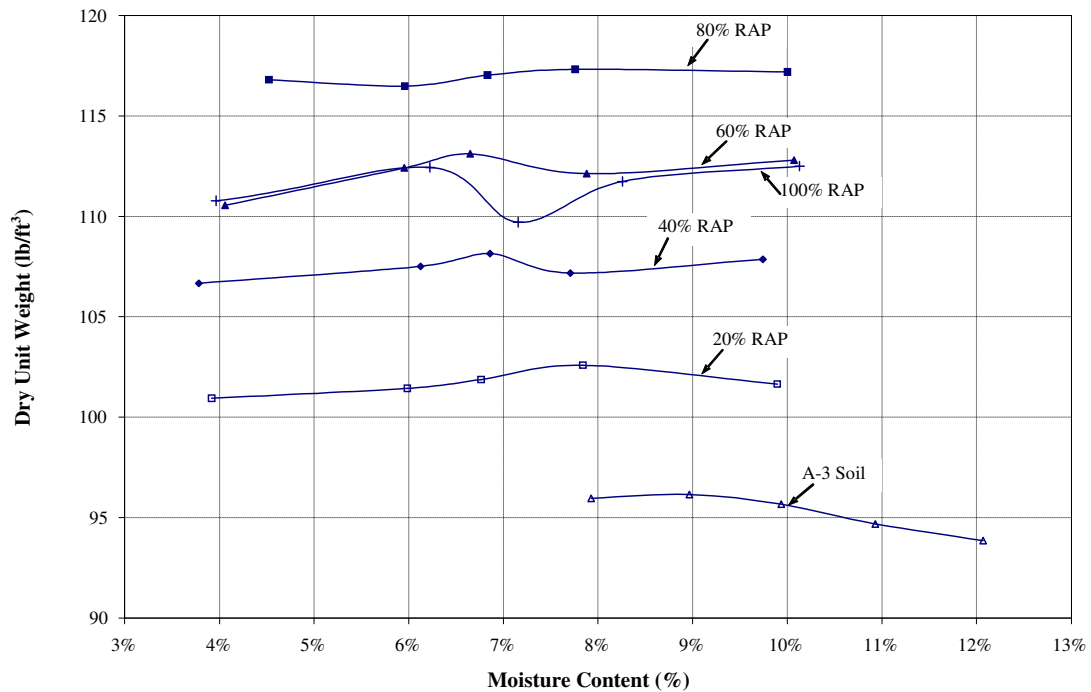
approximately 1, while typically the specific gravity of most aggregates used in hot mix applications range between 2.6 and 2.7 (Holtz and Kovacs, 1981).

#### 4.1.4 Moisture-Density

The compaction characteristics of RAP, RAP-soil mixtures and A-3 soil were determined from modified Proctor testing. A summary of the maximum dry unit weights and the optimum moisture contents is given in Table 4.3. The corresponding moisture-density relationships are presented in Figure 4.5.

**Table 4.3 Summary of moisture-density parameters from modified Proctor compaction of RAP, RAP-soil mixtures and A-3 soil**

Mixture	Maximum Dry Unit Weight (lb/ft <sup>3</sup> )	Optimum Moisture Content (%)
100% RAP	112.44	6.2
80% RAP	117.33	7.8
60% RAP	113.12	6.6
40% RAP	108.16	6.9
20% RAP	102.60	7.8
A-3 Soil	96.16	9.0



**Figure 4.5 Moisture-Density Relationships of RAP, RAP-Soil Mixtures, and A-3 Soil**

A typical compaction curve for cohesive soils is parabolic in shape and includes a well defined peak where the maximum dry unit weight is reached at the optimum water content. The curves displayed on Figure 4.5 exhibited shapes representative of cohesionless soil behavior and are relatively flat with unpronounced peaks. Cohesionless soils do not respond to variations in compaction moisture content because their soil structure is less sensitive to moisture than the structure of cohesive soils. Often, the moisture drains rapidly to the bottom of the mold before the test specimen can be weighed and its moisture content determined (Barksdale, 1991). It is not uncommon that in cohesionless soils a maximum dry unit weight is obtained by dry or completely saturated samples (Lambe & Whitman, 1969). The moisture contents in Table 4.3 data confirmed the insensitivity of RAP-soil mixtures to water. The largest maximum dry unit weight of 117.3 lb/ft<sup>3</sup> was achieved by the 80% RAP at approximately 8% moisture content. The addition of more than 20% A-3 soil decreased the maximum dry unit weight of the mixtures. The compaction curves of the mixtures gradually shifted down and to the right acquiring a position and parameters closer to those of the A-3 soil curve, which peaked at 9% moisture content yielding a maximum dry density of 96.2 lb/ft<sup>3</sup>.

The maximum dry unit weight for 100% RAP was 112.4 lb/ft<sup>3</sup> and was obtained at approximately 6% moisture. This value compared well to some values reported by Cleary (2005), Gomez (2003), and Taha et al. (1999), and was identical to the value obtained by Montemayor (1998) at the same compaction effort, as listed in Table 2.1. The 100% RAP compaction curve displayed a unique undulating shape, encountered also by Montemayor (1998) and Cleary (2005). This unusual compaction behavior could be attributed to the high drainability of RAP and its failure to hold moisture due to deficiency of fines in its gradation and the asphalt coating covering the particles. During the compaction process, water drained from the bottom of the mold distorting the moisture content and consequently the dry unit weight. This effect gradually diminished with increasing proportions of A-3 soil. The problem of moisture migration and loss was also encountered while attempting to obtain samples for moisture content determination immediately following compaction. The conventional approach of collecting samples from the material in the mold resulted in erratic data. This problem necessitated the collection of moisture content samples directly from material in the bins at several samplings during compaction.

Compared to A-3 soil compacted at the same effort, 100% RAP produced a 17% higher maximum dry unit weight. The 80% RAP mixture, however, demonstrated a dry unit weight increase of approximately 4% and 22% compared to 100% RAP and A-3 soil, respectively. This finding confirmed that the addition of A-3 soil improved the dry density together with the gradation of RAP.

The moisture-density results did not coincide with the findings from dry rodded unit weight test that the 60% RAP mixture possessed the optimum RAP-soil gradation. It should be emphasized, however, that the procedures for the two tests differ significantly, the main difference being the presence of moisture in one of the tests and absence in the other, as well as the level of compaction energy, which allowed for an additional yield of the particles in the 80% RAP.

#### **4.1.5 LBR Before Creep**

LBR is a test method used to evaluate the potential soil strength of base, subbase and subgrade material in highway construction. The FDOT Standard Specifications require aggregate to exceed a minimum LBR values specified according to the application. For instance, base materials are required to show a minimum LBR of 100, while subbase and subgrade materials should have LBR of at least 40.

In this study LBR was employed to assess and compare the load-bearing capacity of RAP and RAP-soil mixtures before and after experiencing long-term deformations. The unsoaked method FM 5-515 was used to expedite testing. Some of the factors influencing the LBR values are maximum particle size, gradation, and density. Materials containing larger particles are expected to exhibit higher LBR values. A proper material gradation defined by good representation of various

grain-size fractions typically also yields an increased LBR. When the density of a granular material is improved by compaction, the internal friction between particles increases and a higher LBR is achieved.

The strength of RAP, RAP-soil mixtures and A-3 soil before creep as measured by unsoaked LBR testing are summarized in Table 4.4. LBR was conducted on the sample with the highest dry unit weight during moisture-density testing for each mixture. Thus the samples were tested at 100% relative compaction. The maximum LBR value of 19 was achieved by the 80% RAP mixture and represented an increase in LBR of 58% and 36% relative to the 100% RAP and A-3 soil respectively. The strength of the 40%, 60%, and 80% RAP mixtures was not influenced significantly by the difference in the densities as they yielded very similar LBR values.

**Table 4.4 Summary of unsoaked LBR values for RAP, RAP-soil mixtures, and A-3 soil before creep testing**

Compaction procedure	Modified Proctor					
Compactive effort (ft-lb/ft <sup>3</sup> )	56,000					
Mixture	100% RAP	80% RAP	60% RAP	40% RAP	20% RAP	A-3 Soil
Unsoaked LBR	12	19	18	18	15	14

Compared to the LBR values reported in Table 2.1, the bearing strength of 12 yielded by 100% RAP was in the lower range. At the same compactive effort, similar values were obtained by Taha et al (1999) and Maher et al (1997), but other reported values were significantly higher. It should be noted, however, that sample preparation, mostly application of surcharge, while not always reflected in the technical papers could substantially improve the LBR results. The samples tested during this study were not pre-soaked or subjected to surcharge, as explained in Section 3.2.8. Additionally, the 100% RAP strength could have been affected by asphalt content and by the age of the material. The LBR of 14 achieved by the A-3 soil compared well with the lower limit of LBR range for natural sand (Barksdale, 1991) and exceeded the LBR of 100% RAP.

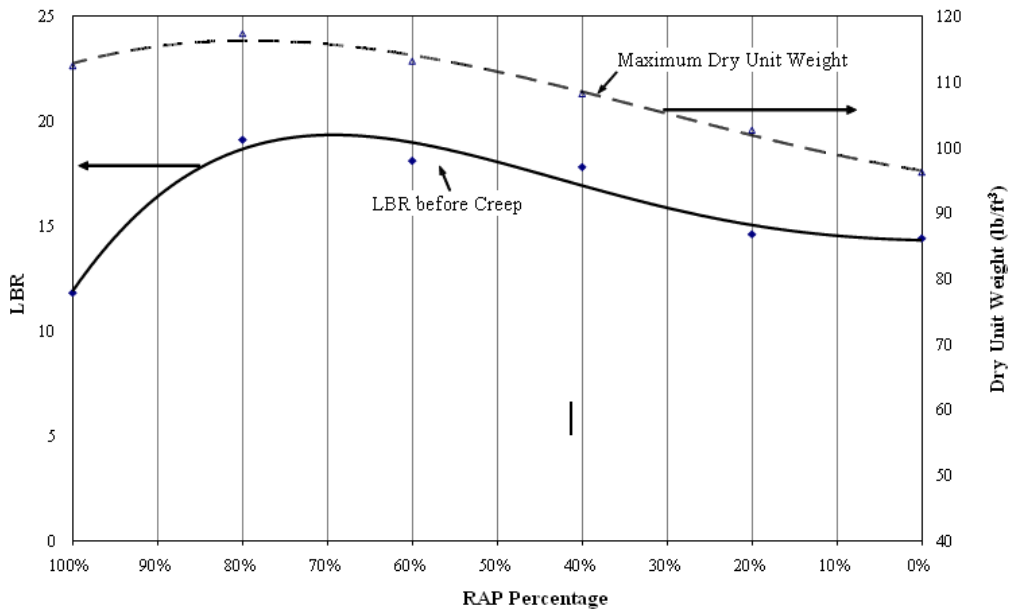
The factors influencing the bearing strength of the materials tested includes 1) maximum grain size, 2) overall gradation and 3) compaction. The maximum grain size of the mixtures appeared to have little effect on their strength. Based on its classification (see Table 4.2), the 100% RAP contained the highest percentages of largest particles, but demonstrated lower strength than the A-3 soil. Conversely, gradation, particularly fines content, could have had a substantial influence on LBR. The relatively low bearing strength demonstrated by both 100% RAP and A-3 soil, could be partially attributed to the insignificant amount of fines contained by the two materials. Investigations show that bearing strength of aggregates increases with increasing fines content until an optimum level is reached and later

decreases with amount of fines above that level (Barksdale, 1991). According to the Talbot curve, an optimum level of fines for 100% RAP should be 6%, and similarly for A-3 soil – 15%, whereas the actual percent of fines did not surpass 1% in neither material.

To evaluate the relationship between bearing strength and compaction, Figure 4.6 was developed. It depicts the LBR values and maximum dry unit weights versus RAP percentage. The two relationships demonstrated a similar behavior revealing that improved gradation leads to denser material of higher strength. Both curves peaked at the 80% RAP confirming that a maximum strength is achieved by the material of highest dry unit weight. The 80% RAP also compared well with the optimum gradation indicated by Talbot’s curve. Linear trends were observed between the peaks and 0% RAP content, (i.e. A-3 soil), indicating that the incremental addition of soil led to proportional reductions of the two properties.

## 4.2 Laboratory Creep Behavior

The creep behavior of RAP and RAP-soil mixtures was the focus of the experimental work in this study. The concern for long-term deformations has been a major obstacle to the use of RAP in earthwork applications. The magnitude of these long-term deformations, however, has not been quantified or limited by specifications. One method to reduce the creep in RAP is to mix with soil, yet the creep behavior of RAP-soil mixtures has been the subject of a very limited research.



**Figure 4.6 LBR before creep and maximum dry unit weight curves for RAP, RAP-soil mixtures, and A-3 soil**



## 4.2.1 Relative Compaction of Creep Samples

Following Cleary's (2005) recommendations, creep samples were prepared at controlled densities based on relative compaction (RC). For RAP, RAP-soil mixtures, and A-3 soil the RC was calculated as the ratio of the dry unit weight of a creep sample to the maximum dry unit weight from the moisture-density test for the specific mixture. To match FDOT criteria the goal of the creep compaction was to maintain a RC between 95 and 100% (Sections 283-4.3, 548-7.4, and 200-7.2.1 of the Standard Specifications). The mean and standard deviation RC values for three replicas at each stress level were summarized in Table 4.5. Appendix A-2 contains the creep testing data.

**Table 4.5 Quality control for RC of creep samples tested at 6, 12 and 18 psi stress levels**

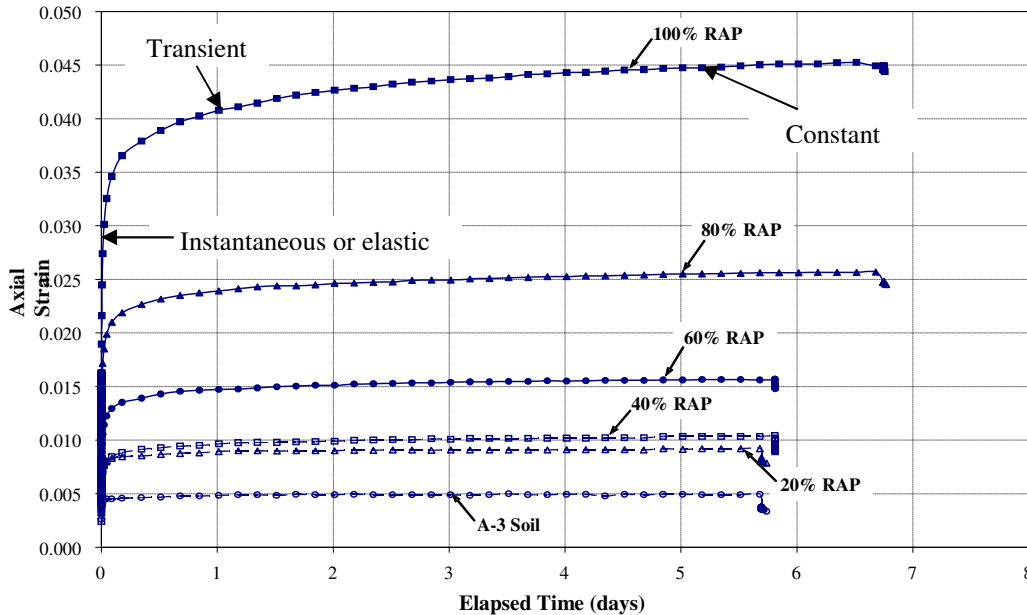
Mixture	RC @ 6 psi		RC @ 12 psi		RC @ 18 psi	
	Mean	St Dev	Mean	St Dev	Mean	St Dev
100% RAP	97.4%	0.4%	98.8%	0.1%	98.3%	0.7%
80% RAP	96.8%	0.1%	97.9%	0.4%	97.9%	0.5%
60% RAP	98.4%	0.7%	98.2%	0.4%	97.5%	0.7%
40% RAP	98.4%	0.9%	98.8%	0.3%	98.8%	0.6%
20% RAP	98.5%	0.6%	98.3%	0.2%	97.1%	0.2%
0% RAP	100.0%	0.6%	100.9%	0.4%	98.5%	0.7%
EA	96.5%		98.0%		96.0%	

During the actual compaction approximately 90% of the RC values fell within the narrower range between 97 and 100% of the modified Proctor maximum density. The targeted maximum difference between the individual RC values of each three replicas loaded at one of the stress levels was 2%, yet only one of the actual differences exceeded 1.5%.

## 4.2.2 Axial Strain

To investigate the long-term behavior of RAP and RAP-soil mixtures, three replicas of each mixture were loaded in the pneumatic one-dimensional oedometric (i.e. PLD) setup at stress levels of 6, 12 and 24 psi. Two replicas of A-3 soil and one sample prepared from extracted aggregate (EA) were loaded as control materials at each stress level. During creep testing the deflections experienced by

the samples with time were recorded and converted to axial strains. A plot of the average axial strain versus elapsed time for RAP, RAP-soil mixtures, and A-3 soil, at the 18 psi stress level is presented in Figure 4.7. The strain-time relationships for the mixtures at 6 and 12 psi, and a plot for the EA at all three stress levels are given in Appendix A-4.

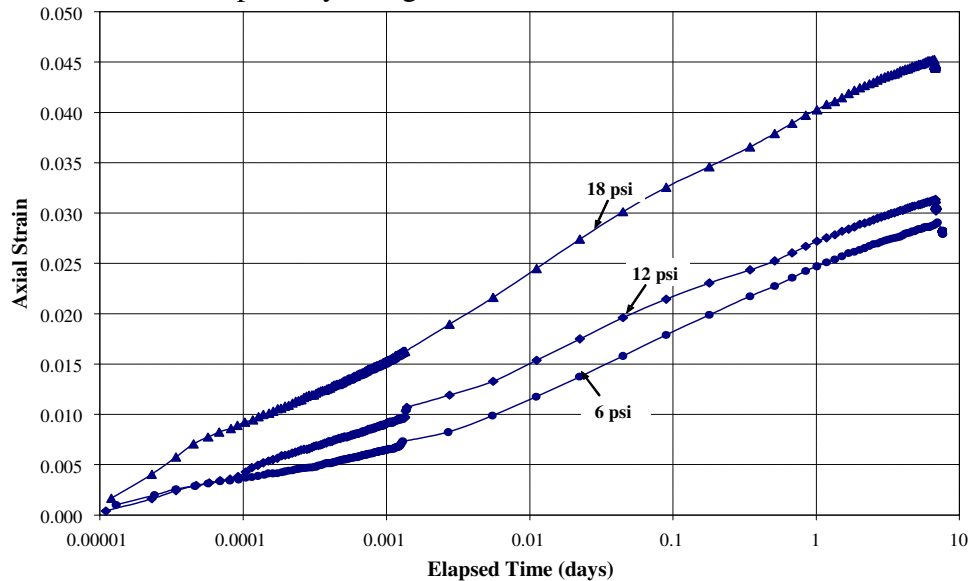


**Figure 4.7 Average axial strain vs. time relationship during creep of RAP, RAP-soil mixtures, and A-3 soil at 18 psi stress level**

The strain-time relationships in Figure 4.7 were characterized by three phases: initially a phase of instantaneous deformations occurs; followed by a transient phase, during which the strain rate progressively decreases with time; and finally a phase of constant or nearly constant strain rate (Mitchell, 1993). These components are displayed for the 100% RAP data in the figure, which also produced the highest overall strain, the largest instantaneous deformation, the longest transition, and the steepest slope of strain curve during strain rate decrease. A constant rate of strain in 100% RAP was not achieved for a period of more than one day. It was concluded that if the stress was maintained past 7 days, the deformations may have continued at an ever so slightly decreasing rate. The A-3 soil, used as control material, experienced only about 14% of the strain produced by the 100% RAP, exhibiting predominantly instantaneous deflection, short transition, and a leveling curve after the first day of loading. The RAP-soil mixtures displayed a behavior similar to the 100% RAP at higher RAP contents and gradually adopted the curve character of A-3 soil at lower RAP contents. The total strain decreased with increasing soil content with the most significant decrease occurring between 100% and 80% RAP. At the two lower stress levels of 6 and

12 psi, Figures A-4.1 and A-4.2 in Appendix A-4, similar behaviors were observed at respectively lower total strain and faster convergence to the steady state of constant strain rate.

To further evaluate the influence of stress level on strain-time, the 6, 12, and 18 psi strain curves for 100% RAP were plotted versus logarithm of time in Figure 4.8. The relatively steep slope for the 18-psi data indicates that it has a higher strain rate than the 6 and 12 psi. For the blends the slopes at the three levels had smaller differences, especially at higher A-3 contents.



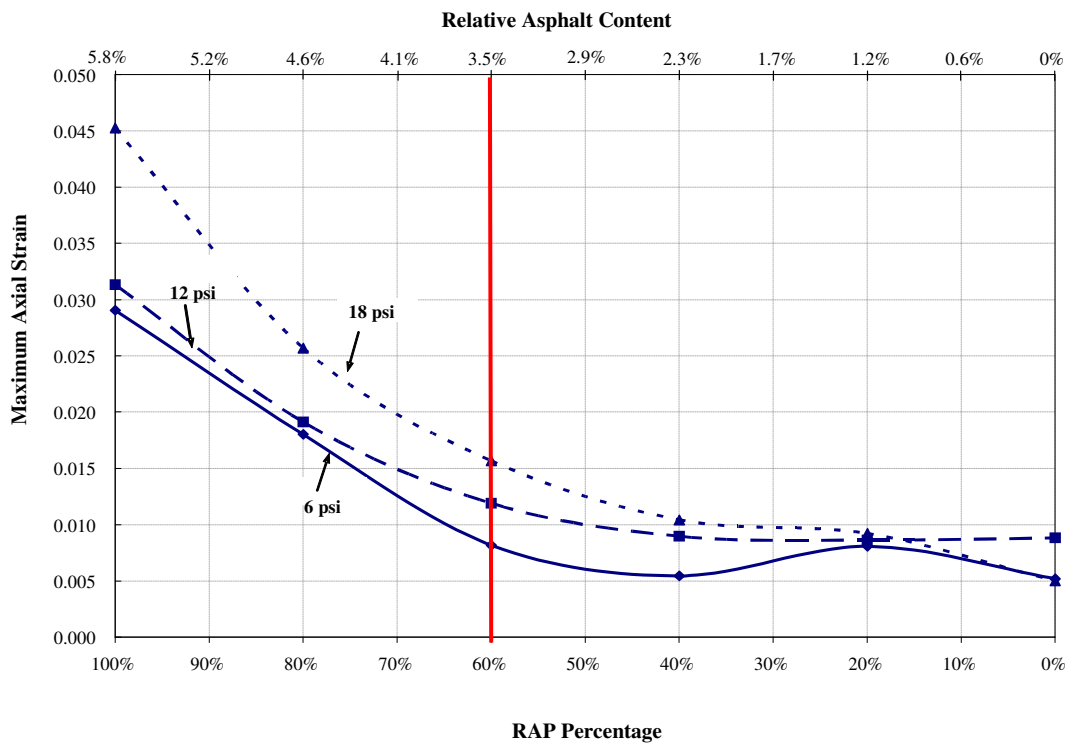
**Figure 4.8 Average axial strain vs. time relationships during creep of 100% RAP at 6, 12 and 18 psi stress levels**

The extracted aggregate displayed a behavior comparable to the A-3 soil with somewhat higher strain, Figure A-4.4, Appendix A-4. It should be noted that the ignition method used to obtain EA from RAP involved subjecting the aggregate to elevated temperatures. These temperatures alter the chemical and structural properties resulting in reduced soundness and possibly leading to higher strains.

The maximum axial strains achieved by the materials during loading at the three stress levels after 5 to 7 days were plotted in Figure 4.9 and yielded two general trends. The maximum axial strain decreased with decreasing RAP content and increased with higher stress level. The most significant finding from this figure, however, was the leveling mode of the curves below 60% RAP. Somewhere between 40 and 60% RAP the A-3 soil started controlling the creep behavior of the mixtures producing smaller long-term deformations regardless of the actual RAP content.

There were several means by which A-3 soil could have influenced the mixture behavior. First, A-3 improved the gradation of the mixtures producing a

material of higher density, as was the case with 80% RAP. Secondly, more than 40% A-3 soil produces a shift in gradation curves away from the recommended Talbot curve. The dry unit weights of the mixtures containing 60% RAP or lower did not show any improvement compared to the 100% RAP. Thirdly, adding A-3 soil influences the behavior of RAP by indirectly decreasing the asphalt content in the mixtures, (Table 4.1), and thus reduces the viscous deformations. As evident from Figure 4.9, the magnitude of axial strains in mixtures containing less than 60% RAP is similar irrespective of the particular RAP content and even of the stress level. Applying the values from Table 4.1, it was concluded that RAP blends with asphalt contents less than 3.5% would not undergo significant long-term deformations while loaded at comparable stress levels.

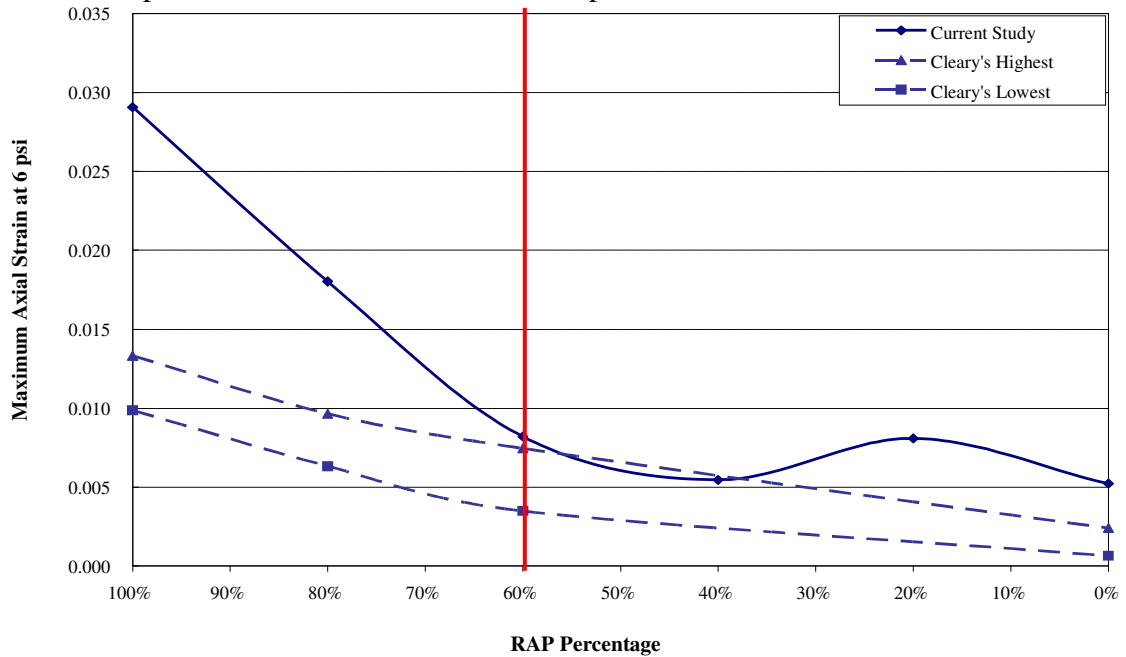


**Figure 4.9 Maximum axial strain of RAP, RAP-soil mixtures, and A-3 soil during creep at stress levels of 6, 12 and 18 psi**

Two data inconsistencies were revealed from inspection of Figure 4.9. At 6 psi the strain for 20% RAP exceeded the strain at 40% RAP. Also, the strain of A-3 soil at 18 psi was lower than the strain at 12 psi. In both cases the strains were rather small. It is believed that both inconsistencies could be attributed to the relative compaction of the particular samples. Comparing the RC numbers in the 6 psi column of Table 4.5 it was concluded that 20% RAP produced a slightly higher mean RC value than the 40%, but more importantly the 40% samples

registered the highest standard deviation indicating irregularity in the quality of compaction of the 40% samples. Analogously, a comparison between the values in the 0% RAP in

Table 4.5 revealed that the 18 psi samples achieved a significantly lower level of compaction than the 6 and 12 psi samples. This assessment of the data inconsistencies confirmed the conclusion reported by Cleary (2005) on importance of the compaction level to the collection of representative data.



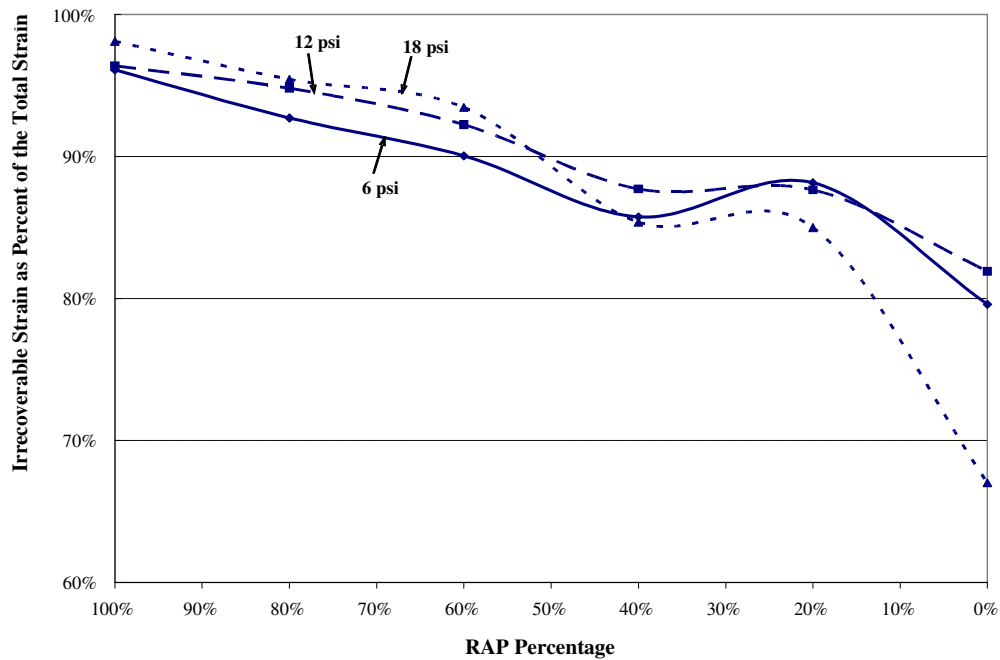
**Figure 4.10 Maximum axial strain comparison with Cleary (2005) of RAP, RAP-soil mixtures and A-3 soil tested at the 6 psi stress level**

In Figure 4.10 the average maximum axial strains were compared to Cleary's (2005) values. The comparison was only conducted at the 6 psi stress level, since after Cleary (2005) completed the 6 psi loading; the same sample was loaded to the higher two load increments, while in this study they were applied on new samples. The mixtures tested by Cleary (2005) contained 80%, 60%, and 0% RAP, whereas the remaining mixture strains were interpolated.

In Figure 4.10 the average maximum axial strain from the three replicas were compared with the highest and lowest axial strains for each mixture obtained from the three samples loaded by Cleary (2005). The axial strains at higher RAP contents differed and could be attributed to the testing methodology, the RAP properties, and possibly to the relative compaction. The 60% RAP values,

however, were nearly identical and the data from the two studies remained comparable for lower RAP contents.

After achieving a stable long-term axial strain or after seven days of sustained loading, the samples were unloaded. The displacement recorded upon unloading represented the permanent strain of the mixtures. Table A-3.2 in Appendix A-3 summarizes for 100% RAP, RAP-soil mixtures, A-3 soil, and EA the maximum attained axial strain during loading, the irrecoverable strain after unloading, and the ratio between them representing the proportion of irrecoverable strain to total strain in percent. Figure 4.11 depicts the irrecoverable strain in RAP, RAP-soil mixtures, and A-3 soil. In general, the irrecoverable strain decreases with decreasing RAP content while stress level had a very small effect on it.



**Figure 4.11 Irrecoverable strain of RAP, RAP-soil mixtures, and A-3 soil during creep at 6, 12 and 18 psi stress levels**

### 4.2.3 Strain Rate and Settlement Calculations

Strain rate calculations were based on the constitutive relationship proposed by Singh and Mitchell (1968) and presented as Equation 2.3 in Chapter 2.

$$\dot{\epsilon} = Ae^{\alpha D} \left( \frac{t_1}{t} \right)^m \quad \text{Equation 2.3}$$

This equation allows determination of the strain rate  $\dot{\epsilon}$  at some projected time  $t$  given the strain rate at a unit time  $t_1$  from a creep test. The time-dependant

distortions of the soil structure and relative movements between particles cause continuous changes in creep rate making the parameters  $A$ ,  $\alpha$  and  $m$  time dependent.

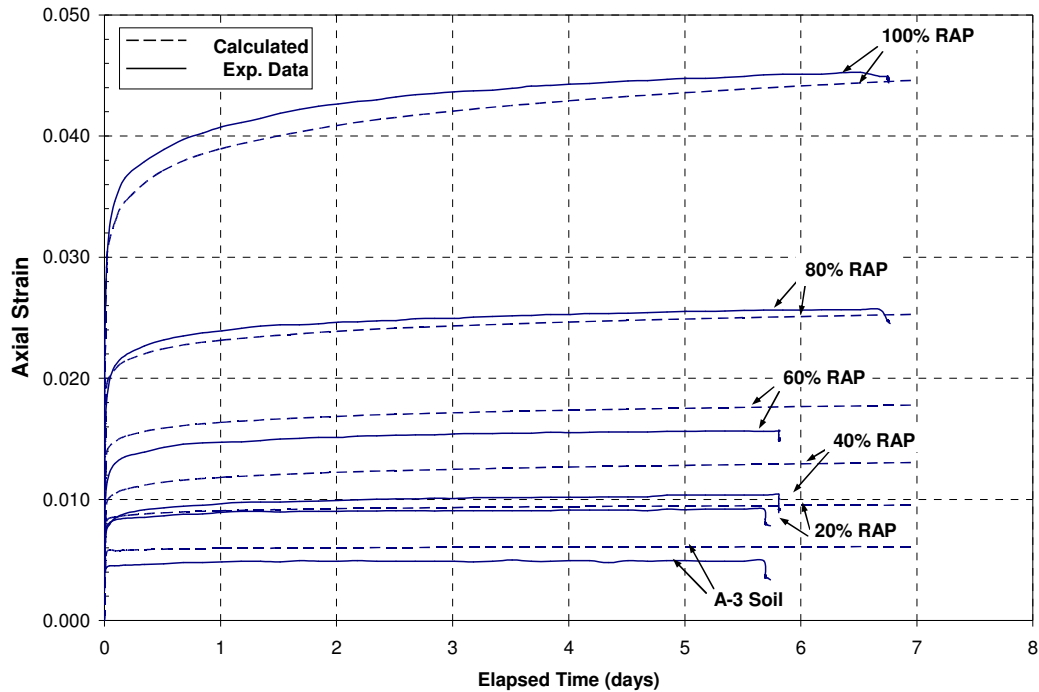
The Singh and Mitchell (1968) equation was used along with the test data for each material and the results are given in Appendix A-5. Parameters  $A$  and  $\alpha$  were obtained from plots of log strain rate at unit time  $t_1$  versus the three stress intensities of 6, 12 and 18 psi at which strain was measured. The parameters  $A$  and  $\alpha$  represent the intercept and the slope respectively of a linear trendline fitted to the three points in the plot.

Parameter  $m$  was the slope of the linear trendline fitted to the three data series obtained at the 6, 12 and 18 psi stress levels and depicted on a log strain rate versus log time plot. The three trendlines were parallel which matched Singh and Mitchell's (1968) assumption; therefore, for each blend,  $m$  was chosen as the average of the three slopes.

Equation 2.3 proved to be very sensitive to the selection of unit time  $t_1$ . To evaluate this parameter, the strain rate in 100% RAP at 12 psi was calculated at 50 years using three unit times – 1 minute, 1 day, and 7 days. The 7-day calculation yielded a strain rate equal to 90% of the value obtained using 1 day. The 1-minute calculation resulted in a strain rate approximately 200% higher than the 1-day value and therefore was not used. To choose between one and seven days, the theoretical strain rates were compared to the 1-day and 7-day laboratory strain rates. The best comparison corresponded to the 1-day calculations and therefore it was adopted as unit time for all materials.

An important consideration for the accuracy of the strain rate and subsequent settlement calculations was to ensure the applicability of Equation 2.3. Singh and Mitchell (1968) specified that this equation could be used to characterize creep behavior of soils loaded over the range of stresses between 30% and 90% of their initial strength determined from a conventional strength test. During this research, however, the stress levels were not selected based on a strength test, but were rather theoretically determined by performing bearing capacity calculations. The calculations were performed for 100% RAP, which was the material with a) the lowest bearing strength according to the LBR and b) based on the RAP shear strength from triaxial compression testing by Gomez on 100% RAP. Gomez's (2001) triaxial compression data was used to estimate the ratio of the applied stress to the material strength before creep, Appendix A-8. Although the material tested by Gomez (2001) was not identical to the RAP used in this study, it was assumed that incorporating these results would lead to calculations of reasonable engineering approximation. For 100% RAP Gomez (2001) determined the cohesion to be 4.9 psi and the friction angle to be 44 degrees. Based on this Mohr-Coulomb failure criterion, the ratios between the 6, 12, and 18 psi stress levels and the calculated shear strength were 56%, 72%, and 80%, respectively. All three fell within the range specified by Singh and Mitchell (1968), signifying overall the applicability of Equation 3.2 for the analysis.

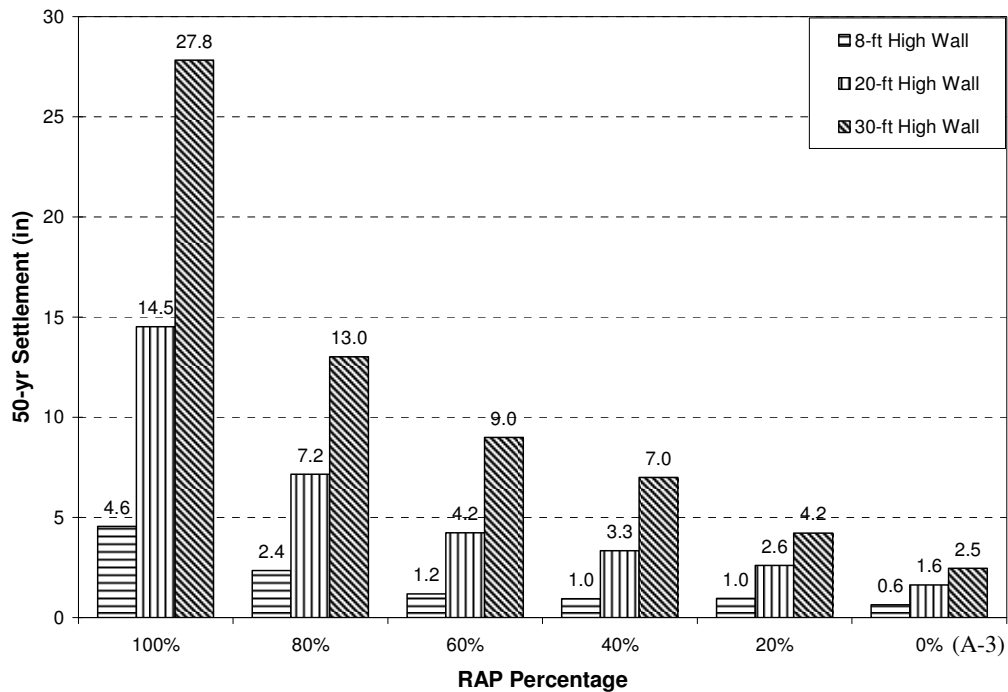
Figure 4.12 shows a comparison of the theoretical and measured strains for 5 to 7 day test durations. The figure presents the strains at 18 psi stress level, while the remaining plots for 6 and 12 psi are included as Figures A-5.1 and A-5.2 in Appendix A-5. The calculated and experimentally obtained strains at all three stress levels plotted with identical curve character and close fit in magnitude. The agreement between the two sets of curves proved the applicability of Equation 2.3 for strain determination in RAP and RAP-soil mixtures, as well as the correctness of the conducted strain rate calculations.



**Figure 4.12 Comparison between experimentally collected and calculated using strain rate theory strains in RAP, RAP-soil mixtures, and A-3 soil tested in creep at stress level of 18 psi for a period of 5-7 days**

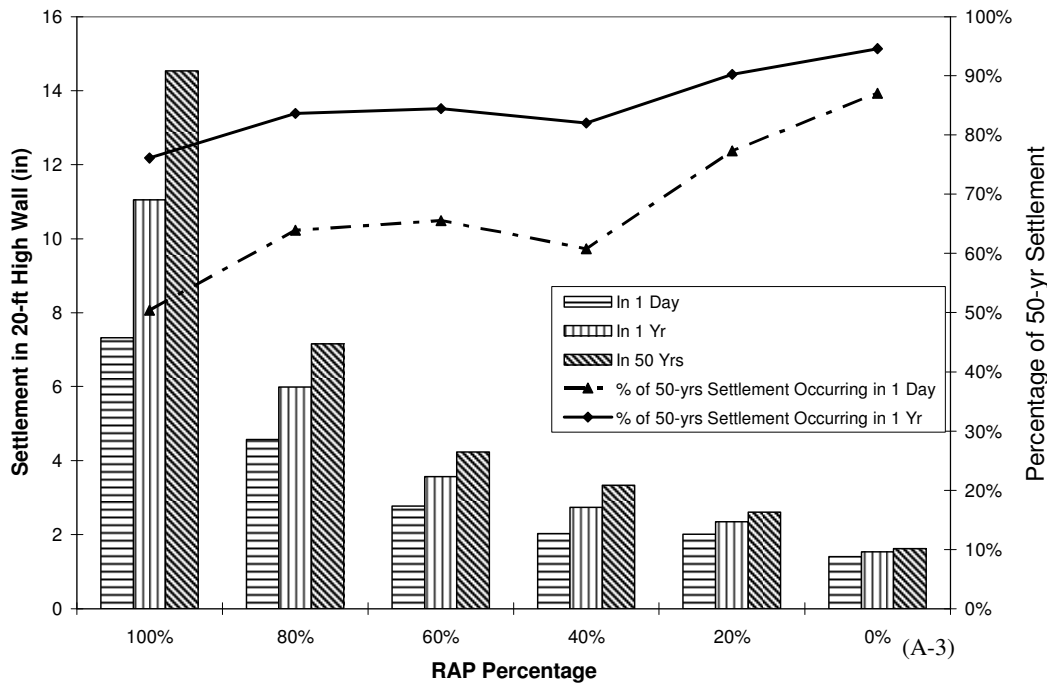
Using the strain rates, the strains and ultimately the settlements associated with the assumed wall heights for periods of 1 day, 7 days, 1 year, and 50 years were calculated, Appendix A-7. Based on the external stability calculations, Appendix A-1, these stress levels corresponded to MSE wall heights of 8, 20, and 30 ft. respectively. The data in Figure 4.13 summarizes the 50-year settlements at the three wall heights for all tested materials. The settlement increased with increasing wall height and RAP content.





**Figure 4.13 Creep settlement after 50 years in RAP, RAP-soil mixtures, and A-3 soil For MSE wall heights of 8, 20 and 30 ft.**

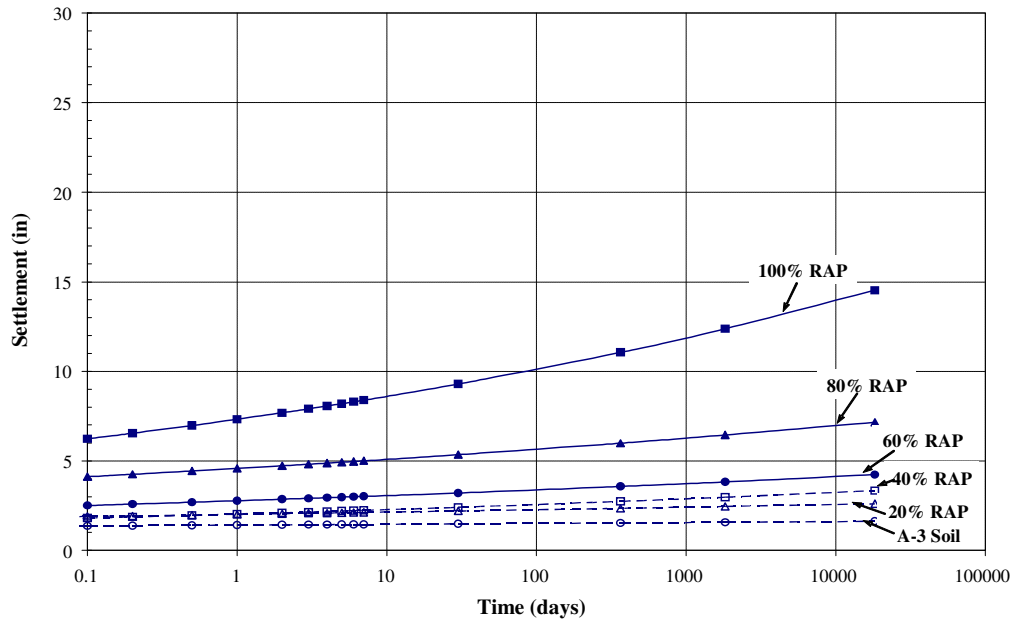
The minimal long-term settlement in the A-3 soil was used as a control. The large settlement estimated for 100% RAP is a clear indication that it would not function in real world applications. Based on this data, a general conclusion was formed. No more than about 20% RAP could be used behind MSE walls if settlement was not a major issue. The settlements determined were totals over 50 years, but the Singh and Mitchell (1968) approach allowed this settlement to be determined at any times desired. In Figure 4.14 the settlements in RAP, RAP-soil mixtures, and A-3 soil for an assumed 20-ft high wall for 1 day and 1 year were compared to the 50-yr values. With the exception of 40% RAP, for which the experimental data showed some abnormalities, the general trend in Figure 4.14 was that settlements varying from 50% in 100% RAP to 87% in A-3 soil, occurred during the first day of load application. One year after load application the settlement reached 76% of its 50-yr value in 100% RAP and 95% in A-3 soil. It was assumed that only short-term settlement occurred in the A-3 soil.



**Figure 4.14 Backfill settlement of RAP, RAP-soil mixtures, and A-3 soil for 20-ft high wall, occurring at different time periods**

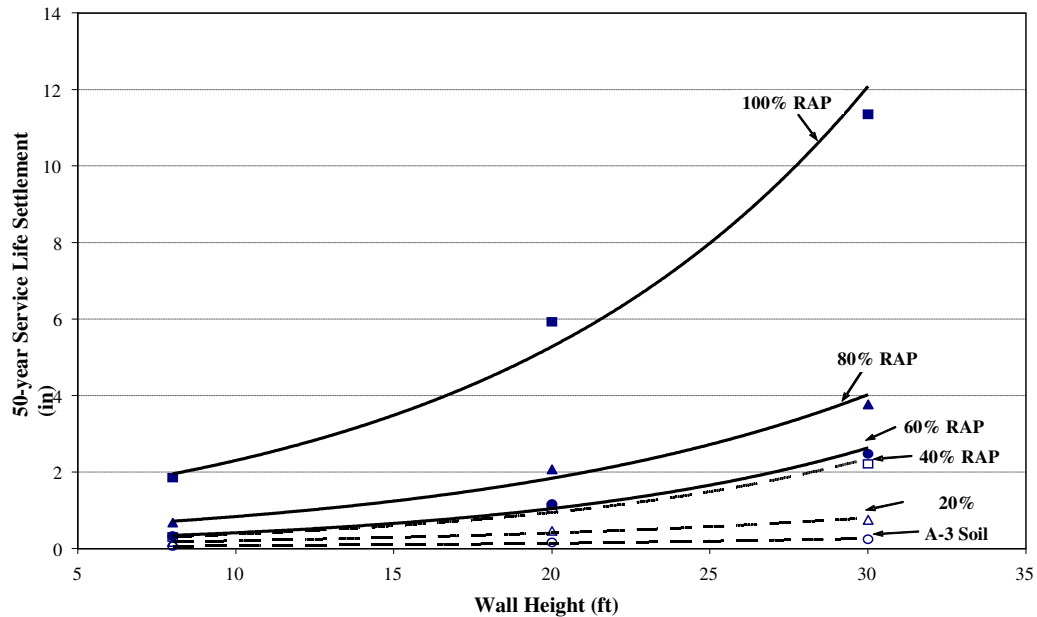
To summarize, if a 20-ft high wall was to be built with backfill of 50% RAP mixture, the expected settlement in 50 years would be approximately 3.8 inches. Nearly 70% of the settlement would occur during the first day and 85% would take place in the first year of service. Thus the estimated settlement for the remaining 49 years would amount to the relatively insignificant 0.6 inches.

The relationship between estimated settlement and time is presented in more detail in Figure 4.15, for the settlement of a backfill behind a 20-ft high wall for the RAP, RAP-soil mixtures, and A-3 soil. Analogous plots for 8-ft and 30-ft wall heights are given in Appendix A-7 as Figures A-7.1 and A-7.2. The settlement associated with 100% RAP significantly exceeded those in the remaining blends. The 80% RAP mixture showed a moderate magnitude, whereas the mixtures of 60% RAP or less grouped together displaying relatively small variations in magnitude and slope.



**Figure 4.15 Backfill settlement with time for RAP, RAP-soil mixtures, and A-3 soil behind 20-ft high wall**

Figure 4.16 represents the increase in 50 year settlement for the various wall heights. For the purpose of settlement calculation, a construction period of 10 days was assumed; the settlement associated with that period was subtracted from the 50-year values. Once again, regardless of the wall height, a grouping of the curves characterizing RAP contents 60% or lower was produced. Another observation from this figure is the non-linear increase of the long-term settlement with respect to wall height. This growth was exponential and is faster at higher RAP contents.



**Figure 4.16 50-year backfill settlement in RAP, RAP-soil mixtures, and A-3 soil for various MSE wall heights**

#### 4.2.4 Creep Correlations

For each mixture and load increment, the creep testing was performed on series of three replicas. If all three samples within a series were identical, their creep curves would have overlapped. This was not the case and a certain spread of test data was observed. In order to examine the relation between the creep spread and the test parameters, correlation analyses were conducted for each mix using two properties – RC and LBR after creep. Appendix A-9 contains the LBR data collected after creep testing and Appendix A-6 contains the statistical analysis on the data. The calculated correlation coefficients had values between 0.75 and 0.80 for the relationship between strain and LBR, and significantly higher absolute values, around 0.95, for the strain and RC. The negative correlation between strain and RC showed that small values of the latter parameter tend to be associated with large values of the former. Demonstrating a high degree of correlation, RC was included in an equation expressing the strain as function of the stress level and relative compaction.

Regression analysis in Excel was performed for every mixture. The regression analysis in Excel uses the least squares method to fit a line through a set of observations. This method allowed for an analysis of the impact, which the independent variables stress level and RC have on the strain as a single dependent variable. The result was an equation representing the strain as a function of stress level and relative compaction. A summary of the resulting equations is presented

in Table 4.6. The equations in the table were developed from the maximum strains achieved at the end of loading, at approximately 5 to 7 day test duration. As discussed in previous paragraphs, for this duration in most of the tested mixtures the majority of the deformation had taken place. The equations could be used as a direct method for calculation of the strain at various mixture proportions, as a function of the stress level and RC.

**Table 4.6 Summary of regression equations for RAP, RAP-soil mixtures, and A-3 soil after 5 to 7 days of creep testing**

$\varepsilon = f(p ; RC)$	
Mixture	Equation
100% RAP	$\varepsilon = e^{0.050*p-19.26*RC+14.93}$
80% RAP	$\varepsilon = e^{0.045*p-17.59*RC+12.73}$
60% RAP	$\varepsilon = e^{0.053*p+1.74*RC+3.40}$
40% RAP	$\varepsilon = e^{0.037*p+28.51*RC-33.43}$
20% RAP	$\varepsilon = e^{0.025*p+5.43*RC-10.45}$
0% RAP	$\varepsilon = e^{0.009*p+21.50*RC-26.58}$
<b>Overall Correlations</b>	
$\varepsilon = f(p; Mix)$	$\varepsilon = e^{0.023*p+0.016*Mix-5.5}$
$\varepsilon = f(p; AC)$	$\varepsilon = e^{0.023*p+27.43*Mix-5.5}$

$\varepsilon$  - Strain

p - Stress Level

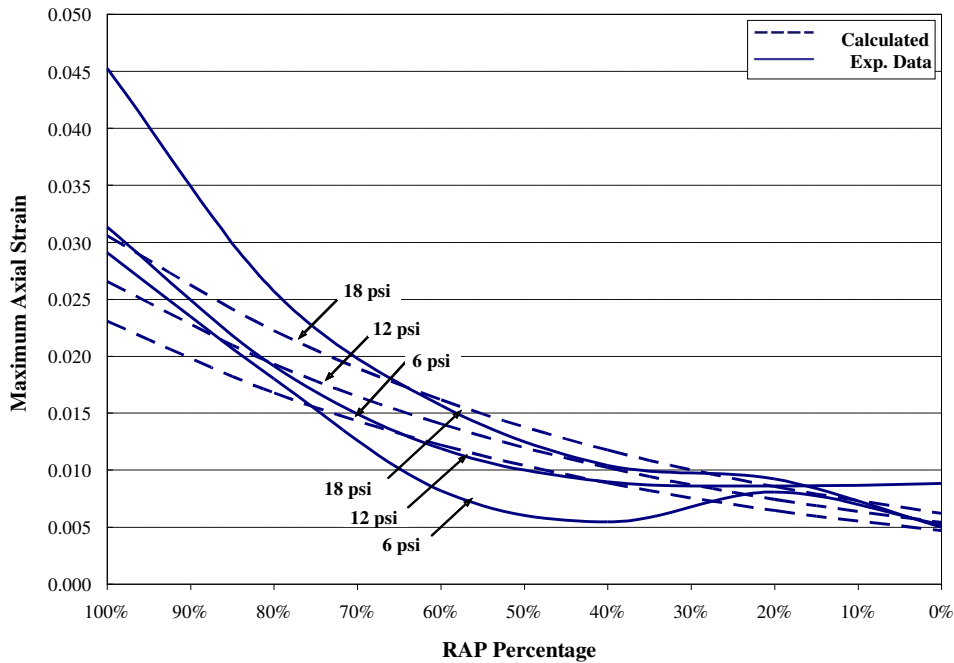
RC - Relative Compaction

Mix - Percentage RAP in Mixture

AC - Asphalt Content

In Figure 4.17 the experimentally collected strain data is compared to the strain values calculated by applying the last equation in Table 4.6, which relates strain, pressure, and asphalt content. Two sets of three curves were plotted, representing the experimental and calculated maximum strains at the end of testing at the three used stress levels of 6, 12, 18 psi. At the higher RAP contents the calculated trends resembled better the experimental strains obtained at the two lower stress levels of 6 and 12 psi, whereas at midrange RAP contents there was a better agreement with the strains obtained at 12 and 18 psi stress levels. Overall, it could be concluded that the regression equations generated a close fit to the

experimental data values, yielding strain predictions of reasonable engineering approximation for the particular RAP gradation and asphalt content.

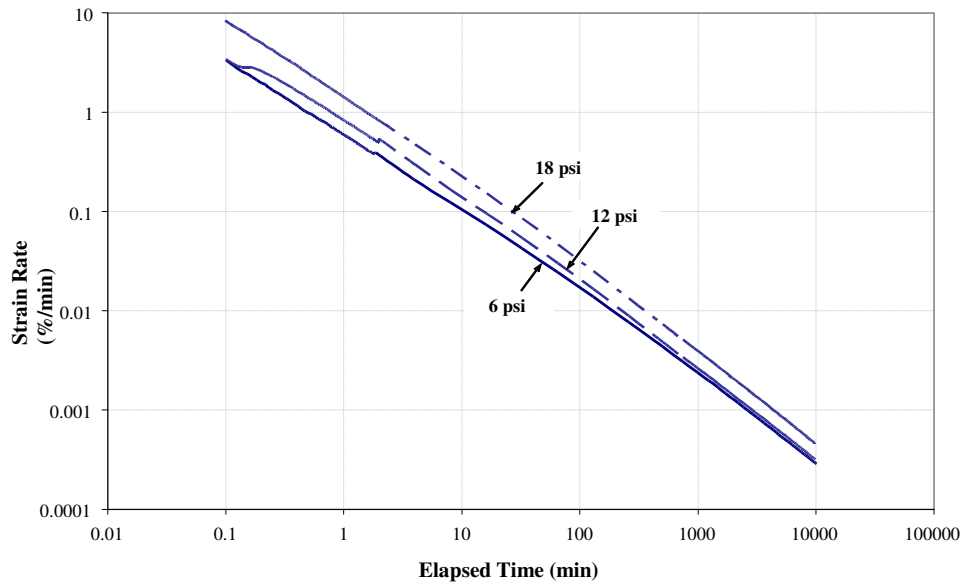


**Figure 4.17 Maximum strain comparisons between experimental and empirical regression equations in RAP, RAP-soil mixtures, and A-3 soil at 6, 12, and 18 psi stress levels**

#### 4.2.5 Physical Factors Affecting Creep

The analysis of the physical factors initiating creep was based on the discussions on soil time-dependent behavior by Augustesen et al (2004) and Mitchell (1993). Primary, secondary and tertiary creep phases are possible.

In Figure 4.18 the 100% RAP data was plotted in log strain rate versus log time. The shape of the plot is similar to the linear portion of the relationship, identified as secondary creep by Augustesen et al (2004) and the linear relationship identified as a combination of primary and secondary creep by Singh and Mitchell (1968).



**Figure 4.18 Strain rate vs. time relationship during creep of 100% RAP at 6, 12 and 18 psi stress levels**

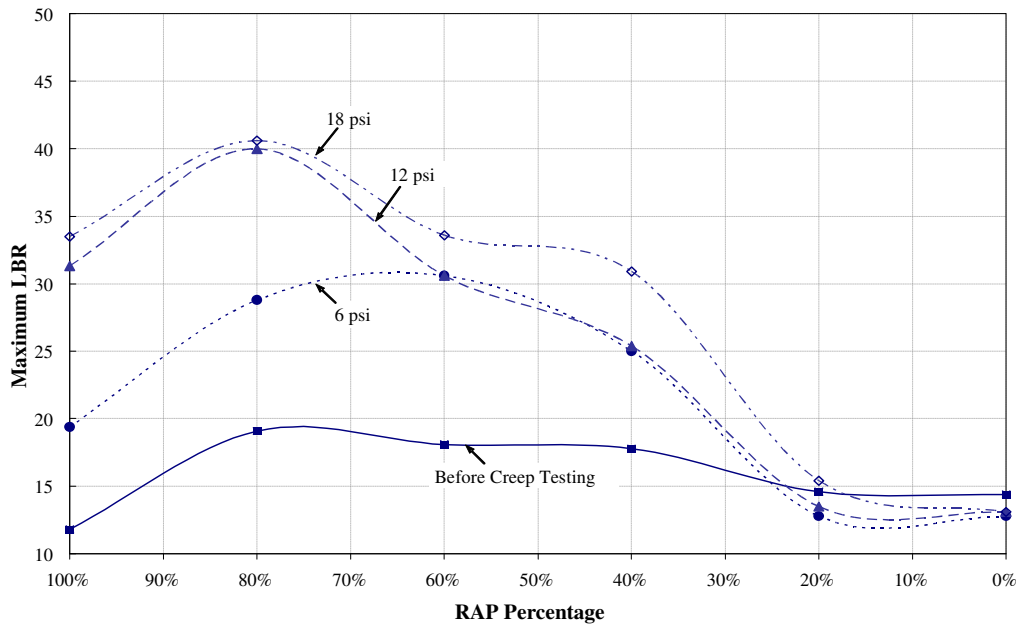
It was concluded that at the 6, 12, and 18 psi stress levels the 100% RAP experienced primary and secondary compression, where voids were eliminated and some soil skeleton deformations occurred. It could not be determined from the plot at what time the primary compression ended and secondary compression began. In the literature this time is referred to as reference time. It was assumed that both phases started simultaneously at the beginning of loading. For the purpose of settlement analysis it was considered practical to perform the calculations, without separating movements into instantaneous and long-term components.

During primary compression void reduction takes place. According to Mitchell (1993), the secondary compression is a creep of the soil skeleton and in oedometer testing is accompanied by volume reduction as the particles adjust to more stable arrangements. The main factor influencing the volume change at constant stress level is the physical interaction between the particles, consisting of particle sliding, rolling, and crushing. Particle sliding and rolling is resisted by the material friction. Particle crushing occurs at the zones of interparticle contact, where forces are transferred and increases with increasing grain size. Typically more crushing would be observed in gravel than sand, and in angular than subrounded material. Although significant particle crushing is not likely to have occurred at the stress levels used for this study, it could have disintegrated the RAP agglomerations. The applied stresses were insufficient to cause crushing in the A-3 soil.

On the microstructure level, new bonds are formed at the interparticle contacts during compression. The particle sliding force depends on the contact area and the shear strength of contact. The strength of the contact depends also on the nature of the contact area. If the particles are covered by a film, as in RAP, the contact surface is not clean and is prone to plastic deformations, resulting in decreased strength.

### 4.3 LBR after Creep

Bearing values as measured by the LBR were obtained after creep testing. The intention was to evaluate the LBR changes occurring after permanent deformation and to develop a relationship between stress level and LBR. The resultant LBR values are presented in Figure 4.19. It was believed that after experiencing permanent deformation, which forced the particles closer together, the samples would possess a higher density, an increased shear resistance and ultimately, an improved bearing capacity.



**Figure 4.19 LBR of RAP, RAP-soil mixtures and A-3 soil after creep testing at 6, 12 and 18 psi stress levels**

The post creep LBR values were higher for 100% RAP and blends from 40 to 80% RAP. Although not with a consistent rate, in general LBR increased with increasing stress level. The largest LBR value of 41 was achieved in 80% RAP, while the largest improvement to LBR before and after creep of 183% occurred in 100% RAP. The 20% RAP blend and the A-3 soil, while exhibiting increased LBR with increased stress level, did not show improvement compared to their strength before creep.



In general, it was concluded that after creep the material exhibited an improved strength above 40% RAP and that this effect should be further investigated in the field.

#### 4.4 Engineering Properties of Tested Materials

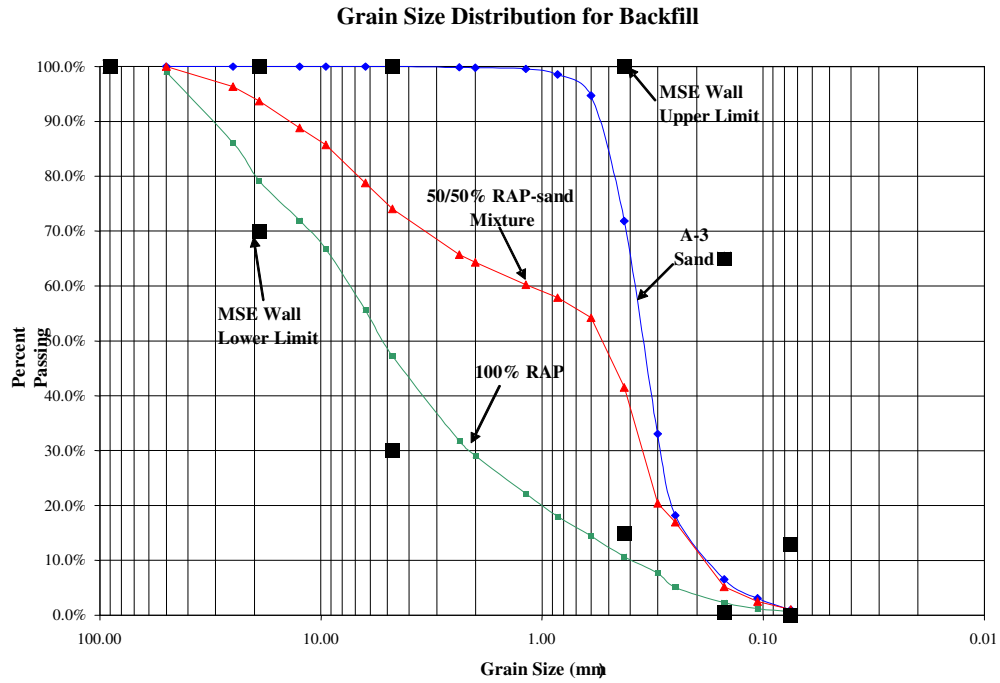
The engineering properties of the materials used in this investigation were evaluated at either FIT-ARL or FDOT SMO. Grain size distribution, moisture-density properties and asphalt content for 100% RAP, 50/50% blend and A-3 sand were determined using the standardized tests presented in Table 4.7. All test data associated with this portion of the research is presented in Appendix B.

Asphalt content testing using the ignition method resulted in an asphalt content of 6.9% for 100% RAP. Asphalt content testing performed at the FDOT-SMO reported a range of 4.9% to 8.5% for milled RAP. The difference in asphalt content is attributed the original mix design of RAP.

**Table 4.7 Laboratory test methods to evaluate engineering properties of RAP, A-3 sand and RAP-sand blend**

Laboratory Test	Description	Standard
Sampling	Sampling Coarse and Fine Aggregate	FM 1-T 002
Quartering	Reducing Field Samples of Aggregate to Testing Size	FM 1-T 248
Grain Size Analysis	Sieve Analysis of Fine and Coarse Aggregate	FM 1-T 027 / AASHTO T27
Moisture Density	Moisture Density Relations of Soils Using 10-lb. Rammer and 18-in. Drop	FM 5-521 [FM 1-T 180] / AASHTO T 180-74, Method D
Asphalt Content	Quantitative Determination of Asphalt Content from Asphalt Paving Mixtures by the Ignition Method	FM 5-563

Sieve analyses were conducted for each material yielding the grain size distribution curves presented in Figure 4.20. Figure 4.20 also includes the upper and lower MSE wall particle gradation limits as specified by Section 548 of the FDOT Standard Specifications for Road and Bridge Construction (2007). The materials were AASHTO classified as A-1-a for the 100%RAP, A-1-b for the 50/50% blend, and A-3 for the sand. The materials were USCS classified as well graded gravel (GW) for the 100% RAP and poorly graded sand (SP) for 50/50% blend and 100% sand. The A-3 sand and 50/50% blend met the specified limits for MSE walls; however for 100% RAP, the percent passing the No. 40 sieve was below the acceptable range.



**Table 4.8 Grain size distribution parameters for 100% RAP, 50/50% blend and A-3 sand**

	A-3 Sand	50-50% Blend	100% RAP
<b>D<sub>10</sub> (mm)</b>	<b>0.19</b>	<b>0.19</b>	<b>0.40</b>
<b>C<sub>u</sub></b>	<b>2.0</b>	<b>6.2</b>	<b>17.9</b>
<b>C<sub>c</sub></b>	<b>1.2</b>	<b>0.6</b>	<b>1.5</b>
<b>USCS</b>	<b>SP</b>	<b>SP</b>	<b>GW</b>
<b>AASHTO</b>	<b>A-3</b>	<b>A-1-b</b>	<b>A-1-a</b>

In order to determine compaction requirements to be used in the FDOT SMO test pit, compaction characteristics of each material were analyzed using the results from modified Proctor testing. The materials were prepared at moisture contents ranging from 2 to 14% and compacted. Based on the data collected from laboratory tests performed at FDOT-SMO, average moisture density results were determined and are summarized in Table 4.9. The average represents the results of 3 trials. The maximum dry density for RAP in this study (115.5 pcf) was similar to the values identified by Dikova (2006) of 112.4 pcf at 6.2% moisture and Cleary (2005) of 116.8 pcf at 7.0% moisture.

**Table 4.9 Average moisture-density characteristics of materials used in FDOT-SMO test pit**

	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
<b>100% RAP</b>	<b>115.5</b>	<b>5.1</b>
<b>50/50% Blend</b>	<b>115.6</b>	<b>8.1</b>
<b>A-3 Sand</b>	<b>110.2</b>	<b>12.6</b>

#### **4.4.1 Pullout Testing**

Pullout testing was conducted using ribbed steel soil reinforcement embedded in 100% RAP, 50/50% blend and A-3 sand. Vertical pressures of 6, 12 and 18 psi were applied at the testing surface to the materials as the strips were pulled.

Due to stress distribution from the applied vertical pressure, the vertical stress at the 8 inch strip depth was less than the stress applied on top of the material in the test box. An I-beam was used to load the surface of the soil. The vertical stress at strip depth was estimated for each stress level using Boussinesq equation for a point below a rectangular loaded area (Holtz and Kovacs 1981). Table 4.10

presents the vertical stresses at a depth of 8 inches below the rectangular I-beam used for the maximum and incremental pullout tests.

**Table 4.10 Estimated stresses at depth of 8 inches as a result of applied vertical stresses at surface**

Applied Stress at Surface (psi)	Estimated Stress on Strip (psi)
6	4.9
12	8.9
18	12.8

Moisture and density samples were obtained for each of the three compacted lifts at various locations in the test box. Table 4.11 presents the relative compaction for each lift of the maximum and incremental pullout test setups (Table 3.2). As described in Chapter 3, lift 1 was placed first, followed by lift 2 and 3 respectively. The reinforcing strip was placed between lifts 2 and 3. The average density of each lift varied by 5.3% for the 100% RAP, 5.4% for 50-50% blend, and 4.6% for A-3 sand. Both RAP and sand are well drained non-cohesive materials; therefore it was difficult to maintain steady moisture content throughout each lift. This condition had little effect on the density where the relative compaction of each material was between 95 and 99%.

**Table 4.11 Relative compaction of each lift for the maximum and incremental pullout tests**

		Vertical Stress		
		6 psi	12 psi	18 psi
		Average* Dry Density (pcf)	Average* Dry Density (pcf)	Average* Dry Density (pcf)
100% RAP	Lift 1	97%	97%	97%
	Lift 2	98%	98%	98%
	Lift 3	96%	97%	97%
50/50% Blend	Lift 1	95%	95%	95%
	Lift 2	96%	96%	96%
	Lift 3	97%	98%	99%
A-3 Sand	Lift 1	96%	96%	96%
	Lift 2	97%	97%	97%
	Lift 3	97%	96%	97%

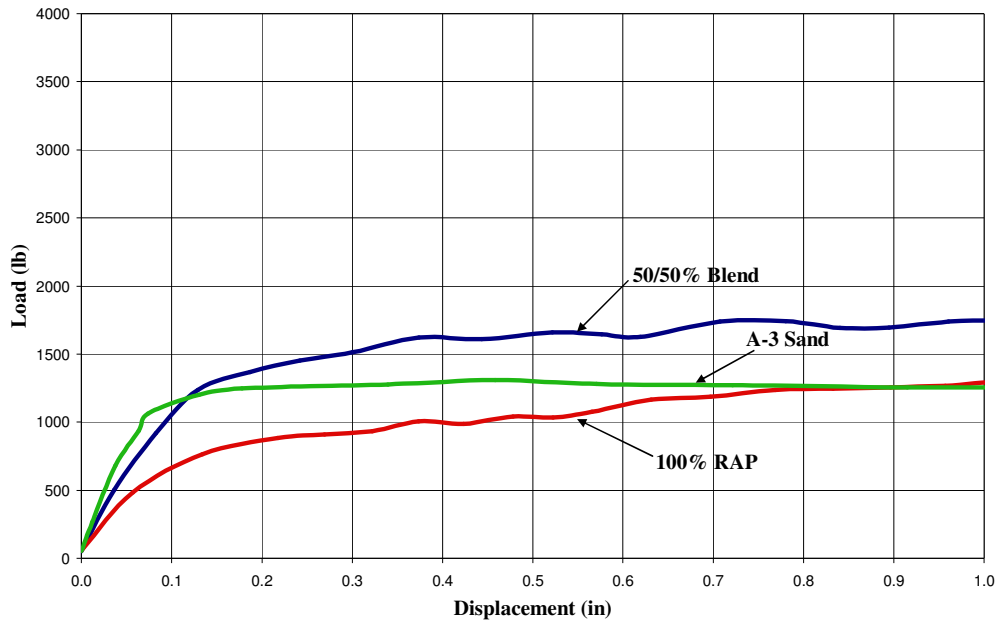
\* Averages - based on 8 tests per lift

#### 4.4.2 Maximum Pullout Test Results

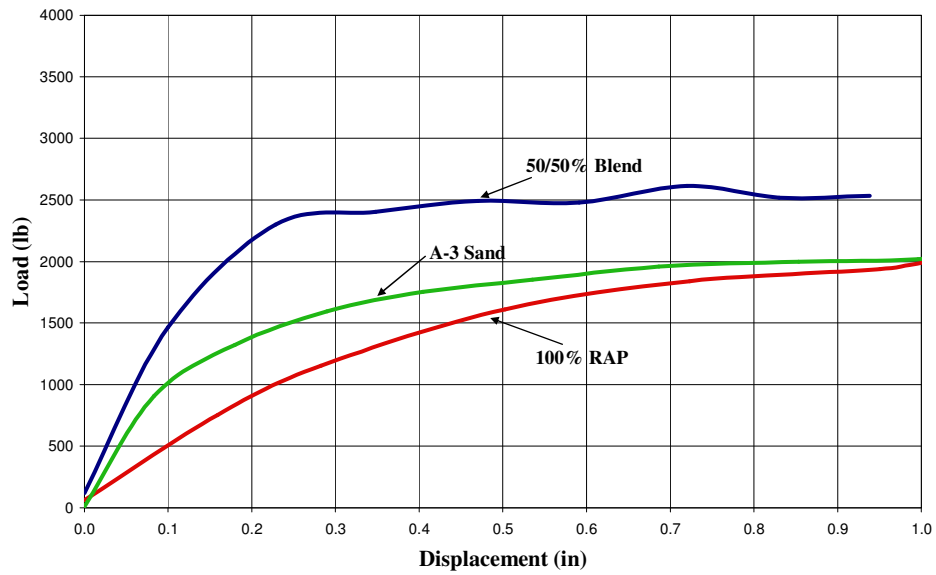
One maximum pullout test was performed on the reinforcing strip in each material at every applied vertical stress resulting in a total of nine tests. The reinforcing strip was pulled at a rate of 0.05 inches per minute until a deflection of approximately 1.0 inch was obtained or soil failure occurred. Figure 4.21, Figure 4.22, and Figure 4.23 present the pullout load versus horizontal displacement of the MSE reinforcement strip in each material with an applied vertical stress of 6, 12 or 18 psi at the soil surface. The maximum pullout loads were chosen as the maximum load from the horizontal portion of each curve.

The load versus displacement relationship shows an increase in pullout resistance with an increase in vertical pressure. An increase in confining stress caused by the increased vertical stress increases the friction on the strip consequently producing a greater pullout capacity. The materials displayed similar characteristics in maximum pullout loads of the reinforcing strip at every vertical confining stress. The 50-50% blend produced the highest pullout capacity followed by the A-3 sand and then 100% RAP. Figure 4.24 presents a summary of the maximum pullout loads. On average 50-50% blend had a pullout force 1.3 times that of the A-3 sand and 1.4 times that of the 100% RAP. The maximum pullout curves presented in Figures 4.21 to 4.24 display results similar to Rathje et al. (2006) where an increase in applied vertical stress increased the maximum pullout load of the soil reinforcement.

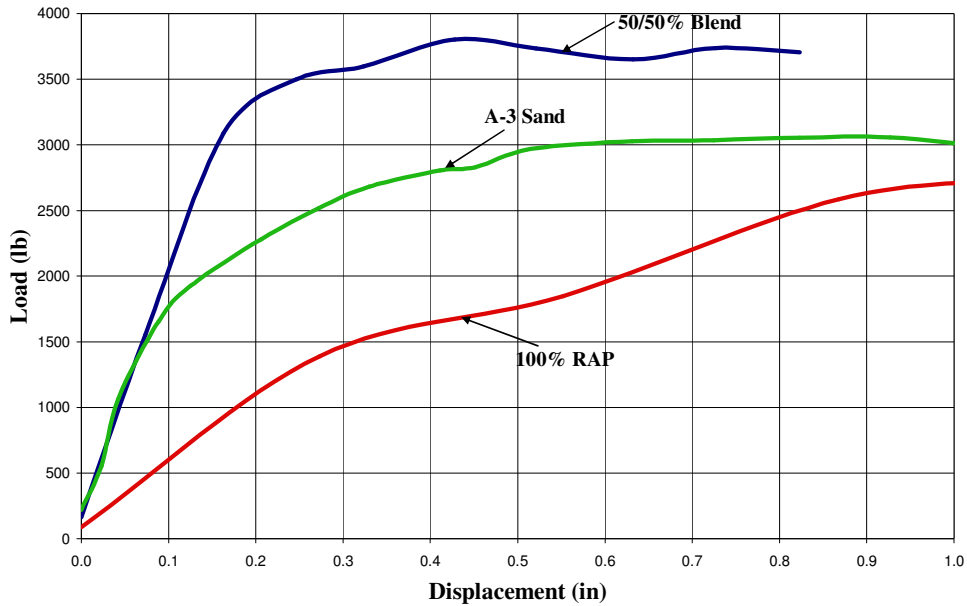
The increase of pullout strength of the 50/50% blend may be attributed to both the granular nature of the RAP and the increase in the amount of material passing the #40 sieve compared to the 100% RAP resulting in a decrease in voids and greater soil friction. Gomez (2003) research found that the density and LBR of a material improved with the addition of material passing #40 sieve. Asphalt content in the 50/50% blend was also reduced by mixing with A-3 sand as compared to the 100% RAP resulting in less displacement of the reinforcing strip.



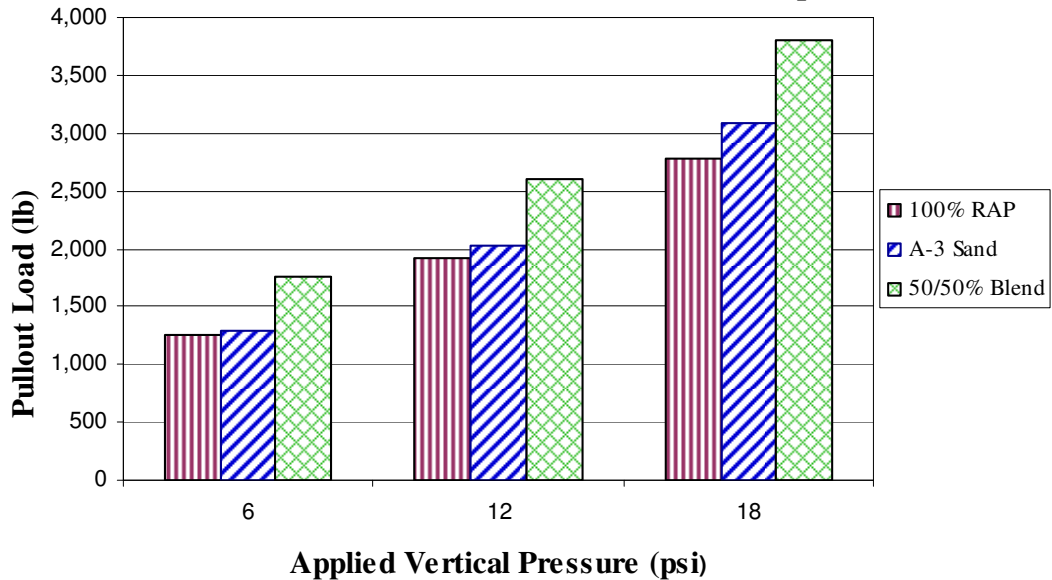
**Figure 4.21 Load versus displacement results for MSE reinforcement in backfill materials with vertical stress of 6 psi**



**Figure 4.22 Load versus displacement results for MSE reinforcement in backfill materials with vertical stress of 12 psi**



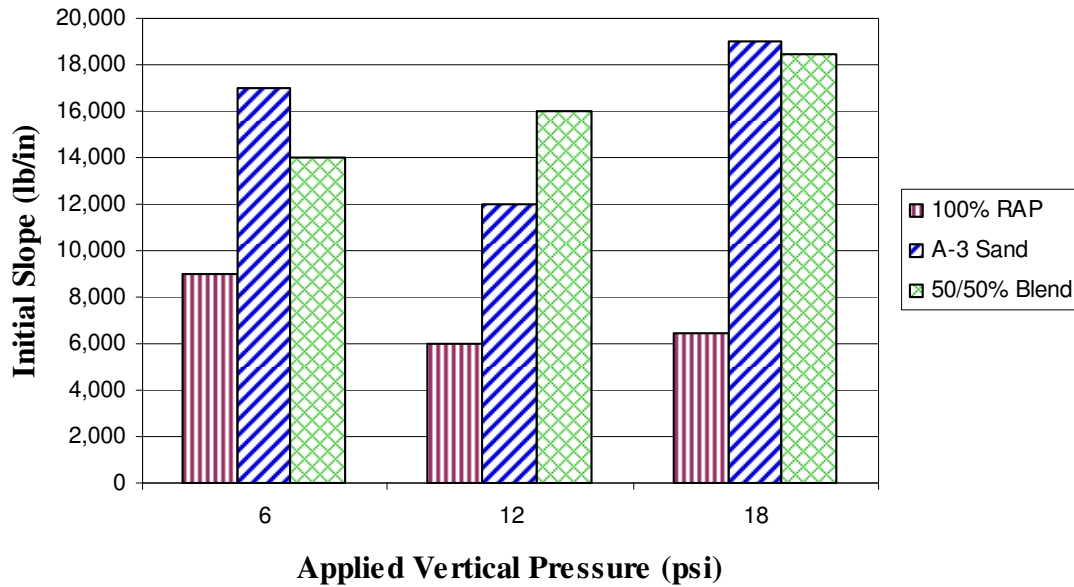
**Figure 4.23 Load versus displacement results for MSE reinforcement in backfill materials with vertical stress of 18 psi**



**Figure 4.24 Maximum pullout loads for soil reinforcement embedded in backfill with applied vertical stresses of 6, 12 and 18 psi**

Initial slopes of the load versus displacement curves shown in Figure 4.21, Figure 4.22, and Figure 4.23 were also compared for each material and are presented in Figure 4.25. The initial slopes for 50/50% blend and A-3 sand were

similar compared to 100% RAP which had a consistently lower slope at each applied vertical stress. Though the 50/50% blend and the A-3 sand had similar slopes, the maximum pullout values for each material were not similar. The 50/50% blend consistently had higher maximum pullout values than the A-3 sand (Figure 4.24). Except for the 50/50% blend, the slopes did not increase as a result of an increase in the applied vertical stress. This may be a result in variation of the pullout rate of the strip from one test to another.

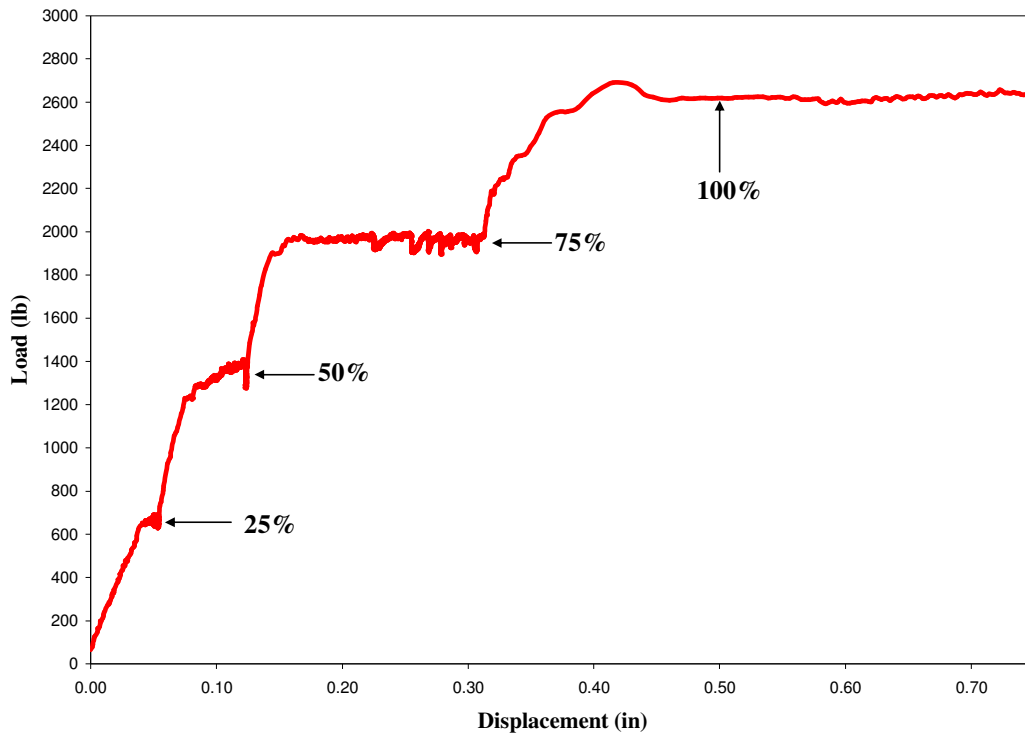


**Figure 4.25 Initial slopes for maximum pullout test of each material with an applied vertical stress of 6, 12 or 18 psi**

#### 4.4.3 Incremental Pullout Test Results

The maximum pullout loads presented in Figure 4.24 were used as a basis to calculate four incremental loads for the incremental pullout test. The maximum pullout load was divided into increments of 25, 50, 75 and 100% of this value and these loads were held constant on the reinforcing strip for 15 minutes while the strip displaced. After the data was recorded, the load was increased to the next level. Figure 4.26 presents a typical incremental pullout load versus displacement chart for the 50/50% blend at an applied vertical stress of 12 psi. Appendices B-1 and B-2 presents data for all incremental pullout tests.

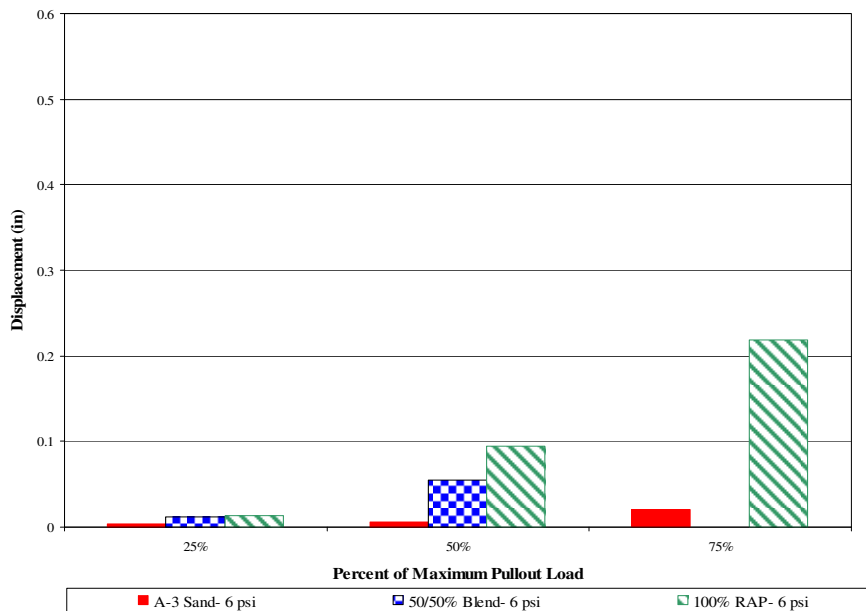




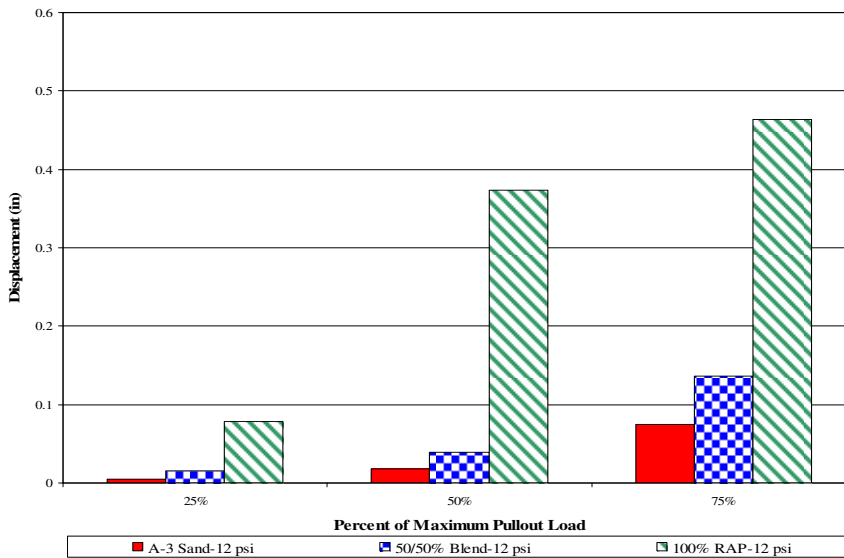
**Figure 4.26 Incremental pullout load versus displacement chart for 50/50% blend with an applied vertical stress of 12 psi**

Figure 4.26 displays the total horizontal strip displacement which includes the creep occurring at each increment. To quantify the creep displacement, Figure 4.27, Figure 4.28, and Figure 4.29 present the creep displacement that was measured during each 15 minute increment of 25, 50 and 75% of the maximum pullout load. The load on the reinforcing strip was held constant. The displacement values for 100% of the maximum pullout load are not presented due to failure of the soil reinforcement.

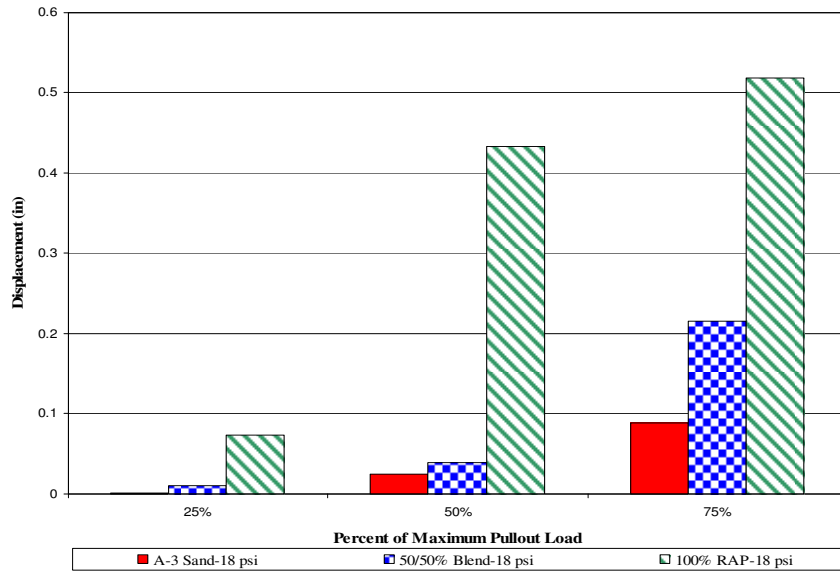
The strip creep displacement at all applied vertical stresses was largest in the 100% RAP. The 50/50% blend data is not present in Figure 4.27 due to the failure of the testing equipment. Figures 4.27 to 4.29 show similar trends for all the materials. In each figure the A-3 sand yields the least creep displacement followed by the 50/50% blend and then the 100% RAP as the percentage of maximum pullout load is increased. As the applied vertical stress increased from 6, 12 to 18 psi, the 50/50% blend produced strip displacement values closer to A-3 sand than 100% RAP; suggesting that the decrease in asphalt content due to mixing produced less displacement of the soil reinforcement.



**Figure 4.27 Creep displacement of soil reinforcement from incremental pullout test at 6 psi applied vertical stress for each 15 minute load increment**

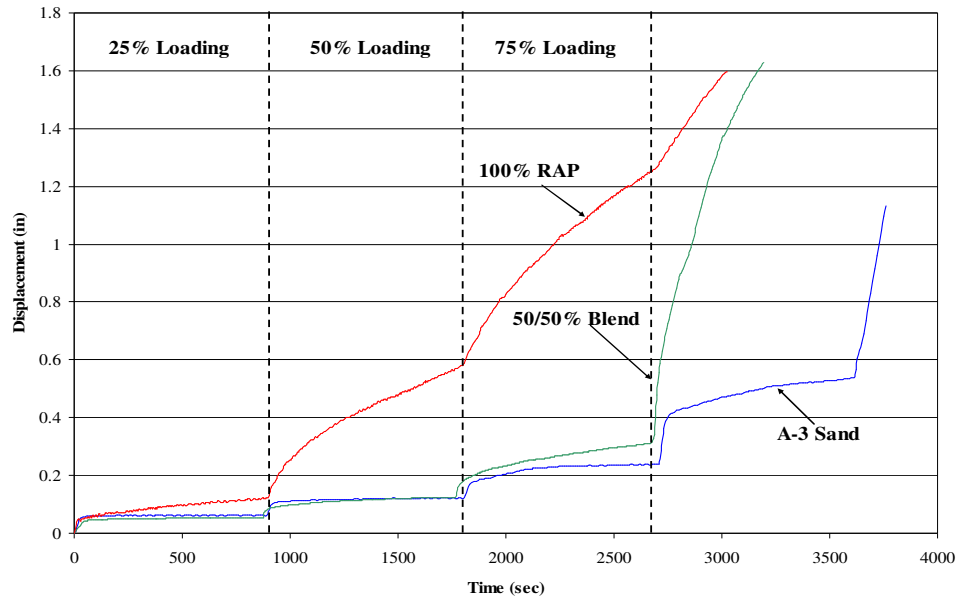


**Figure 4.28 Creep displacement of soil reinforcement from incremental pullout test at 12 psi applied vertical stress for each 15 minute load increment**



**Figure 4.29 Creep displacement of soil reinforcement from incremental pullout test at 18 psi applied vertical stress for each 15 minute load increment**

The total creep displacement of the reinforcing strip was not measured in this study because the incremental loads were only held for 15 minutes then increased to the next increment. To measure the total creep displacement for the soil reinforcement in each backfill material, the pullout loads on the strip would need to be held constant until zero displacement occurred for that load increment. The A-3 sand was used as a control in this study to compare with the 100% RAP and 50/50% blend. Figure 4.30 presents displacement versus time plots for each backfill with an applied vertical stress of 12 psi. It is apparent that the 100% RAP continues to display creep displacement at each load increment. The 50/50% blend and the A-3 sand present the less creep displacement as compared to the 100% RAP; however, a closer examination of the figure shows that the 50/50% blend also continues to creep at each increment which is especially evident at 75% loads. The A-3 sand presents the least amount of displacement of all the materials and the curves level off as time increases in each increment.



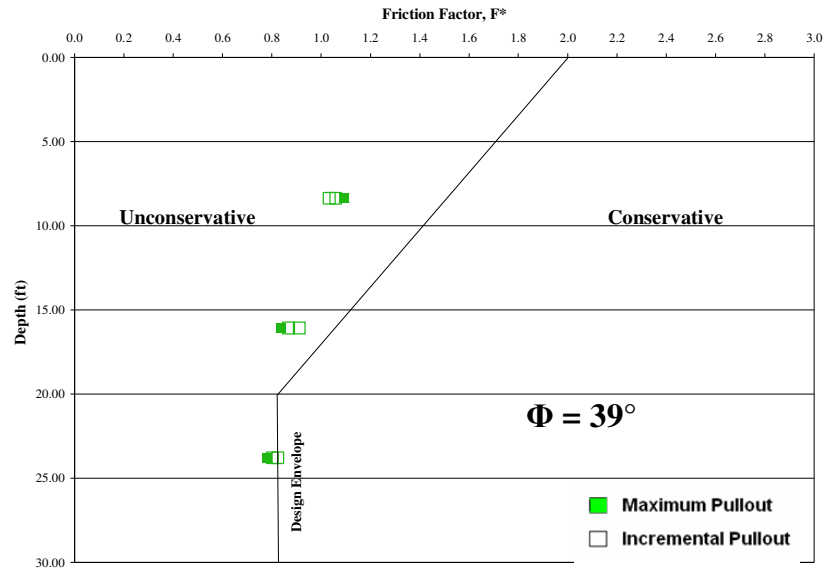
**Figure 4.30 Creep displacement versus time plot from incremental pullout test for all materials with an applied vertical stress of 12 psi**

#### 4.4.4 Friction Factor ( $F^*$ ) Results for Pullout Testing

The friction factors,  $F^*$  for 100% RAP, 50/50% blend and A-3 sand were calculated from the maximum pullout force obtained from both the maximum and incremental pullout tests. Christopher et al (1990) and Elias et al. (2001) presented a standardized expression to calculate  $F^*$ . Using the calculated  $F^*$  and the corresponding design envelope defined by Equations 2.4 and 2.5, Figure 4.31, Figure 4.32, and Figure 4.33 were developed for each material. The calculation of the design envelope required values for the friction angle ( $\phi$ ) and uniformity coefficient ( $C_u$ ). The  $C_u$  values for each material were presented in Section 4.1. Friction angle values were determined by triaxial test performed at the FDOT SMO and were found to be  $39^\circ$ ,  $34.5^\circ$  and  $33^\circ$  for 100% RAP, 50/50% blend and A-3 sand respectively. An embedment length ( $L_e$ ) of 48 inches was used in the calculation of  $F^*$ . The figures compare  $F^*$  values calculated from the pullout tests on each material to the equivalent overburden depth estimated from the applied vertical surface stresses of 6, 12 and 18 psi.

The majority of the  $F^*$  values calculated for 100% RAP are less conservative than the minimum values defined by the design envelope; therefore not recommended for use in MSE walls (Figure 4.31). The 50/50% blend presented in Figure 4.32 displayed a majority of the  $F^*$  values on the conservative side of the friction envelope as did the A-3 sand presented in Figure 4.33; therefore, validating

their use as MSE wall backfill based on pullout load. The results from this study also coincide with the results presented in Chapter 2 from Rathje et al. (2006), Lai and Moore (1990), and RECO (1998). The low friction factors may be associated with the overall low strength of 100% RAP.



**Figure 4.31 Friction factor with the design envelope versus equivalent overburden depth for 100% RAP**

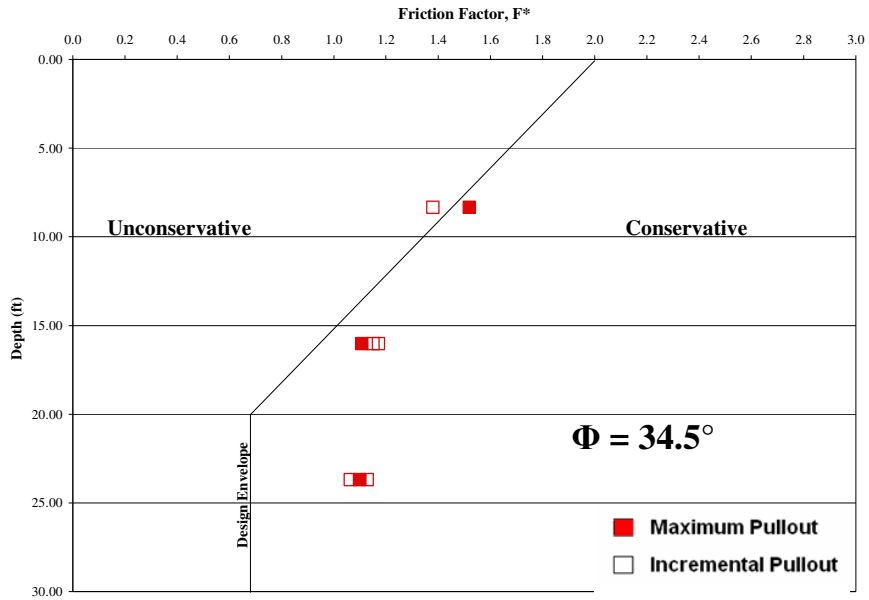


Figure 4.32 Friction factor with the design envelope versus equivalent overburden depth for 50/50% blend

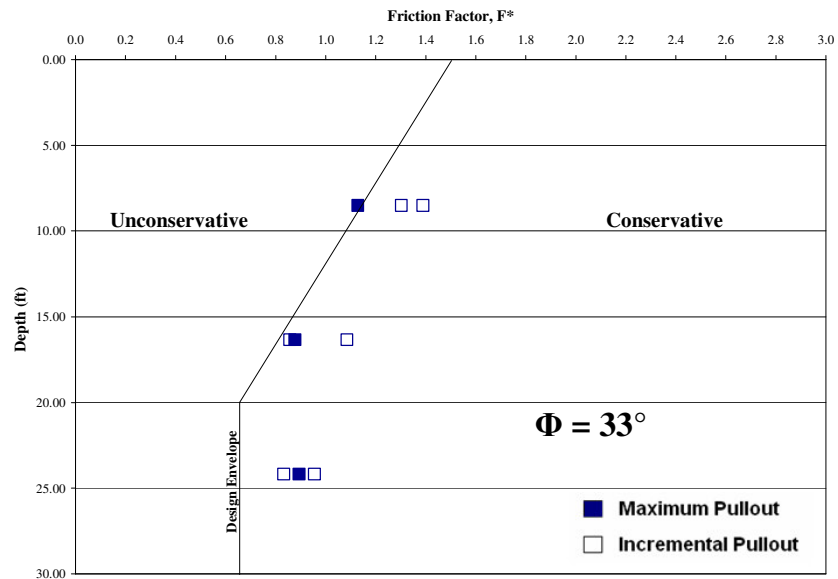


Figure 4.33 Friction factor with the design envelope versus equivalent overburden depth for A-3 sand

## 4.5 Vertical Creep Testing

A series of vertical creep tests were performed at FIT and FDOT-SMO. These tests yielded vertical displacement for 100% RAP, 50/50% blend and A-3 when loaded at an applied vertical stress of 6, 12 or 18 psi.

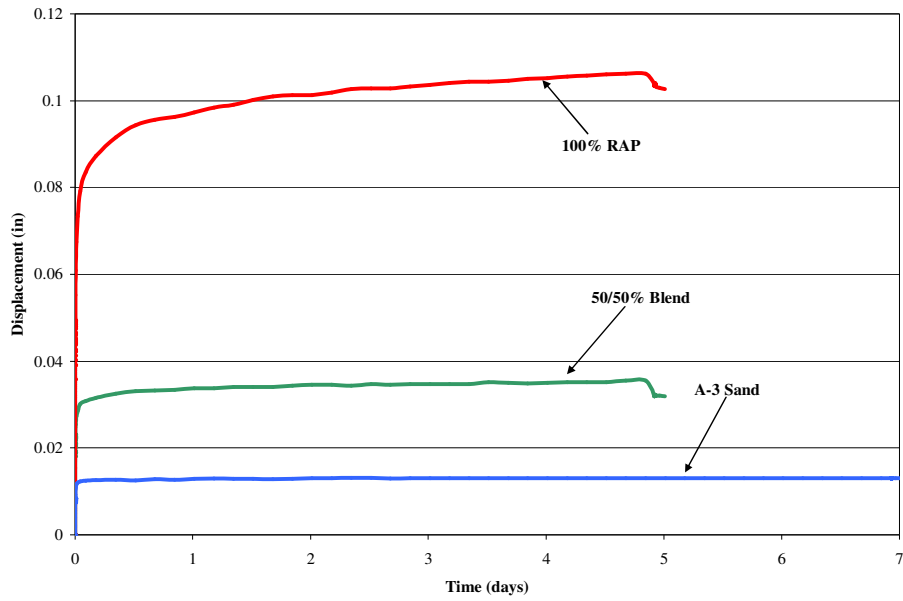
The tests conducted at FIT were small scale tests that measured the displacement of a 4.5 inch tall and 6.0 inch in diameter sample using the procedure developed by Dikova (2006). The vertical creep of these materials was evaluated at time increments of 2 to 7 days. A total of 27 vertical creep tests were performed at FIT.

The tests performed at FDOT-SMO test pit were larger scale creep tests. The first test was a 45 minute vertical creep test performed prior to MSE pullout testing using an I-beam to apply the vertical stress. A total of 27 creep tests were conducted in this set-up with applied vertical stresses of 6, 12 and 18 psi. The second large scale vertical creep test was performed on backfill occupying 1/3 of the FDOT-SMO pit area. These tests were performed for time increments ranging from 1 to 7 days using a 12 inch diameter load plate. A total of 18 creep tests were conducted in this set-up with vertical stresses of 12 and 18 psi.

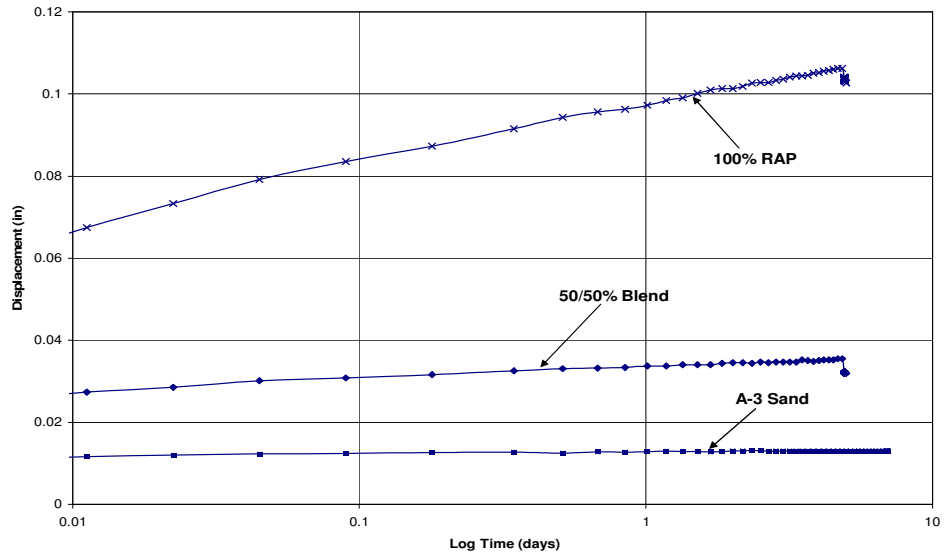
### 4.5.1 Laboratory Testing Results

Vertical displacement due to creep versus time is presented from the data collected from the small scale laboratory creep tests. Figure 4.34 presents the vertical creep displacement versus time for the materials loaded with a vertical stress of 12 psi. The plot for each material is an average of three trials. In these discussions, displacement occurring from 0 to 15 minutes was considered as elastic deformation while plastic, or creep, displacement was considered from 15 minutes to 4.5 days. Analysis of these curves shows that the 100% RAP had the largest elastic deformation followed by the 50/50% blend and the A-3 sand. Appendices B-3 and B-4 presents figures for all vertical displacement versus time tests.

To more accurately analyze creep of each material, creep displacements were compared to the logarithm of time as displayed in Figure 4.35. The linear relationship shown in Figure 4.35 between displacement and time identifies creep due to the constant loading. These slopes represent the plastic deformation and are the greatest for 100% RAP and decrease for the 50/50% blend and A-3 sand. These slopes can be used to identify the displacement due to creep of each material in a specified time interval. Using the data from Figure 4.35, the time increment of 15 minutes (0.01 days) to 4.5 days was used to compare creep caused by constant stress for each backfill material. Figure 4.36 presents the total vertical creep displacement of each backfill material under constant vertical stresses of 6, 12 or 18 psi between 15 minutes and 4.5 days. The displacement increased as the applied vertical stress increased for all materials. The 100% RAP yielded the largest creep displacement; on average 97% greater than the A-3 sand.

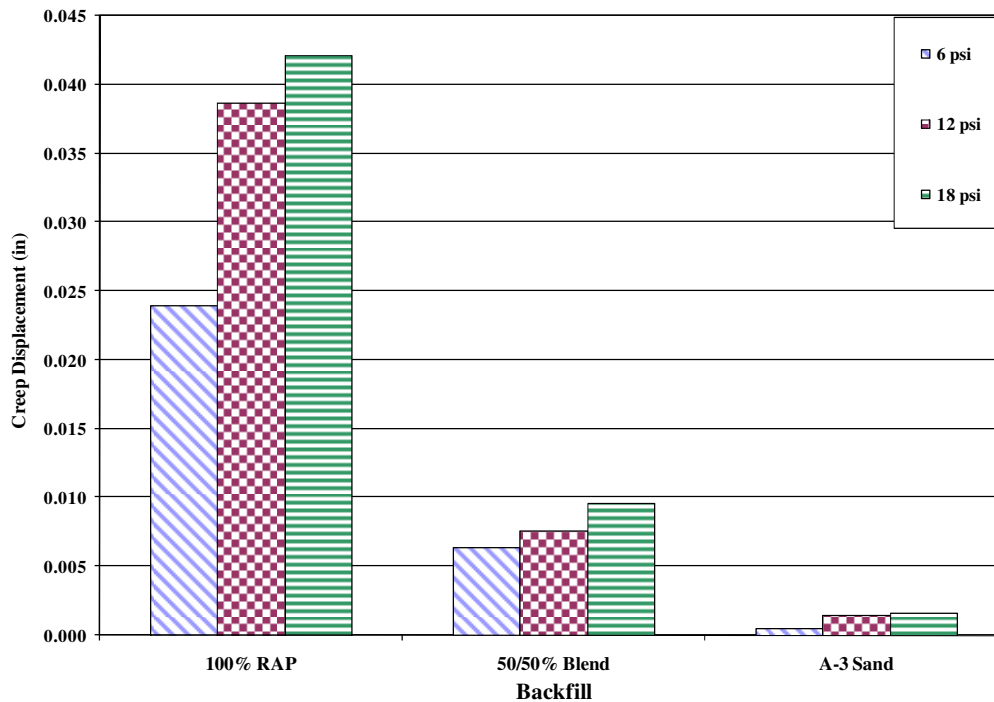


**Figure 4.34 Vertical displacement versus time plots for all materials with applied vertical stress of 12 psi**



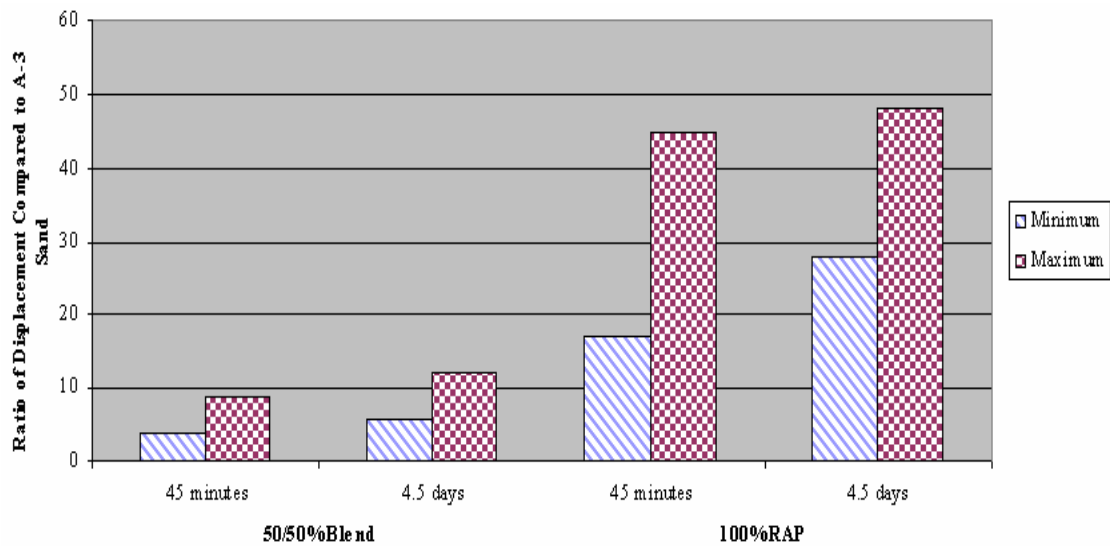
**Figure 4.35 Vertical creep displacement versus log time chart for all materials with applied vertical stress of 12 psi from small scale laboratory tests**





**Figure 4.36 Vertical creep displacement for all materials between 15 minutes and 4.5 days under constant vertical stress of 6, 12 and 18 psi from small scale laboratory tests**

The creep displacements of the 50/50% blend and 100% RAP were compared to the A-3 sand as a ratio of the displacement of a given material to A-3 sand in Figure 4.37. The figure presents the ranges of minimum to maximum ratios for the increase in displacement caused by creep for 50/50% blend and 100% RAP compared to the A-3 sand between 15 and 45 minutes, and 15 minutes and 4.5 days. The 100% RAP presents the greatest creep displacement ratios for both time increments as compared to A-3 sand. The 50/50% blend had at least 10 times the less creep displacement than the 100% RAP.



**Figure 4.37 Minimum and maximum ratios of displacement cause by creep as compared to A-3 sand for 50/50% blend and 100% RAP between 15 and 45 minutes, and 15 minutes and 4.5 days from the small scale creep tests**

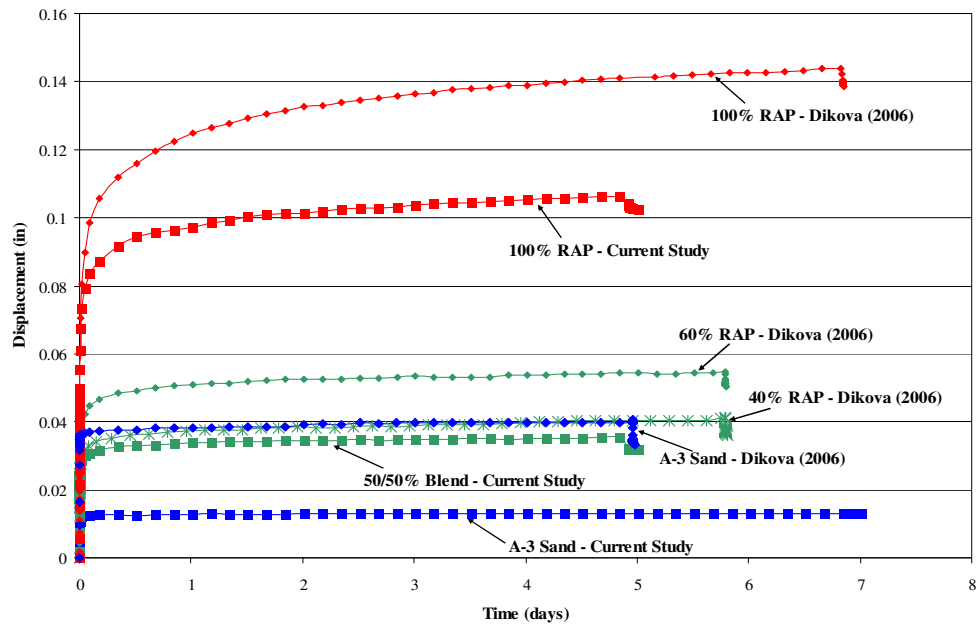
Dikova (2006) tested vertical creep displacement of RAP-sand mixtures at the vertical stresses of 6, 12 and 18 psi. Dikova (2006) concluded that the relative compaction of the samples affected the displacement that occurred as a result of the creep test. So that the results from this study could be compared to Dikova's (2006), samples were compacted to RC values between 95 and 100%.

Table 4.12 present the RC values of this study compared to Dikova's (2006). Vertical creep tests performed for each material in this study were compared to Dikova's (2006) 100, 40/60, 60/40% RAP-sand mixtures and A-3 sand loaded at a vertical stress of 12 psi as presented in Figure 4.38. The figure shows that Dikova's (2006) results present similar trends to the results of this study. The 100% RAP displacements are greater than the 50/50% blend (40% and 60% for Dikova (2006)) and A-3 sand for both studies as presented in Figure 4.39. Dikova's (2006) yielded slightly larger displacements for each backfill material than the creep displacements obtained from this study. This increase in displacement can only be attributed to the difference in the source of the RAP and sand; however, overall the differences in the total displacements were rather small.

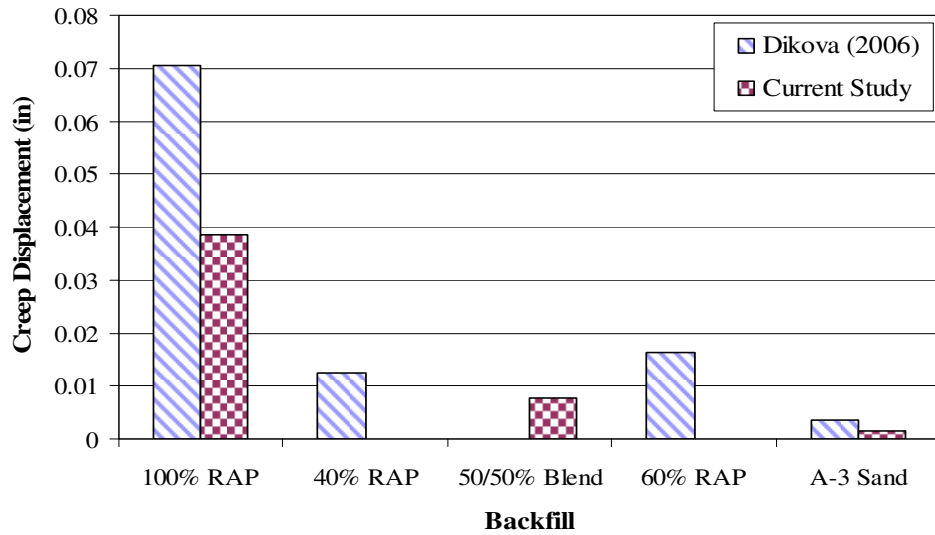
**Table 4.12 Relative compaction values of backfill material comparing this study to Dikova (2006) from the small scale laboratory test**

	Relative Compaction				
	A-3 Sand	40% RAP	50/50% Blend	60% RAP	100% RAP
Dikova (2006)	99.8%	98.7%	x	98.0%	98.2%
Current Study	99.4%	x	100.6%	x	98.4%

x - Material not tested in study



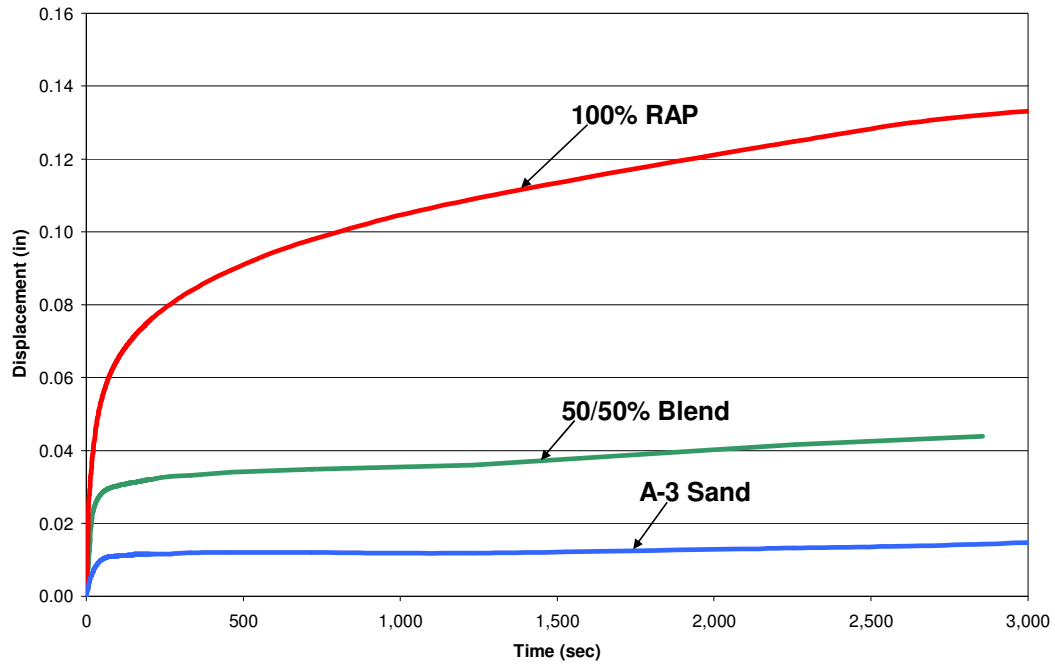
**Figure 4.38 Vertical displacement versus time plots comparing Dikova's (2006) RAP-sand mixtures to current study's backfill with vertical applied stress of 12 psi in small scale laboratory tests**



**Figure 4.39 Vertical creep displacements of backfill material from this study and Dikova (2006) from the small scale laboratory test**

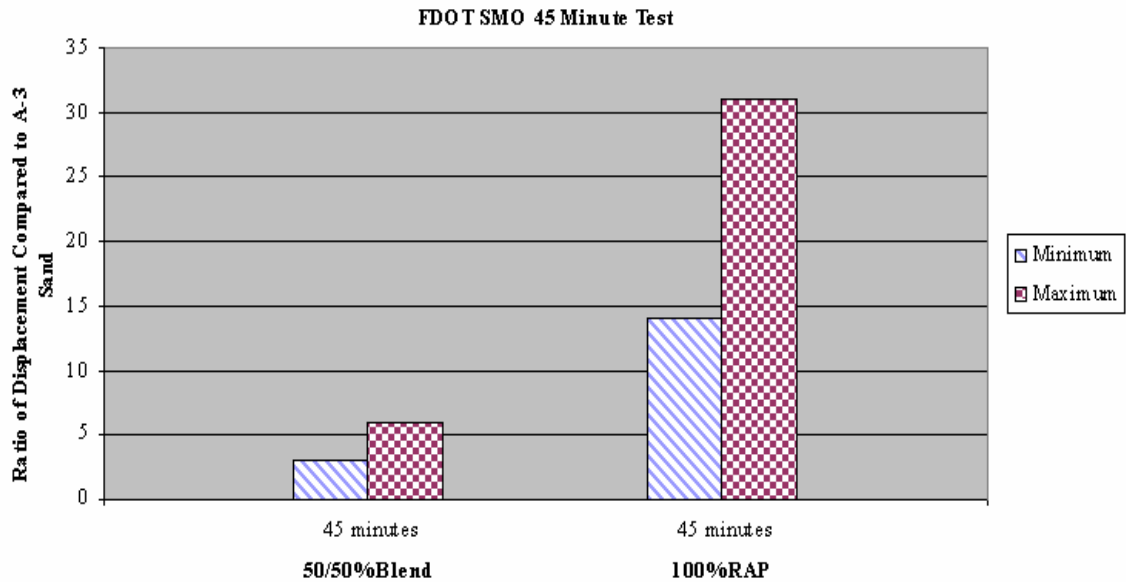
#### **4.5.2 45-Minute Pit Creep Test Results**

A 45-minute creep test was conducted prior to pullout testing on each material with applied vertical stresses of 6, 12 and 18 psi. The vertical stress was applied using a rectangular I-beam that was aligned over the reinforcing strip. Figure 4.40 presents displacement versus time for each material with an applied vertical stress of 12 psi. The resulting plots have similar curves to the test conducted at FIT where the 100% RAP displays the largest displacement followed by the 50/50% blend and A-3 sand. The total displacements that were measured at 45 minutes (2,700 seconds) are similar to the total displacements at the same time period for the laboratory creep tests (Figure 4.34). The displacement versus time plots for 6 and 18 psi vertical stresses are presented in Appendix B-5.



**Figure 4.40 Displacement versus time for 45 minute creep test with applied vertical stress of 12 psi from the 45 minute creep test**

Figure 4.41 displays ratios for the minimum and maximum displacement caused by creep in 50/50% blend and the 100% RAP as compared to the A-3 sand's displacement between 15 and 45 minutes of testing. The 50/50% blend had 3 to 6 times the displacement of the A-3 sand and the 100% RAP had 14 to 31 times the displacement of the A-3 sand. The ratios from the 45 minute test and the FIT small scale laboratory test (Figure 4.37) are very similar, therefore indicating a large decrease in creep displacement, at least 10 times, as a result of mixing RAP and sand.



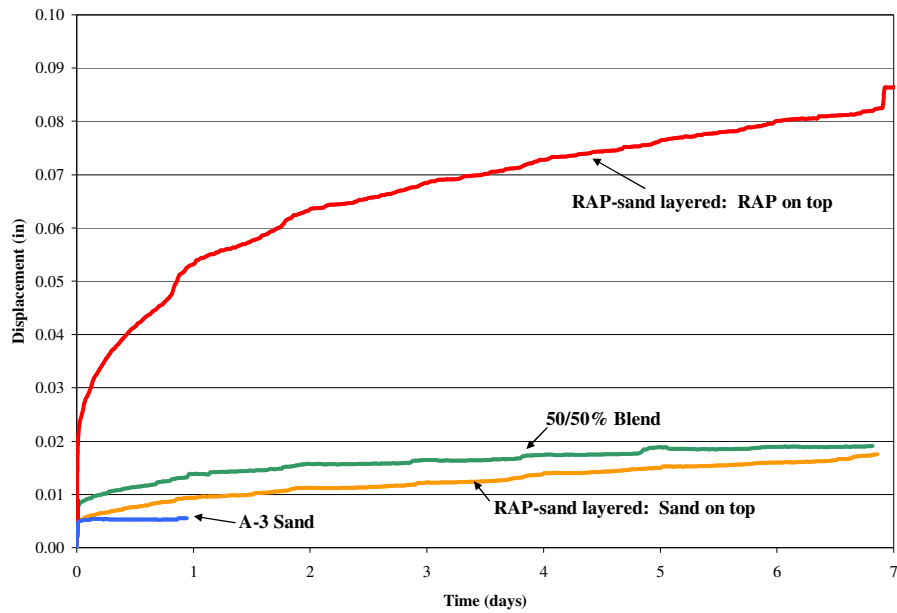
**Figure 4.41 Minimum and maximum ratios of displacement cause by creep as compared to A-3 sand for 50/50% blend and 100% RAP between 15 and 45 minutes from the 45 minute creep test**

### 4.5.3 Large Scale Pit Creep Testing Results

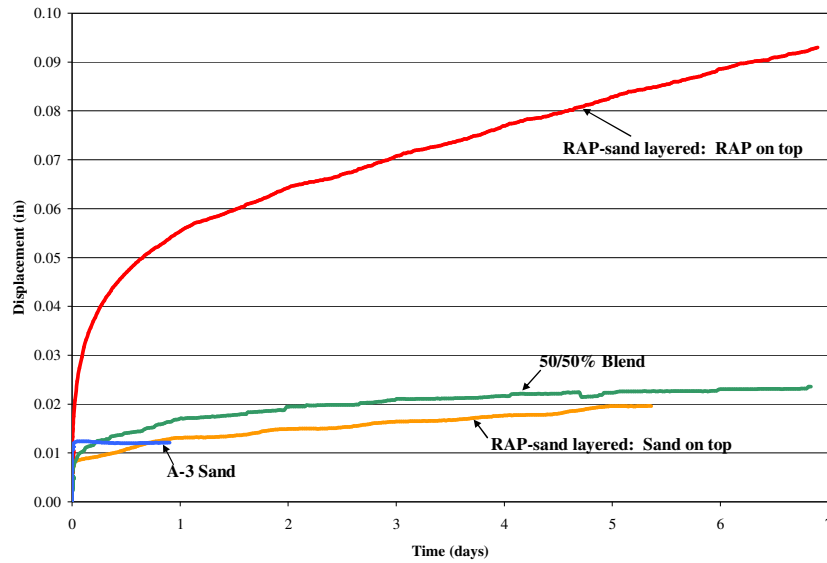
To evaluate the performance of the backfill materials in large scale, or similar to field conditions, vertical creep tests were performed on an 8 ft. wide by 8 ft. long by 3 ft. deep section of the test pit. The materials were tested for vertical creep by applying a load to a 12 inch circular plate which produced stresses of 12 and 18 psi. Four materials were testing in this testing sequence including 50/50% blend, A-3 sand, RAP-sand layers with RAP on the top surface and RAP-sand layers with sand on the top surface. The RAP-sand layered approach was tested in this sequence in accordance with an FDOT memorandum by Malerk and Xanders (2000) for the use of alternative layers of RAP and soil for embankment construction. Layering of the material was not practical in the FIT laboratory creep tests and the 45 minute creep tests due to the small sample thicknesses and the confinement from the 6 inch mold and pit test box.

Figure 4.42 and Figure 4.43 present the displacement versus time plots for all of the materials with applied vertical stresses of 12 and 18 psi respectively. Similar to the behavior in the FIT laboratory tests and 45 minute creep tests, the large scale creep test produced plots in which the 50/50% blend had more displacement than the A-3 sand. The RAP-sand layering with the RAP on the surface had the largest amount of displacement after 4.5 days. The RAP-sand layering with the sand on the

surface had less displacement than the 50/50% blend and the RAP-sand layered with the RAP on the top surface. The results from the layered creep tests suggest that the top layer influences the majority of the initial creep behavior of the entire backfill material.



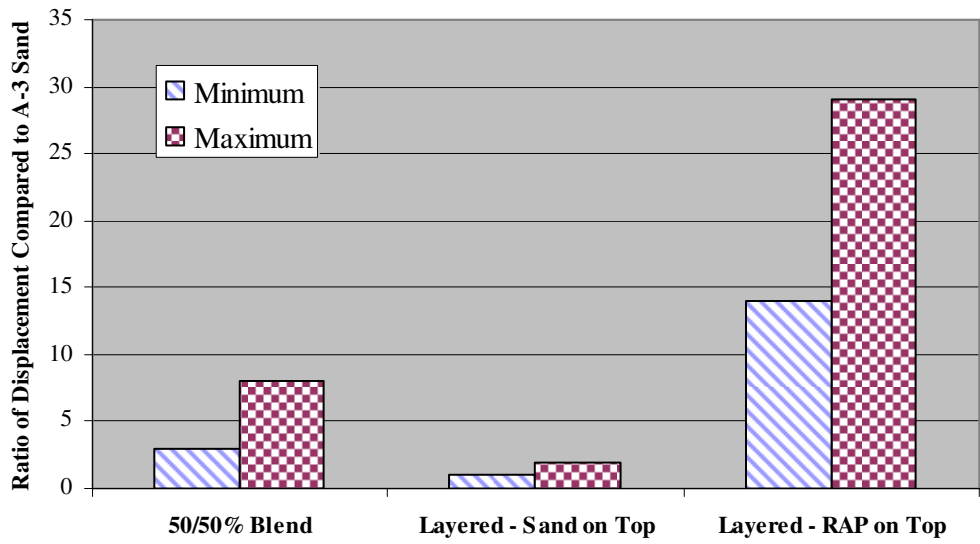
**Figure 4.42 Displacement versus time plot for all backfill materials with applied vertical stress of 12 psi from the large-scale pit creep tests**



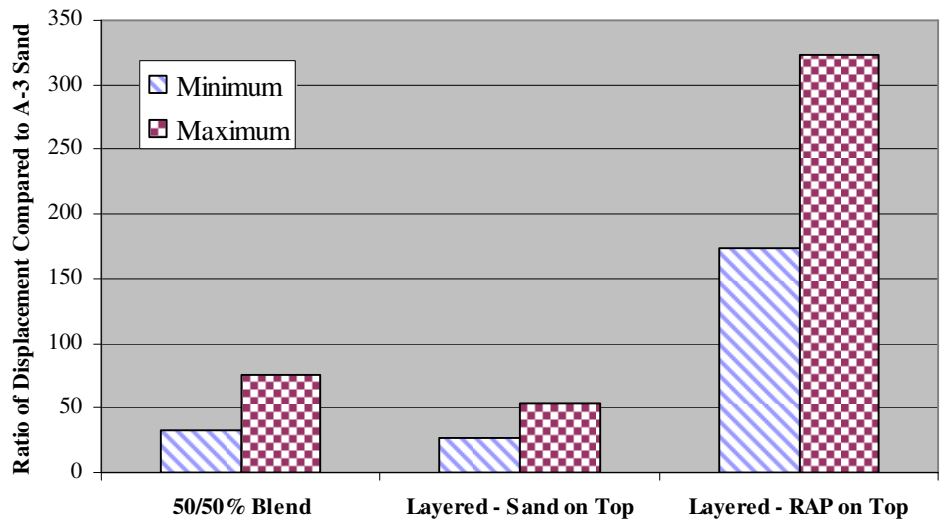
**Figure 4.43 Displacement versus time plot for all backfill materials with applied vertical stress of 18 psi from the large-scale pit creep tests**

Figure 4.44 presents the minimum and maximum ratios for the increase in displacement caused by creep for 50/50% blend and the RAP-sand layers as compared to the A-3 sand between 15 and 45 minutes, and 15 minutes and 4.5 days in Figure 4.45. Both figures support the fact that the material placed on the top for the RAP-sand layered test will influence the displacement caused by creep where the RAP-sand layered with RAP on top had at least 10 times the displacement of the RAP-sand layered with sand on top for both time increments. The 50/50% blend also had at least 10 times the creep displacement of the RAP-sand layered with RAP on top for both time increments. By placing the sand of the top layer of the RAP-sand layered setup, the material had slightly less displacement than the 50/50% blend as compared to the A-3 sand shown in Figure 4.44 and Figure 4.45.





**Figure 4.44** Minimum and maximum ratios of displacement caused by creep for 50/50% blend and RAP-sand layers as compared to A-3 sand between 15 and 45 minutes from the large-scale creep test



**Figure 4.45** Minimum and maximum ratios of displacement caused by creep for 50/50% blend and rap-sand layers as compared to a-3 sand between 15 minutes and 4.5 days from the large scale creep test

#### 4.5.4 Creep Testing Summary

The three vertical creep tests discussed above measured the vertical displacement of each backfill material when influenced by an applied vertical stress of 6, 12 or 18 psi. Though each tests measured vertical displacement, the tests cannot be compared to one another due to several major differences in testing method. First, the thickness of the material in each test varied from 4.5 inches in the small scale FIT laboratory test, to 20 inches in the 45 minute test, to 36 inches in the large scale test. The large material thicknesses in the FDOT-SMO pit creep tests made it difficult to compare strain values unless finite thicknesses were assumed for each test.

Second, the confinement of the material differed for each of the tests. The FIT laboratory test had confined compression due to the 6 inch tall, 6 inch diameter mold which resulted in uni-axial compression produced by the stress applied by the circular load plate. The 45-minute test material was confined by the 8 ft. wide by 4 ft. long by 20 inch deep box; however the I-beam created a plane strain condition. The large scale creep test material had minimum confinement resulting in tri-axial compression created from the stress applied by the 12 inch circular load plate.

A creep index ( $C_1$ ) was developed from the secondary compression index ( $C_\alpha$ ) to where the change in void ratio is replaced with the change in displacement with in a time interval. The  $C_1$  value is calculated by the slope of the displacement versus logarithm time plot where creep displacement occurs.

To form a comparison from the three creep tests, the  $C_1$  values for each test were calculated as presented in Table 4.13. The 45 minute test  $C_1$  values are greater when comparing the values of like materials. This increase can be attributed to the plane strain conditions created by the I-beam which displaces the material at a greater rate resulting in a larger slope. The 100% RAP displays the greatest  $C_1$  values followed by the 50/50% blend then A-3 sand. The  $C_1$  increased by an order of magnitude from A-3 sand to 100% RAP. The table also shows an increase of  $C_1$  values with an increase of applied vertical stress for the majority of the creep tests. The 50/50% blend and A-3 sand display a large range of  $C_1$  values for each applied vertical stress where the 100% RAP index values are consistently within a small range of 0.01 to 0.06.

**Table 4.13 Creep Index (CI) values for each material tested for vertical creep at FIT laboratory and FDOT SMO**

		Creep Index, $C_1$									
Vertical Creep Test Sequence	Applied Vertical Stress (psi)	A-3 Sand		50/50% Blend		100% RAP		Layered - RAP on Top		Layered - Sand on Top	
		0.03 days (45 mins)	4.5 days	0.03 days (45 mins)	4.5 days	0.03 days (45 mins)	4.5 days	0.03 days (45 mins)	4.5 days	0.03 days (45 mins)	4.5 days
FIT Laboratory -Dikova (2006)	6	0.0003	0.0010	0.005	0.004	0.03	0.03				
	12	0.0007	0.0014	0.007	0.005	0.03	0.03				
	18	0.0012	0.0011	0.010	0.007	0.04	0.03				
FIT Laboratory -Current Study	6	0.0002	0.0002	0.002	0.002	0.01	0.01				
	12	0.0011	0.0005	0.004	0.003	0.02	0.01				
	18	0.0007	0.0004	0.006	0.005	0.02	0.02				
FDOT SMO Pit - 45 minute	6	0.0022		0.013		0.03					
	12	0.0042		0.018		0.06					
	18	0.0021		0.007		0.06					
FDOT SMO Pit - Large scale	6										
	12	0.0007	0.0001	0.002	0.004			0.01	0.02	0.001	0.004
	18	0.0004	0.0001	0.003	0.006			0.01	0.02	0.001	0.004

## **4.6 Statewide Variability Results**

Critical variables were evaluated to characterize the engineering behavior of each sample. Laboratory and field data were compared to determine the validity of sample test results and the reuse potential of each sample. Statistical methods were implemented to analyze results to develop statewide conclusions.

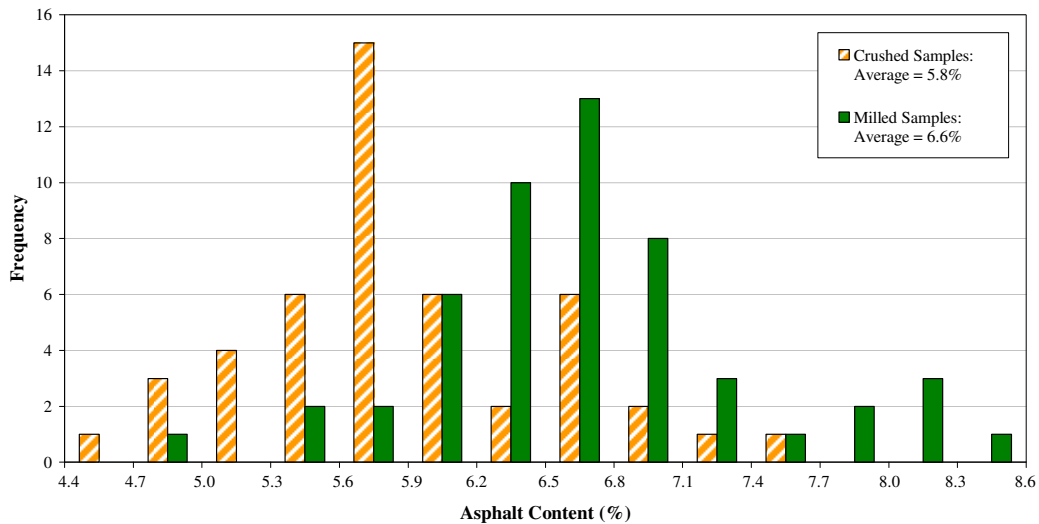
### **4.6.1 Asphalt Content**

Asphalt content is most highly influenced by the properties associated with the original pavement from which RAP was reclaimed. Factors that influence this ingredient of RAP include the mix design of the origin pavement, the quality of the origin pavement, and the accumulated traffic loading. In addition, asphalt content can be influenced by the mixing of soils or granular debris with RAP during reclamation or processing, as well as the mixing of different RAP sources in the same stockpile.

In this study, the asphalt content of RAP was analyzed using two data sets, laboratory and field data. Laboratory testing was conducted on samples based on processing type and geography. Field asphalt content data was compiled from composition reports of existing pavements on state highways. These samples represented the source condition of RAP, before reclamation by milled or processing by crushing. Unlike laboratory asphalt content tests of collected RAP samples, pavement composition reports were used to evaluate the variability in asphalt content in a much larger statewide sample population. The asphalt content in both laboratory and field data was determined by the ignition furnace method following FM 5-563 as the percentage by weight.

#### **4.6.1.1 Laboratory Testing Results**

The greatest variability in material properties between post-milling processed samples (crushed RAP) and un-processing raw milled samples (milled RAP) was exhibited by the asphalt content and gradation. Figure 4.46 shows a frequency histogram of the asphalt content test results for both crushed and milled RAP, divided into 0.3% intervals. The asphalt content of crushed RAP samples is typically lower than milled RAP samples, although the range overlaps substantially. Crushed RAP typically has lower asphalt content due to the stockpiling of RAP before crushing, compared to milled RAP, which is typically directly stockpiled after reclamation. Crushed RAP is more often compiled from multiple sources as well as minor amounts of base aggregate or soil extracted during reclamation. Table 4.14 shows summary statistics of asphalt content test results.



**Figure 4.46 Histogram of asphalt content test results**

**Table 4.14 Summary statistics of asphalt content test results**

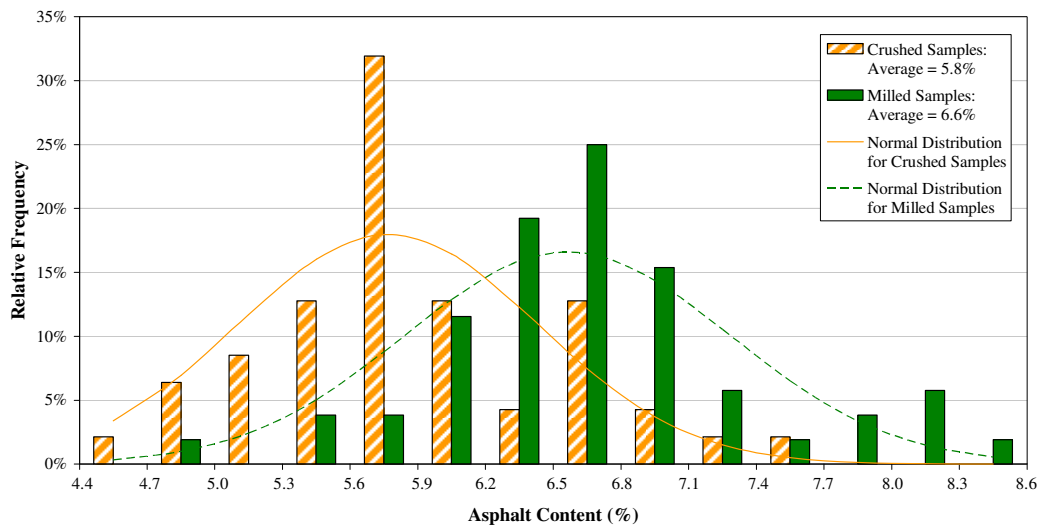
	Number of Samples Tested	Mean	Minimum	Maximum	Range	Standard Deviation
All Samples	99	6.2%	4.5%	8.5%	4.0%	0.8%
Milled Samples	52	6.6%	4.9%	8.5%	3.6%	0.7%
Crushed Samples	47	5.8%	4.5%	7.4%	2.9%	0.7%

The range in each data set was relatively wide, indicating that the variability in asphalt content was wide for both material types. The overlap between the histograms in each data set also indicated differences in asphalt content between crushed and milled samples were not constant. The higher frequency of samples within the median interval indicated a disproportionate number of samples close to the mean. The median interval was the interval on the histogram where the mean was located and ranges from 6.5% to 6.8% for milled samples and from 5.6% to 5.9% for crushed samples. In addition, six milled samples had results in the 7.7% to 8.6% range, suggesting a subset of samples that exceeds the mean by a large margin.

#### **4.6.1.1.1 Analysis of Normality**

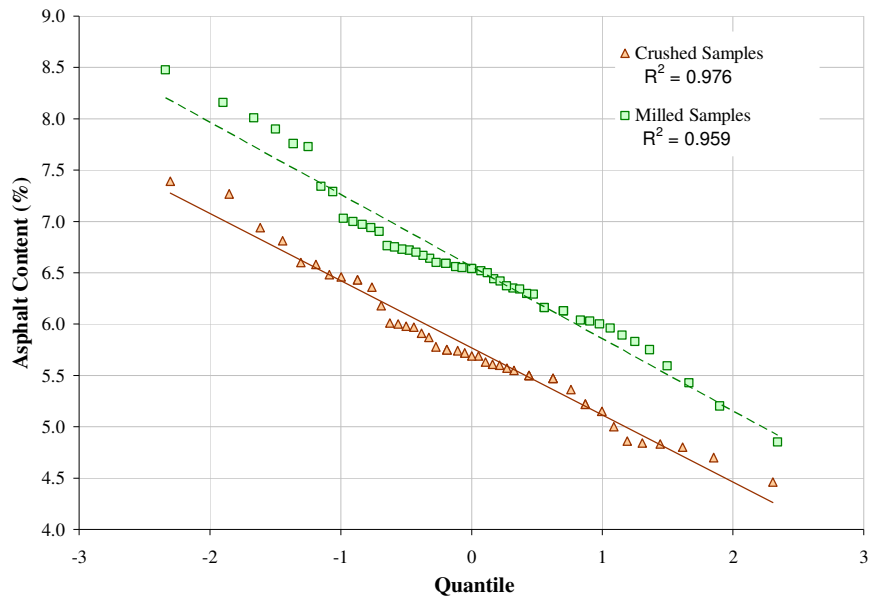
The fundamental assumption of this study was the asphalt binder in RAP influences engineering behavior, causing RAP to act significantly different in earthwork application than conventional aggregates or soils. Therefore, the

measure of the amount of asphalt binder in RAP, the asphalt content, was the most critical parameter for analyzing the statewide variability in engineering behavior of RAP materials. To validate the applicability of sample test results with the larger population of candidate RAP materials, it was assumed that the tested asphalt content results were normally distributed. To analyze the normality of the sample distribution, an overlay of normal distributions was placed over relative frequency histograms in Figure 4.47. Deviations between the sample distribution and the corresponding normal distribution suggest non-normal behavior. Given that non-normal test results may not provide the desired representative distribution for the entire RAP population, additional statistical tests were required to evaluate the normality of the asphalt content test results.



**Figure 4.47 Relative histogram of asphalt content results**

The normality of the asphalt content test results can be verified by constructing a normal probability plot of all asphalt content data. Instead of grouping samples into artificial ranges to produce a histogram, a normal probability plot compares each value in a data set with the standard normal quantile corresponding to a data set of the same size. When normally-distributed data was plotted against the corresponding normal quantile, an approximate straight line should form. Large deviations or non-linear trends in the plot suggest non-normal characteristics in the underlying data set. Figure 4.48 shows the normal probability plots for crushed and milled RAP samples, both producing near-linear relationships, therefore indicating normal distributions.



**Figure 4.48 Normal probability plot of asphalt content test results**

To evaluate the hypothesis of normality in a normal probability plot, the correlation coefficient ( $R^2$ ) was compared to critical values at given confidence intervals (Johnson 2002). Low correlation coefficients indicate that the null hypothesis was correct and the samples were not from a normal distribution. Crushed and milled data sets were tested at the 99% and 95% confidence levels. The correlation coefficients exceeded the critical values; therefore, the null hypothesis was rejected for both data sets at the 99% and 95% confidence levels. It was concluded that asphalt content test results provide an acceptable normal distribution. Any non-normal characteristics observed in the histograms in Figure 4.46 and Figure 4.47 can be attributed to the nature of the histogram's development, where the scale of intervals used to categorize a small data set can distort the frequency near the mean. These histograms show a summary of the sample data distribution and a normal probability plot should not be used to evaluate the true normality of a data set.

#### **4.6.1.1.2 Geographic Differences**

To evaluate regional differences in asphalt content, samples were further categorized by processing method and source location. Samples were grouped based on the FDOT District where the source facility was located. Table 4.15 shows the average asphalt content of samples collected from each FDOT District for each processing method. District average asphalt content was generally close to

the overall statewide average, no single District average deviated from the statewide sample average by more than 0.3% for milled or 0.4% for crushed.

**Table 4.15 Summary of asphalt content by district and processing method**

FDOT District & Geographic Region		Milled			Crushed		
		Number of Samples Tested	Mean	Standard Deviation	Number of Samples Tested	Mean	Standard Deviation
1	Southwest & South Central	8	6.7%	1.7%	10	6.2%	0.5%
2	Northeast & North Central	4	6.6%	1.2%	7	5.7%	0.5%
3	Northwest	10	6.7%	2.4%	9	5.5%	0.3%
4	Southeast	7	6.3%	0.8%	4	5.5%	0.1%
5	East Central	16	6.5%	1.4%	8	5.6%	0.4%
6	South	3	6.5%	0.3%	7	6.1%	0.3%
7	West Central	4	6.4%	1.7%	2	5.4%	0.1%
Statewide		52	6.6%	0.7%	47	5.8%	0.7%

In general, the means for crushed samples in neighboring Districts 1 and 6 was larger than the statewide average and the averages for all other Districts, indicating that crushed RAP materials at the southern end of the state may have higher asphalt content. District means from milled RAP samples did not exhibit any significant geographic differences. In summary, most samples collected in this study did not exhibit statistical differences in asphalt content between FDOT Districts. Only crushed RAP samples from District 1, in the southwest region of the state, exhibited higher asphalt content than the rest of the state. Overall, the low number of samples available in Districts 2, 4, 6 and 7 indicate that averages in these regions may not be representative. Laboratory sample results should be used as examples of typical values found in each District, but data available cannot be used to evaluate precise differences between most regions of the state. It was concluded that dividing samples by FDOT Districts does not indicate that geography was a key factor to influence asphalt content of milled RAP.

#### **4.6.1.2 Pavement Composition Reports**

Pavement composition reports were used to evaluate the variability in asphalt content for a statewide sample population. They represent the in-situ condition of RAP before reclamation and processing. The asphalt content of existing pavements more closely represents milled RAP samples, since these samples contain material from the pavement surface layers before reclamation and blending with soil or



aggregates. Pavement composition reports were compiled by the Central Asphalt Laboratory at the SMO from FDOT field offices in all Districts and the FTE. Reports were compiled from projects throughout the state dated from 2000 to 2007.

Asphalt content for each project was analyzed separately. Any asphalt content data available for a project was assumed to be representative of the entire road segment length. As a result, the asphalt content for road segments with a higher sampling frequency, or number of cores per mile, will typically have a larger range and a smaller standard deviation. The average, standard deviation and asphalt content range were determined for each project based on the data shown in Appendix C-2.

**Table 4.16 Pavement composition reports from Bay County, FL**

State Road and US Highway Number	Project Number (F.P.N.)	Project Length (Miles)	Number of Lanes	Project Area (Lane-Miles)	Number of Cores Tested	Asphalt Content (%)	
						Average	Range
SR 30/30A (US 98)	411384-1-52-01	5.57	4	22.3	24	7.63	7.20 - 8.34
SR 30 (US 98)	416917-1-52-01	2.34	5	11.7	25	5.93	5.49 - 6.38
SR 30A (US 98A)	411401-1-52-01	2.52	5	12.6	12	6.67	6.15 - 7.20
SR 75 (US 231)	413430-1-52-01	1.13	4	4.5	8	6.37	5.91 - 6.68
SR 75 (US 231)	413431-1-52-01	5.83	4	23.3	24	10.04	8.35 - 11.52
SR 75 (US 231)	413433-1-52-01	9.38	4	37.5	36	6.98	5.52 - 10.27
SR 75 (US 231)	413434-1-52-01	9.61	4	38.5	20	5.17	4.68 - 5.78
SR 390	413438-1-52-01	5.00	2	10.0	10	6.13	5.38 - 6.70
County Total		41.38		160.4	158		

As an example, all results from Bay County are shown in Table 4.16. State highways in the county include low-volume rural roads as well as two high-volume US Highways, routes 98 and 231. Bay County was also chosen because it has the largest range of project-specific average asphalt contents, indicating high variability in material properties within a single county. The road segments with the lowest and highest average asphalt content within Bay County were located within 10

miles of each other on US-231, illustrating the high variability of asphalt content within some localized areas.

#### ***4.6.1.2.1 County Averages***

Weighted averages were calculated from to determine county and regional averages. Weighted averages were considered representative of typical material properties within each geographic area, because of the large quantity of materials tested. Weighted averages were calculated based on the estimated area of pavement for each project. The project limit mileposts were used to estimate the project length in miles. The number of lanes for each road segment was used to estimate the project width and was multiplied by the project length to estimate the area of pavement, in lane-miles. This area estimate excludes pavement area dedicated to turn lanes, driveways and shoulders. Turn lanes were only counted when oriented as continuous bi-directional left turn lanes; this was noted wherever a road segment was designated as having an odd number of lanes. Examples of county averages are shown in Table 4.17 and Table 4.18.

County averages were compiled using the ten largest counties in Table 4.17 and the ten smallest counties in Table 4.18, based on the number of lane-miles analyzed. The range of project averages for each county was compiled from the project-specific average. This list shows significant variation in asphalt content within some counties, but less overall variation around the state. In general, the counties listed in both tables have average asphalt content within 1.0% of the statewide weighted field sample average of 6.8%. This shows that a majority of projects were close to statewide average.

**Table 4.17 Summary of asphalt content for the ten largest counties**

County	FDOT District	Number of Projects	Total Project Length (Miles)	Total Project Area (Lane-Miles)	Asphalt Content (%)	
					Weighted Average	Range of Project Averages
Alachua	2	19	79	303	7.3	6.5 - 8.3
Brevard	5	15	82	251	6.3	5.1 - 7.4
Broward	4	28	80	392	6.7	6.1 - 7.6
Columbia	2	14	105	343	6.6	5.6 - 6.6
Duval	2	52	200	813	6.5	5.1 - 7.7
Marion	5	14	98	371	7.0	6.5 - 8.0
Orange	5	18	93	370	6.7	6.4 - 7.0
Palm Beach	4	19	80	261	6.5	5.6 - 7.2
Taylor	2	8	82	262	7.3	6.6 - 7.9
Volusia	5	19	119	318	6.6	6.0 - 7.1
Statewide Total		496	2,518	8,198		
Average (Ten Largest)					6.7	5.1 - 8.3
Statewide Average (All Projects)					6.8	4.6 - 10.0

**Table 4.18 Summary of asphalt content for the ten smallest counties**

County	FDOT District	Number of Projects	Total Project Length (Miles)	Total Project Area (Lane-Miles)	Asphalt Content (%)	
					Weighted Average	Range of Project Averages
Gulf	3	1	2	6	6.8	N/A
Hendry	1	1	5	22	7.7	N/A
Holmes	3	2	7	18	6.1	6.1 - 6.2
Jefferson	3	1	7	15	7.5	N/A
Monroe	6	2	5	9	7.4	6.8 - 8.1
Okaloosa	3	3	6	23	6.0	6.0 - 6.1
Pasco	7	4	13	27	6.4	6.2 - 6.6
Pinellas	7	3	5	24	6.6	6.2 - 7.2
Union	2	1	3	5	5.8	N/A
Wakulla	3	1	13	26	5.8	N/A
Average (Ten Smallest)					6.6	6.0 - 8.1
Statewide Average (All Projects)					6.8	4.6 -

#### 4.6.1.2.2 Minimum Sample Size

Statistical methods were utilized to analyze geographic differences in asphalt content. Regional boundaries were determined using statistical similarities in project average asphalt content between adjoining counties. To compare county averages at a reasonable level of significance, a minimum number of projects per county were required. The minimum number of projects or sample size,  $n$ , was determined based the desired margin of error ( $\delta$ ), confidence level ( $\alpha$ ) and population standard deviation ( $\sigma$ ), using Equation 4.2. The critical value was  $z_{\alpha/2}$ , from two-tailed limits of the normal distribution at a given confidence level,  $\alpha$ . For a 90% confidence level,  $\alpha = 0.10$  and  $z_{\alpha/2} = -1.645$ .

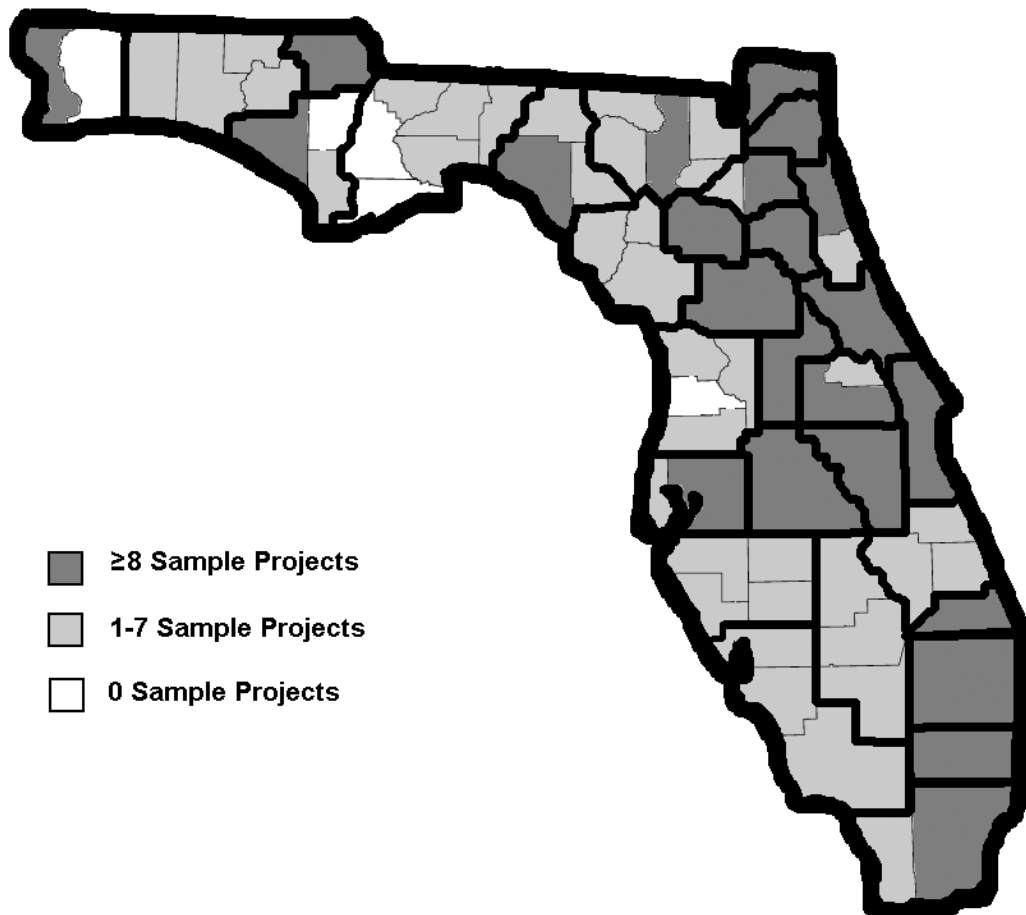
$$n = \left( \frac{z_{\alpha/2}}{\delta} \right)^2 * \sigma^2 \quad \text{Equation 4.2}$$

The margin of error is the maximum difference between the observed mean and true mean for a given sample. Laboratory results in Table 4.15 indicate the minimum standard deviation for milled RAP samples within a single FDOT District was 0.3%. The 0.3% standard deviation occurred in District 6, the District with the smallest geographic area, encompasses only Miami-Dade and Monroe counties. In a normal distribution, 68% of values will be within one standard deviation above or below the average. Therefore the standard deviation in District 6 reflects the relative variability within a smaller geographic area, dominated by one county. It was decided that the margin of error for pavement composition samples within any single county should exhibit a similar level of variability. At 90% confidence, the margin of error in District 6 of the laboratory samples is approximately 0.4%. Therefore the margin of error,  $\delta$ , was chosen to be 0.4%. The statewide arithmetic project average asphalt content was 6.73%, with a standard deviation of 0.63%. The statewide weighted project average asphalt content was 6.78%, with a weighted standard deviation of 0.65%. Given that the weighted average for each county was used for comparison, the statewide population weighted standard deviation was used ( $\sigma = 0.65\%$ ). Therefore,  $n = 7.2$  and a minimum of 8 samples per county was required to determine the county average by a margin of 0.4% at the 90% confidence level.

#### 4.6.1.2.3 Determination of County Groups

Pavement composition reports were typically conducted on highways before milling for resurfacing or reconstruction projects. Therefore, asphalt content data was only available from highways where milling has taken place between 2000 and 2007. Available data does not include every state highway in every county. Data was extremely limited in most rural counties and some urban counties. Based on

the minimum sample size, a minimum of 8 projects were required to independently analyze each county. Of the 67 counties in Florida, 49 counties had less than 8 projects and the remaining 23 counties had greater than 8 projects. It was also noted that 5 counties had no pavement composition reports available. Counties with less than 8 projects represent 66% of the state's counties and contain 39% of the total lane-miles of state highways. Therefore, these generally smaller and rural counties have less data available; however can still represent a considerable proportion of the state's pavement materials. To compare county averages, counties with fewer than 8 projects were grouped with adjoining counties to form "county groups" meeting the minimum sample size. County group boundaries were drawn based on geography and similar asphalt content results. Counties with no results were also grouped with adjoining counties. Figure 4.49 shows the county group boundaries and the number of projects per county. Counties with less than 8 sample projects are indicated with either no or light shading.



**Figure 4.49** Number of projects per county and county group boundaries

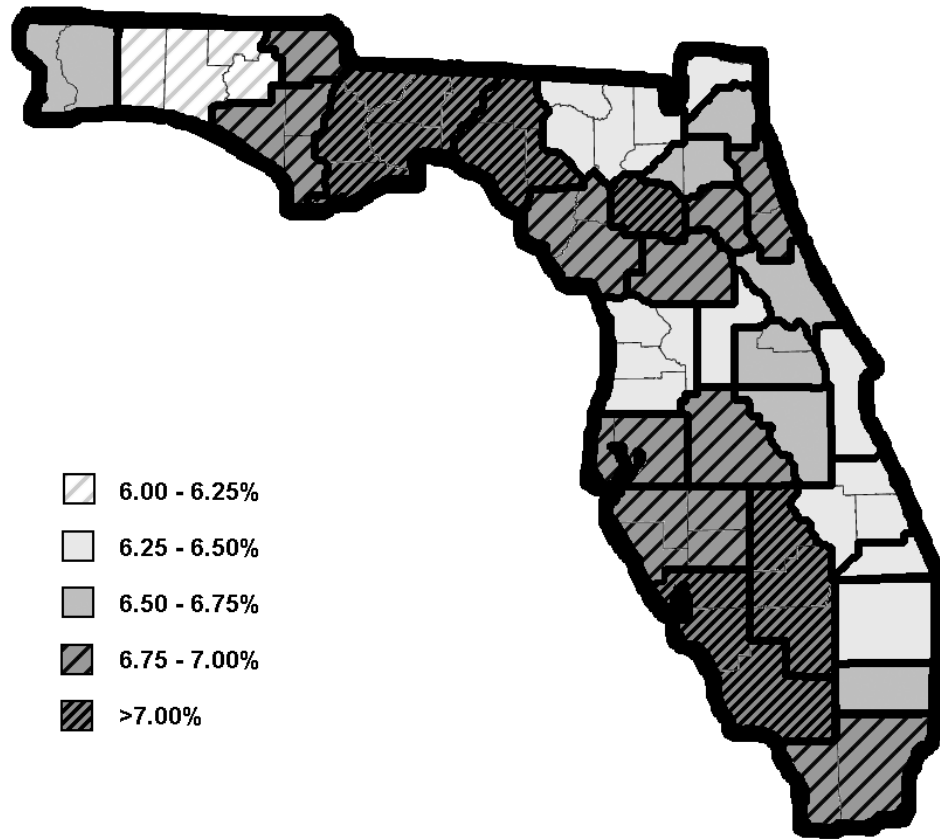
A total of 31 county groups were identified across the state. The county groups with largest number of counties occurred in the northwest, north-central and southwest regions of the state, as these regions have less available data and a high proportion of rural areas. By comparison, several counties contained sufficient data to be analyzed separately, despite their small area, population or rural characteristics. Specifically, Nassau, Putnam, Columbia, Taylor and Jackson counties each had 8 or more projects, while having a 2005 population of less than 75,000 people (U.S. Census Bureau). These counties were generally concentrated in the northeast region of the state. Likewise, several counties with large populations did not have a comparable level of projects available for analysis. Collier, Lee, Leon, Manatee, Pasco, Pinellas, Sarasota, Seminole, and St. Lucie counties each had less than 8 projects available for analysis and a 2005 population of greater than 200,000 people, (U.S. Census Bureau). In addition, Hernando and Santa Rosa counties each have a population of greater than 100,000 and had no available asphalt content data. These lightly-sampled counties were primarily concentrated in the southwest region of the state, suggesting that this region may be underrepresented in the source data set. Differences in the number of projects highlight the limitations of the available data. Asphalt content data from pavement composition reports was limited in some counties, although combining data based on county groups allowed for generalized comparisons of regional differences in asphalt content, within the wide range of typical values. The weighted average of asphalt content was used for comparison between county groups; therefore, differences in the quantity of available data between county groups did not influence regional comparisons.

#### ***4.6.1.2.4 County Group Results***

The weighted average, weighted standard deviation and range were calculated for each county group. Table 4.19 shows a summary of results for the 31 county groups. County groups were identified based on the largest town or city within each group. Figure 4.50 shows a map with each county group, shaded based on the average asphalt content.

**Table 4.19 Pavement composition results by county group**

Region	County Group (Major Cities)	Counties	Pavement Composition Reports				
			Number of Projects	Total Lane Miles Analyzed	Weighted Average Asphalt Content (%)	Weighted Standard Deviation (%)	Range (%)
<b>NW</b>			<b>47</b>	<b>589</b>	<b>6.7</b>	<b>1.1</b>	<b>5.2 - 10.0</b>
	Pensacola	Escambia, Santa Rosa	17	128	6.7	0.6	5.6 - 7.9
	Ft. Walton Beach	Okaloosa, Walton, Holmes, Washington	8	105	6.1	0.1	5.9 - 6.3
	Marianna	Jackson	13	189	6.8	1.2	5.6 - 9.8
	Panama City	Bay, Gulf, Calhoun	9	167	6.9	1.6	5.2 - 10
<b>Tal/BB</b>	<b>BB</b>		<b>25</b>	<b>586</b>	<b>7.4</b>	<b>0.6</b>	<b>5.8 - 8.5</b>
	Perry	Taylor, Lafayette, Madison	16	363	7.4	0.5	6.5 - 7.9
	Tallahassee	Leon, Gadsden, Wakulla, Jefferson, Liberty, Franklin	9	223	7.5	0.8	5.8 - 8.5
<b>NE</b>	<b>NE</b>		<b>131</b>	<b>2216</b>	<b>6.6</b>	<b>0.5</b>	<b>4.6 - 8.2</b>
	Fernandina Beach	Nassau	10	157	6.4	0.5	6.3 - 7.3
	Jacksonville	Duval	52	813	6.6	0.5	5.1 - 7.7
	Lake City	Columbia, Suwannee, Hamilton, Baker, Union	26	634	6.5	0.6	5.4 - 7.4
	Orange Park	Bradford, Clay	16	190	6.7	0.6	6.1 - 8.2
	Palatka	Putnam	12	158	6.8	0.6	5.4 - 7.5
	St Augustine	St. Johns, Flagler	15	264	7.3	0.5	4.6 - 7.3
<b>NC</b>	<b>NC</b>		<b>44</b>	<b>910</b>	<b>7.1</b>	<b>0.6</b>	<b>6.3 - 8.3</b>
	Chiefland	Levy, Dixie, Gilchrist	11	236	7.0	0.7	6.3 - 8
	Gainesville	Alachua	19	303	7.5	0.5	6.5 - 8.3
	Ocala	Marion	14	371	6.9	0.5	6.5 - 8
<b>WC</b>	<b>WC</b>		<b>52</b>	<b>643</b>	<b>6.7</b>	<b>0.4</b>	<b>5.9 - 8.2</b>
	Brooksville	Pasco, Citrus, Sumter, Hernando	12	120	6.3	0.5	5.9 - 7.8
	Tampa Bay	Hillsborough, Pinellas	15	228	6.8	0.3	6.2 - 7.2
	Lakeland	Polk	10	89	6.8	0.6	6.2 - 8.2
	Sarasota	Sarasota, Manatee, Hardee, DeSoto	15	206	6.8	0.3	6.2 - 7.3
<b>EC</b>	<b>EC</b>		<b>77</b>	<b>1441</b>	<b>6.5</b>	<b>0.4</b>	<b>5.1 - 7.7</b>
	Daytona Beach	Volusia	19	318	6.6	0.3	6 - 7.1
	Melbourne	Brevard	15	251	6.3	0.5	5.1 - 7.4
	Leesburg	Lake	10	229	6.5	0.5	6 - 7.7
	Orlando	Orange, Seminole	23	425	6.6	0.2	5.9 - 7
	Kissimmee	Osceola	10	218	6.5	0.3	5.9 - 6.1
<b>SW</b>	<b>SW</b>		<b>26</b>	<b>644</b>	<b>7.4</b>	<b>0.6</b>	<b>6.8 - 9.1</b>
	Ft. Myers	Charlotte, Lee, Collier	18	469	7.6	0.5	6.8 - 9.1
	Sebring	Highlands, Glades, Hendry	8	175	7.3	0.5	6.9 - 8.1
<b>SE</b>	<b>SE</b>		<b>94</b>	<b>1171</b>	<b>6.6</b>	<b>0.3</b>	<b>5.6 - 8.1</b>
	Ft. Lauderdale	Broward	28	392	6.7	0.3	6.1 - 7.6
	Ft. Pierce	Indian River, St Lucie, Okeechobee	15	214	6.4	0.3	6.1 - 7.3
	Stuart	Martin	10	92	6.5	0.3	5.8 - 7.1
	Miami	Miami-Dade, Monroe	22	212	6.8	0.3	6.1 - 8.1
	West Palm Beach	Palm Beach	19	261	6.5	0.3	5.6 - 7.2



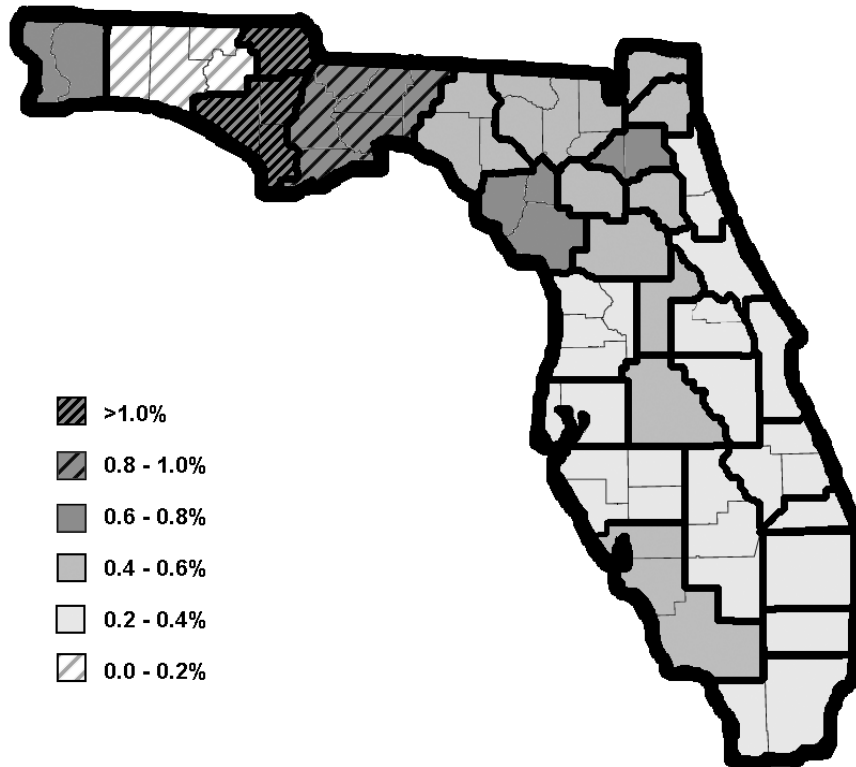
**Figure 4.50 Weighted average asphalt content by county group**

Comparing asphalt content between county groups illustrated distinct regional characteristics across the state. Based on the weighted average asphalt content, the county groups with the highest asphalt content were Ft. Myers, Sebring, Tallahassee, Perry and Gainesville; each having an average asphalt content greater than 7.0%. These counties were concentrated into three distinct areas; the region surrounding Tallahassee, Alachua County, and the southwest corner of the state. Alachua County, containing the City of Gainesville, was unique, having significantly higher average asphalt content than all adjoining counties. The Ft. Walton Beach county group had the lowest average asphalt content. This group was located in the northwest region of the state, covering the rural counties between Panama City and Pensacola; also includes the cities of Niceville, Destin, and Crestview. Only 8 projects were available for the 4 counties in this group, yet the average asphalt content of 6.1% was lower than the surrounding groups, with averages from 6.7% to 6.9%. It was also noted that several other groups across the state also had average asphalt content significantly lower than the statewide average.



#### 4.6.1.2.5 Analysis of Variability

The weighted standard deviation of asphalt content results was calculated for each county and county group, values were used to compare the relative variability of different regions of the state. Table 4.19 shows the number of projects analyzed, the weighted standard deviation and the range of values for each county group. Distinct characteristics were evident in these results. Figure 4.51 shows a map of county groups, shaded based on the weighted standard deviation of asphalt content for each county group.



**Figure 4.51 Standard deviation of asphalt content by county group**

The county groups with the highest and lowest weighted standard deviation were located adjacent to each other in the northwest region of the state. The highest standard deviation was located in the Panama City and Marianna groups and the lowest in the Ft. Walton Beach group. In general, most county groups exhibited moderate variability with the weighted standard deviation ranging from 0.2% to 0.6%. In general, the northern and southwest regions of the state exhibited slightly higher variability than the central and southeast regions. Higher variability in pavement composition samples indicated moderate variability should be anticipated in earthwork applications, yet regions with distinctly higher variability in asphalt content could produce higher variable results with the use of RAP in earthwork applications.

### 4.6.1.3 Regional Differences

Regional differences in asphalt content were identified based on results from milled RAP laboratory samples and pavement composition field data from throughout the state. The FDOT Districts were used as boundaries for geographic regions for development of the sampling plan and comparing laboratory sample results, yet results from the field data has lead to identification of additional regions of distinct asphalt content, independent of the FDOT District boundaries. The regions are summarized below.

#### 4.6.1.3.1 Northwest Florida

A summary of results for the northwest region of Florida are shown in Table 4.20. As defined in this study, the northwest region of Florida encompasses all counties west of the Apalachicola River, containing the majority of FDOT District 3.

**Table 4.20 Asphalt content results from Northwest Florida**

County Group (Largest City)	Counties	Field Samples (Pavement Composition Reports)				Laboratory Samples (Milled RAP)		
		Number of Projects	Weighted Average Asphalt Content (%)	Weighted Standard Deviation (%)	Asphalt Content Range (%)	Number of Samples	Average Asphalt Content (%)	Asphalt Content Range (%)
Pensacola	Escambia, Santa Rosa	17	6.7	0.6	5.6-7.9	2	6.5	6.4-6.6
Ft. Walton Beach	Okaloosa, Walton, Holmes, Washington	8	6.1	0.1	5.9-6.3	4	6.5	6.0-7.3
Marianna	Jackson	13	6.8	1.2	5.6-9.8	1	5.2	N/A
Panama City	Bay, Gulf, Calhoun	9	6.9	1.6	5.2-10.0	1	7.8	N/A
Total for Region		47	6.7	1.1	5.2-10.0	8	6.4	5.2-7.8

Field data from the Pensacola, Panama City and Marianna county groups indicated that the weighted average asphalt content was close to the statewide field sample average of 6.8%. The weighted average asphalt content of the Ft. Walton Beach group was 6.1%, the lowest asphalt content of any group in the state and significantly lower than the average in each adjacent group.

The northwest region exhibited the highest variability and largest range in asphalt content field results of any region. The large range and weighted standard deviation was observed within individual county group averages as well as the

regional average. In three of the four groups in this region, the weighted standard deviation was above the statewide weighted standard deviation of 0.7%. Numerous projects throughout this region exhibited project averages significantly above and below the regional average, contributing to the large range and high standard deviation. These deviations illustrate the distinct differences in asphalt content across the northwest region of Florida. The Ft. Walton Beach group, just west of the Panama City and Marianna groups, had a very low weighted standard deviation and low asphalt content of 6.1%. The Tallahassee group, just east of the Panama City and Marianna groups, had a high average asphalt content of 7.5%. This suggests there were distinctly different supplies of aggregate materials across this region, leading to large differences in mix design requirement for asphalt content of pavements. The Panama City and Marianna groups had weighted averages of 6.9% and 6.8%, acting as a zone between the high and low values to the east and west, respectively. In general, these county groups may exhibit high variability due to multiple aggregate supplies from the adjoining groups, leading to high variability in the asphalt content of pavements.

Milled RAP samples in the region had asphalt content slightly lower than the field data average, although values were within the range identified by the field data. The laboratory samples from the Ft. Walton Beach group had an average of 6.5%, close to the regional field average of 6.5%, contradicting the low asphalt content of 6.1% observed in the field samples. The laboratory samples from the Pensacola group were also close to the regional average of 6.5%, confirming the field sample results. The Panama City and Marianna groups only had one milled RAP sample each and both asphalt content results differed from the regional field average of 6.5% by more than one standard deviation (1.1%). In summary, both field and laboratory samples indicate a large range of measured asphalt content in this region, with results ranging from 5.2% to 10.0%. In addition, the high variability observed indicated that pavements in this region exhibit a wide distribution of asphalt content. Overall, this suggests local and project-specific conditions in the northwest region of Florida provide the primary influence on the asphalt content of RAP materials.

#### ***4.6.1.3.2 Big Bend Region of Florida***

A summary of results for the Big Bend region of Florida are shown in Table 4.21. As defined in this study, the Big Bend region of Florida includes the counties surrounding Tallahassee and extends along the Gulf Coast to Taylor County; containing portions of FDOT Districts 2 and 3.

**Table 4.21 Asphalt content results from the Big Bend Region of Florida**

County Group (Largest City)	Counties	Field Samples (Pavement Composition Reports)				Laboratory Samples (Milled RAP)		
		Number of Projects	Weighted Average Asphalt Content (%)	Weighted Standard Deviation (%)	Asphalt Content Range (%)	Number of Samples	Average Asphalt Content (%)	Asphalt Content Range (%)
Perry	Taylor, Lafayette, Madison	16	7.4	0.4	6.5-7.9	0	N/A	N/A
Tallahassee	Leon, Gadsden, Wakulla, Jefferson, Liberty, Franklin	9	7.5	0.8	5.8-8.5	2	7.8	7.7-7.9
Total for Region		25	7.4	0.6	5.8-8.5	2	7.8	7.7-7.9

Pavement composition reports indicated that samples in these counties had the highest average asphalt content in the state and had significantly higher asphalt content than neighboring regions. The regional standard deviation of 0.6% was close to the statewide weighted standard deviation of 0.7%, indicating that results in this region exhibit a typical level of variability. The standard deviation in the Big Bend region was lower than the northwest region to the west. This indicated that the high asphalt content observed represent a distinct region of higher asphalt content.

Overall, field projects analyzed in the Big Bend region indicate that typical asphalt content ranges from 7.0% to 8.0%. The two milled RAP samples collected in this region had asphalt content close to the regional average, confirming the regional trends exhibited by the field samples. This suggests there were distinctly different supplies of aggregate materials in this region, leading differences in mix design requirements that require additional asphalt binder for pavement mixes. In general, RAP materials in the Big Bend region should consistently exhibit asphalt content significantly above the statewide average.

#### 4.6.1.3.3 Northeast Florida

A summary of results for the northeast region of Florida are shown in Table 4.22. As defined in this study, the northeast region of Florida includes Lake City, Jacksonville, and extends along the Atlantic Coast to Flagler County, containing portions of FDOT Districts 2 and 5.

**Table 4.22 Asphalt content results from Northeast Florida**

County Group (Largest City)	Counties	Field Samples (Pavement Composition Reports)				Laboratory Samples (Milled RAP)		
		Number of Projects	Weighted Average Asphalt Content (%)	Weighted Standard Deviation (%)	Asphalt Content Range (%)	Number of Samples	Average Asphalt Content (%)	Asphalt Content Range (%)
Fernandina Beach	Nassau	10	6.4	0.6	6.3-7.3	0	N/A	N/A
Jacksonville	Duval	52	6.6	0.4	5.1-7.7	2	6.4	6.3-6.4
Lake City	Columbia, Suwannee, Hamilton, Baker, Union	26	6.5	0.2	5.4-7.4	1	6.9	N/A
Orange Park	Bradford, Clay	16	6.7	0.3	6.1-8.2	0	N/A	N/A
Palatka	Putnam	12	6.8	0.4	5.4-7.5	0	N/A	N/A
St. Augustine	St. Johns, Flagler	15	6.8	0.5	4.6-7.3	0	N/A	N/A
Total for Region		131	6.6	0.5	4.6-8.2	3	6.6	6.3-6.9

Pavement composition reports indicated that samples in the northern counties in this region had slightly higher asphalt content than the southern counties. The Lake City, Jacksonville, and Fernandina Beach groups had weighted averages slightly below the statewide average, from 6.4% to 6.6%. The Orange Park, Palatka, & St. Augustine groups had weighted averages close to the statewide field sample average, from 6.7% to 6.8%. This suggests small differences in material properties between the northern and southern counties may be influencing asphalt content.

The large range of values observed in this region suggests higher variability, although the regional weighted standard deviation of 0.5% was slightly lower than the statewide weighted standard deviation of 0.7%, indicating that results in this region exhibit a typical level of variability. The weighted standard deviation in the Lake City and Orange City county groups was 0.3% or lower, indicating that results in those counties exhibit a lower level of variability than the rest of the state and the asphalt content in RAP materials in these counties was less likely to deviate

from the local average. Several small projects throughout the northeast region exhibited project averages significantly above and below the regional average. This suggests that the distribution of asphalt content was wide and local or project-specific conditions provide a significant influence on the asphalt content of RAP materials.

Field projects analyzed in this region indicate that typical asphalt content ranges from 6.0% to 7.0%. The three milled RAP samples collected in this region had measured asphalt content close to the field data average, confirming the regional trends exhibited by the field samples. In general, RAP materials in the northeast region should consistently exhibit asphalt content at and above the statewide average.

#### 4.6.1.3.4 North-Central Florida

A summary of results for the north-central region of Florida are shown in Table 4.23. As defined in this study, the north-central region of Florida includes Gainesville, Ocala, and extends to the Gulf Coast of Levy and Dixie Counties, containing portions of FDOT Districts 2 and 5.

**Table 4.23 Asphalt content results from North-Central Florida**

County Group (Largest City)	Counties	Field Samples (Pavement Composition Reports)				Laboratory Samples (Milled RAP)		
		Number of Projects	Weighted Average Asphalt Content (%)	Weighted Standard Deviation (%)	Asphalt Content Range (%)	Number of Samples	Average Asphalt Content (%)	Asphalt Content Range (%)
Gainesville	Alachua	19	7.5	0.5	6.5-8.3	1	6.7	N/A
Chiefland	Levy, Dixie, Gilchrist	11	7.0	0.7	6.3-8.0	0	N/A	N/A
Ocala	Marion	14	6.9	0.5	6.5-8.0	5	6.6	5.8-8.2
Total for Region		44	7.1	0.6	6.3-8.3	6	6.6	5.8-8.2
Total for Region (without outlier)						5	6.8	6.1-8.2

Pavement composition reports indicated that field samples in Alachua County had higher asphalt content than all adjoining counties, and exhibited asphalt content more characteristic of the non-contiguous Big Bend region to the west. The Ocala and Chiefland county groups exhibited values slightly above the statewide average and more characteristic of counties in the northeast region. The Ocala and Chiefland county groups were combined with the Gainesville group instead of the northeast region because these counties were geographically contiguous with Alachua County and exhibited slightly higher average asphalt content than the

groups in the northeast region. The regional standard deviation of 0.6% was close to the statewide weighted standard deviation of 0.7%, indicating that results in this region exhibit a typical level of variability.

Field projects analyzed in Alachua County indicate that typical asphalt content ranges from 7.0% to 8.0%. Field projects analyzed in the Ocala and Chiefland groups indicate that typical asphalt content ranges from 6.5% to 7.5%. Table 4.23 shows that milled RAP samples collected in this region had average asphalt content of 6.6%; this was lower than the field average asphalt content of 7.1% by approximately one standard deviation (0.6%). This difference contradicts the field sample results and suggests that the laboratory samples may not be representative of asphalt content within the region. Overall, the range of values from the laboratory samples (5.8% to 8.2%) was within the range of the field data (6.3% to 8.3%), with the exception of one sample from Marion County, with an asphalt content of 5.8%. This sample was the lowest value within all adjacent county groups and differs from the statewide milled RAP sample average of 6.6% by more than one standard deviation (0.7%). This sample was considered an outlier and not representative of the regional asphalt characteristics. If the sample was removed from data set, the sample average for the region rises to 6.8%, equal to the statewide field sample average. In general, RAP materials in the north-central region should consistently exhibit asphalt content above the statewide average.

#### ***4.6.1.3.5 West-Central Florida***

A summary of results for the west-central region of Florida are shown in Table 4.24. As defined in this study, the west-central region of Florida includes Tampa Bay, Lakeland and Sarasota. This region covers all Gulf Coast counties from Sarasota to Citrus and interior counties of Sumter, Polk, Hardee and DeSoto. This region contains portions of FDOT Districts 1 and 5 as well as all of District 7.

**Table 4.24 Asphalt content results from West-Central Florida**

County Group (Largest City)	Counties	Field Samples (Pavement Composition Reports)				Laboratory Samples (Milled RAP)		
		Number of Projects	Weighted Average Asphalt Content (%)	Weighted Standard Deviation (%)	Asphalt Content Range (%)	Number of Samples	Average Asphalt Content (%)	Asphalt Content Range (%)
Brooksville	Pasco, Hernando, Citrus, Sumter	12	6.3	0.5	5.9-7.8	1.5 *	5.9	5.6-6.6
Tampa Bay	Hillsborough, Pinellas	15	6.8	0.3	6.2-7.2	1.5 *	6.7	6.6-6.8
Lakeland	Polk	10	6.8	0.6	6.2-8.2	2	6.3	6.0-6.7
Sarasota	Sarasota, Manatee, DeSoto, Hardee	15	6.8	0.3	6.2-7.3	2	6.4	4.9-8.0
Total for Region		52	6.7	0.4	5.9-8.2	7	6.3	4.9-8.0
Total for Region (without outlier)						6	6.6	5.6-8.0

\* Note: One sample collected in southern Pasco County was from a stockpile containing materials from projects in both Pasco County and Hillsborough County, and was partially counted in both the Tampa Bay and Brooksville Groups

Pavement composition reports in the Brooksville group exhibited average asphalt content lower than the statewide average and all adjoining counties. The remaining county groups exhibited asphalt content close to the statewide field sample average of 6.8%. The regional weighted standard deviation of 0.4% was slightly lower than the statewide weighted standard deviation of 0.7%. In addition, the Tampa Bay and Sarasota county groups had weighted standard deviations of 0.3%. This indicated that results in those counties exhibit a slightly lower level of variability than the rest of the state and the asphalt content of RAP materials in this region were less likely to deviate from the regional weighted average.

Field projects analyzed in the Brooksville group indicate that typical asphalt content ranges from 5.8% to 6.8%. Field projects analyzed in the remaining counties in the west-central region indicate that typical asphalt content ranges from 6.2% to 7.2%. Milled RAP samples collected in this region had average asphalt content of 6.3%, approximately one standard deviation (0.4%) lower than the field weighted average of 6.7%. This suggests that the samples collected may not be representative of asphalt content within the region. The range of values from the laboratory samples was within the range of the field data, with the exception of one sample from Sarasota County, with an asphalt content of 4.9%. This sample was the lowest sample value within all adjacent county groups and differs from the



statewide milled RAP average of 6.6% by over two standard deviations (1.4%). This sample was considered an outlier and not representative of the regional asphalt characteristics. In addition, another sample from Sarasota County had an asphalt content of 8.0% and one sample from Hernando County had an asphalt content of 5.6%; both of these values differ from the regional and statewide field averages by more than one standard deviation of 0.7%. If the 4.9% sample was removed from data set, the sample average for the region increases to 6.6%. This suggests that rather than regional trends, local and project-specific conditions in this region provide the primary influence on the asphalt content of RAP. In addition, the large range of laboratory sample results from Sarasota County may indicate that the southern counties in this region were also influenced by the same regional trends affecting the Ft. Myers group to the south. The Ft. Myers group had a higher average asphalt content and slightly higher standard deviation than adjacent groups.

Overall, the milled RAP samples collected in this region exhibited a wide range of asphalt content, suggesting the regional average calculated was dependent on these outlying values and not based on a regional trend. When compared to the field samples, the laboratory samples in this region exhibited high variability in asphalt content. In summary, RAP materials in the west-central region should consistently exhibit asphalt content at and slightly below the statewide average.

#### 4.6.1.3.6 East-Central Florida

A summary of results for the east-central region of Florida are shown in Table 4.25. As defined in this study, the east-central region of Florida includes the interior counties surrounding Orlando and extends along the Atlantic Coast to Volusia and Brevard Counties, containing the majority of FDOT District 5.

**Table 4.25 Asphalt content results from East-Central Florida**

County Group (Largest City)	Counties	Field Samples (Pavement Composition Reports)				Laboratory Samples (Milled RAP)		
		Number of Projects	Weighted Average Asphalt Content (%)	Weighted Standard Deviation (%)	Asphalt Content Range (%)	Number of Samples	Average Asphalt Content (%)	Asphalt Content Range (%)
Daytona Beach	Volusia	19	6.6	0.3	6.0-7.1	5	6.8	6.2-7.3
Melbourne	Brevard	15	6.3	0.5	5.1-7.4	2	5.9	5.5-6.3
Leesburg	Lake	10	6.5	0.5	6.0-7.7	1	6.9	N/A
Orlando	Orange, Seminole	23	6.6	0.2	5.9-7.0	4	6.4	5.9-6.7
Kissimmee	Osceola	10	6.5	0.3	5.9-6.1	0	N/A	N/A
Total for Region		77	6.5	0.4	5.1-7.7	12	6.5	5.5-7.3

Field samples in this region had asphalt contents at and below the statewide field sample average of 6.8%. The Melbourne county group, containing Brevard County, measured slightly lower values, with an average of 6.3%. The regional weighted standard deviation of 0.4% was slightly lower than the statewide field weighted standard deviation of 0.7%, indicating that results in this region exhibit a slightly lower level of variability. In addition, the Daytona Beach, Orlando, and Kissimmee county groups had standard deviations of 0.3% or lower, indicating that results in these counties exhibited a slightly lower level of variability than the rest of the state and asphalt content was less likely to deviate from the regional average.

Field projects analyzed in the east-central region indicate that typical asphalt content ranges from 6.0% to 7.0%. The milled RAP samples collected had measured asphalt content close to the field data average, confirming the regional trends exhibited by the field data. Field and laboratory samples from Brevard County exhibited averages significantly below the regional and statewide average. This suggests that rather than regional trends, local and project-specific conditions in Brevard County provide a more critical influence on the asphalt content of RAP than the regional trends in the project data suggests. In summary, RAP materials in the east-central region should consistently exhibit asphalt content at and slightly below the statewide average.

#### 4.6.1.3.7 Southwest Florida

A summary of results for the southwest region of Florida are shown in Table 4.26. As defined in this study, the southwest region of Florida includes the cities of Ft. Myers, Naples and Punta Gorda. This region covers the interior counties of Highlands, Glades, and Hendry and extends along the Gulf Coast from Charlotte to Collier County, containing a portion of FDOT District 1.

**Table 4.26 Asphalt content results from Southwest Florida**

County Group (Largest City)	Counties	Field Samples (Pavement Composition Reports)				Laboratory Samples (Milled RAP)		
		Number of Projects	Weighted Average Asphalt Content (%)	Weighted Standard Deviation (%)	Asphalt Content Range (%)	Number of Samples	Average Asphalt Content (%)	Asphalt Content Range (%)
Ft. Myers	Charlotte, Lee, Collier	18	7.6	0.5	6.8-9.1	4	7.1	5.8-8.5
Sebring	Highlands, Glades, Hendry	8	7.3	0.5	6.9-8.1	0	N/A	N/A
Total for Region		26	7.4	0.6	6.8-9.1	4	7.1	5.8-8.5

Pavement composition reports indicated that all samples in this region had asphalt content higher than the statewide field sample average of 6.8%, producing a regional weighted average of 7.4%. The regional standard deviation of 0.5% was

slightly lower than the statewide weighted standard deviation of 0.7%, indicating that results in this region exhibit a typical level of variability, despite the higher asphalt content.

Field projects analyzed in the southwest region indicate that typical asphalt content ranges from 7.0% to 8.0%. The four milled RAP samples collected in this region also had asphalt content above the statewide average, confirming the regional trends exhibited by the field data. Within this region, the two samples collected in Charlotte County had the highest and lowest regional values of 5.8% and 8.5% respectively. Both of these values differ from regional average of 7.1% by large margins, and differ from the field weighted average of 7.4% by nearly two standard deviations (1.2%). Due to the low number of samples in this region, eliminating one or both or one of these samples as an outlier would limit the quality of the remaining regional average. In addition, laboratory sample from Sarasota County, immediately north of Charlotte County, also exhibited highly variable asphalt content results. This suggests that rather than regional trends, local and project-specific conditions provide the primary influence on the asphalt content of RAP in Sarasota and Charlotte Counties.

Overall, field data and laboratory samples confirm that the southwest region of the state exhibits distinct characteristics of high asphalt content, similar to the Big Bend region. In summary, RAP materials in this region should consistently exhibit asphalt content significantly above the statewide average.

#### ***4.6.1.3.8 Southeast Florida***

A summary of results for the southeast region of Florida are shown in Table 4.27. As defined in this study, the southeast region of Florida includes the urban corridor of cities between Miami and Vero Beach. This region covers Okeechobee County and all counties along the Atlantic Coast from Indian River County to Monroe County, containing a portion of FDOT District 1 and all of Districts 4 and 6.

**Table 4.27 Asphalt content results from Southeast Florida**

County Group (Largest City)	Counties	Field Samples (Pavement Composition Reports)				Laboratory Samples (Milled RAP)		
		Number of Projects	Weighted Average Asphalt Content (%)	Weighted Standard Deviation (%)	Asphalt Content Range (%)	Number of Samples	Average Asphalt Content (%)	Asphalt Content Range (%)
Ft. Lauderdale	Broward	28	6.7	0.3	6.1-7.6	0	N/A	N/A
Ft. Pierce	Indian River, St Lucie, Okeechobee	15	6.4	0.3	6.1-7.3	4	6.2	6.0-6.4
Stuart	Martin	10	6.5	0.3	5.8-7.1	0	N/A	N/A
Miami	Miami-Dade, Monroe	22	6.8	0.3	6.1-8.1	3	6.5	6.1-6.8
West Palm Beach	Palm Beach	19	6.5	0.3	5.6-7.2	3	6.4	6.0-6.6
Total for Region		94	6.6	0.3	5.6-8.1	10	6.3	6.0-6.8

Pavement composition reports indicated that samples in the southern counties within the region had slightly higher asphalt content than the northern counties. The Miami and Ft. Lauderdale groups had weighted averages close to the statewide field sample average, at 6.8% and 6.7%. The West Palm Beach, Stuart and Ft. Pierce groups had weighted averages slightly below the statewide average, from 6.4% to 6.5%. The regional standard deviation of 0.3% was below the statewide weighted standard deviation of 0.7%, indicating that results in this region exhibit a generally lower level of variability than the rest of the state. The variability in each county group was also 0.3%, indicating that results within each county group also exhibited a lower level of variability.

Field projects analyzed in the southeast region indicate that typical asphalt content ranges from 6.0% to 7.0%. The milled RAP samples collected in this region had asphalt content slightly lower than the regional field data average of 6.6%. In addition, the differences between the southern and northern counties were not confirmed in the laboratory samples; the four milled RAP samples from Miami-Dade County exhibited values similar to the regional field average (6.6%). Similarly, the milled RAP averages from the Ft. Pierce and West Palm Beach groups were also lower than the county group field averages. Overall, these samples were still within one standard deviation (0.3%) of their county group field averages, indicating that the milled RAP samples collected in the southeast region represent the lower range of asphalt content typically found in that region. In summary, RAP materials in the southeast region should consistently exhibit asphalt content slightly below the statewide average.

#### 4.6.1.3.9 Summary of Regional Differences

In summary, eight regions were identified with distinct asphalt content characteristics. Table 4.28 shows a summary of results compiled by region.

**Table 4.28 Summary of regional differences**

Region	Field Samples (Pavement Composition Reports)				Laboratory Samples (Milled RAP)			
	Number of Projects	Weighted Average Asphalt Content (%)	Weighted Standard Deviation (%)	Asphalt Content Range (%)	Number of Samples	Average Asphalt Content (%)	Standard Deviation (%)	Asphalt Content Range (%)
Northwest	47	6.7	1.1	5.2-10.0	8	6.4	1.0	5.2-7.8
Big Bend	9	7.4	0.6	5.8-8.5	2	7.8	0.1	7.7-7.9
Northeast	131	6.6	0.5	4.6-8.2	3	6.6	0.3	6.3-6.9
North-Central	44	7.1	0.6	6.3-8.3	6	6.6	0.8	5.8-8.2
West-Central	52	6.7	0.4	5.9-8.2	7	6.3	0.9	5.6-6.8
East-Central	77	6.5	0.4	5.1-7.7	12	6.5	0.5	5.5-7.3
Southwest	26	7.4	0.6	6.8-9.1	4	7.1	1.1	5.8-8.5
Southeast	94	6.6	0.3	5.6-8.1	10	6.3	0.3	6.0-6.8
Statewide	496	6.8	0.7	4.9-10.0	52	6.6	0.7	4.9-8.5

Field and laboratory samples confirmed that the Big Bend and Southwest regions of state had asphalt contents higher than the rest of the state. Field data from Alachua County in the North-Central region also exhibited significantly higher asphalt content; yet this trend was contradicted by laboratory sample results. Similarly, laboratory samples from the Ft. Walton Beach county group in the Northwest region exhibited results lower than the statewide average, although this was not observed in the field data. Field and laboratory samples confirmed that asphalt content in the east-central region was slightly lower than the statewide average. The remaining regional averages did not differ from the statewide weighted average by more than 0.3% in either field or laboratory sample sets.

It was assumed that large geographic differences in asphalt content were a result of regional factors that influence FDOT mix design requirements for asphalt binder. Regional factors primarily influence the aggregate mix components, and may be due to unique locally available aggregate materials, differences in aggregate processing equipment and access to imported aggregates from domestic or international sources. In conclusion, regional and statewide results indicate that there were few significant geographic-dependent differences in asphalt content. Therefore, typical RAP samples found in the state should exhibit asphalt content results within 1.0% of the statewide field sample average of 6.8%.

Geographic differences were also observed in the variability in asphalt content results. The Northwest region exhibited the highest variability in asphalt

content results, in both the field and laboratory sample sets. The Southeast region exhibited the lowest variability in asphalt content field data and was confirmed by the relatively small range in laboratory sample results. Overall, the wide range of individual test results measured within projects, within counties or across large regions did not translate to large variability in county or regional weighted averages. In conclusion, outside of the two regions identified, regional and statewide results indicated that variability was approximately the same level throughout the state. Therefore, RAP materials found in most regions of the state should exhibit moderate variability in asphalt content.

#### **4.6.2 Sieve Analysis**

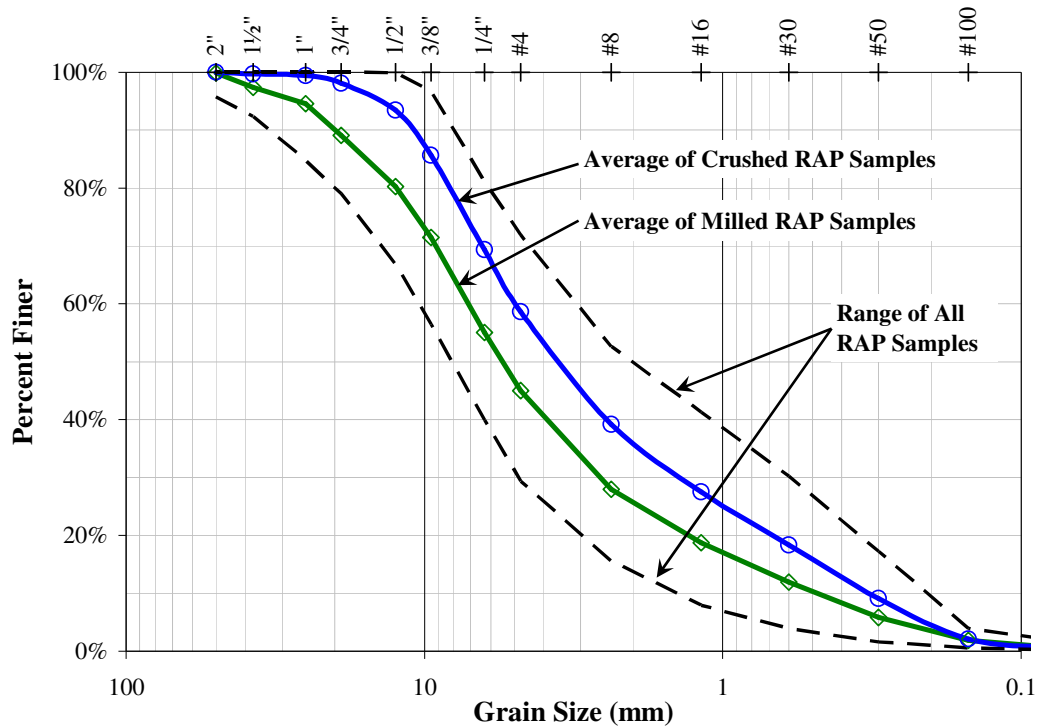
The grain size distribution is the property of RAP most influenced by the combination of the origin pavement as well as reclamation and processing method. In addition, the gradation can be influenced by the mixing of soil or granular debris with RAP during reclamation or processing, as well as the mixing of different RAP sources in the same stockpile. The grain size distribution of all RAP samples was determined by dry-sieving, following the procedure in FM 1-T 027 and AASHTO T27. The percent passing was calculated for each trial and the average was used to represent the gradation of each sample.

Several variables were used to measure the size and quantity of a sample's largest particles. This study applies variable definitions consistent with the Superpave asphalt mix design method and introduces additional variables relevant to earthwork applications. The nominal maximum particle size ( $D_{nom}$ ) was defined as one sieve size larger than the largest sieve to retain more than 10% of the material. The maximum particle size ( $D_{max}$ ) was defined as one sieve size larger than the  $D_{nom}$ ; typically soils and aggregate blends have 100% passing this size (Roberts et al., 1996). The top size ( $D_{top}$ ) was defined as the sieve size with 100% passing, as measured in a sieve analysis; this value was typically the same as the  $D_{max}$ .

**Table 4.29 Summary of RAP gradation parameters**

	Milled			Crushed		
	Average	Min.	Max.	Average	Min.	Max.
Number of Samples Tested	52	-	-	43	-	-
Nominal Maximum Particle Size, $D_{nom}$ (mm)	23.6	12.5	37.5	12.9	9.5	25.0
Percent Passing #4	45%	29%	64%	59%	30%	72%
Percent Passing #30	12%	4%	19%	19%	4%	30%
Percent Passing #200	0.6%	0.2%	1.6%	0.5%	0.1%	1.8%
$D_{10}$ (mm)	0.58	0.29	1.43	0.40	0.21	1.90
$C_U$	13	5	23	13	4	22
$C_C$	1.8	0.7	4.2	1.2	0.4	2.9

The average gradation parameters for milled and crushed samples are presented in Table 4.29. Overall, the crushed RAP samples had slightly finer gradation than the milled RAP samples, as indicated by the larger average  $D_{nom}$  in milled RAP samples. The range of values for each processing type overlaps; therefore some milled samples may be finer than some crushed samples. The range of values for percent passing the #4 and #30 sieves was also large; indicating that regardless of processing type, there can be large variability in this portion of the grain size distribution. The range of values for percent passing the #200 sieve was approximately the same for both material types, indicating that despite the processing method, the amount of fines in RAP was relatively consistent. Milled samples had higher  $D_{10}$  values than crushed samples, indicating that permeability would be substantially better in compacted milled RAP than crushed RAP. Both  $C_U$  and  $C_C$  values also exhibited significant variability, indicating that compaction characteristics could also exhibit variability.



**Figure 4.52 Grain-size distribution of milled and crushed RAP**

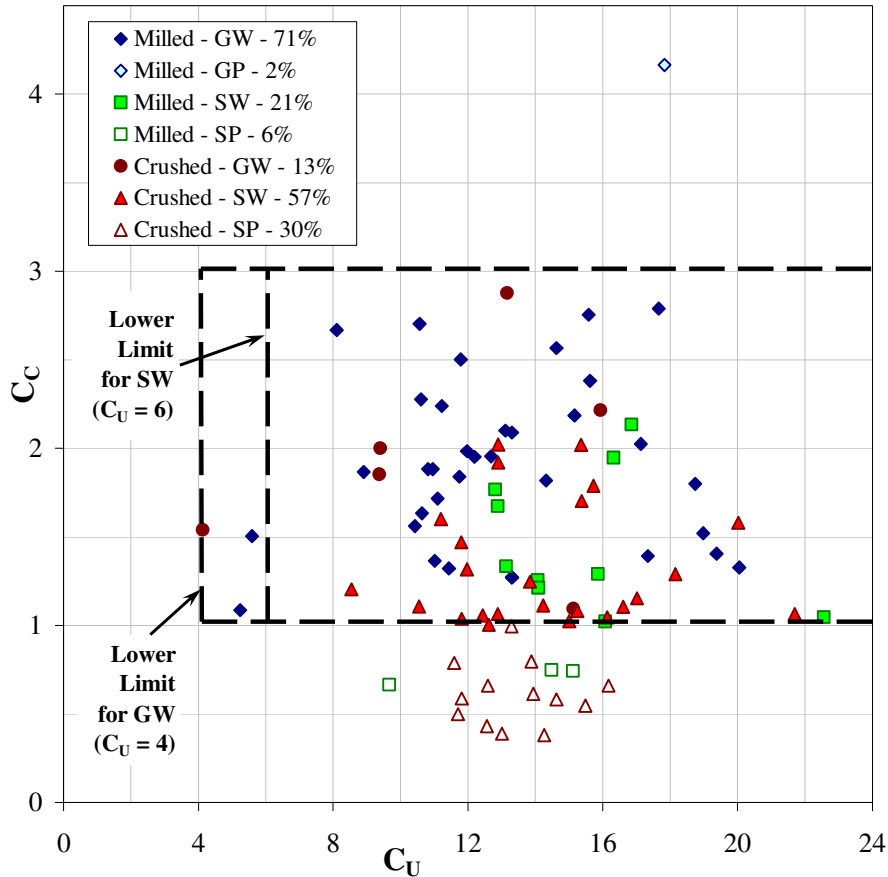
The average gradation curves for milled and crushed RAP samples was shown in Figure 4.52, as well as the minimum and maximum gradation curves from all samples. This chart shows that crushed RAP typically has smaller particles, with an average of 8% more passing each sieve than milled RAP. Samples from both material types had a wide range of gradations, as shown by the minimum and maximum curves. No sample had more than 6% passing the #100 sieve or 2% passing the #200 sieve. The composite action of the asphalt binder in RAP material differentiates RAP from soil or aggregate blends of similar gradations, producing to significantly fewer fines.

#### 4.6.2.1 Classification

Gradation data was used to apply soil classifications to all RAP samples. Based on conclusions from previous research (Rathje, et al 2006; Sayed, et al 1993); it was assumed that all RAP samples had non-plastic fines, and therefore Atterberg Limits testing was deemed unnecessary. Using the AASHTO classification system, all samples were classified as A-1-a, indicating any RAP samples could produce an adequately graded base or subbase material. Using the USCS classification system, distinct differences emerged between crushed and milled RAP samples. Milled RAP samples were typically classified as gravel



(GW), while crushed RAP samples, with smaller particle sizes, were typically classified as sand (SW or SP). Both types had a large majority of well-graded samples, with 92% milled RAP and 70% of crushed RAP samples being classified as GW or SW. However, a significant amount of crushed RAP samples, 30%, were classified as SP, indicating the crushing process leads to an overall reduction in particle size as well as a concentration of particles within a more narrow range of sizes.



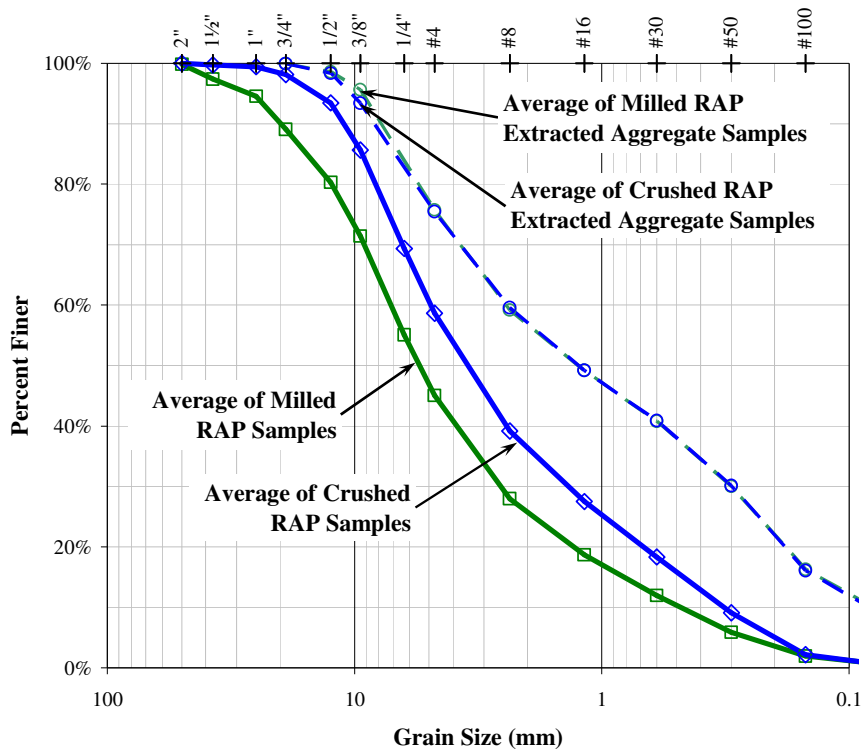
**Figure 4.53 USCS classifications of RAP samples**

By comparing  $C_c$  to  $C_u$ , a graphical representation of USCS classifications for all RAP samples is shown in Figure 4.53. A majority of samples were located within the dashed box, indicating well-graded classification (GW or SW). Only three milled samples (6%) and fourteen crushed samples (30%) lie below the lower  $C_c$  limit (GP or SP). One milled sample had a  $C_c$  value significantly higher than the rest of the sample population, at 4.2, and was considered a non-representative outlier.

#### **4.6.2.2 Extracted Aggregate**

The grain size distribution of aggregates is a primary factor in the development of pavement mix designs and is measured and balanced against the proportions of virgin fine and coarse aggregates to provide an acceptable gradation. The properties of aggregates in RAP are critical to allowing or restricting the quantity of RAP that can be recycled into new hot-mix asphalt. These properties are primarily influenced by the mix design of the original pavement layers as well as particle breakdown due to traffic loading and reclamation processes. After the asphalt content of RAP was determined by the ignition furnace method, the grain size distribution of the remaining aggregates was determined by dry-sieving, following the procedure in FM 1-T 027. The properties of the remaining aggregate particles, or extracted aggregates, can be used to make inferences about the composition of the aggregates in the original pavement mix design.

The extracted aggregates were not tested on every RAP sample collected, resulting in fewer samples of extracted aggregates than RAP. Extracted aggregate samples were collected from sites throughout the state and from RAP samples of varying asphalt content. These characteristics permit the tested extracted aggregate samples to be effective in developing comparisons between RAP and RAP aggregate. These samples were considered representative of the statewide sample distribution and conclusions were applied to all RAP samples collected. Figure 4.54 shows a comparison of the average gradation curves for milled and crushed RAP to the extracted aggregate.



**Figure 4.54 Grain-size distribution of RAP and RAP extracted aggregate**

Figure 4.54 shows the gradation curves for extracted aggregate from both processing types, milled and crushed RAP, were nearly equivalent. The largest difference in percent passing between milled and crushed RAP samples was 2.5%, occurring on the 3/8 inch (9.5 mm) sieve. Therefore, it was concluded that processing type has no influence on the gradation characteristics of extracted aggregate from RAP. This confirms the fact that the method used to reclaim and process RAP was independent of the original pavement mix design and aggregate properties. As a result, further analysis of extracted aggregate samples was performed on a single data set composed of samples from both processing types.

**Table 4.30 Comparison of nominal maximum particle sizes**

Standard Sieve Sizes for $D_{nom}$	RAP			RAP Extracted Aggregate
	Overall	Milled	Crushed	
Number of Samples Tested	95	52	43	61
1 1/2 inch (37.5 mm)	7%	13%	-	-
1 inch (25.0 mm)	20%	35%	2%	-
3/4 inch (19.0 mm)	35%	48%	19%	-
1/2 inch (12.5 mm)	25%	4%	51%	10%
3/8 inch (9.5 mm)	13%	-	28%	87%
#4 (4.75 mm)	-	-	-	3%
Average $D_{nom}$ (mm)	18.8	23.6	12.9	9.6

A comparison of the  $D_{nom}$  for all RAP and RAP extracted aggregate samples is shown in Table 4.30. The distribution and average of test results for each material type is shown. In general, the removal of the asphalt binder during extraction causes the bonded aggregate particles to be separated, creating a distribution of significantly smaller particles. On average, the  $D_{nom}$  of extracted aggregate was approximately half the size of RAP. The removal of asphalt binder causes an average decrease of 9.2 mm. The distribution of  $D_{nom}$  for extracted aggregate was concentrated near the average of 3/8 inch (9.5 mm) as compared to the wider distribution of  $D_{nom}$  sizes in RAP samples. This narrow concentration indicated that aggregate blends with a  $D_{nom}$  of 3/8 inch (9.5 mm) are commonly used in Florida. The RAP sample data suggests a large majority of aggregate blends have a  $D_{nom}$  of 3/8 inch (9.5 mm), yet it was anticipated that substantial quantities of RAP materials were composed of aggregates with a  $D_{nom}$  of 1/2 inch (12.5 mm) and 3/4 inch (19 mm). Future RAP samples could contain larger aggregate  $D_{nom}$  sizes, due to specification changes which require use of Superpave mix design as well as gap-graded and stone-mix asphalt blends.

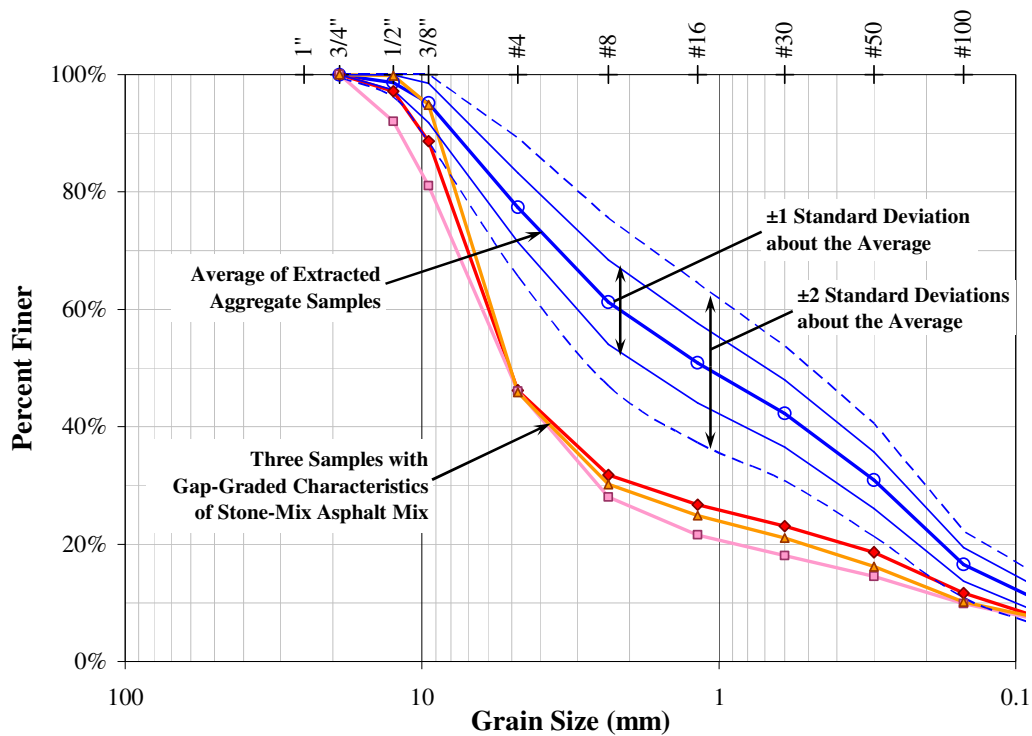
Approximately 87% of RAP aggregate samples had a  $D_{nom}$  equal to 3/8 inch (9.5 mm). An additional 10% of samples had a  $D_{nom}$  of 1/2 inch (12.5 mm). This range was consistent with the standard for coarse aggregates in AASHTO and FDOT Standard Specifications for hot-mix asphalt design (FDOT, 2007; FDOT, 2004). Nominal maximum particle sizes used in Superpave mix design are 3/8 inch (9.5 mm), 1/2 inch (12.5 mm), and 3/4 inch (19.0 mm). The remaining two samples, representing 3% of all samples, had a  $D_{nom}$  equal to the #4 sieve size (4.75 mm). The percent passing the #4 sieve (4.75 mm) on these two samples was within 3% of having a  $D_{nom}$  of 3/8 inch (9.5 mm). It was reasonable to assume that the  $D_{nom}$  in the original aggregate blend of these two extracted samples was

3/8 inch (9.5 mm) and the reduction in  $D_{nom}$  was due to aggregate degradation. Aggregate degradation was the breakdown in solid aggregate particles typically occurring during reclamation and processing of RAP. This process may also cause some samples originating from aggregate blends with a  $D_{nom}$  of 1/2 inch (12.5 mm) or 3/4 inch (19.0 mm) to exhibit an extracted  $D_{nom}$  of 3/8 inch (9.5 mm). Test results conducted in this study cannot be used to determine the content and gradation of the original aggregate blend in any RAP sample.

**Table 4.31 Summary of RAP extracted aggregate gradation parameters**

	RAP			RAP Extracted Aggregate		
	Average	Min.	Max.	Average	Min.	Max.
Number of Samples Tested	94	-	-	64	-	-
$D_{nom}$ (mm)	18.8	9.5	37.5	9.6	4.75	12.5
Percent Passing #200	0.6%	0.1%	1.8%	9%	3%	13%
$D_{10}$ (mm)	0.50	0.21	1.90	0.09	0.08	0.18
$C_U$	13.5	4.1	22.6	27.6	8.7	50.3
$C_C$	1.6	0.4	4.2	0.9	0.3	6.7

A comparison of gradation parameters for all RAP and RAP extracted aggregate samples was shown in Table 4.31. Extracted aggregate from RAP contains significantly more fines than RAP, as indicated by both the value of  $D_{10}$  and the percent passing the #200 sieve. The  $D_{10}$  of extracted aggregate was less than a fifth of the  $D_{10}$  of RAP and the percent passing the #200 sieve was over ten times that of RAP. Extracted aggregates were not typically classified using the USCS system, yet values of  $C_U$  and  $C_C$  were calculated for comparison. The values of  $C_U$  for extracted aggregate was approximately double that of RAP. Therefore, if the ratio of larger to smaller particles ( $C_U = D_{60}/D_{10}$ ) in extracted aggregate was much larger than for RAP, then the extraction process produces both an overall reduction in particle size and a higher proportion of fines. The range of  $C_C$  values for extracted aggregate samples did not differ significantly from RAP; however the average  $C_C$  for extracted aggregate was below 1.0, indicating that a majority of samples were poorly graded. Based on values of  $C_U$ ,  $C_C$ , and percent passing the #200 sieve, typical RAP extracted aggregate would be classified as poorly graded sand with fines (SP-SM).



**Figure 4.55 Summary of RAP extracted aggregate grain-size distribution**

Analysis of the range of values in the extracted aggregate gradation revealed that three samples exhibited characteristics of gap-graded particle distributions. Figure 4.55 shows the grain size distribution of these three samples relative to the rest of the average of all extracted aggregate samples. Based on conversations with staff at the asphalt plants where these specific samples were collected, it was determined that all three came from interstate highways that contained surface courses of gap-graded stone matrix asphalt (SMA). If these three samples were removed from the sample distribution, the range shifts significantly closer to the average. In addition, each of these three samples exceeds the overall average by greater than two standard deviations on all sieve sizes between the #4 (4.75 mm) and #100 sieves (0.150 mm); therefore these samples were considered outliers of the sample distribution. This highlights the low proportion of mid-sized particles in the SMA gradations, where additional coarse particles were used to provide stone-on-stone contact to maximize resistance and durability on high traffic roadways. These three samples do not appear to be representative of typical RAP aggregate characteristics, yet they denote a significant distinct subset of RAP materials.

**Table 4.32 Superpave aggregate gradation control limits**

Sieve Size	SP-12.5			SP-9.5		
	Min. Limit	Max. Limit	Average Percent Passing	Min. Limit	Max. Limit	Average Percent Passing
3/4 inch (19.0 mm)	100%	-	100%	-	-	100%
1/2 inch (12.5 mm)	90%	100%	96%	100%	-	99%
3/8 inch (9.5 mm)	-	90%	86%	90%	100%	96%
#4 (4.75 mm)	-	-	61%	-	90%	78%
#8 (2.36 mm)	28%	58%	47%	32%	67%	61%
#200 (0.075 mm)	2%	10%	8.4%	2%	10%	9.4%

Extracted aggregates were also compared to the Superpave gradation control limits for each  $D_{nom}$  sieve size, SP-12.5 and SP-9.5. Each sample was compared to the Superpave control limits for the corresponding  $D_{nom}$  size. The two samples with a  $D_{nom}$  equal to the #4 sieve (4.75 mm) were analyzed using the SP-9.5 gradation requirements. Control limits and sample averages are shown in Table 4.32. The Superpave control limits were determined at sieve sizes based on the  $D_{nom}$  and  $D_{max}$ , not the  $D_{top}$ . Given that the  $D_{max}$  was one sieve larger than the  $D_{nom}$ , the  $D_{max}$  may not have a percent passing of 100% and therefore, was not necessarily equal to the  $D_{top}$ . The control limits specify that the  $D_{max}$  sieve size have a percent passing of 100%; therefore, some samples may meet the  $D_{nom}$  limits and not the  $D_{max}$  limit. This effect was observed in a majority of extracted aggregate samples as 86% of SP-12.5 samples and 78% of SP-9.5 samples exceeded the  $D_{max}$  limit, while all samples fell within the  $D_{nom}$  limits.

The Superpave mid-range limits, set at the #8 sieve for SP-9.5 and SP-12.5, provide wider ranges to allow for well-graded and gap-graded aggregate blends. The average percent passing the #8 sieve for SP-9.5 samples was 61%, which was relatively close to the maximum limit of 67%, as shown in Table 4.32. This shows that most samples with a  $D_{nom}$  equal to 3/8 inch (9.5 mm) had results at or near the maximum limit on the mid-range sieve. SP-12.5 samples were generally within #8 limits, with an average percent passing the #8 sieve of 47%. SP-9.5 samples were generally close to or above the maximum limit of 67%, with an average percent passing of 61%. Overall, of the 64 samples tested, 51 met the #8 limits, while 12 samples were above the maximum limit and one below the minimum limit. The sample below the lower limit was one of the gap-graded SMA samples, which exhibited a gradation curve significantly coarser than typical aggregate blends and most likely the original aggregate blend did not have to meet the Superpave gradation requirements.

The smallest Superpave control limit was set at the #200 sieve, to restrict the use of aggregates with excessive fines. Extracted aggregate samples had an average of 8.4% and 9.4% passing the #200 sieve, indicating that a majority of samples met the Superpave limit of 10%. The proximity of the average to the limit also indicated that a significant minority of samples did not. The aggregate blend in the original pavement mix designs should have been limited to less than 10% fines. This difference suggests that aggregate degradation during the pavement service life, reclamation or processing may be occurring in all samples, leading to a fines above the #200 limit in aggregate extracted from RAP. This highlights the fact that fines content can be a primary factor to restrict the amount of RAP recycled in HMA. Overall, only 6 samples met all Superpave gradation limits, indicating that when analyzed alone, extracted aggregate from RAP was insufficient to meet the gradation requirements for aggregate in hot mix asphalt.

#### 4.6.2.3 Gradation Analysis

RAP particle gradations were compared to theoretical maximum density models for granular materials, known as Fuller’s model and its derivative, the Talbot equation. These are allometric models; models that produce curves based on the exponential growth formula. These models create an exponential gradation curve starting from the maximum particle size ( $D_{max}$ ) and extend to the theoretical minimum particle size of 0. Theory suggests that coarse-grained soil and aggregate materials with a grain size distribution equal to or close to these curves should produce the optimum particle packing. Furthermore, theory states that optimum particle packing can allow a material to produce its maximum compacted density, shear and bearing strength.

$$P_i = \left( \frac{d_i}{D_{max}} \right)^n \quad \text{Equation 4.3}$$

The generic form of the Fuller’s model formula is shown in Equation 4.3. The variable “i” represents a given sieve size,  $d_i$  is the diameter for the given sieve,  $P_i$  is the percent passing at the diameter  $d_i$ , and  $n$  is the gradation coefficient or shape factor (Asphalt Institute, 2001). Talbot’s equation is a specific form of Fuller’s model, where the shape factor,  $n$ , ranges from 0.33 to 0.50, with an optimum value of 0.45. For Talbot’s equation,  $D_{top}$  is used in place of  $D_{max}$ .



#### 4.6.2.3.1 Gradation Analysis Comparison to Talbot Equation Curves

The grain-size distribution curves for milled and crushed RAP samples were compared to the corresponding Talbot equation curves in Figure 4.56 and Figure 4.57.

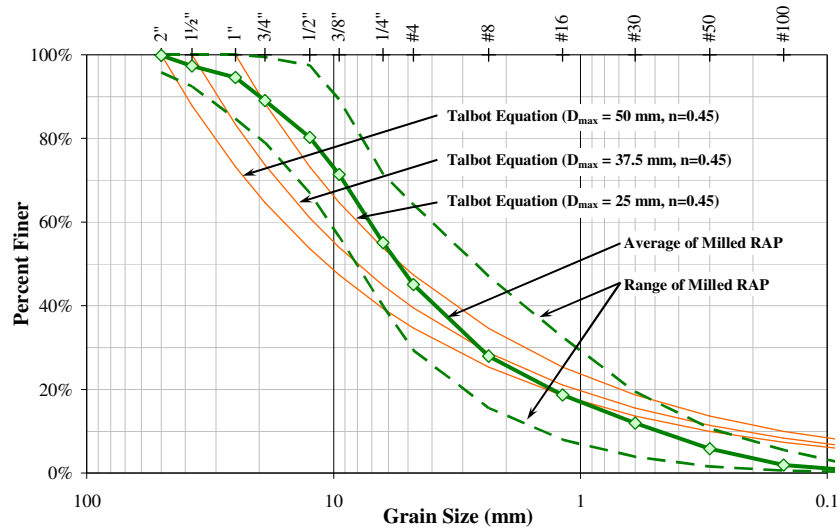


Figure 4.56 Grain-size distribution curves for milled RAP samples

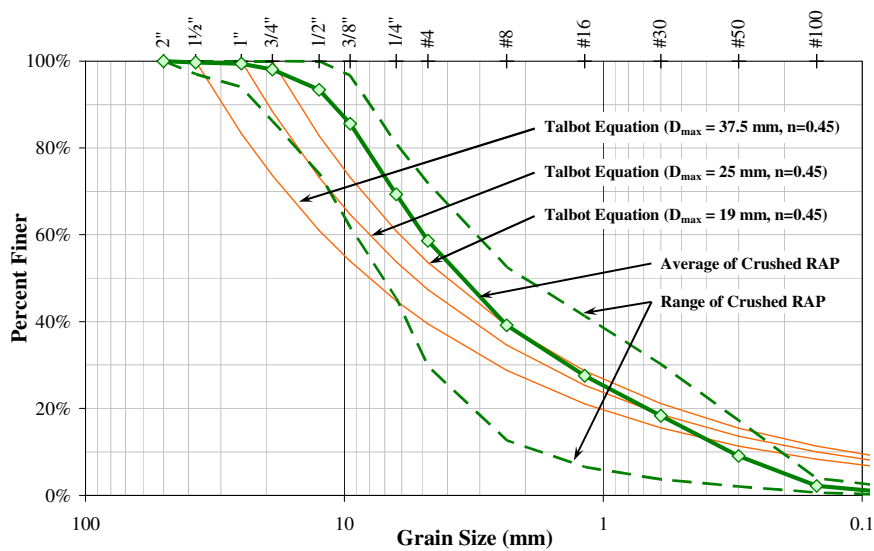


Figure 4.57 Grain-size distribution curves for crushed RAP samples

The most significant differences between RAP gradation curves and the corresponding Talbot equation curves occur at the ends of the range; on all sieve sizes greater than the 3/8 inch (9.5 mm) and less than the #30 (0.60 mm), as shown

in Figure 4.56 and Figure 4.57. According to the Talbot equation, for a soil with  $D_{max}$  of 1/2 to 2 inches, the optimum fines content should be approximately 5 to 11%. The RAP samples tested had a maximum of 1.8% fines. The USCS classification system assigns dual classes to samples with 5 to 12% fines, indicating that these materials contain enough fines to influence their behavior. According to Barksdale (1991), a base material is not free draining if the amount of material passing the #200 sieve is more than about 2.0%. Unlike larger agglomerated particles, aggregate fines in RAP remain bonded to larger particles during milling or crushing processes, leading to the deficiency of particle sizes smaller than the #30 sieve (0.60 mm), including the fines. When RAP is utilized in earthwork applications, the fines content controls the drainage characteristics. Based on gradation testing, all RAP samples were in this study were classified as free-draining soils, confirming previous research identifying RAP as a free-draining material in earthwork applications (Montemayor, 1998; Gomez, 2003). Talbot's model suggests that a soil with 5 to 15% fines would produce the maximum compacted density, shear and bearing strength; yet the lack of fines in RAP provides a substantial benefit of improved drainage and may not significantly influence density or strength.

RAP samples also significantly deviated from the Talbot equation curves on all sizes greater than 3/8 inch (9.5 mm), where values ranged between 55% and 90% passing. According to current FDOT Standard Specifications, coarse aggregates in hot-mix asphalt typically have a  $D_{nom}$  of 3/4 inch (19 mm) to 3/8 inches (9.5 mm) (FDOT, 2007). Figure 4.55 shows that the percent passing the 3/8 inches (9.5 mm) sieve for extracted aggregate from RAP was typically 85% and higher. Therefore, a majority of RAP particles larger than 3/8 inch were not single aggregate particles coated in asphalt; instead they were agglomerated particles of bonded smaller diameter aggregate particles. These particles contain aggregates from the entire particle size range not separated during processing. In conclusion, when RAP is milled and crushed, the weakest asphaltic bonds were broken down, forming a generally well-graded distribution of aggregate particles coated in asphalt, leaving a significant portion of particles larger than 3/8 inch (9.5 mm).

The correlation coefficient between the gradation data and the Talbot equation was calculated for each sample to measure the accuracy of fit. A summary of results comparing sample gradation to the Talbot equation curve is shown in Table 4.33. Most samples had reasonably good correlations to their respective Talbot equation curve, with the lowest value of 0.65 for a crushed RAP and 0.73 for a milled RAP sample. Overall, 63%, a majority of crushed RAP samples had a correlation coefficient of 0.90 or higher, and 15% had a correlation of 0.95 or higher. Milled RAP samples were slightly less likely to fit the Talbot equation curve, as 8% of samples had a correlation of 0.95 or higher and 46% had correlation of 0.90 or higher.

**Table 4.33 Summary of Talbot's equation results**

	Milled			Crushed		
	Average	Min.	Max.	Average	Min.	Max.
Top Size, $D_{top}$ (mm)	45.9	25.0	50.0	29.4	19.0	50.0
Correlation Coefficient, $R^2$	0.88	0.73	0.97	0.90	0.65	0.97

In summary, a perfectly well-graded grain size distribution that follows the Talbot equation was not observed in any of the RAP samples. However, the relatively good correlations suggest that most crushed and milled RAP materials would produce reasonable levels of compaction. The gradation characteristics of RAP materials deviates significantly at the upper and lower size limits, suggesting the compacted density, shear and bearing strength may not be maximized.

#### 4.6.2.3.2 Gradation Analysis Comparison to Fuller's Model

To develop a well-graded model more representative of the low fines content and low proportion of large particles, a Fuller's model equation was also developed for each sample. Similar to the procedure utilized in Sánchez-Leal (2007), statistical software was used to calculate the unique pair of  $n$  and  $D$  values for each sample that produced a curve best fit to the real gradation results, therefore maximizing the correlation coefficient. The diameter was calculated independent of the  $D_{\max}$  or  $D_{\text{nom}}$  measured in the sieve analysis, and was distinguished as the optimal maximum particle size ( $D_{\text{opt}}$ ). The Fuller's model produced curves that provided substantially better correlations to the actual sample gradation than the Talbot equation curves, as no sample had a correlation coefficient less than 0.95.

**Table 4.34 Summary of Fuller's model parameters**

		Milled RAP			Crushed RAP		
		Average	Min.	Max.	Average	Min.	Max.
Sieve Analysis	$D_{\max}$ (mm)	32.8	19.0	50.0	18.3	12.5	37.5
	$D_{\text{nom}}$ (mm)	23.6	12.5	37.5	12.9	9.5	25.0
Fuller's Model	$D_{\text{opt}}$ (mm)	18.5	11.5	29.0	11.9	8.4	22.0
	$n$	0.64	0.46	0.96	0.62	0.48	1.13
	$R^2$	0.99	0.98	1.00	0.99	0.98	1.00
$D_{\max} - D_{\text{opt}}$ (mm)		27.4	-3.6	38.5	17.5	7.1	38.7

A summary of results comparing sample gradation data to the Talbot equation and Fuller's model is shown in Table 4.34. A majority of samples had an excellent correlation of 0.99 or higher; 65% of milled RAP samples and 76% of crushed RAP samples, indicating that all RAP samples could produce compacted densities close to their theoretical maximum density. The calculated  $D_{\text{opt}}$  values ranged from 8.4 to 29.0 mm, which was significantly lower than the measured  $D_{\max}$  values ranging from 12.5 to 50.0 mm. The calculated  $D_{\text{opt}}$  values were within the range of the  $D_{\text{nom}}$  measured for the extracted aggregates, from 3/4 inch (19 mm) to 3/8 inch (9.5 mm). This difference indicated that RAP gradation curves provide a better fit to maximum density models when based on the maximum particle size of the aggregate rather than the agglomerated particle in RAP. The shape factor,  $n$ , ranged from 0.46 to 1.13; these values were significantly larger than the 0.30 to 0.50 range used in the Talbot equation. The shape factor determines how the model draws the curve toward the lower diameter limit. The higher shape factors indicate that the low fines content in RAP samples causes the Fuller's model curve to shift below the Talbot equation curve.

The difference between the sample's maximum and optimal maximum particle size from the Fuller's model was on average 17.7 mm for crushed RAP

samples and 27.4 mm for milled RAP samples. The lower average difference in crushed RAP indicated that these samples were more likely to provide a better gradation fit to the Fuller's model than milled RAP. This finding confirms previous research indicating that the smaller  $D_{max}$  in crushed RAP can lead to higher compacted densities (Montemayor, 1998). It also suggests that the portion of particles larger than the  $D_{opt}$  or aggregate  $D_{nom}$  sieve sizes may be a critical factor influencing the compacted density. In summary, a majority of RAP samples were classified as well-graded by USCS, yet comparison to theoretical maximum density gradation models indicated that all RAP samples exhibit characteristics similar to well-graded soils.

#### **4.6.2.4 Sieve Analysis Correlations to Asphalt Content**

RAP particle gradations were compared to measured asphalt contents and statistical correlations were evaluated. It was assumed that asphalt content would influence the gradation of RAP materials in three ways; fines content, maximum particle size and coefficient of uniformity ( $C_U$ ). It was assumed asphalt binder caused smaller aggregate particles to bind to larger particles; therefore increased asphalt content should cause reduced fines content and  $C_U$ , creating a strong negative correlation. The fines content was assessed using the variables: percent passing the #200 sieve (0.075 mm) and  $D_{10}$ . Likewise, it was assumed increased asphalt content would increase maximum particle size, creating a strong negative correlation. The variables  $D_{top}$ ,  $D_{nom}$ , and  $D_{opt}$  were used as indices of the maximum particle size. Lastly, the percent passing the #30 sieve (0.60 mm) was also used for comparison. It was also assumed that asphalt content would produce higher correlation coefficients to gradation variables in milled RAP samples than crushed RAP samples, since crushed RAP has been subjected to mechanical crushing processes that modify the gradation regardless of asphalt content. Overall, seven gradation variables were identified for comparison to sample asphalt content.

Variables were compared for three sample data sets; milled RAP, crushed RAP, and all RAP samples. A best-fit linear regression trend line was drawn to compare each variable to the asphalt content. To determine the equation of the trend line, the slope ( $m$ ) and intercept ( $b$ ) were calculated. To evaluate the precision of fit, the correlation coefficient ( $R^2$ ) was calculated for each trend line. A summary of results is shown in Table 4.35.

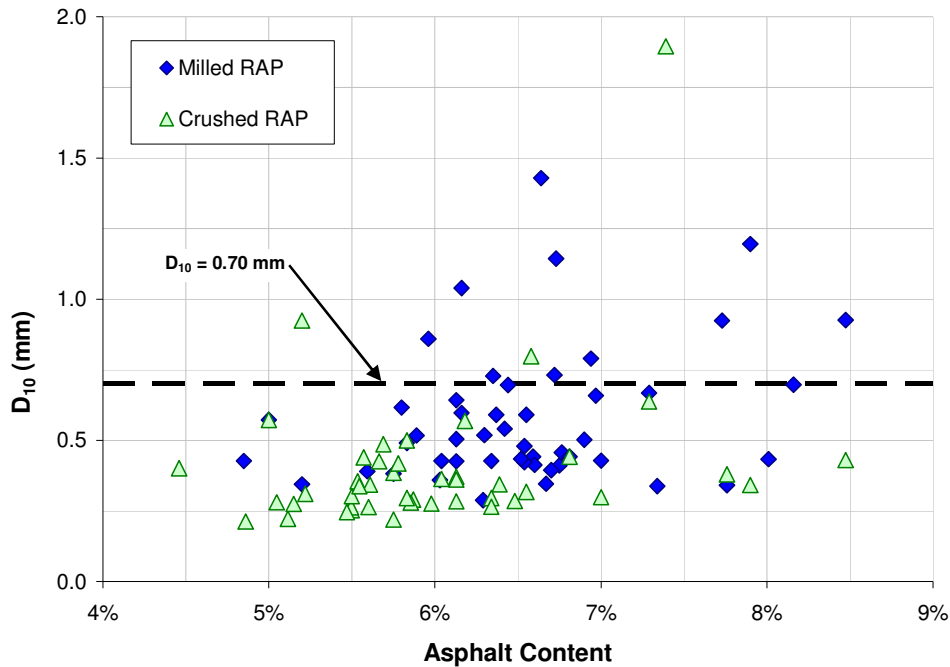
**Table 4.35 Linear regression coefficients between RAP gradation variables and asphalt content**

	Asphalt Content (%) of All RAP Samples			Asphalt Content (%) of Milled RAP			Asphalt Content (%) of Crushed RAP		
	R <sup>2</sup>	m	b	R <sup>2</sup>	m	b	R <sup>2</sup>	m	b
C <sub>U</sub>	0.09	-119.01	21.00	0.09	-142.96	22.97	0.13	-132.60	21.04
Percent Passing #200 Sieve (0.075 mm)	0.06	-0.094	0.011	0.08	-0.115	0.013	0.10	-0.116	0.012
D <sub>10</sub> (mm)	0.13	11.686	-0.237	0.08	9.245	-0.025	0.08	9.198	-0.146
D <sub>top</sub> (mm)	0.08	534.63	6.88	0.04	249.34	34.44	0.00	-1.21	29.33
D <sub>nom</sub> (mm)	0.11	448.37	-0.18	0.03	152.91	26.86	0.00	27.81	16.64
D <sub>opt</sub> (mm)	0.05	144.10	6.12	0.01	70.94	13.5	0.00	-20.84	13.19
Percent Passing #30 Sieve (0.60 mm)	0.13	-2.59	0.31	0.06	-1.22	0.20	0.05	-1.65	0.28

Based on the correlation coefficients of the samples analyzed, no strong or weak linear correlations between gradation variables and asphalt content were evident. Overall, the correlation coefficient on any data set or variable did not exceed 0.2. Therefore, the assumptions that increased asphalt content were directly correlated to decreased fines content and C<sub>U</sub> or increased maximum particle size were not confirmed by either milled RAP or crushed RAP samples. In addition, the assumption that gradation variables in milled RAP samples produced higher correlation coefficients to asphalt content than crushed RAP was not confirmed.

One variable, D<sub>10</sub>, exhibited a notable increase in dispersion of results with increased asphalt content. It was noted that most D<sub>10</sub> values greater than 0.7 mm were in samples with asphalt content of greater than 6.5%. Table 4.29 shows the average D<sub>10</sub> was 0.58 mm and 0.40 mm for milled and crushed RAP samples, while the standard deviation for D<sub>10</sub> was 0.24 mm and 0.27 mm, respectively. Therefore, the limit for one standard deviation above the average was 0.71 mm and 0.67 mm, or approximately 0.7 mm for all RAP samples. A total of 13 samples had a D<sub>10</sub> value greater than 0.70 mm and 9 of these samples had asphalt content greater than 6.5%. Figure 4.58 visually shows these 13 samples above the dashed line. The overall average asphalt content was 6.2% for RAP samples. In general, a majority of samples exhibited D<sub>10</sub> values were close to the average, yet asphalt content values above the average had appreciably larger range of D<sub>10</sub> values. An increase in D<sub>10</sub> indicated an overall decrease in fines content; therefore, a noticeable decrease in the dispersion of results of percent passing the #200 sieve with increasing asphalt content was also observed. In summary, there was no strong

linear correlation between the variables for fines content, yet trends in the dispersion of results indicate some RAP samples with asphalt content greater than 6.5% may result in decreased fines content.



**Figure 4.58 Comparison of  $D_{10}$  to asphalt content**

### 4.6.3 Dry Rodded Unit Weight

The dry rodded unit weight test was utilized as a simple method to measure the unit weight of RAP under loosely-compacted conditions. Tests were conducted on all crushed and milled RAP samples. Given that no moisture-density or bearing strength tests were conducted on crushed RAP samples, the results of this test were used to analyze the unit weight of crushed RAP samples and compare to milled RAP samples. The dry rodded unit weight test was conducted following FM 1-T 019 and AASHTO T-19, “Bulk Density and Voids in Aggregate.” The entire aggregate blend, including particles smaller than #4 was tested. A minimum of five trials allows for the calculation of the average and range for each sample.

The average dry rodded unit weight was 91 lb/ft<sup>3</sup> for milled RAP and 94 lb/ft<sup>3</sup> for crushed RAP. The standard deviation was 5 lb/ft<sup>3</sup> for both milled and crushed RAP data sets. This shows the finer gradation in crushed RAP can produce a slightly higher density than milled RAP. In addition, the unit weight of RAP was lower than soil or aggregate blends due to the presence of asphalt cement. These

values were consistent with previous research; selected values of dry rodded unit weight in RAP are shown in Table 4.36.

**Table 4.36 Selected values of dry rodded unit weight for RAP**

Reference	Location	Type of RAP	Dry Rodded Unit Weight (lb/ft <sup>3</sup> )		Asphalt Content
			Average	Range	
Current Study	Statewide	Milled	91	75 – 101	4.9 - 8.5%
Current Study	Statewide	Crushed	94	80 – 102	4.5 - 7.4%
Patel 2007	Gainesville, FL	Milled	96	94 – 96	6.7%
Dikova 2006	Melbourne, FL	Milled	85	84 – 85	5.8%
Cleary 2005	Melbourne, FL	Crushed	95	92 – 96	5.4%
Gomez 2003	Melbourne, FL	Crushed	92	90 – 94	5.2%
MacGregor 1999	Massachusetts	Crushed	93	N/A	N/A

It was likely that the variations in the unit weight of RAP were caused by differences in the gradation and aggregate type for each processing method. Overall, the average dry rodded unit weight for each processing type was comparable to previous studies on RAP, although the range of values observed in this study was significantly larger.

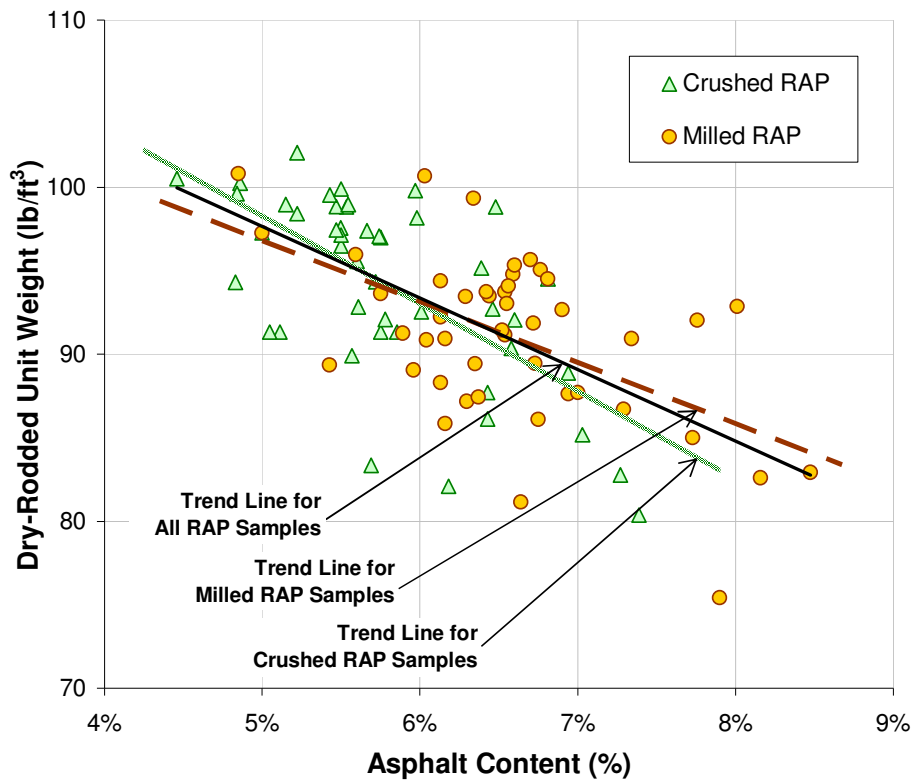
#### **4.6.3.1 Dry Rodded Unit Weight Correlations to Asphalt Content**

Dry rodded unit weight was compared to measured asphalt contents and statistical correlations were evaluated. It was assumed increased asphalt content would decrease the dry rodded unit weight of RAP materials by reducing the individual particle density. In addition, it was assumed asphalt content would influence the packing of particles during rodding and decrease the density by increasing the voids content. Dry rodded unit weight was compared for three sample data sets; milled RAP, crushed RAP, and all RAP samples. A best-fit linear regression trend line was drawn to compare dry rodded unit weight to the asphalt content. It was also assumed that crushed RAP samples would produce a noticeably different trend line and be less correlated to asphalt content than milled RAP due to additional particle breakdown during processing. A summary of results is shown in Table 4.37. Figure 4.59 shows sample results and trend lines for dry rodded unit weight and asphalt content.



**Table 4.37 Linear regression coefficients between dry-rodded unit weight and asphalt content**

	Asphalt Content (%) of All RAP Samples			Asphalt Content (%) of Milled RAP			Asphalt Content (%) of Crushed RAP		
	R <sup>2</sup>	m	b	R <sup>2</sup>	m	b	R <sup>2</sup>	m	b
Dry-Rodded Unit Weight (lb/ft <sup>3</sup> )	0.41	-4.34	119.34	0.42	-5.29	124.74	0.29	-3.66	114.98



**Figure 4.59 Comparison of dry rodded unit weight to asphalt content**

Based on the correlation coefficients of the samples analyzed, weak negative linear correlations were evident between dry rodded unit weight and asphalt content. As asphalt content increased, dry rodded unit weight decreased. The assumption that the crushed RAP samples would produce a better correlation between unit weight and asphalt content appears incorrect. Crushed RAP produced a correlation coefficient of 0.29, which was slightly lower than milled RAP (0.41). Overall, the trend lines for both data sets were close to each other, indicating there

was no significant difference in the relationship between asphalt content and dry rodded unit weight by processing type.

The relatively steep slope of the trend lines in Figure 4.59 suggest the asphalt content influences the dry rodded unit weight of RAP directly and indirectly, impacting the individual particle density as well as the packing of particles. A majority of RAP samples had asphalt contents between 5.5% and 7.0%, producing unit weights between 85 lb/ft<sup>3</sup> and 100 lb/ft<sup>3</sup>. Over the entire range of asphalt content, between 4.5% and 8.5%, the dry-rodded unit weight decreased by an average of 17%, from 100 lb/ft<sup>3</sup> to 83 lb/ft<sup>3</sup>. This equals a decrease of approximately 4.3% of unit weight per 1.0% increase in asphalt content. If it was assumed that a typical RAP material has asphalt binder with a specific gravity of 1.02 and average aggregate specific gravity was 2.65, an increase of 1.0% asphalt content by weight causes the particle density to decrease an average of 0.6%. Therefore, for every 1.0% increase in asphalt content, there was an average decrease in unit weight of 4.3%; approximately 0.6% directly due to the effect of increasing asphalt content decreasing the average particle density and the remaining 3.7% indirectly due to the effect of looser particle packing creating voids in the compacted sample. This shows the indirect effect of asphalt content on compacted density of RAP is approximately six times larger than the direct effect. Overall, this conclusion was made based on typical specific gravity values for aggregates and asphalt binder, not the actual specific gravity of tested samples. The specific gravity of aggregates may vary considerably across the state of Florida and change the exact ratio between the direct and indirect effects; yet there should remain a significantly larger indirect effect between asphalt content and compacted unit weight of RAP. In summary, there were no strong linear correlation between these variables, yet trends indicate that increased asphalt content typically results in decreased unit weight of RAP materials.

#### **4.6.3.2 Gradation Correlations to Dry Rodded Unit Weight**

Dry rodded unit weight was compared to gradation variables and statistical correlations were evaluated. The same six gradation variables displayed in Table 4.35 were evaluated for dry rodded unit weight. It was assumed increased fines content and coefficient of uniformity improve the packing of particles and increase the dry rodded unit weight, creating a positive correlation. It was also assumed increased maximum particle size creates increased voids and decreases dry rodded unit weight, creating a minor negative correlation.

Variables were compared for three sample data sets; milled RAP, crushed RAP, and all RAP samples. A best-fit linear regression trend line was drawn to compare dry rodded unit weight to the gradation variables. D<sub>10</sub>, percent passing #30 sieve (0.60 mm) and C<sub>U</sub> were the only variables to exhibit weak relationships, with correlation coefficients of 0.2 or greater. A summary of results for these variables was shown in Table 4.38. Correlation results for the percent passing the

#200 sieve are also shown for comparison. All variable results can be found in Appendices C-3 and C-4. Based on low correlation coefficients and inconclusive slope coefficients, the assumption that increased maximum particle size creates voids and decreases dry rodded unit weight was not confirmed in any RAP samples.

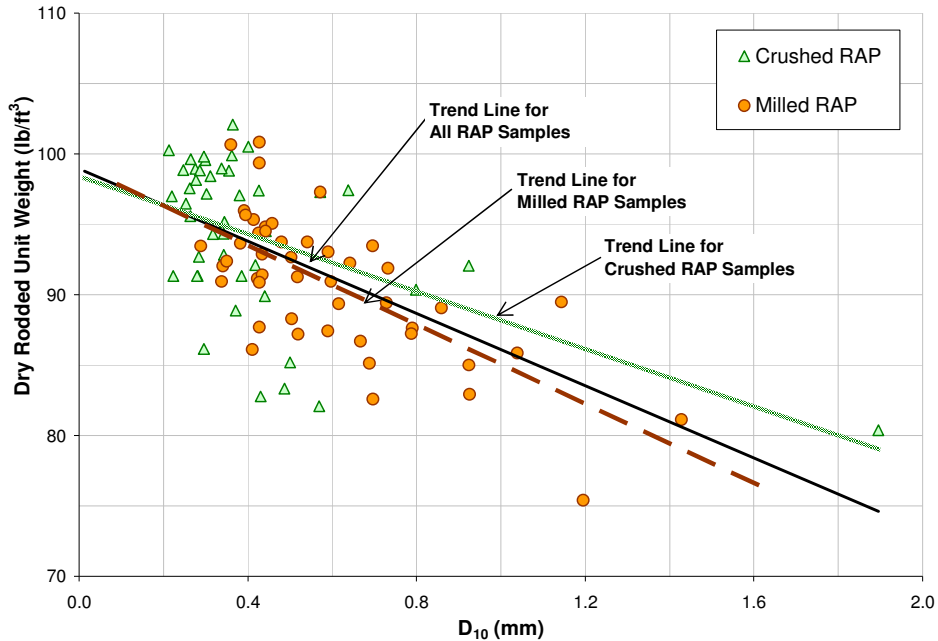
**Table 4.38 Linear regression coefficients between RAP gradation variables and dry rodded unit weight**

	Dry Rodded Unit Weight of All RAP Samples			Dry Rodded Unit Weight of Milled RAP Samples			Dry Rodded Unit Weight of Crushed RAP Samples		
	R <sup>2</sup>	m	b	R <sup>2</sup>	m	b	R <sup>2</sup>	m	b
C <sub>U</sub>	0.37	0.371	-20.847	0.50	0.504	-32.461	0.33	0.321	-16.768
Percent Passing #200 Sieve (0.075 mm)	0.07	0.014%	-0.772%	0.04	0.012%	-0.516%	0.15	0.021%	-1.475%
D <sub>10</sub> (mm)	0.41	-0.032	3.430	0.48	-0.034	3.722	0.25	-0.025	2.735
Percent Passing #30 Sieve (0.60 mm)	0.38	0.674%	-47.348%	0.43	0.484%	-32.097%	0.27	0.582%	-36.370%

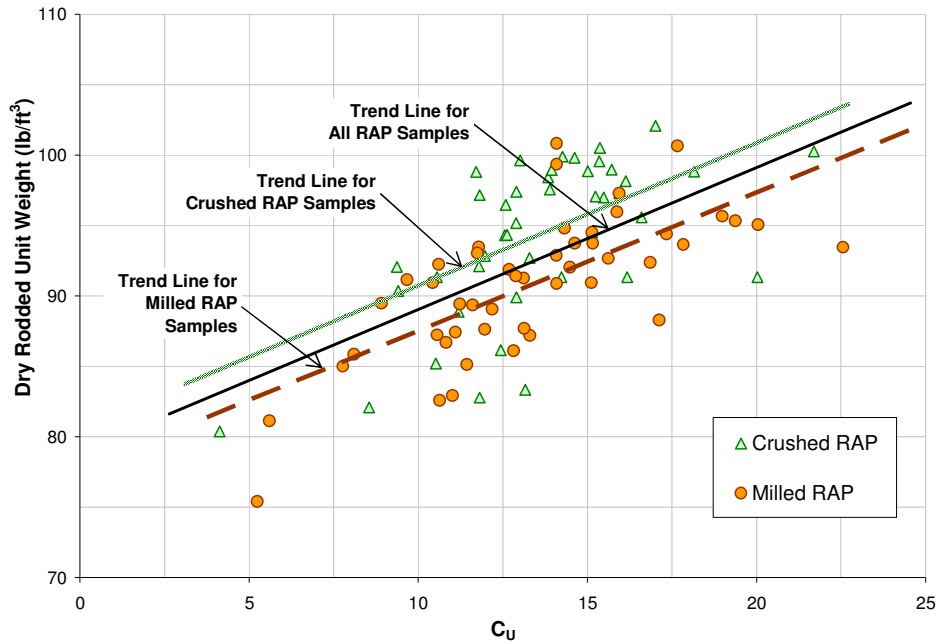
Based on the correlation coefficients, weak linear correlations between dry rodded unit weight and gradation were evident. The assumption that C<sub>U</sub> is positively correlated to dry rodded unit weight was confirmed in both milled and crushed RAP samples. In summary, as the size ratio of the largest and smallest particles in a blend increases ( $C_U = D_{60}/D_{10}$ ), RAP materials will typically produce a more uniform particle distribution capable of filling void spaces and producing a higher bulk unit weight.

The assumption that fines content was positively correlated to dry rodded unit weight was confirmed in both milled and crushed RAP samples. Increased fines content reduces D<sub>10</sub>; therefore higher D<sub>10</sub> values indicate increased voids in RAP, creating a reduced unit weight. This relationship was not observed in the other variable analyzed for fines content, the percent passing the #200 sieve. The #30 sieve typically had a percent passing slightly higher than 10%, with an average of 19% for crushed and 12% for milled RAP samples; therefore, the percent passing the #30 sieve should exhibit a weak positive linear correlation with dry rodded unit weight, the inverse of D<sub>10</sub>. Trends between dry rodded unit weight and D<sub>10</sub> or C<sub>U</sub> were in agreement, as a decrease in D<sub>10</sub> causes an increase in C<sub>U</sub>. The percent passing the #30 sieve ranged from 4% to 30%. The D<sub>10</sub> ranged from 0.21 mm to 1.90 mm, with an average of 0.40 mm for crushed RAP and 0.58 mm for milled RAP, slightly lower than the #30 sieve (0.60 mm). RAP particles finer than the #30 sieve typically represent from 8% to 25% of the total sample mass,

while particles passing the #200 sieve (0.075 mm) typically only represent 1% to 2% of the total sample mass. In summary, these correlations suggest the particles in the  $D_{10}$  and #30 sieve size ranges supply a critical proportion and particle size to fill the void spaces between coarser particles and influence RAP unit weight. Dry rodded unit weight sample results and trend lines for variables  $D_{10}$  and  $C_U$  are shown in Figure 4.60 and Figure 4.61.



**Figure 4.60 Comparison of dry rodded unit weight to  $D_{10}$**

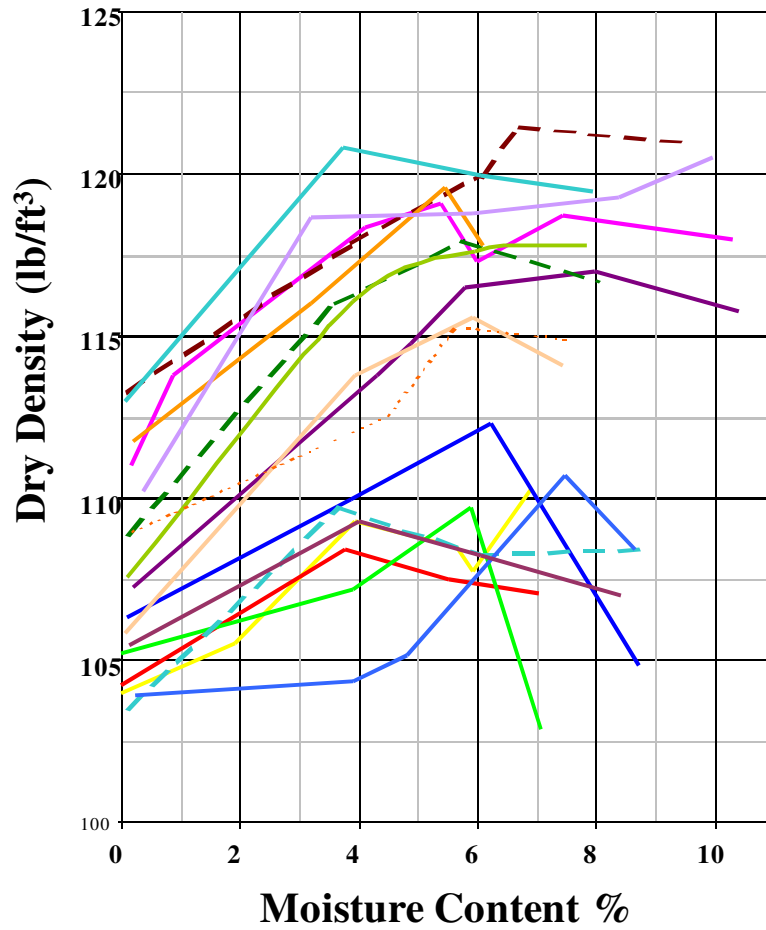


**Figure 4.61 Comparison of dry rodded unit weight to  $C_U$**

The relationship between dry rodded unit weight and the three gradation variables was similar for both crushed and milled RAP samples. For the variables analyzed, crushed RAP produced correlation coefficients from 0.25 to 0.33, slightly lower than milled RAP (0.43 to 0.50); although the trend lines for both data sets were close to each other. Overall, there was no strong linear correlation between dry rodded unit weight and gradation variables; yet trends indicate that increased fines content in RAP typically results in decreased unit weight in both crushed and milled RAP.

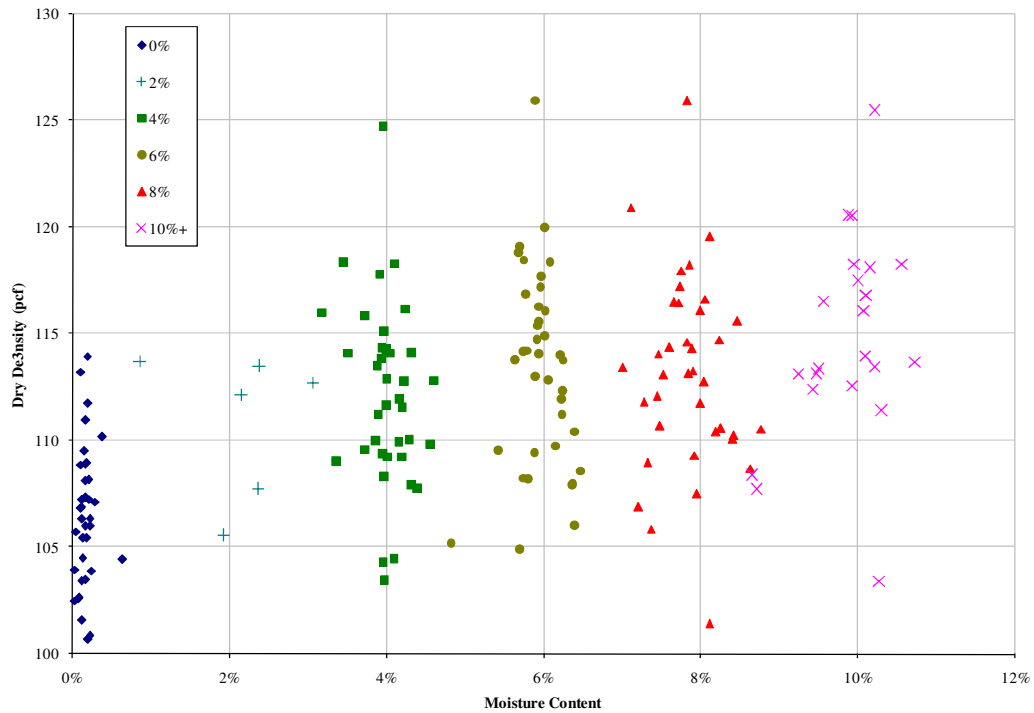
#### **4.6.4 Moisture Density Results**

The modified Proctor moisture density testing, conducted on all crushed and milled RAP samples, is summarized in Appendix C-6. Moisture contents were varied in 2% increments from zero to 12%. Figure 4.62 shows typical moisture density relationships for 17 RAP samples. In general, the classical Proctor shape is not evident. This results matches data from previous reports (Cosentino and Kalajian, 2001, Cosentino et al. 2003) indicating that RAP compaction behavior does not follow the classical Proctor shape.



**Figure 4.62 Typical moisture density relationships**

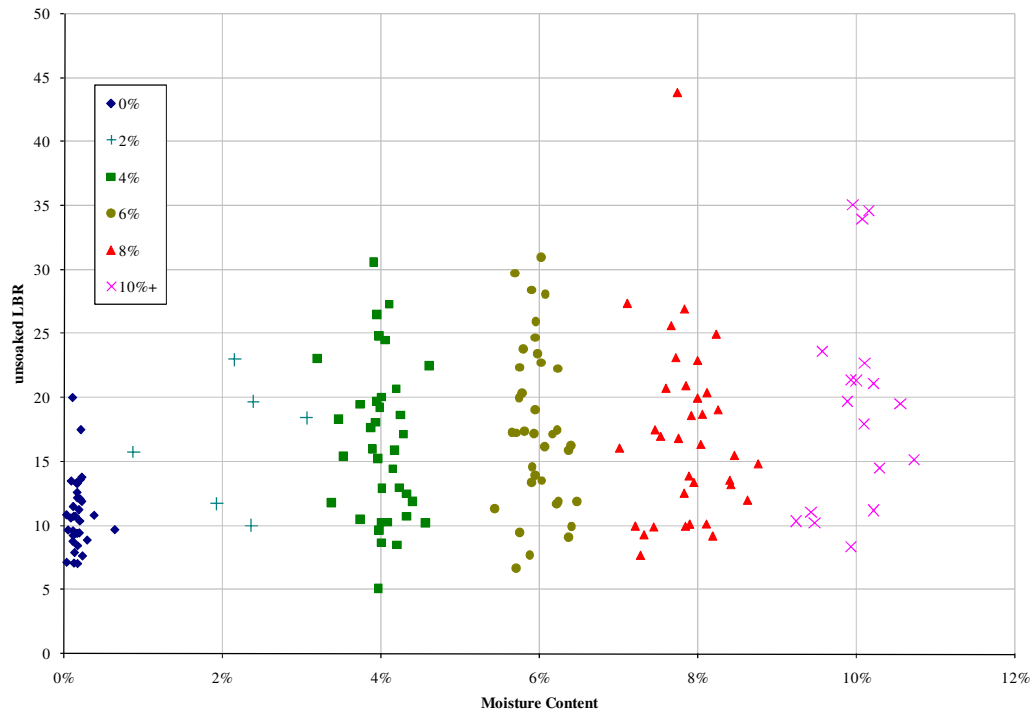
Figure 4.63 shows a summary of all the RAP compaction data. In general, there is a very slight increase in dry density with increasing moisture content. Evaluation of the data between 2 and 10% does not show much change, implying that the lowest and highest moisture contents have the greatest effect on RAP.



**Figure 4.63 Summary of RAP compaction data from modified Proctor testing**

### 4.6.5 LBR Results

Appendix C-7 contains the LBR test results for moisture contents from zero to 12%. The average LBR at zero percent was 11 while the averages for the 2 to 10% range varied between 15 and 17. The average for the 12% moisture was 20. Figure 4.64 shows all the unsoaked LBR data plotted versus the various moisture contents between zero and 12%. Of the 181 tests, 135 or 74.6% of the values were below 20, 39 or 21.5% were between 20 and 30, 6 or 4.1% were between 30 and 40 and one was above 40. In conclusion none of the RAP tested statewide met the current base course standards of 100 for LBR and only one sample would meet the subbase requirements of 43 for unsoaked LBRs.

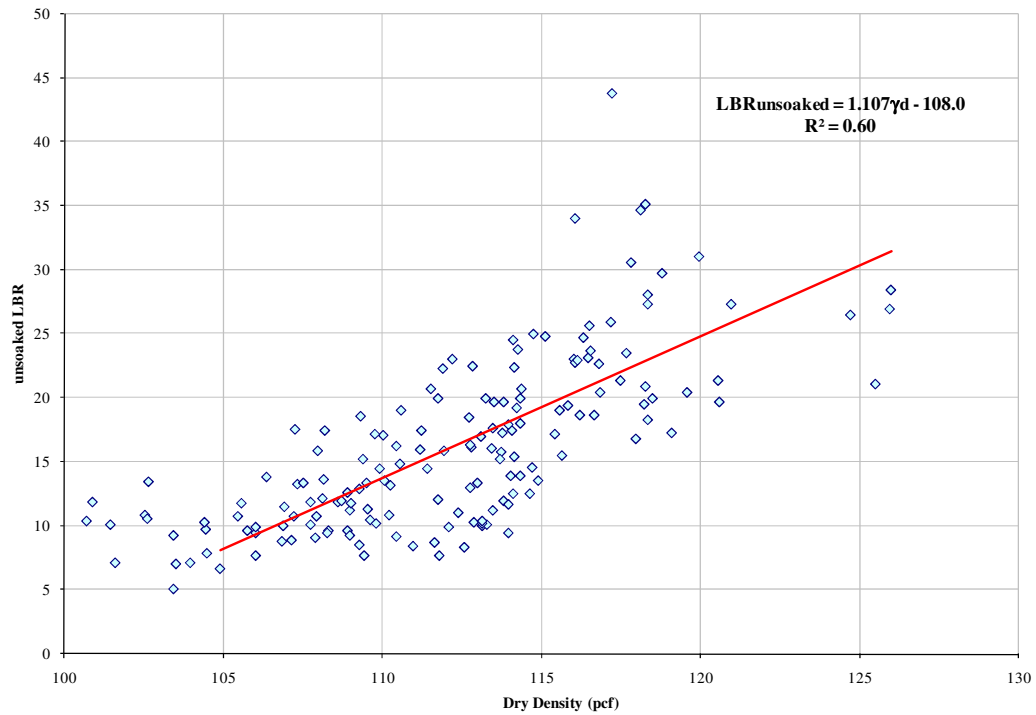


**Figure 4.64 Summary of RAP LBR versus moisture content data**

### 4.6.6 Moisture Density and LBR Correlations

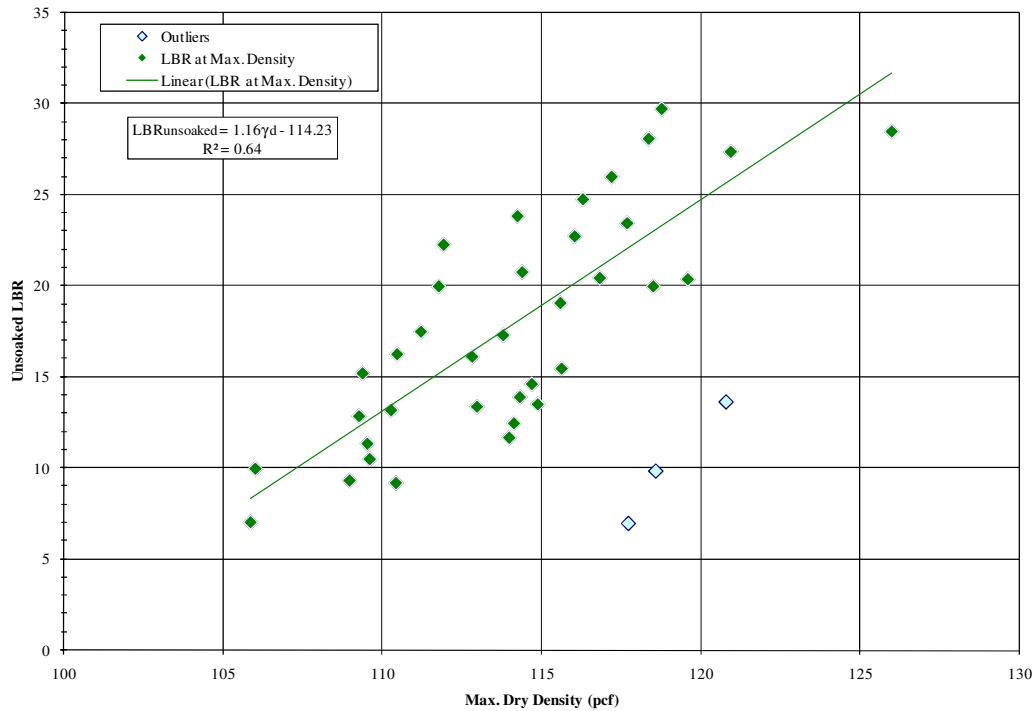
The Proctor dry densities were plotted against the corresponding LBR values to produce the data shown in Figure 4.65. A linear regression analysis was performed to produce the equation shown. Interestingly, there is nearly a one to one correlation between LBR and Dry Density. The trend line produced a correlation coefficient of 0.6; while the actual scatter of data at the low and high densities suggest that the equation would be most suitable between LBR values ranging from 10 to 25.





**Figure 4.65 LBR versus dry density**

Using only the data from an assumed optimum moisture content and associated maximum dry density, Figure 4.66 was developed. Again a nearly one to one, linear correlation was developed and the correlation coefficient was slightly improved over the previous data. These two figures lead to the conclusion that the LBR and the corresponding dry densities were linearly correlated.



**Figure 4.66 LBR versus maximum dry density**

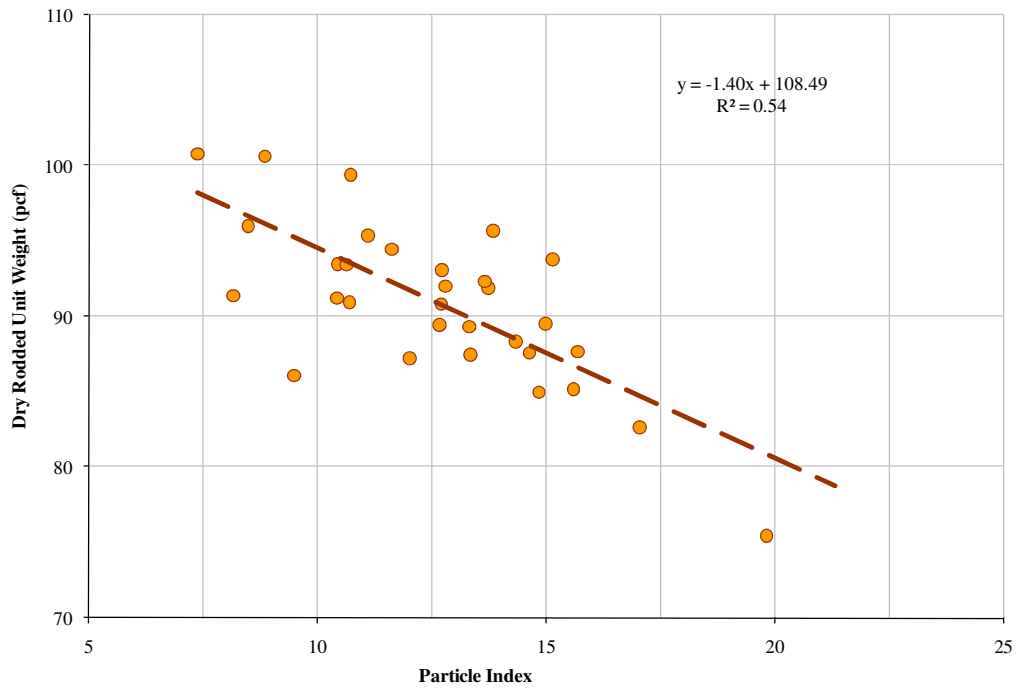
### 4.6.7 Particle Shape Index

The particle shape index analysis performed on select milled samples subjected to moisture-density and LBR testing is summarized in Table 4.39. As this index increases the particles tend to be more angular with more voids. Data calculated both using the ASTM D 3398 equation and the modified Michigan Test equation is presented. The Index for Florida tends to be two to three times higher than Vermont's (Janoo and Bayer, 2001) and Michigan's MTC (1983) reported values. Also the coarse values tend to be higher than the fine values for the Florida RAP.

Correlations between particle shape index and several variables were developed. Poor correlations resulted between the index and asphalt content, percent passing and the gradation parameters; percent passing the #200 sieve and  $D_{10}$ . The only correlation that showed promise was between the dry rodded unit weight and the particle index as shown in Figure 4.67. This plot shows a linear decrease in the density as the index increases, implying that more voids are present as the density decreases.

**Table 4.39 Comparison of particle index values from Florida, Vermont, and Michigan (Janoo and Bayer, 2001, Michigan Transportation Commission, 1983)**

Location	Type of Material	Particle Index (ASTM D3398)		Michigan Modified Particle Index					
		Average	Range	Average	Range	Coarse		Fine	
Florida	Selected Milled RAP	19	16 - 22	19	14 - 26	21	16 - 27	19	13 - 24
	100% Crushed	---	---	8.6	---	7.6	6.0 - 9.1	10.9	---
	75% Crushed	---	---	7.8	---	7.0	5.9 - 9.1	9.0	---
	50% Crushed	---	---	5.7	---	5.7	5.3 - 6.0	11.0	---
	25% Crushed	---	---	4.8	---	4.6	4.2 - 5.4	9.6	---
	Natural Aggregate (0% Crushed)	---	---	2.9	---	3.2	2.2 - 4.9	10.9	---
Vermont	100% Crushed	---	---	11	9 - 14	---	---	---	---
Michigan	Natural Aggregate (0% Crushed)	---	---	5	3 - 7	---	---	---	---



**Figure 4.67 Particle index versus dry rodded unit weight**

## **5 Conclusions & Recommendations**

### **5.1 Conclusions**

The two main objectives of this project were met. Based on the results from the laboratory and large scale creep tests, the excessive creep of RAP and RAP-Soil blends poses a significant engineering concern. Based on the data obtained from over 50 locations, the statewide variability assessment of RAP indicates that some regional variability exists with the asphalt-content, specifically in the Southwestern portion of Florida, however, there were no clear variability issues concerning the engineering behavior.

#### **5.1.1 Lab Creep Testing Conclusions**

1. The 80/20 blend yields nearly optimum gradation and highest density.
2. Singh and Mitchell's (1968) model accurately characterizes RAP and RAP-soil blends.
3. Oedometer testing with the PLDs proved to be reliable for evaluating creep in RAP and RAP-Sand blends.
4. At constant stress, strain decreased with increasing sand content.
5. As the asphalt content decreases the creep is limited.
6. For blends with less than 60% RAP more than 2/3rds of the settlement may occur during construction and 85% may occur within the 1st year.

#### **5.1.2 Large Scale Pullout Testing Conclusions**

1. The ultimate pullout capacity of the MSE metallic strip increased due to an increase in the amount of material passing the #40 sieve and a decrease in asphalt content resulting from blending 50% RAP with 50% sand.
2. The increase of vertical stresses from 6, 12 to 18 psi produced higher pullout capacities in all test materials and the highest pullout capacity in the 50/50 blend consistently.
3. The MSE reinforcement displayed greater horizontal creep when embedded in 100% RAP and 50/50% blend than the A-3 sand at all vertical stress levels. Straps in 100% RAP experienced 16 times more creep displacement than straps in A-3 sand at 50% of the maximum pullout capacity. Straps in the 50/50 mix experienced three times more creep displacement than A-3 sand at 25% of the maximum pullout and eight times more creep than A-3 at 50 % of the maximum pullout. Therefore, RAP and RAP-soil blends produce wall movements that exceed permitted values at service loads.
4. The 100% RAP yielded low friction factors compared to the 50/50% blend and A-3 sand. The lower friction factors are due to a reduced resistance between the soil reinforcement and backfill material which ultimately reduced the maximum pullout capacity.

5. The 50/50 RAP-sand blend increases maximum pullout capacity of MSE wall strips.
6. The maximum pullout capacity increases with overburden.
7. Both the 100% RAP and 50/50 RAP-sand blends display more creep than the A-3 control material during all pullout load increments.

### **5.1.3 Large Scale Vertical Creep Testing Conclusions**

1. For all vertical creep tests (i.e., FIT laboratory (6 to 7days), 45 minute and large scale (4.5 days)) 100% RAP displayed the largest vertical creep displacement, followed by the 50/50% blend; however, both were significantly greater than the A-3 sand.
2. The 100% RAP produced significantly more displacement (i.e., from 14 to 47 times) than the A-3 sand in vertical creep testing.
3. When RAP was used as the top layer of the RAP-sand layered system, with each layer being 6-inches in the large scale vertical creep tests, larger creep displacements (i.e., from 14 times for the 45 minute test, to as high as 325 times for the 4.5 day test) were produced than when the sand was used as the top layer.
4. Placing the sand as the top 6 inch lift produced creep displacements slightly less than the 50/50% blend; however, the layered material still resulted in at least 20 times the displacement of the A-3 sand.

### **5.1.4 RAP Variability Conclusions**

1. The asphalt content of RAP in Florida varied from 3.5% to 11.0%. Typical unprocessed or milled RAP will have 5.5% to 8.0% asphalt. Typical processed or crushed RAP will have 4.5% to 7.0% asphalt. The reduction in asphalt content from milled RAP to crushed RAP is a result of the blending within a facility during the post milling processes.
2. During compaction, RAP will behave like a granular soil. Improving the gradation of well-graded RAP samples, improves the compacted densities.
3. Typical crushed or milled RAP materials are well-graded.
4. The Particle Index Test does not indicate large differences between various RAP sources. The quantity of air voids determined by the Particle Index Test is weakly correlated to the engineering behavior of RAP. In general, for every 1% increase in air voids, there is a decrease of 1.3 lb/ft<sup>3</sup> in compacted density and a decrease of 1.6 in the LBR value.
5. No RAP materials tested in this study met the minimum FDOT base course unsoaked LBR value of 43. RAP should not be used as a base or subbase course material under highway pavements.
6. Asphalt content is correlated to the engineering behavior of RAP. In general, for every 1% increase in asphalt content, there is a decrease of 3.8 lb/ft<sup>3</sup> in compacted density and a decrease of 4.5 in the LBR value. RAP materials with high asphalt content should not be used in earthwork applications where bearing strength is required or long term settlements are a concern.

7. Gradation variability of RAP samples is primarily dependent on processing method. Florida's statewide data did not indicate any geographic differences in gradation, density, or bearing strength.
8. Florida's geographic trends strongly influence the asphalt content of RAP in two separate regions. RAP in the Big Bend region surrounding Tallahassee, FL and the Southwest region exhibit higher asphalt content than the rest of the state. Typical RAP in the Big Bend and southwest region range from 6.8% to 8.0%.
9. These geographic trends also influence the variability in asphalt content of RAP. The Northwest region exhibits the highest variability of asphalt content results. The Southeast region exhibited the lowest variability of asphalt content results.

## 5.2 Recommendations

It is recommended that the existing FDOT Standard Specifications regarding RAP be modified as follows:

1. Remove the current requirement in Section 283 of a minimum 4% asphalt content. For applications where RAPs' low bearing strength or excessive creep would not be a concern, permit the use of RAP and RAP-soil mixtures imposing a requirement for an upper limit of asphalt content.
2. When used as backfill, specify a maximum asphalt content of RAP and RAP-soil mixture of 4%.
3. In order to control the stress level and the expected settlement, limit the height of the embankment fill for which RAP is used, or the depth to which RAP is placed, to 10 feet.
4. Due to large horizontal creep displacements of the soil reinforcement observed during pullout testing and large vertical displacements observed during creep testing, relative to A-3 sand; the 100% RAP and the 50/50% blend are not recommended for use as backfill for MSE walls.
5. RAP-sand layering is not recommended for use behind MSE walls due to large vertical creep displacements.
6. When layering is used in embankments, sand should be placed as the top lift of the layered material.
7. When used in embankments, mixing or blending RAP with sand is recommended over layering RAP and sand.
8. Conduct field testing to evaluate the long-term behavior of RAP and RAP-soil mixtures when compacted using conventional construction techniques. Compare the stress-strain-time characteristics of RAP and a mixture of 50% RAP and 50% A-3 soil.
9. If additional incremental pullout tests are conducted, each load on the soil reinforcement should be held until displacement caused by that load reaches a minimal rate. Following this procedure will produce a time increment in which the entire horizontal displacement of the strip will occur for a particular vertical stress and incremental load.
10. Based on the extensive variability testing program, it is recommended that asphalt-content tests be performed on representative samples of RAP.

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## **Appendix A-1: Stress Level Calculations**

**DATA SHEET FOR STRESS LEVEL CALCULATION**

Example for Wall Height of 15 feet and Surcharge of 2psi

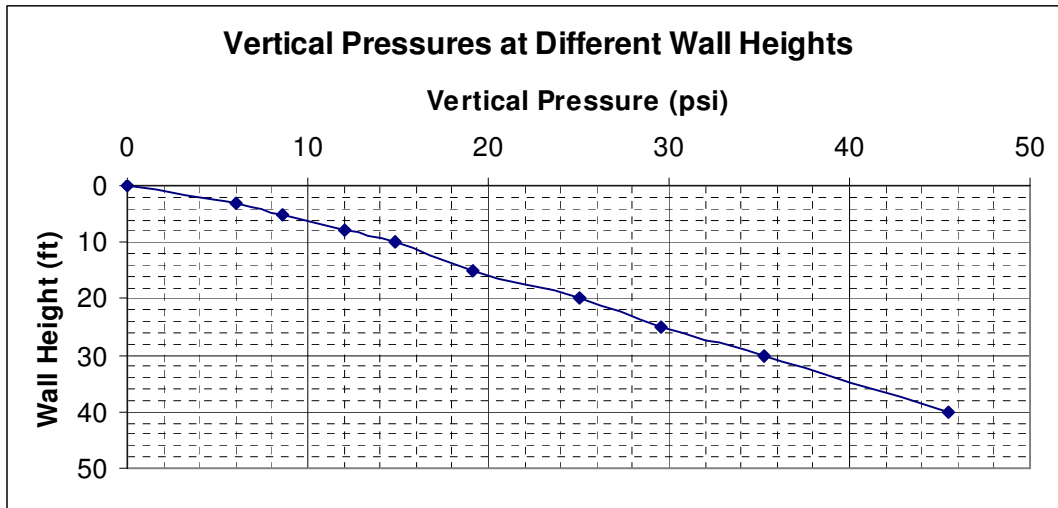
<b>Design Method:</b> Bearing Capacity Failure	<b>Wall Height, H =</b> 20 ft
	<b>Surcharge, q =</b> 2 psi

Backfill	
Type	100% RAP
$\phi_1=$	44 °
$c_1=$	705.6 pcf
$\gamma_1=$	115 pcf
$\phi_1/2=$	22
$k_{a1}=\tan(45-\phi_1/2)^2=$ 0.18	

Retained Fill	
Type	Sandy soil
$\phi_2=$	28 °
$c_2=$	1000 pcf
$\gamma_2=$	105 pcf
$\phi_2/2=$	14
$k_{a2}=\tan(45-\phi_2/2)^2=$ 0.36	

Reinforcement	
Type	Galvanized ribbed strip
w=	3 in
$S_v=$	2 ft
$S_H=$	3 ft
$f_y=$	35000 psi
$\phi_\mu=$	20 °
FS(B)=	3
FS(P)=	3
$L=0.7*H=$ 14 ft	

Forces			
Reinforced Soil Weight	$V_1=$	32200 lb/ft	
Surcharge	$V_2=$	4032 lb/ft	
Resultant from Vertical Forces, $V_1+V_2$	$R=$	36232 lb/ft	
Resultant from Retained Fill	$F_1=$	7582 lb/ft	
Resultant from Surcharge over Fill	$F_2=$	2080 lb/ft	
Sum Resultants from Fill, $F_1+F_2$	$O=$	9661 lb/ft	
FS			
External Stability Factor of Safety	FS=	1.77 >1.5	Check: YES
Moments			
Moment of R about Center Line	$M_R=$	253624 lb.ft/ft	
Moment of O about Center Line	$M_O=$	71340 lb.ft/ft	
Eccentricity	e=	1.97 <L/6	Check: YES
Bearing pressure at base:			
$\sigma_v=$	3601 lb/ft <sup>2</sup>	25 psi	



**Appendix A-2: Data Collected During Creep Testing**



**DATA SHEET FOR CREEP TESTING**

<b>Description of Soil:</b>	Gradation modified 100% RAP at 5.5% MC	<b>Sample No.:</b>	100-A6;100-B6;100-C6
<b>Stress Level:</b>	6 psi	<b>Date Mixed:</b>	9/5/2005
<b>Number of Samples:</b>	3	<b>Date Compacted:</b>	9/7/2005
<b>Tested by:</b>	DD	<b>Compaction:</b>	Modified/D/Mechanical
		<b>Mold Size:</b>	4.59 in

Mix Proportions			
RAP	100%	36	lbs
A-3 Sand	0%	0	lbs

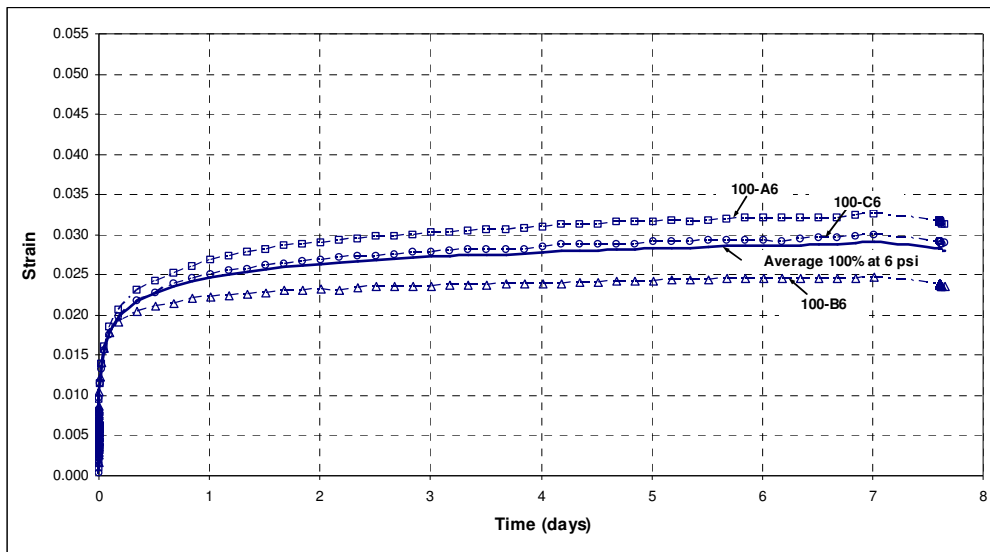
Moisture Content		
Mass of water	1.98	lb

Proctor Max Dry Unit Weight		
At 6%	112.44	pcf

**Date Start Creep:** 9/7/2005  
**Date End Creep:** 9/15/2005  
**Load Frame:** A, B, C

Compaction Moisture Content										
Can Number Date	B1		B2		B3		B4		B5	
	9/7/2005									
Mass of Can	42.78	g	44.10	g	45.81	g	42.96	g	42.96	g
Mass of Wet Soil & Can	290.14	g	339.74	g	281.09	g	274.12	g	274.12	g
Mass of Dry Soil & Can	280.47	g	327.72	g	271.26	g	264.59	g	264.59	g
Mass of Wet Soil	237.69	g	283.62	g	225.45	g	221.63	g	221.63	g
Mass of Water	9.67	g	12.02	g	9.83	g	9.53	g	9.53	g
<b>w (%)</b>	<b>4.07%</b>		<b>4.24%</b>		<b>4.36%</b>		<b>4.30%</b>		<b>4.30%</b>	
<b>Weighted Average w (%)</b>	<b>4.25%</b>		<b>St Dev = 0.11%</b>							

Density Computations						
	Sample1		Sample2		Sample3	
Mass of Mold	14.4	lb	14.4	lb	14.65	lb
Mass of Mold and Wet Soil	22.94	lb	23.01	lb	23.20	lb
Vol of Mold	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>
Mass of Wet Soil	8.540	lb	8.610	lb	8.550	lb
Unit Weight	113.8	lb/ft <sup>3</sup>	114.8	lb/ft <sup>3</sup>	114.0	lb/ft <sup>3</sup>
<b>Dry Unit Weight</b>	<b>109.2</b>	<b>lb/ft<sup>3</sup></b>	<b>110.1</b>	<b>lb/ft<sup>3</sup></b>	<b>109.3</b>	<b>lb/ft<sup>3</sup></b>
<b>Relative Compaction</b>	<b>97.1%</b>		<b>97.9%</b>		<b>97.2%</b>	
<b>Ave Relative Compaction</b>	<b>97.4%</b>					
<b>St Deviation</b>	<b>0.43%</b>					











**DATA SHEET FOR CREEP TESTING**

**Description of Soil:** Gradation modified 100% RAP at 5.5% MC

**Sample No.:** 100-A12;100-B12;100-C12

**Stress Level:** 12 psi  
**Number of Samples:** 3  
**Tested by:** DD

**Date Mixed:** 9/14/2005  
**Date Compacted:** 9/16/2005  
**Compaction:** Modified/D/Mechanical  
**Mold Size:** 4.59 in

Mix Proportions			
RAP	100%	36	lbs
A-3 Sand	0%	0	lbs

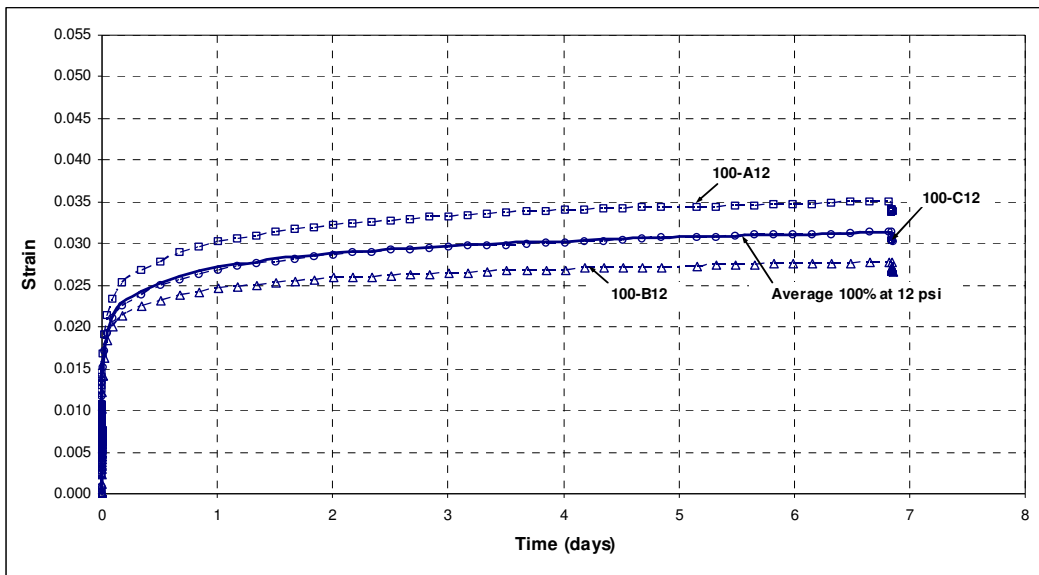
**Date Start Creep:** 9/16/2005  
**Date End Creep:** 9/23/2005  
**Load Frame:** A, B, C

Moisture Content		
Mass of water	1.98	lb

Proctor Max Dry Unit Weight		
At 6%	112.44	pcf

Compaction Moisture Content											
Can Number Date	B1		B2		B3		B4		B5		
	9/16/2005		9/16/2005		9/16/2005		9/16/2005		9/16/2005		
Mass of Can	42.77	g	44.14	g	45.84	g	42.98	g	43.43	g	
Mass of Wet Soil & Can	336.48	g	336.35	g	316.18	g	336.58	g	295.15	g	
Mass of Dry Soil & Can	320.82	g	320.04	g	301.86	g	321.93	g	282.70	g	
Mass of Dry Soil	278.05	g	275.90	g	256.02	g	278.95	g	239.27	g	
Mass of Water	15.66	g	16.31	g	14.32	g	14.65	g	12.45	g	
w (%)	5.63%		5.91%		5.59%		5.25%		5.20%		
<b>Weighted Average w (%)</b>	<b>5.52%</b>		<b>St Dev = 0.29%</b>								

Density Computations						
	Sample1		Sample2		Sample3	
Mass of Mold	14.41	lb	14.65	lb	14.41	lb
Mass of Mold and Wet Soil	23.19	lb	23.45	lb	23.20	lb
Vol of Mold	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>
Mass of Wet Soil	8.780	lb	8.800	lb	8.790	lb
Unit Weight	117.0	lb/ft <sup>3</sup>	117.3	lb/ft <sup>3</sup>	117.2	lb/ft <sup>3</sup>
<b>Dry Unit Weight</b>	<b>110.9</b>	<b>lb/ft<sup>3</sup></b>	<b>111.2</b>	<b>lb/ft<sup>3</sup></b>	<b>111.0</b>	<b>lb/ft<sup>3</sup></b>
<b>Relative Compaction</b>	<b>98.6%</b>		<b>98.9%</b>		<b>98.8%</b>	
<b>Ave Relative Compaction</b>	<b>98.8%</b>					
<b>St Deviation</b>	<b>0.11%</b>					













**DATA SHEET FOR CREEP TESTING**

<b>Description of Soil:</b>	Gradation modified 100% RAP at 5.5% MC	<b>Sample No.:</b>	100-A18;100-B18;100-C18
<b>Stress Level:</b>	18 psi	<b>Date Mixed:</b>	9/21/2005
<b>Number of Samples:</b>	3	<b>Date Compacted:</b>	9/23/2005
<b>Tested by:</b>	DD	<b>Compaction:</b>	Modified/D/Mechanical
		<b>Mold Size:</b>	4.59 in

Mix Proportions			
RAP	100%	36	lbs
A-3 Sand	0%	0	lbs

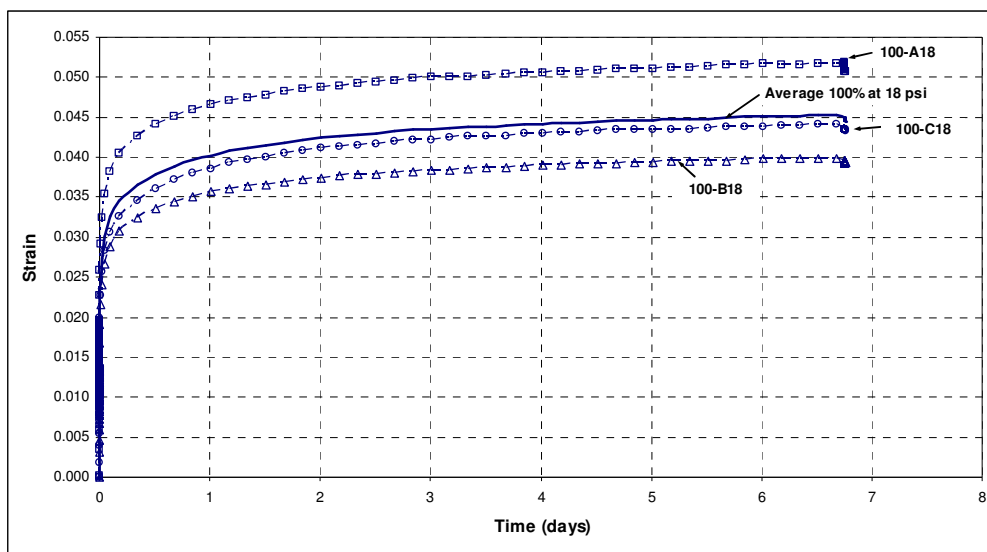
**Date Start Creep:** 9/23/2005  
**Date End Creep:** 9/30/2005  
**Load Frame:** A, B, C

Moisture Content		
Mass of water	1.98	lb

Proctor Max Dry Unit Weight		
At 6%	112.44	pcf

Compaction Moisture Content										
Can Number	B11		B12		B9		B10		B15	
	9/23/2005		9/23/2005		9/23/2005		9/23/2005		9/23/2005	
Date										
Mass of Can	18.33	g	18.33	g	18.21	g	18.29	g	18.06	g
Mass of Wet Soil & Can	334.79	g	337.30	g	307.23	g	326.71	g	323.98	g
Mass of Dry Soil & Can	319.44	g	323.29	g	293.41	g	312.96	g	309.98	g
Mass of Dry Soil	301.11	g	304.96	g	275.20	g	294.67	g	291.92	g
Mass of Water	15.35	g	14.01	g	13.82	g	13.75	g	14.00	g
w (%)	5.10%		4.59%		5.02%		4.67%		4.80%	
<b>Weighted Average w (%)</b>	<b>4.84%</b>		<b>St Dev = 0.22%</b>							

Density Computations						
	Sample1		Sample2		Sample3	
	Mass of Mold	14.42	lb	14.65	lb	14.41
Mass of Mold and Wet Soil	23.04	lb	23.40	lb	23.11	lb
Vol of Mold	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>
Mass of Wet Soil	8.620	lb	8.750	lb	8.700	lb
Unit Weight	114.9	lb/ft <sup>3</sup>	116.6	lb/ft <sup>3</sup>	116.0	lb/ft <sup>3</sup>
<b>Dry Unit Weight</b>	<b>109.6</b>	<b>lb/ft<sup>3</sup></b>	<b>111.3</b>	<b>lb/ft<sup>3</sup></b>	<b>110.6</b>	<b>lb/ft<sup>3</sup></b>
<b>Relative Compaction</b>	<b>97.5%</b>		<b>98.9%</b>		<b>98.4%</b>	
<b>Ave Relative Compaction</b>	<b>98.3%</b>					
<b>St Deviation</b>	<b>0.74%</b>					











**DATA SHEET FOR CREEP TESTING**

<b>Description of Soil:</b>	Gradation modified 80% RAP at 7% MC	<b>Sample No.:</b>	80-D6;80-E6;80-F6
<b>Stress Level:</b>	6 psi	<b>Date Mixed:</b>	9/5/2005
<b>Number of Samples:</b>	3	<b>Date Compacted:</b>	9/7/2005
<b>Tested by:</b>	DD	<b>Compaction:</b>	Modified/D/Mechanical
		<b>Mold Size:</b>	4.59 in

Mix Proportions			
RAP	80%	28.8	lbs
A-3 Sand	20%	7.2	lbs

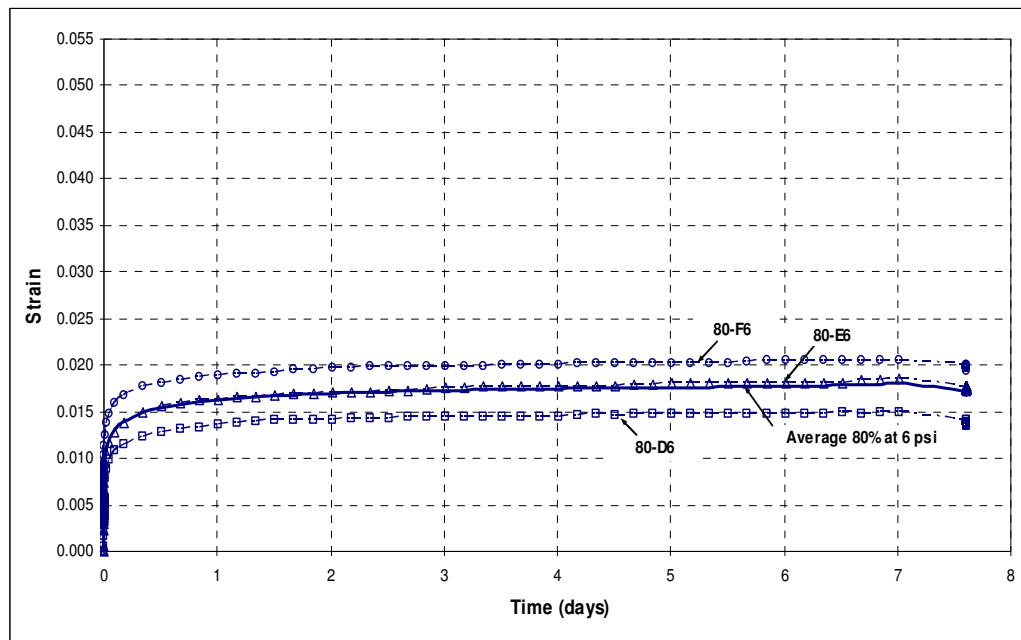
**Date Start Creep:** 9/7/2005  
**Date End Creep:** 9/15/2005  
**Load Frame:** D, E, F

Moisture Content		
Mass of water	2.52	lb

Proctor Max Dry Unit Weight	
At 7.8%	117.33 pcf

Compaction Moisture Content										
Can Number	B6		B7		B8		B41		B42	
	9/7/2005		9/7/2005		9/7/2005		9/7/2005		9/7/2005	
Date										
Mass of Can	42.43	g	43.21	g	56.60	g	14.54	g	14.46	g
Mass of Wet Soil & Can	323.18	g	335.06	g	314.55	g	343.60	g	331.85	g
Mass of Dry Soil & Can	308.54	g	319.83	g	301.36	g	326.78	g	314.35	g
Mass of Dry Soil	266.11	g	276.62	g	244.76	g	312.24	g	299.89	g
Mass of Water	14.64	g	15.23	g	13.19	g	16.82	g	17.50	g
w (%)	5.50%		5.51%		5.39%		5.39%		5.84%	
<b>Weighted Average w (%)</b>	5.52%		St Dev =		0.18%					

Density Computations						
	Sample1		Sample2		Sample3	
Mass of Mold	14.41	lb	14.21	lb	14.56	lb
Mass of Mold and Wet Soil	23.41	lb	23.21	lb	23.54	lb
Vol of Mold	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>
Mass of Wet Soil	9.000	lb	9.000	lb	8.980	lb
Unit Weight	120.0	lb/ft <sup>3</sup>	120.0	lb/ft <sup>3</sup>	119.7	lb/ft <sup>3</sup>
<b>Dry Unit Weight</b>	<b>113.7</b>	<b>lb/ft<sup>3</sup></b>	<b>113.7</b>	<b>lb/ft<sup>3</sup></b>	<b>113.4</b>	<b>lb/ft<sup>3</sup></b>
<b>Relative Compaction</b>	<b>96.9%</b>		<b>96.9%</b>		<b>96.7%</b>	
<b>Ave Relative Compaction</b>	<b>96.8%</b>					
<b>St Deviation</b>	<b>0.12%</b>					













**DATA SHEET FOR CREEP TESTING**

<b>Description of Soil:</b>	Gradation modified 80% RAP at 7% MC	<b>Sample No.:</b>	80-D12;80-E12;80-F12
<b>Stress Level:</b>	12 psi	<b>Date Mixed:</b>	9/14/2005
<b>Number of Samples:</b>	3	<b>Date Compacted:</b>	9/16/2005
<b>Tested by:</b>	DD	<b>Compaction:</b>	Modified/D/Mechanical
		<b>Mold Size:</b>	4.59 in

Mix Proportions			
RAP	80%	28.8	lbs
A-3 Sand	20%	7.2	lbs

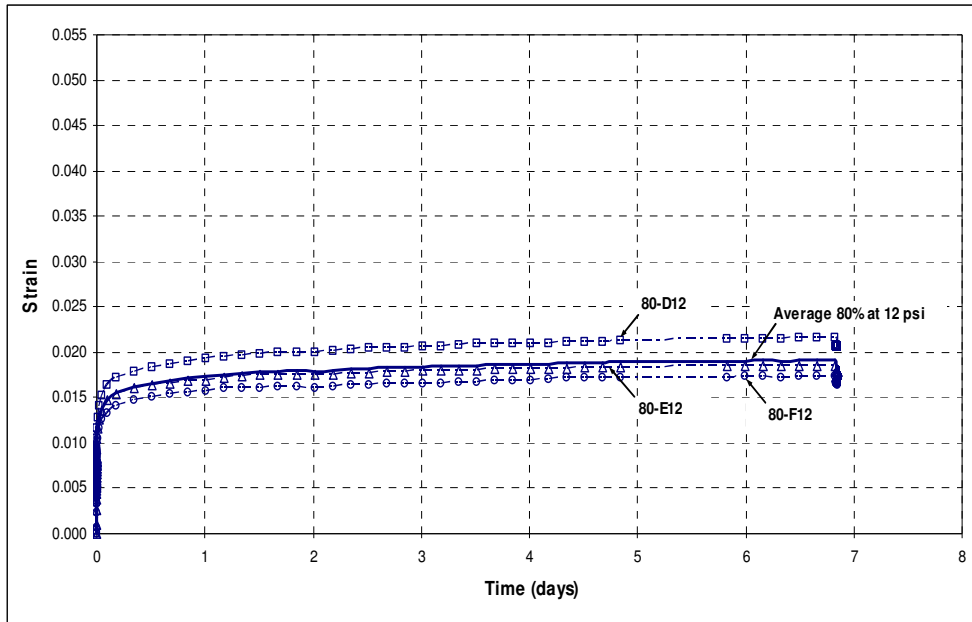
**Date Start Creep:** 9/16/2005  
**Date End Creep:** 9/23/2005  
**Load Frame:** D, E, F

Moisture Content		
Mass of water	2.52	lb

Proctor Max Dry Unit Weight		
At 7.8%	117.33	pcf

Compaction Moisture Content										
Can Number Date	B6		B7		B8		B9		B10	
	9/16/2005		9/16/2005		9/16/2005		9/16/2005		9/16/2005	
Mass of Can	42.43	g	43.16	g	56.60	g	157.46	g	60.12	g
Mass of Wet Soil & Can	343.96	g	335.40	g	325.13	g	340.88	g	300.30	g
Mass of Dry Soil & Can	323.59	g	316.35	g	308.22	g	328.37	g	286.32	g
Mass of Dry Soil	281.16	g	273.19	g	251.62	g	170.91	g	226.20	g
Mass of Water	20.37	g	19.05	g	16.91	g	12.51	g	13.98	g
w (%)	7.24%		6.97%		6.72%		7.32%		6.18%	
<b>Weighted Average w (%)</b>	<b>6.89%</b>		<b>St Dev = 0.46%</b>							

Density Computations						
	Sample1		Sample2		Sample3	
Mass of Mold	14.42	lb	14.22	lb	14.56	lb
Mass of Mold and Wet Soil	23.60	lb	23.43	lb	23.81	lb
Vol of Mold	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>
Mass of Wet Soil	9.180	lb	9.210	lb	9.250	lb
Unit Weight	122.4	lb/ft <sup>3</sup>	122.8	lb/ft <sup>3</sup>	123.3	lb/ft <sup>3</sup>
<b>Dry Unit Weight</b>	<b>114.5</b>	<b>lb/ft<sup>3</sup></b>	<b>114.9</b>	<b>lb/ft<sup>3</sup></b>	<b>115.4</b>	<b>lb/ft<sup>3</sup></b>
<b>Relative Compaction</b>	<b>97.6%</b>		<b>97.9%</b>		<b>98.3%</b>	
<b>Ave Relative Compaction</b>	<b>97.9%</b>					
<b>St Deviation</b>	<b>0.37%</b>					









Date_Time mm/dd/yr_h:m	Load Frame A			Load Frame B			Load Frame C			Average Strain	
	Time (days)	Deflection (in)	Strain	Time (days)	Deflection (in)	Strain	Time (days)	Deflection (in)	Strain	Ave Strain	St Dev
9/19/2005 9:21	3.679200	0.095797	0.020871	3.679214	0.083089	0.018102	3.679226	0.077713	0.016931	0.018635	0.002023
9/19/2005 13:21	3.845868	0.095797	0.020871	3.845879	0.083089	0.018102	3.845890	0.077713	0.016931	0.018635	0.002023
9/19/2005 17:21	4.012534	0.096285	0.020977	4.012546	0.083089	0.018102	4.012557	0.077713	0.016931	0.018670	0.002082
9/19/2005 21:21	4.179200	0.096285	0.020977	4.179212	0.083578	0.018209	4.179224	0.078201	0.017037	0.018741	0.002023
9/20/2005 1:21	4.345868	0.096774	0.021084	4.345880	0.083578	0.018209	4.345890	0.078690	0.017144	0.018812	0.002038
9/20/2005 5:21	4.512534	0.097263	0.021190	4.512546	0.084066	0.018315	4.512558	0.078690	0.017144	0.018883	0.002082
9/20/2005 9:21	4.679201	0.097263	0.021190	4.679212	0.084066	0.018315	4.679223	0.078690	0.017144	0.018883	0.002082
9/20/2005 13:21	4.845868	0.097752	0.021297	4.845880	0.084066	0.018315	4.845890	0.078690	0.017144	0.018919	0.002141
9/20/2005 17:21	5.024455	0.098241	0.021403	5.024470	0.084555	0.018422	5.024481	0.079178	0.017250	0.019025	0.002141
9/20/2005 21:21	5.199112	0.098241	0.021403	5.199127	0.084555	0.018422	5.199137	0.079667	0.017357	0.019060	0.002098
9/21/2005 1:21	6.157788	0.098729	0.021510	6.157603	0.084555	0.018422	6.157464	0.079667	0.017357	0.019096	0.002157
9/21/2005 5:21	6.324455	0.098729	0.021510	6.324270	0.084555	0.018422	6.324131	0.079178	0.017250	0.019060	0.002200
9/21/2005 9:21	6.491122	0.099218	0.021616	6.490937	0.084555	0.018422	6.490797	0.079667	0.017357	0.019131	0.002217
9/21/2005 20:52	6.657788	0.099218	0.021616	6.657603	0.084555	0.018422	6.657464	0.079667	0.017357	0.019131	0.002217
9/22/2005 0:52	6.824455	0.099218	0.021616	6.824270	0.084555	0.018422	6.824131	0.079667	0.017357	0.019131	0.002217
9/22/2005 4:52	6.842580	0.095308	0.020764	6.842465	0.081622	0.017783	6.842371	0.076735	0.016718	0.018422	0.002098
9/22/2005 8:52	6.842591	0.095308	0.020764	6.842476	0.081622	0.017783	6.842383	0.076735	0.016718	0.018422	0.002098
9/22/2005 12:52	6.842591	0.095308	0.020764	6.842488	0.081622	0.017783	6.842395	0.076735	0.016718	0.018422	0.002098
9/22/2005 16:52	6.842603	0.094819	0.020658	6.842499	0.081622	0.017783	6.842418	0.076735	0.016718	0.018386	0.002038
9/22/2005 20:52	6.842603	0.094819	0.020658	6.842511	0.081622	0.017783	6.842429	0.076735	0.016718	0.018386	0.002038
9/23/2005 0:52	6.842615	0.094819	0.020658	6.842522	0.081622	0.017783	6.842429	0.076735	0.016718	0.018386	0.002038
9/23/2005 4:52	6.842615	0.094819	0.020658	6.842534	0.081622	0.017783	6.842441	0.076735	0.016718	0.018386	0.002038
9/23/2005 8:52	6.842626	0.094819	0.020658	6.842546	0.081622	0.017783	6.842452	0.076735	0.016718	0.018386	0.002038
9/23/2005 9:17	6.842626	0.094819	0.020658	6.842557	0.081622	0.017783	6.842464	0.076735	0.016718	0.018386	0.002038
9/23/2005 9:17	6.842638	0.094819	0.020658	6.842569	0.081622	0.017783	6.842487	0.076735	0.016718	0.018386	0.002038
9/23/2005 9:17	6.842638	0.094819	0.020658	6.842580	0.081622	0.017783	6.842499	0.076735	0.016718	0.018386	0.002038
9/23/2005 9:17	6.842649	0.094819	0.020658	6.842592	0.081622	0.017783	6.842499	0.076735	0.016718	0.018386	0.002038
9/23/2005 9:17	6.842661	0.094819	0.020658	6.842603	0.081622	0.017783	6.842522	0.076735	0.016718	0.018386	0.002038
9/23/2005 9:17	6.842661	0.094819	0.020658	6.842615	0.081622	0.017783	6.842522	0.076735	0.016718	0.018386	0.002038
9/23/2005 9:18	6.842672	0.094819	0.020658	6.842627	0.081622	0.017783	6.842533	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.842684	0.094819	0.020658	6.842638	0.081622	0.017783	6.842557	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.842684	0.094819	0.020658	6.842650	0.081622	0.017783	6.842568	0.076735	0.016718	0.018386	0.002038
9/23/2005 9:18	6.842696	0.094819	0.020658	6.842661	0.081622	0.017783	6.842568	0.076735	0.016718	0.018386	0.002038
9/23/2005 9:18	6.842707	0.094819	0.020658	6.842673	0.081622	0.017783	6.842591	0.076735	0.016718	0.018386	0.002038
9/23/2005 9:18	6.842719	0.094819	0.020658	6.842684	0.081622	0.017783	6.842603	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.842730	0.094819	0.020658	6.842696	0.081622	0.017783	6.842603	0.076735	0.016718	0.018386	0.002038
9/23/2005 9:18	6.842742	0.094819	0.020658	6.842719	0.081622	0.017783	6.842614	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.842753	0.094819	0.020658	6.842719	0.081622	0.017783	6.842626	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.842765	0.094819	0.020658	6.842731	0.081622	0.017783	6.842638	0.076735	0.016718	0.018386	0.002038
9/23/2005 9:18	6.842777	0.094819	0.020658	6.842742	0.081622	0.017783	6.842661	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.842788	0.094819	0.020658	6.842754	0.081622	0.017783	6.842672	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.842811	0.094819	0.020658	6.842777	0.081622	0.017783	6.842672	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.842823	0.094819	0.020658	6.842789	0.081622	0.017783	6.842695	0.076735	0.016718	0.018386	0.002038
9/23/2005 9:18	6.842835	0.094819	0.020658	6.842800	0.081622	0.017783	6.842695	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.842835	0.094819	0.020658	6.842800	0.081622	0.017783	6.842719	0.076735	0.016718	0.018386	0.002038
9/23/2005 9:18	6.842846	0.094819	0.020658	6.842823	0.081622	0.017783	6.842730	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.842858	0.094819	0.020658	6.842823	0.081622	0.017783	6.842742	0.076735	0.016718	0.018386	0.002038
9/23/2005 9:18	6.842869	0.094819	0.020658	6.842835	0.081622	0.017783	6.842753	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.842892	0.094819	0.020658	6.842847	0.081622	0.017783	6.842753	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.842904	0.094819	0.020658	6.842858	0.081622	0.017783	6.842776	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.842904	0.094819	0.020658	6.842881	0.081622	0.017783	6.842788	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.842927	0.094819	0.020658	6.842881	0.081622	0.017783	6.842800	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.842939	0.094819	0.020658	6.842893	0.081622	0.017783	6.842800	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.842939	0.094819	0.020658	6.842904	0.081622	0.017783	6.842823	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.842962	0.094819	0.020658	6.842928	0.081622	0.017783	6.842823	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.842973	0.094819	0.020658	6.842928	0.081622	0.017783	6.842834	0.076735	0.016718	0.018386	0.002038
9/23/2005 9:18	6.842973	0.094819	0.020658	6.842939	0.081622	0.017783	6.842846	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.842997	0.094819	0.020658	6.842951	0.081622	0.017783	6.842858	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.843008	0.094819	0.020658	6.842962	0.081622	0.017783	6.842869	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.843020	0.094819	0.020658	6.842974	0.081622	0.017783	6.842881	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.843020	0.094819	0.020658	6.842985	0.081622	0.017783	6.842892	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.843043	0.094819	0.020658	6.843009	0.081622	0.017783	6.842904	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.843043	0.094819	0.020658	6.843009	0.081622	0.017783	6.842915	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.843054	0.094819	0.020658	6.843020	0.081622	0.017783	6.842927	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.843078	0.094819	0.020658	6.843032	0.081622	0.017783	6.842939	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.843078	0.094819	0.020658	6.843043	0.081622	0.017783	6.842950	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.843101	0.094819	0.020658	6.843055	0.081622	0.017783	6.842962	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:18	6.843101	0.094819	0.020658	6.843066	0.081622	0.017783	6.842985	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:19	6.843124	0.094819	0.020658	6.843078	0.081622	0.017783	6.842985	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:19	6.843124	0.094819	0.020658	6.843090	0.081622	0.017783	6.842996	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:19	6.843147	0.094819	0.020658	6.843101	0.081622	0.017783	6.843008	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:19	6.843159	0.094819	0.020658	6.843124	0.081622	0.017783	6.843020	0.076246	0.016611	0.018351	0.002082
9/23/2005 9:19	6.843170	0.094819	0.020658	6.843136	0.081622	0.017783	6.843031				





**DATA SHEET FOR CREEP TESTING**

<b>Description of Soil:</b>	Gradation modified 80% RAP at 7% MC	<b>Sample No.:</b>	80-D18;80-E18;80-F18
<b>Stress Level:</b>	18 psi	<b>Date Mixed:</b>	9/21/2005
<b>Number of Samples:</b>	3	<b>Date Compacted:</b>	9/23/2005
<b>Tested by:</b>	DD	<b>Compaction:</b>	Modified/D/Mechanical
		<b>Mold Size:</b>	4.59 in

Mix Proportions			
RAP	80%	28.8	lbs
A-3 Sand	20%	7.2	lbs

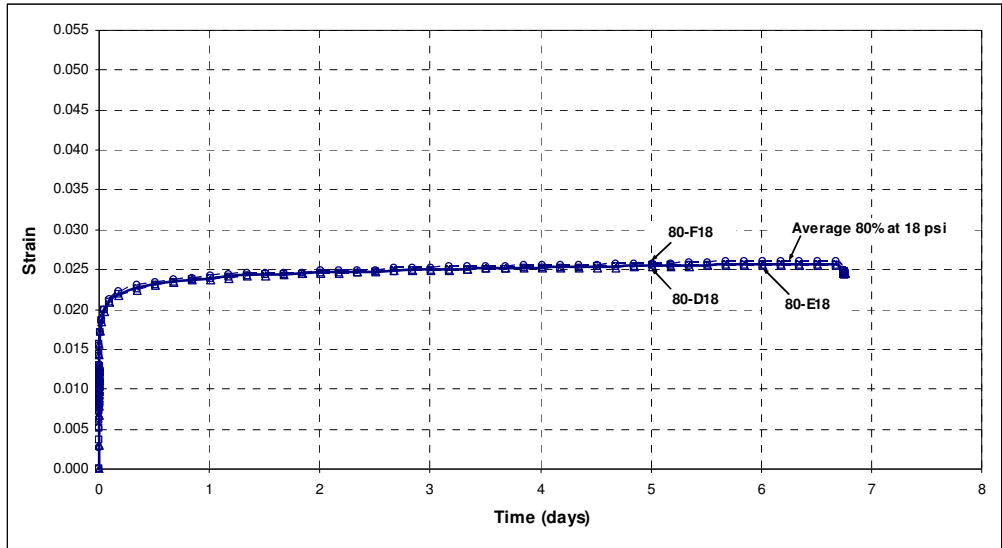
**Date Start Creep:** 9/23/2005  
**Date End Creep:** 9/30/2005  
**Load Frame:** D, E, F

Moisture Content		
Mass of water	2.52	lb

Proctor Max Dry Unit Weight		
At 7.8%	117.33	pcf

Compaction Moisture Content										
Can Number Date	B1		B2		B3		B4		B5	
	9/16/2005		9/16/2005		9/16/2005		9/16/2005		9/16/2005	
Mass of Can	42.76	g	44.12	g	45.84	g	42.95	g	43.42	g
Mass of Wet Soil & Can	250.77	g	298.88	g	311.09	g	301.95	g	340.43	g
Mass of Dry Soil & Can	237.27	g	283.08	g	294.18	g	285.65	g	322.00	g
Mass of Dry Soil	194.51	g	238.96	g	248.34	g	242.70	g	278.58	g
Mass of Water	13.50	g	15.80	g	16.91	g	16.30	g	18.43	g
w (%)	6.94%		6.61%		6.81%		6.72%		6.62%	
<b>Weighted Average w (%)</b>	6.74%		St Dev = 0.14%							

Density Computations						
	Sample1		Sample2		Sample3	
Mass of Mold	14.42	lb	14.22	lb	14.53	lb
Mass of Mold and Wet Soil	23.64	lb	23.45	lb	23.67	lb
Vol of Mold	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>
Mass of Wet Soil	9.220	lb	9.230	lb	9.140	lb
Unit Weight	122.9	lb/ft <sup>3</sup>	123.0	lb/ft <sup>3</sup>	121.8	lb/ft <sup>3</sup>
<b>Dry Unit Weight</b>	115.1	lb/ft <sup>3</sup>	115.3	lb/ft <sup>3</sup>	114.1	lb/ft <sup>3</sup>
<b>Relative Compaction</b>	98.1%		98.2%		97.3%	
<b>Ave Relative Compaction</b>	97.9%					
<b>St Deviation</b>	0.53%					











**DATA SHEET FOR CREEP TESTING**

**Description of Soil:** Gradation modified 60% RAP at 7% MC

**Sample No.:** 60-A6;60-B6;60-C6

**Stress Level:** 6 psi

**Date Mixed:** 9/28/2005

**Number of Samples:** 3

**Date Compacted:** 9/30/2005

**Tested by:** DD

**Compaction:** Modified/D/Mechanical

**Mold Size:** 4.59 in

Mix Proportions			
RAP	60%	21.6	lbs
A-3 Sand	40%	14.4	lbs

**Date Start Creep:** 9/30/2005

**Date End Creep:** 10/7/2005

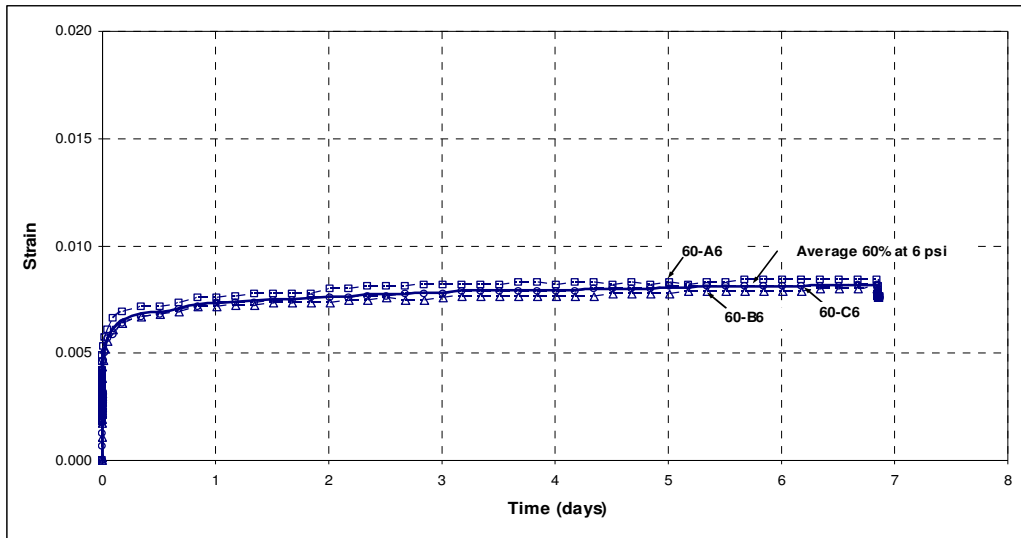
**Load Frame:** A, B, C

Moisture Content		
Mass of water	2.52	lb

Proctor Max Dry Unit Weight		
At 6.7%	113.12	pcf

Compaction Moisture Content										
Can Number Date	B1		B2		B3		B4		B5	
	9/30/2005		9/30/2005		9/30/2005		9/30/2005		9/30/2005	
Mass of Can	42.74	g	44.13	g	45.86	g	42.97	g	43.43	g
Mass of Wet Soil & Can	257.82	g	293.85	g	312.23	g	299.10	g	326.90	g
Mass of Dry Soil & Can	243.29	g	278.30	g	294.06	g	281.31	g	308.32	g
Mass of Dry Soil	200.55	g	234.17	g	248.20	g	238.34	g	264.89	g
Mass of Water	14.53	g	15.55	g	18.17	g	17.79	g	18.58	g
w (%)	7.25%		6.64%		7.32%		7.46%		7.01%	
Weighted Average w (%)	7.14%		St Dev =	0.32%						

Density Computations						
	Sample1		Sample2		Sample3	
Mass of Mold	14.41	lb	14.65	lb	14.41	lb
Mass of Mold and Wet Soil	23.28	lb	23.64	lb	23.39	lb
Vol of Mold	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>
Mass of Wet Soil	8.870	lb	8.990	lb	8.980	lb
Unit Weight	118.2	lb/ft <sup>3</sup>	119.8	lb/ft <sup>3</sup>	119.7	lb/ft <sup>3</sup>
Dry Unit Weight	110.4	lb/ft <sup>3</sup>	111.9	lb/ft <sup>3</sup>	111.7	lb/ft <sup>3</sup>
Relative Compaction	97.6%		98.9%		98.8%	
Ave Relative Compaction	98.4%					
St Deviation	0.73%					













**DATA SHEET FOR CREEP TESTING**

<b>Description of Soil:</b> Gradation modified 60% RAP at 7% MC	<b>Sample No.:</b> 60-A12;60-B12;60-C12
<b>Stress Level:</b> 12 psi	<b>Date Mixed:</b> 10/5/2005
<b>Number of Samples:</b> 3	<b>Date Compacted:</b> 10/7/2005
<b>Tested by:</b> DD	<b>Compaction:</b> Modified/D/Mechanical
	<b>Mold Size:</b> 4.59 in

RAP	60%	21.6	lbs
A-3 Sand	40%	14.4	lbs

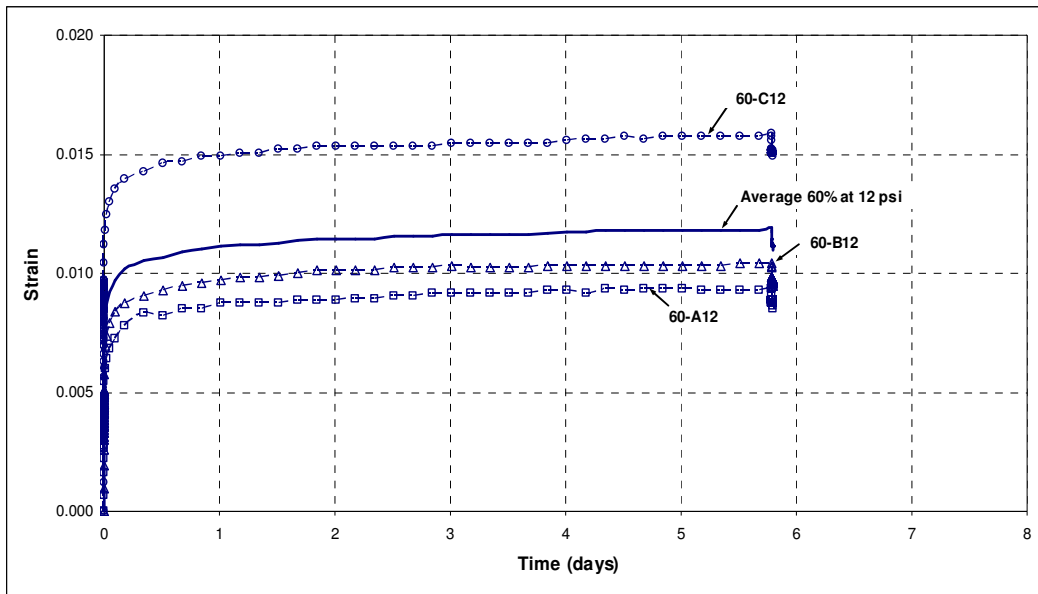
<b>Date Start Creep:</b>	10/7/2005
<b>Date End Creep:</b>	10/13/2005
<b>Load Frame:</b>	A, B, C

Mass of water	2.52	lb
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At 6.7%	113.12	pcf
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Can Number	B1		B2		B3		B4		
	10/7/2005		10/7/2005		10/7/2005		10/7/2005		
Date									
Mass of Can	42.75	g	44.11	g	45.86	g	42.95	g	
Mass of Wet Soil & Can	266.41	g	342.93	g	314.15	g	282.55	g	
Mass of Dry Soil & Can	252.24	g	322.16	g	296.46	g	266.78	g	
Mass of Dry Soil	209.49	g	278.05	g	250.60	g	223.83	g	
Mass of Water	14.17	g	20.77	g	17.69	g	15.77	g	
w (%)	6.76%		7.47%		7.06%		7.05%		
Weighted Average w (%)	7.08%		St Dev =	0.29%					

	Sample1		Sample2		Sample3	
	Mass of Mold	14.41	lb	14.65	lb	14.4
Mass of Mold and Wet Soil	23.32	lb	23.54	lb	23.36	lb
Vol of Mold	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>
Mass of Wet Soil	8.910	lb	8.890	lb	8.960	lb
Unit Weight	118.8	lb/ft <sup>3</sup>	118.5	lb/ft <sup>3</sup>	119.4	lb/ft <sup>3</sup>
Dry Unit Weight	110.9	lb/ft <sup>3</sup>	110.7	lb/ft <sup>3</sup>	111.5	lb/ft <sup>3</sup>
Relative Compaction	98.0%		97.8%		98.6%	
Ave Relative Compaction	98.2%					
St Deviation	0.40%					













## DATA SHEET FOR CREEP TESTING

<b>Description of Soil:</b> Gradation modified 60% RAP at 7% MC	<b>Sample No.:</b> 60-A18;60-B18;60-C18
<b>Stress Level:</b> 18 psi	<b>Date Mixed:</b> 10/11/2005
<b>Number of Samples:</b> 3	<b>Date Compacted:</b> 10/13/2005
<b>Tested by:</b> DD	<b>Compaction:</b> Modified/D/Mechanical
	<b>Mold Size:</b> 4.59 in

Mix Proportions			
RAP	60%	21.6	lbs
A-3 Sand	40%	14.4	lbs

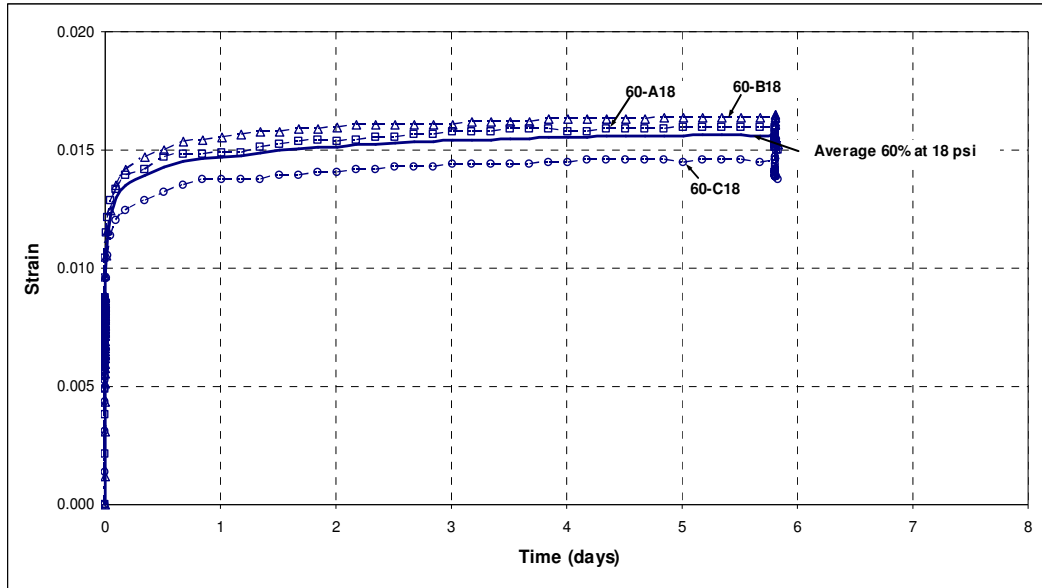
**Date Start Creep:** 10/13/2005  
**Date End Creep:** 10/19/2005  
**Load Frame:** A, B, C

Moisture Content		
Mass of water	2.52	lb

Proctor Max Dry Unit Weight		
At 6.7%	113.12	pcf

Compaction Moisture Content										
Can Number	B1		B2		B3		B4		B5	
	Date 10/13/2005									
Mass of Can	42.73	g	44.12	g	45.85	g	42.97	g	43.41	g
Mass of Wet Soil & Can	288.75	g	290.66	g	287.71	g	273.39	g	276.18	g
Mass of Dry Soil & Can	273.18	g	275.01	g	272.12	g	258.55	g	260.79	g
Mass of Dry Soil	230.45	g	230.89	g	226.27	g	215.58	g	217.38	g
Mass of Water	15.57	g	15.65	g	15.59	g	14.84	g	15.39	g
w (%)	6.76%		6.78%		6.89%		6.88%		7.08%	
<b>Weighted Average w (%)</b>	<b>6.88%</b>		<b>St Dev = 0.13%</b>							

Density Computations						
	Sample1		Sample2		Sample3	
Mass of Mold	14.41	lb	14.65	lb	14.4	lb
Mass of Mold and Wet Soil	23.22	lb	23.45	lb	23.31	lb
Vol of Mold	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>
Mass of Wet Soil	8.810	lb	8.800	lb	8.910	lb
Unit Weight	117.4	lb/ft <sup>3</sup>	117.3	lb/ft <sup>3</sup>	118.8	lb/ft <sup>3</sup>
Dry Unit Weight	109.9	lb/ft <sup>3</sup>	109.8	lb/ft <sup>3</sup>	111.1	lb/ft <sup>3</sup>
<b>Relative Compaction</b>	<b>97.1%</b>		<b>97.0%</b>		<b>98.2%</b>	
<b>Ave Relative Compaction</b>	<b>97.5%</b>					
<b>St Deviation</b>	<b>0.67%</b>					











**DATA SHEET FOR CREEP TESTING**

<b>Description of Soil:</b>	Gradation modified 40% RAP at 7% MC	<b>Sample No.:</b>	40-D6;40-E6;40-F6
<b>Stress Level:</b>	6 psi	<b>Date Mixed:</b>	9/28/2005
<b>Number of Samples:</b>	3	<b>Date Compacted:</b>	9/30/2005
<b>Tested by:</b>	DD	<b>Compaction:</b>	Modified/D/Mechanical
		<b>Mold Size:</b>	4.59 in

Mix Proportions			
RAP	40%	14.4	lbs
A-3 Sand	60%	21.6	lbs

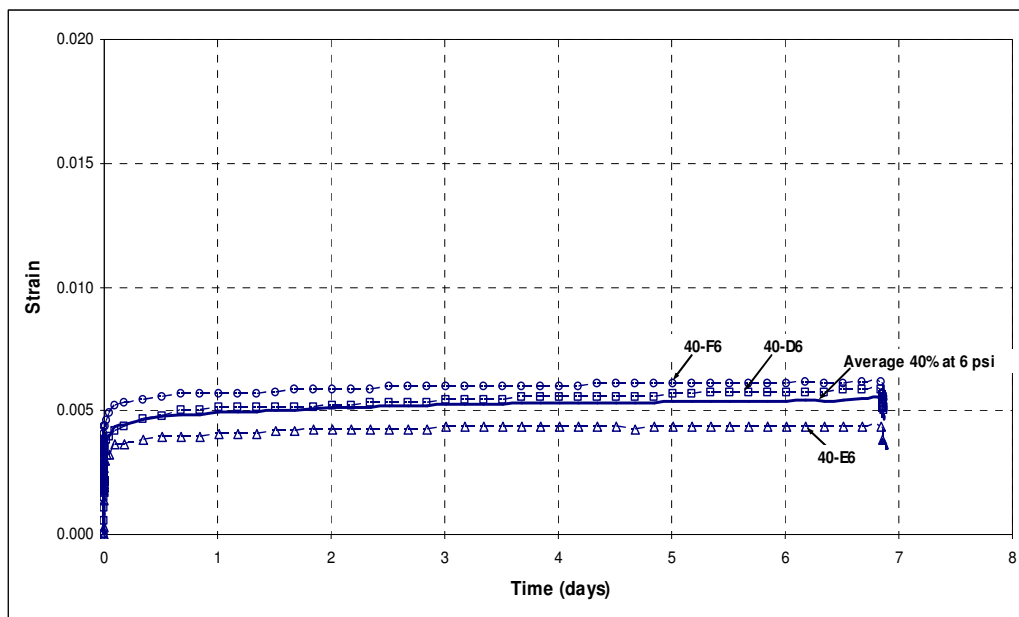
<b>Date Start Creep:</b>	9/30/2005
<b>Date End Creep:</b>	10/7/2005
<b>Load Frame:</b>	D, E, F

Moisture Content		
Mass of water	2.52	lb

Proctor Max Dry Unit Weight		
At 6.9%	108.16	pcf

Can Number Date	B9		B10		B11		B12		B13	
	9/30/2005		9/30/2005		9/30/2005		9/30/2005		9/30/2005	
Mass of Can	18.22	g	18.29	g	18.33	g	18.29	g	18.24	g
Mass of Wet Soil & Can	330.16	g	341.54	g	299.20	g	325.15	g	312.58	g
Mass of Dry Soil & Can	310.95	g	321.45	g	281.25	g	306.15	g	294.39	g
Mass of Dry Soil	292.73	g	303.16	g	262.92	g	287.86	g	276.15	g
Mass of Water	19.21	g	20.09	g	17.95	g	19.00	g	18.19	g
w (%)	6.56%		6.63%		6.83%		6.60%		6.59%	
<b>Weighted Average w (%)</b>	<b>6.64%</b>		<b>St Dev =</b>	<b>0.11%</b>						

Density Computations						
	Sample1		Sample2		Sample3	
Mass of Mold	14.42	lb	14.22	lb	14.56	lb
Mass of Mold and Wet Soil	23.01	lb	22.73	lb	23.00	lb
Vol of Mold	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>
Mass of Wet Soil	8.590	lb	8.510	lb	8.440	lb
Unit Weight	114.5	lb/ft <sup>3</sup>	113.4	lb/ft <sup>3</sup>	112.5	lb/ft <sup>3</sup>
<b>Dry Unit Weight</b>	<b>107.4</b>	<b>lb/ft<sup>3</sup></b>	<b>106.4</b>	<b>lb/ft<sup>3</sup></b>	<b>105.5</b>	<b>lb/ft<sup>3</sup></b>
<b>Relative Compaction</b>	<b>99.3%</b>		<b>98.3%</b>		<b>97.5%</b>	
<b>Ave Relative Compaction</b>	<b>98.4%</b>					
<b>St Deviation</b>	<b>0.87%</b>					













## DATA SHEET FOR CREEP TESTING

<b>Description of Soil:</b>	Gradation modified 40% RAP at 7% MC	<b>Sample No.:</b>	40-D12;40-E12;40-F12
<b>Stress Level:</b>	12 psi	<b>Date Mixed:</b>	10/5/2005
<b>Number of Samples:</b>	3	<b>Date Compacted:</b>	10/7/2005
<b>Tested by:</b>	DD	<b>Compaction:</b>	Modified/D/Mechanical
		<b>Mold Size:</b>	4.59 in

Mix Proportions			
RAP	40%	14.4	lbs
A-3 Sand	60%	21.6	lbs

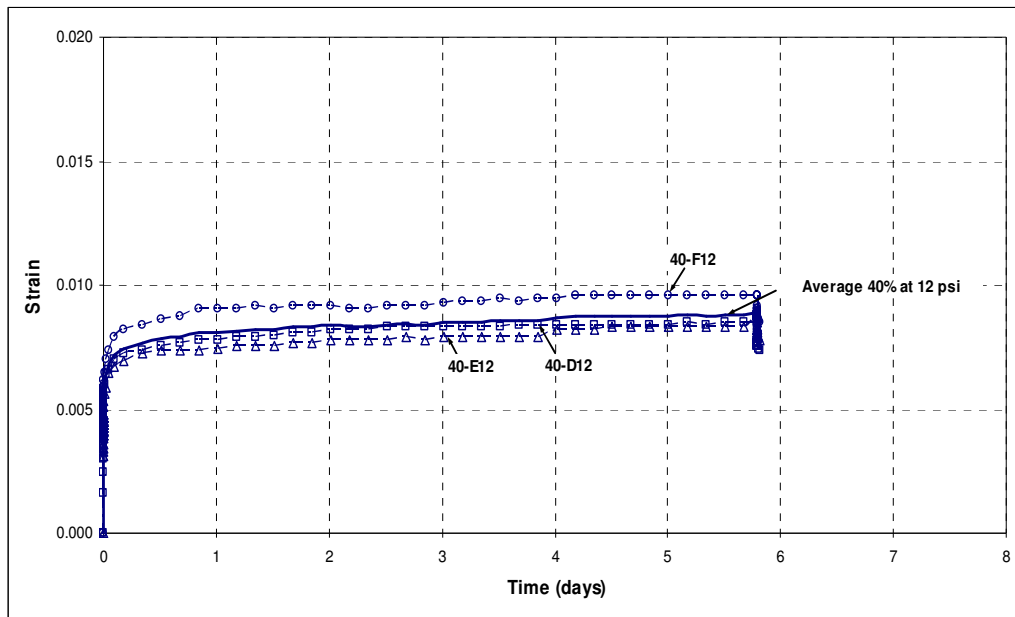
**Date Start Creep:** 10/7/2005  
**Date End Creep:** 10/13/2005  
**Load Frame:** D, E, F

Moisture Content		
Mass of water	2.52	lb

Proctor Max Dry Unit Weight		
At 6.9%	108.16	pcf

Compaction Moisture Content								
Can Number Date	B5		B6		B9		B13	
	10/7/2005		10/7/2005		10/7/2005		10/7/2005	
Mass of Can	43.42	g	42.43	g	18.25	g	18.31	g
Mass of Wet Soil & Can	340.00	g	295.60	g	288.36	g	342.43	g
Mass of Dry Soil & Can	321.30		279.17	g	271.07	g	321.44	g
Mass of Dry Soil	277.88		236.74	g	252.82	g	303.13	g
Mass of Water	18.70	g	16.43	g	17.29	g	20.99	g
w (%)	6.73%		6.94%		6.84%		6.92%	
<b>Weighted Average w (%)</b>	<b>6.86%</b>		<b>St Dev =</b>	<b>0.10%</b>				

Density Computations						
	Sample1		Sample2		Sample3	
Mass of Mold	14.42	lb	14.22	lb	14.56	lb
Mass of Mold and Wet Soil	22.96	lb	22.81	lb	23.12	lb
Vol of Mold	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>
Mass of Wet Soil	8.540	lb	8.590	lb	8.560	lb
Unit Weight	113.8	lb/ft <sup>3</sup>	114.5	lb/ft <sup>3</sup>	114.1	lb/ft <sup>3</sup>
<b>Dry Unit Weight</b>	<b>106.5</b>	<b>lb/ft<sup>3</sup></b>	<b>107.2</b>	<b>lb/ft<sup>3</sup></b>	<b>106.8</b>	<b>lb/ft<sup>3</sup></b>
<b>Relative Compaction</b>	<b>98.5%</b>		<b>99.1%</b>		<b>98.7%</b>	
<b>Ave Relative Compaction</b>	<b>98.8%</b>					
<b>St Deviation</b>	<b>0.29%</b>					









Date_Time mm/dd/yr_h:m	Load Frame A			Load Frame B			Load Frame C			Average Strain	
	Time (days)	Deflection (in)	Strain	Time (days)	Deflection (in)	Strain	Time (days)	Deflection (in)	Strain	Ave Strain	St Dev
10/13/2005 9:35	5.791903	0.035191	0.007667	5.791934	0.036657	0.007986	5.791991	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:35	5.791915	0.035191	0.007667	5.791947	0.036657	0.007986	5.792002	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:35	5.791927	0.035191	0.007667	5.791957	0.036657	0.007986	5.792013	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:35	5.791938	0.035191	0.007667	5.791968	0.036657	0.007986	5.792026	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:35	5.791950	0.035191	0.007667	5.791982	0.036657	0.007986	5.792036	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:35	5.791961	0.035191	0.007667	5.791993	0.036657	0.007986	5.792048	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:35	5.791973	0.035191	0.007667	5.792004	0.036657	0.007986	5.792061	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:35	5.791984	0.035191	0.007667	5.792016	0.036657	0.007986	5.792071	0.040567	0.008838	0.008164	0.000605
10/13/2005 9:35	5.791996	0.035191	0.007667	5.792027	0.036657	0.007986	5.792083	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:35	5.792007	0.035191	0.007667	5.792039	0.036657	0.007986	5.792094	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:35	5.792019	0.035191	0.007667	5.792049	0.036657	0.007986	5.792106	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:35	5.792031	0.035191	0.007667	5.792061	0.036657	0.007986	5.792118	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:35	5.792042	0.035191	0.007667	5.792073	0.036657	0.007986	5.792129	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:35	5.792053	0.035191	0.007667	5.792085	0.036657	0.007986	5.792140	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:35	5.792065	0.035191	0.007667	5.792096	0.036657	0.007986	5.792152	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:35	5.792077	0.035191	0.007667	5.792107	0.036657	0.007986	5.792165	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:35	5.792088	0.035191	0.007667	5.792120	0.036657	0.007986	5.792176	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:35	5.792099	0.035191	0.007667	5.792131	0.036657	0.007986	5.792188	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:35	5.792112	0.035191	0.007667	5.792142	0.036657	0.007986	5.792198	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:35	5.792123	0.035191	0.007667	5.792154	0.036657	0.007986	5.792210	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:36	5.792134	0.035191	0.007667	5.792166	0.036657	0.007986	5.792222	0.039589	0.008625	0.008093	0.000488
10/13/2005 9:36	5.792146	0.035191	0.007667	5.792177	0.036657	0.007986	5.792234	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:36	5.792157	0.035191	0.007667	5.792189	0.036657	0.007986	5.792246	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:36	5.792170	0.035191	0.007667	5.792200	0.036657	0.007986	5.792257	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:36	5.792180	0.035191	0.007667	5.792212	0.036657	0.007986	5.792268	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:36	5.792192	0.035191	0.007667	5.792224	0.036168	0.007880	5.792279	0.040078	0.008732	0.008093	0.000563
10/13/2005 9:36	5.792204	0.034702	0.007560	5.792235	0.036657	0.007986	5.792292	0.040078	0.008732	0.008093	0.000593
10/13/2005 9:36	5.792215	0.035191	0.007667	5.792247	0.036657	0.007986	5.792303	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:36	5.792227	0.034702	0.007560	5.792259	0.036657	0.007986	5.792315	0.040078	0.008732	0.008093	0.000593
10/13/2005 9:36	5.792238	0.035191	0.007667	5.792270	0.036168	0.007880	5.792327	0.039589	0.008625	0.008057	0.000503
10/13/2005 9:36	5.792250	0.035191	0.007667	5.792282	0.036168	0.007880	5.792338	0.040078	0.008732	0.008093	0.000563
10/13/2005 9:36	5.792262	0.035191	0.007667	5.792293	0.036168	0.007880	5.792350	0.040078	0.008732	0.008093	0.000563
10/13/2005 9:36	5.792274	0.035191	0.007667	5.792304	0.036657	0.007986	5.792360	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:36	5.792285	0.035191	0.007667	5.792316	0.036657	0.007986	5.792373	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:36	5.792296	0.034702	0.007560	5.792328	0.036657	0.007986	5.792384	0.039589	0.008625	0.008057	0.000536
10/13/2005 9:36	5.792308	0.034702	0.007560	5.792339	0.036657	0.007986	5.792396	0.039589	0.008625	0.008057	0.000536
10/13/2005 9:36	5.792320	0.034702	0.007560	5.792351	0.036657	0.007986	5.792407	0.040078	0.008732	0.008093	0.000593
10/13/2005 9:36	5.792331	0.034702	0.007560	5.792363	0.036657	0.007986	5.792418	0.040078	0.008732	0.008093	0.000593
10/13/2005 9:36	5.792343	0.035191	0.007667	5.792374	0.036657	0.007986	5.792430	0.040078	0.008732	0.008128	0.000546
10/13/2005 9:36	5.792354	0.034702	0.007560	5.792386	0.036657	0.007986	5.792442	0.040078	0.008732	0.008093	0.000593
10/13/2005 9:36	5.792366	0.034702	0.007560	5.792397	0.036657	0.007986	5.792453	0.039589	0.008625	0.008057	0.000536
10/13/2005 9:36	5.792377	0.034702	0.007560	5.792409	0.036657	0.007986	5.792465	0.039589	0.008625	0.008057	0.000536
10/13/2005 9:36	5.792390	0.034702	0.007560	5.792420	0.036657	0.007986	5.792476	0.039589	0.008625	0.008057	0.000536
10/13/2005 9:36	5.792401	0.034702	0.007560	5.792432	0.036168	0.007880	5.792488	0.039589	0.008625	0.008022	0.000546
10/13/2005 9:36	5.792413	0.035191	0.007667	5.792443	0.036657	0.007986	5.792499	0.039589	0.008625	0.008093	0.000488
10/13/2005 9:36	5.792424	0.034702	0.007560	5.792455	0.036657	0.007986	5.792511	0.040078	0.008732	0.008093	0.000593
10/13/2005 9:36	5.792436	0.035191	0.007667	5.792466	0.036168	0.007880	5.792522	0.039589	0.008625	0.008057	0.000503
10/13/2005 9:36	5.792447	0.034702	0.007560	5.792478	0.036657	0.007986	5.792534	0.039589	0.008625	0.008057	0.000536
10/13/2005 9:36	5.792459	0.035191	0.007667	5.792489	0.036168	0.007880	5.792546	0.040078	0.008732	0.008093	0.000563
10/13/2005 9:36	5.792471	0.034702	0.007560	5.792501	0.036168	0.007880	5.792557	0.040078	0.008732	0.008057	0.000605
10/13/2005 9:36	5.792481	0.035191	0.007667	5.792512	0.036657	0.007986	5.792569	0.039589	0.008625	0.008093	0.000488
10/13/2005 9:36	5.792493	0.035191	0.007667	5.792525	0.036168	0.007880	5.792581	0.040078	0.008732	0.008093	0.000563
10/13/2005 9:36	5.792505	0.034702	0.007560	5.792536	0.036657	0.007986	5.792592	0.039589	0.008625	0.008057	0.000536
10/13/2005 9:36	5.792516	0.035191	0.007667	5.792547	0.036168	0.007880	5.792604	0.039589	0.008625	0.008057	0.000503
10/13/2005 9:36	5.792527	0.034702	0.007560	5.792559	0.036168	0.007880	5.792615	0.039589	0.008625	0.008022	0.000546
10/13/2005 9:36	5.792539	0.034702	0.007560	5.792571	0.036168	0.007880	5.792627	0.040078	0.008732	0.008057	0.000605
10/13/2005 9:36	5.792551	0.034702	0.007560	5.792582	0.036168	0.007880	5.792639	0.039589	0.008625	0.008022	0.000546
10/13/2005 9:36	5.792563	0.034702	0.007560	5.792593	0.036168	0.007880	5.792650	0.039589	0.008625	0.008022	0.000546
10/13/2005 9:36	5.792574	0.034702	0.007560	5.792605	0.036657	0.007986	5.792662	0.039589	0.008625	0.008057	0.000536
10/13/2005 9:36	5.792586	0.034702	0.007560	5.792617	0.036657	0.007986	5.792673	0.039589	0.008625	0.008057	0.000536
10/13/2005 9:36	5.792597	0.034702	0.007560	5.792629	0.036657	0.007986	5.792685	0.039589	0.008625	0.008057	0.000536
10/13/2005 9:38	5.793998	0.034702	0.007560	5.794029	0.036168	0.007880	5.794086	0.039589	0.008625	0.008022	0.000546
10/13/2005 9:42	5.796799	0.034702	0.007560	5.796830	0.035679	0.007773	5.796886	0.039589	0.008625	0.007986	0.000563
10/13/2005 9:50	5.802400	0.034213	0.007454	5.802432	0.036168	0.007880	5.802488	0.039101	0.008519	0.007951	0.000536
10/13/2005 10:06	5.813604	0.033724	0.007347	5.813635	0.035679	0.007773	5.813692	0.039101	0.008519	0.007880	0.000593

## DATA SHEET FOR CREEP TESTING

**Description of Soil:** *Gradation modified 40% RAP at 7% MC*  
**Stress Level:** *18 psi*  
**Number of Samples:** *3*  
**Tested by:** *DD*

**Sample No.:** *40-A18;40-B18;40-C18*  
**Date Mixed:** *10/11/2005*  
**Date Compacted:** *10/13/2005*  
**Compaction:** *Modified/D/Mechanical*  
**Mold Size:** *4.59 in*

Mix Proportions			
RAP	40%	14.4	lbs
A-3 Sand	60%	21.6	lbs

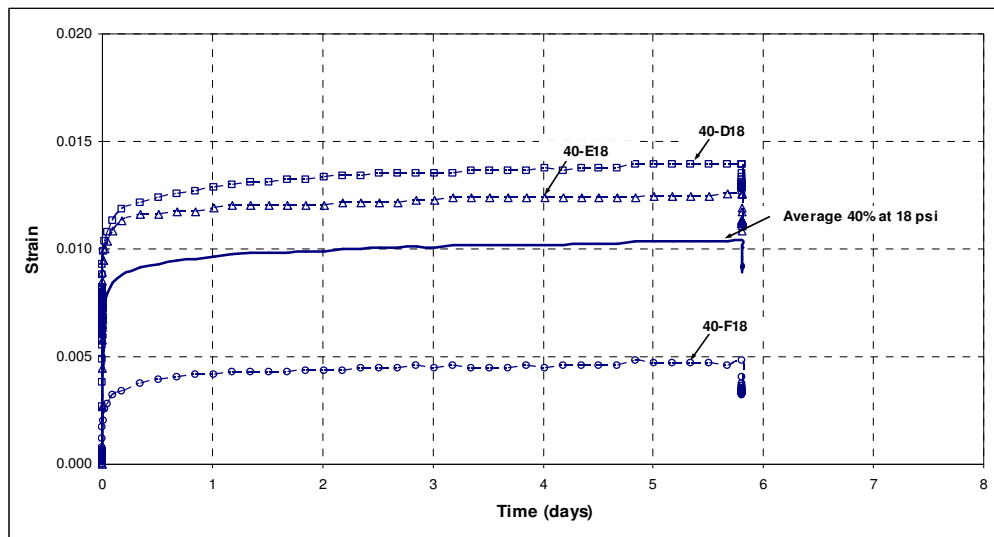
**Date Start Creep:** *10/13/2005*  
**Date End Creep:** *10/19/2005*  
**Load Frame:** *D, E, F*

Moisture Content		
Mass of water	2.52	lb

Proctor Max Dry Unit Weight		
At 6.9%	108.16	pcf

Compaction Moisture Content										
Can Number	B6		B7		B8		B31		B32	
	Date	10/13/2005		10/13/2005		10/13/2005		10/13/2005		10/13/2005
Mass of Can	42.43	g	43.17	g	56.63	g	14.80	g	14.83	g
Mass of Wet Soil & Can	312.81	g	303.10	g	300.37	g	284.21	g	254.69	g
Mass of Dry Soil & Can	294.80	g	285.83	g	284.18	g	266.05	g	239.44	g
Mass of Dry Soil	252.37	g	242.66	g	227.55	g	251.25	g	224.61	g
Mass of Water	18.01	g	17.27	g	16.19	g	18.16	g	15.25	g
w (%)	7.14%		7.12%		7.11%		7.23%		6.79%	
<b>Weighted Average w (%)</b>	<b>7.08%</b>		<b>St Dev =</b>		<b>0.17%</b>					

Density Computations						
	Sample1		Sample2		Sample3	
	Mass of Mold	14.42	lb	14.22	lb	14.56
Mass of Mold and Wet Soil	23.05	lb	22.81	lb	23.09	lb
Vol of Mold	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>
Mass of Wet Soil	8.630	lb	8.590	lb	8.530	lb
Unit Weight	115.0	lb/ft <sup>3</sup>	114.5	lb/ft <sup>3</sup>	113.7	lb/ft <sup>3</sup>
<b>Dry Unit Weight</b>	<b>107.4</b>	<b>lb/ft<sup>3</sup></b>	<b>106.9</b>	<b>lb/ft<sup>3</sup></b>	<b>106.2</b>	<b>lb/ft<sup>3</sup></b>
<b>Relative Compaction</b>	<b>99.3%</b>		<b>98.9%</b>		<b>98.2%</b>	
<b>Ave Relative Compaction</b>	<b>98.8%</b>					
<b>St Deviation</b>	<b>0.58%</b>					













**DATA SHEET FOR CREEP TESTING**

<b>Description of Soil:</b>	Gradation modified 20% RAP at 8% MC	<b>Sample No.:</b>	20-A6;20-B6;20-C6
<b>Stress Level:</b>	6 psi	<b>Date Mixed:</b>	10/24/2005
<b>Number of Samples:</b>	3	<b>Date Compacted:</b>	10/26/2005
<b>Tested by:</b>	DD	<b>Compaction:</b>	Modified/D/Mechanical
		<b>Mold Size:</b>	4.59 in

Mix Proportions			
RAP	20%	7.2	lbs
A-3 Sand	80%	28.8	lbs

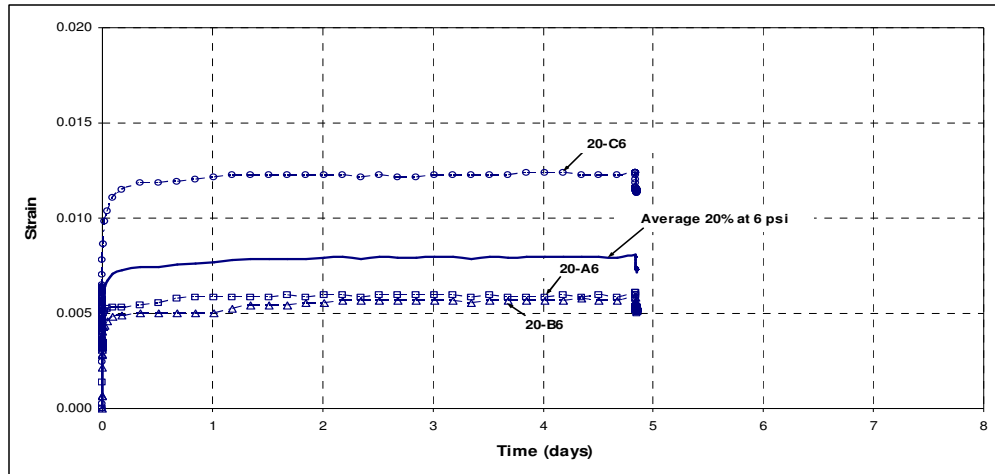
**Date Start Creep:** 10/26/2005  
**Date End Creep:** 10/31/2005  
**Load Frame:** A, B, C

Moisture Content		
Mass of water	2.88	lb

Proctor Max Dry Unit Weight		
At 7.84%	102.60	pcf

Compaction Moisture Content										
Can Number	B1		B2		B3		B5		B6	
	10/26/2005		10/26/2005		10/26/2005		10/26/2005		10/26/2005	
Date	10/26/2005		10/26/2005		10/26/2005		10/26/2005		10/26/2005	
Mass of Can	42.77	g	44.14	g	45.87	g	43.45	g	42.45	g
Mass of Wet Soil & Can	291.24	g	320.24	g	302.02	g	294.66	g	302.56	g
Mass of Dry Soil & Can	273.52	g	300.09	g	283.51	g	277.15	g	284.36	g
Mass of Dry Soil	230.75	g	255.95	g	237.64	g	233.70	g	241.91	g
Mass of Water	17.72	g	20.15	g	18.51	g	17.51	g	18.20	g
w (%)	7.68%		7.87%		7.79%		7.49%		7.52%	
<b>Weighted Average w (%)</b>	7.67%		St Dev =		0.16%					

Density Computations						
	Sample1		Sample2		Sample3	
Mass of Mold	14.4	lb	14.64	lb	14.4	lb
Mass of Mold and Wet Soil	22.60	lb	22.75	lb	22.58	lb
Vol of Mold	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>
Mass of Wet Soil	8.200	lb	8.110	lb	8.180	lb
Unit Weight	109.3	lb/ft <sup>3</sup>	108.1	lb/ft <sup>3</sup>	109.0	lb/ft <sup>3</sup>
<b>Dry Unit Weight</b>	<b>101.5</b>	<b>lb/ft<sup>3</sup></b>	<b>100.4</b>	<b>lb/ft<sup>3</sup></b>	<b>101.3</b>	<b>lb/ft<sup>3</sup></b>
<b>Relative Compaction</b>	98.9%		97.9%		98.7%	
<b>Ave Relative Compaction</b>	98.5%					
<b>St Deviation</b>	0.57%					













## DATA SHEET FOR CREEP TESTING

**Description of Soil:** Gradation modified 20% RAP at 8% MC **Sample No.:** 20-A12;20-B12;20-C12  
**Stress Level:** 12 psi **Date Mixed:** 10/30/2005  
**Number of Samples:** 3 **Date Compacted:** 11/1/2005  
**Tested by:** DD **Compaction:** Modified/D/Mechanical  
**Mold Size:** 4.59 in

Mix Proportions			
RAP	20%	7.2	lbs
A-3 Sand	80%	28.8	lbs

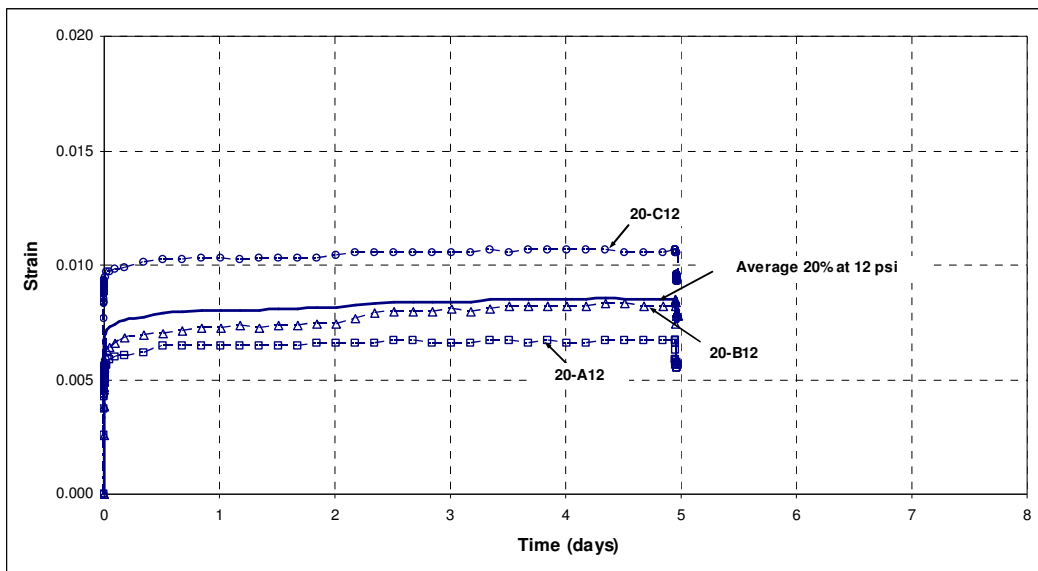
**Date Start Creep:** 11/1/2005  
**Date End Creep:** 11/6/2005  
**Load Frame:** A, B, C

Moisture Content		
Mass of water	2.88	lb

Proctor Max Dry Unit Weight		
At 7.84%	102.60	pcf

Compaction Moisture Content										
Can Number	B1		B2		B3		B5		B6	
	Date		Date		Date		Date		Date	
Date	11/1/2005		11/1/2005		11/1/2005		11/1/2005		11/1/2005	
Mass of Can	42.74	g	44.11	g	45.86	g	43.44	g	42.44	g
Mass of Wet Soil & Can	296.72	g	303.32	g	319.07	g	302.39	g	336.87	g
Mass of Dry Soil & Can	277.64	g	283.73	g	298.64	g	283.27	g	315.36	g
Mass of Dry Soil	234.90	g	239.62	g	252.78	g	239.83	g	272.92	g
Mass of Water	19.08	g	19.59	g	20.43	g	19.12	g	21.51	g
w (%)	8.12%		8.18%		8.08%		7.97%		7.88%	
Weighted Average w (%)	8.05%		St Dev = 0.12%							

Density Computations						
	Sample1		Sample2		Sample3	
Mass of Mold	14.4	lb	14.64	lb	14.4	lb
Mass of Mold and Wet Soil	22.58	lb	22.80	lb	22.59	lb
Vol of Mold	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>
Mass of Wet Soil	8.180	lb	8.160	lb	8.190	lb
Unit Weight	109.0	lb/ft <sup>3</sup>	108.8	lb/ft <sup>3</sup>	109.2	lb/ft <sup>3</sup>
Dry Unit Weight	100.9	lb/ft <sup>3</sup>	100.7	lb/ft <sup>3</sup>	101.0	lb/ft <sup>3</sup>
Relative Compaction	98.4%		98.1%		98.5%	
Ave Relative Compaction	98.3%					
St Deviation	0.18%					









Date_Time mm/dd/yr_h:m	Load Frame A			Load Frame B			Load Frame C			Average Strain	
	Time (days)	Deflection (in)	Strain	Time (days)	Deflection (in)	Strain	Time (days)	Deflection (in)	Strain	Ave Strain	St Dev
11/6/2005 13:09	4.959364	0.025904	0.005644	4.959245	0.036168	0.007880	4.959195	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:09	4.959377	0.025904	0.005644	4.959257	0.036168	0.007880	4.959208	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:09	4.959389	0.025904	0.005644	4.959269	0.036168	0.007880	4.959220	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:09	4.959401	0.025904	0.005644	4.959281	0.036168	0.007880	4.959232	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:09	4.959413	0.025904	0.005644	4.959293	0.036168	0.007880	4.959244	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:09	4.959425	0.025904	0.005644	4.959306	0.036168	0.007880	4.959256	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:09	4.959438	0.025904	0.005644	4.959318	0.036168	0.007880	4.959269	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:09	4.959450	0.025904	0.005644	4.959330	0.036168	0.007880	4.959281	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:09	4.959462	0.025904	0.005644	4.959342	0.036168	0.007880	4.959293	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:09	4.959474	0.025904	0.005644	4.959354	0.036168	0.007880	4.959305	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:09	4.959486	0.025904	0.005644	4.959367	0.036168	0.007880	4.959317	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:09	4.959499	0.025904	0.005644	4.959379	0.036168	0.007880	4.959329	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:09	4.959511	0.025415	0.005537	4.959391	0.036168	0.007880	4.959339	0.042522	0.009264	0.007560	0.001884
11/6/2005 13:09	4.959523	0.025904	0.005644	4.959403	0.036168	0.007880	4.959351	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:09	4.959535	0.025904	0.005644	4.959415	0.036168	0.007880	4.959364	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:09	4.959547	0.025904	0.005644	4.959425	0.036168	0.007880	4.959376	0.042522	0.009264	0.007596	0.001827
11/6/2005 13:10	4.959557	0.025904	0.005644	4.959437	0.036168	0.007880	4.959387	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959569	0.025904	0.005644	4.959449	0.036168	0.007880	4.959400	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959581	0.025904	0.005644	4.959462	0.036168	0.007880	4.959412	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959594	0.025904	0.005644	4.959473	0.036168	0.007880	4.959425	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959605	0.025904	0.005644	4.959486	0.036168	0.007880	4.959437	0.042522	0.009264	0.007596	0.001827
11/6/2005 13:10	4.959617	0.025904	0.005644	4.959498	0.036168	0.007880	4.959449	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959630	0.025415	0.005537	4.959511	0.036168	0.007880	4.959461	0.043011	0.009371	0.007596	0.001932
11/6/2005 13:10	4.959642	0.025904	0.005644	4.959523	0.036168	0.007880	4.959473	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959655	0.025904	0.005644	4.959535	0.036168	0.007880	4.959486	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959667	0.025904	0.005644	4.959547	0.036168	0.007880	4.959498	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959679	0.025904	0.005644	4.959559	0.036168	0.007880	4.959510	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959691	0.025904	0.005644	4.959572	0.036168	0.007880	4.959522	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959703	0.025904	0.005644	4.959584	0.036168	0.007880	4.959533	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959716	0.025904	0.005644	4.959596	0.036168	0.007880	4.959545	0.042522	0.009264	0.007596	0.001827
11/6/2005 13:10	4.959728	0.025904	0.005644	4.959608	0.036168	0.007880	4.959557	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959740	0.025415	0.005537	4.959619	0.036168	0.007880	4.959569	0.043011	0.009371	0.007596	0.001932
11/6/2005 13:10	4.959751	0.025415	0.005537	4.959631	0.036168	0.007880	4.959581	0.043011	0.009371	0.007596	0.001932
11/6/2005 13:10	4.959763	0.025415	0.005537	4.959643	0.036168	0.007880	4.959594	0.043011	0.009371	0.007596	0.001932
11/6/2005 13:10	4.959775	0.025904	0.005644	4.959655	0.036168	0.007880	4.959606	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959787	0.025904	0.005644	4.959667	0.036168	0.007880	4.959617	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959799	0.025904	0.005644	4.959680	0.035679	0.007773	4.959629	0.042522	0.009264	0.007560	0.001820
11/6/2005 13:10	4.959811	0.025904	0.005644	4.959692	0.036168	0.007880	4.959640	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959824	0.025904	0.005644	4.959703	0.036168	0.007880	4.959653	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959835	0.025904	0.005644	4.959715	0.036168	0.007880	4.959665	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959847	0.025904	0.005644	4.959726	0.036168	0.007880	4.959677	0.042522	0.009264	0.007596	0.001827
11/6/2005 13:10	4.959858	0.025904	0.005644	4.959739	0.036168	0.007880	4.959689	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959870	0.025904	0.005644	4.959751	0.036168	0.007880	4.959702	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959883	0.025904	0.005644	4.959763	0.036168	0.007880	4.960700	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959895	0.025904	0.005644	4.959775	0.036168	0.007880	4.960786	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:10	4.959907	0.025904	0.005644	4.959787	0.036168	0.007880	4.963501	0.042522	0.009264	0.007596	0.001827
11/6/2005 13:10	4.960912	0.025904	0.005644	4.960786	0.036168	0.007880	4.969103	0.043011	0.009371	0.007631	0.001876
11/6/2005 13:16	4.963713	0.025415	0.005537	4.963587	0.035679	0.007773	4.969189	0.043011	0.009371	0.007560	0.001926
11/6/2005 13:24	4.969315	0.025904	0.005644	4.969189	0.035679	0.007773	4.980306	0.042522	0.009264	0.007560	0.001820
11/6/2005 13:40	4.980519	0.025904	0.005644	4.980392	0.035679	0.007773	4.980392	0.042522	0.009264	0.007560	0.001820

**DATA SHEET FOR CREEP TESTING**

<b>Description of Soil:</b>	Gradation modified 20% RAP at 8% MC	<b>Sample No.:</b>	20-A18;20-B18;20-C18
<b>Stress Level:</b>	18 psi	<b>Date Mixed:</b>	10/17/2005
<b>Number of Samples:</b>	3	<b>Date Compacted:</b>	10/19/2005
<b>Tested by:</b>	DD	<b>Compaction:</b>	Modified/D/Mechanical
		<b>Mold Size:</b>	4.59 in

Mix Proportions			
RAP	20%	7.2	lbs
A-3 Sand	80%	28.8	lbs

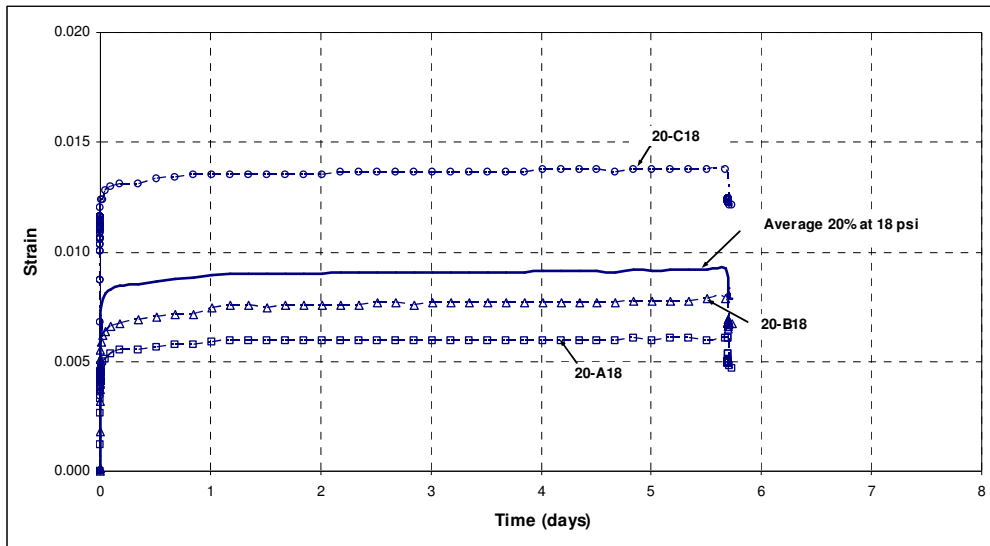
<b>Date Start Creep:</b>	10/19/2005
<b>Date End Creep:</b>	10/26/2005
<b>Load Frame:</b>	A, B, C

Moisture Content		
Mass of water	2.88	lb

Proctor Max Dry Unit Weight		
At 7.84%	102.60	pcf

Compaction Moisture Content										
Can Number Date	B1		B2		B3		B4		B5	
	10/19/2005		10/19/2005		10/19/2005		10/19/2005		10/19/2005	
Mass of Can	42.73	g	44.12	g	45.83	g	42.98	g	43.43	g
Mass of Wet Soil & Can	298.66	g	298.64	g	301.00	g	332.31	g	312.14	g
Mass of Dry Soil & Can	280.13	g	279.62	g	282.79	g	310.73	g	292.59	g
Mass of Dry Soil	237.40	g	235.50	g	236.96	g	267.75	g	249.16	g
Mass of Water	18.53	g	19.02	g	18.21	g	21.58	g	19.55	g
w (%)	7.81%		8.08%		7.68%		8.06%		7.85%	
<b>Weighted Average w (%)</b>	<b>7.89%</b>		<b>St Dev =</b>	<b>0.17%</b>						

Density Computations						
	Sample1		Sample2		Sample3	
Mass of Mold	14.41	lb	14.64	lb	14.41	lb
Mass of Mold and Wet Soil	22.49	lb	22.70	lb	22.45	lb
Vol of Mold	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>
Mass of Wet Soil	8.080	lb	8.060	lb	8.040	lb
Unit Weight	107.7	lb/ft <sup>3</sup>	107.4	lb/ft <sup>3</sup>	107.2	lb/ft <sup>3</sup>
<b>Dry Unit Weight</b>	<b>99.8</b>	<b>lb/ft<sup>3</sup></b>	<b>99.6</b>	<b>lb/ft<sup>3</sup></b>	<b>99.3</b>	<b>lb/ft<sup>3</sup></b>
<b>Relative Compaction</b>	<b>97.3%</b>		<b>97.1%</b>		<b>96.8%</b>	
<b>Ave Relative Compaction</b>	<b>97.1%</b>					
<b>St Deviation</b>	<b>0.24%</b>					











Date_Time mm/dd/yr_h:m	Load Frame A			Load Frame B			Load Frame C			Average Strain	
	Time (days)	Deflection (in)	Strain	Time (days)	Deflection (in)	Strain	Time (days)	Deflection (in)	Strain	Ave Strain	St Dev
10/25/2005 13:10	5.694309	0.022972	0.005005	5.694215	0.031281	0.006815	5.693989	0.056696	0.012352	0.008057	0.003828
10/25/2005 13:10	5.694321	0.022483	0.004898	5.694226	0.030792	0.006708	5.694001	0.056696	0.012352	0.007986	0.003888
10/25/2005 13:10	5.694334	0.022483	0.004898	5.694238	0.030792	0.006708	5.694013	0.056207	0.012246	0.007951	0.003828
10/25/2005 13:10	5.694346	0.022483	0.004898	5.694249	0.031281	0.006815	5.694024	0.056696	0.012352	0.008022	0.003871
10/25/2005 13:10	5.694358	0.022483	0.004898	5.694261	0.030792	0.006708	5.694036	0.056696	0.012352	0.007986	0.003888
10/25/2005 13:10	5.694370	0.022483	0.004898	5.694272	0.030792	0.006708	5.694059	0.056696	0.012352	0.007986	0.003888
10/25/2005 13:10	5.694382	0.022972	0.005005	5.694284	0.030792	0.006708	5.694059	0.056696	0.012352	0.008022	0.003846
10/25/2005 13:10	5.694395	0.022972	0.005005	5.694296	0.030792	0.006708	5.694082	0.056696	0.012352	0.008022	0.003846
10/25/2005 13:10	5.694407	0.022483	0.004898	5.694307	0.031281	0.006815	5.694094	0.056696	0.012352	0.008022	0.003871
10/25/2005 13:10	5.694419	0.022972	0.005005	5.694319	0.030792	0.006708	5.694105	0.056207	0.012246	0.007986	0.003786
10/25/2005 13:10	5.694431	0.022972	0.005005	5.694330	0.031281	0.006815	5.694117	0.056696	0.012352	0.008057	0.003828
10/25/2005 13:10	5.694443	0.022483	0.004898	5.694342	0.030792	0.006708	5.694157	0.056207	0.012246	0.007951	0.003828
10/25/2005 13:10	5.694455	0.022483	0.004898	5.694353	0.031281	0.006815	5.694169	0.056207	0.012246	0.007986	0.003811
10/25/2005 13:10	5.694468	0.022483	0.004898	5.694365	0.030792	0.006708	5.694309	0.056207	0.012246	0.007951	0.003828
10/25/2005 13:10	5.694479	0.022483	0.004898	5.694388	0.030792	0.006708	5.694321	0.056207	0.012246	0.007951	0.003828
10/25/2005 13:10	5.694492	0.022972	0.005005	5.694400	0.030792	0.006708	5.694370	0.056207	0.012246	0.007986	0.003786
10/25/2005 13:10	5.694504	0.022483	0.004898	5.694419	0.030792	0.006708	5.694382	0.056207	0.012246	0.007951	0.003828
10/25/2005 13:10	5.694516	0.022972	0.005005	5.694431	0.030792	0.006708	5.694395	0.056207	0.012246	0.007986	0.003786
10/25/2005 13:10	5.694527	0.022972	0.005005	5.694443	0.030792	0.006708	5.694504	0.056207	0.012246	0.007986	0.003786
10/25/2005 13:10	5.694540	0.022972	0.005005	5.694455	0.030792	0.006708	5.694516	0.056207	0.012246	0.007986	0.003786
10/25/2005 13:10	5.694551	0.022483	0.004898	5.694468	0.030792	0.006708	5.694527	0.056207	0.012246	0.007951	0.003828
10/25/2005 13:10	5.694564	0.022483	0.004898	5.694479	0.030792	0.006708	5.694540	0.056207	0.012246	0.007951	0.003828
10/25/2005 13:13	5.695865	0.022483	0.004898	5.695685	0.030792	0.006708	5.695402	0.056207	0.012246	0.007951	0.003828
10/25/2005 13:17	5.698665	0.022483	0.004898	5.698485	0.030792	0.006708	5.698202	0.056207	0.012246	0.007951	0.003828
10/25/2005 13:25	5.704267	0.022483	0.004898	5.704087	0.030792	0.006708	5.703804	0.056207	0.012246	0.007951	0.003828
10/25/2005 13:41	5.715471	0.021994	0.004792	5.715291	0.030792	0.006708	5.715008	0.055718	0.012139	0.007880	0.003811
10/25/2005 14:13	5.737879	0.021505	0.004685	5.737698	0.030792	0.006708	5.737415	0.055718	0.012139	0.007844	0.003855

**DATA SHEET FOR CREEP TESTING**

Description of Soil: EA at 7% MC

Sample No.: EA-D6

Stress Level: 6 psi

Date Mixed: 10/24/2005

Number of Samples: 1

Date Compacted: 10/26/2005

Tested by: DD

Compaction: Modified/D/Mechanical  
Mold Size: 4.59 in

Date Start Creep: 10/26/2005

Date End Creep: 10/31/2005

Load Frame: D

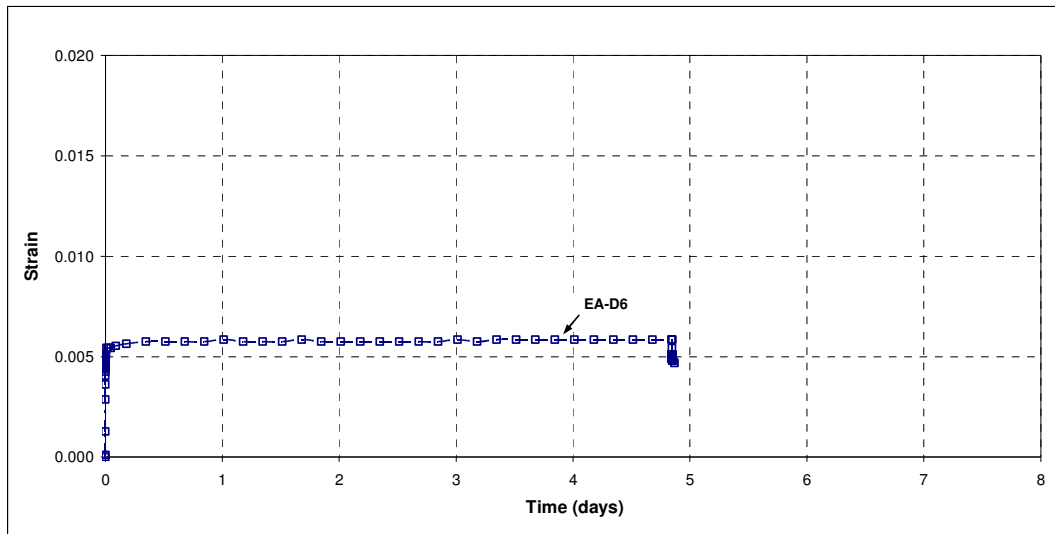
Mix Proportions			
EA	100%	12	lbs

Moisture Content		
Mass of water	0.84	lb

Proctor Max Dry Unit Weight		
At 6.1%	127.82	pcf

Compaction Moisture Content				
Can Number	B31		B32	
	Date	10/26/2005		10/26/2005
Mass of Can	14.76	g	14.81	g
Mass of Wet Soil & Can	297.27	g	289.90	g
Mass of Dry Soil & Can	277.69	g	271.84	g
Mass of Dry Soil	262.93	g	257.03	g
Mass of Water	19.58	g	18.06	g
w (%)	7.45%		7.03%	
Weighted Average w (%)	7.24%		St Dev = 0.30%	

Density Computations		
Sample 1		
Mass of Mold	14.22	lb
Mass of Mold and Wet Soil	24.14	lb
Vol of Mold	0.075	ft <sup>3</sup>
Mass of Wet Soil	9.920	lb
Unit Weight	132.2	lb/ft <sup>3</sup>
Dry Unit Weight	123.3	lb/ft <sup>3</sup>
Relative Compaction	96.5%	
Ave Relative Compaction	96.5%	
St Deviation		



**DATA SHEET FOR CREEP TESTING**

**Description of Soil:** 100% A-3 Soil at 9% MC

**Sample No.:** A3-E6;A3-F6

**Stress Level:** 6 psi

**Date Mixed:** 10/24/2005

**Number of Samples:** 2

**Date Compacted:** 10/26/2005

**Tested by:** DD

**Compaction:** Modified/D/Mechanical

**Mold Size:** 4.59 in

Mix Proportions			
RAP	0%	0	lbs
A-3 Sand	100%	24	lbs

**Date Start Creep:** 10/26/2005

**Date End Creep:** 10/31/2005

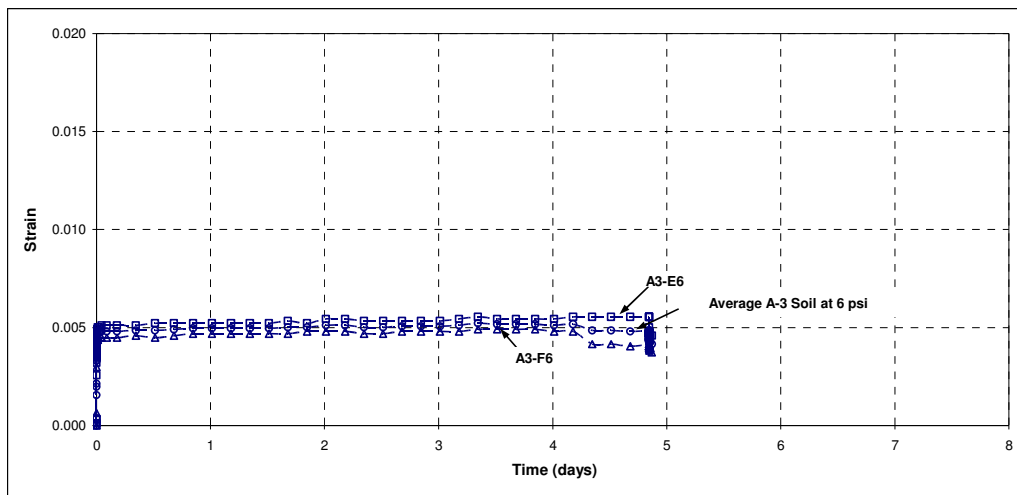
**Load Frame:** E, F

Moisture Content		
Mass of water	2.16	lb

Proctor Max Dry Unit Weight		
At 8.9%	96.33	pcf

Compaction Moisture Content				
Can Number	B7	B51	B52	
Date	10/26/2005	10/26/2005	10/26/2005	
Mass of Can	43.13 g	14.57 g	14.52 g	
Mass of Wet Soil & Can	283.71 g	321.59 g	314.21 g	
Mass of Dry Soil & Can	264.17 g	296.50 g	289.86 g	
Mass of Dry Soil	221.04 g	281.93 g	275.34 g	
Mass of Water	19.54 g	25.09 g	24.35 g	
w (%)	8.84%	8.90%	8.84%	
<b>Weighted Average w (%)</b>	<b>8.86%</b>	<b>St Dev =</b>	<b>0.03%</b>	

Density Computations				
	Sample1		Sample2	
Mass of Mold	14.41 lb	14.56 lb		
Mass of Mold and Wet Soil	22.31 lb	22.39 lb		
Vol of Mold	0.075 ft <sup>3</sup>	0.075 ft <sup>3</sup>		
Mass of Wet Soil	7.900 lb	7.830 lb		
Unit Weight	105.3 lb/ft <sup>3</sup>	104.4 lb/ft <sup>3</sup>		
<b>Dry Unit Weight</b>	<b>96.7 lb/ft<sup>3</sup></b>	<b>95.9 lb/ft<sup>3</sup></b>		
<b>Relative Compaction</b>	<b>100.4%</b>	<b>99.5%</b>		
<b>Ave Relative Compaction</b>	<b>100.0%</b>			
<b>St Deviation</b>	<b>0.63%</b>			













**DATA SHEET FOR CREEP TESTING**

**Description of Soil:** EA at 7% MC  
**Stress Level:** 12 psi  
**Number of Samples:** 1  
**Tested by:** DD

**Sample No.:** EA-D12  
**Date Mixed:** 10/30/2005  
**Date Compacted:** 11/1/2005  
**Compaction:** Modified/D/Mechanical  
**Mold Size:** 4.59 in  
**Date Start Creep:** 11/1/2005  
**Date End Creep:** 11/6/2005  
**Load Frame:** D

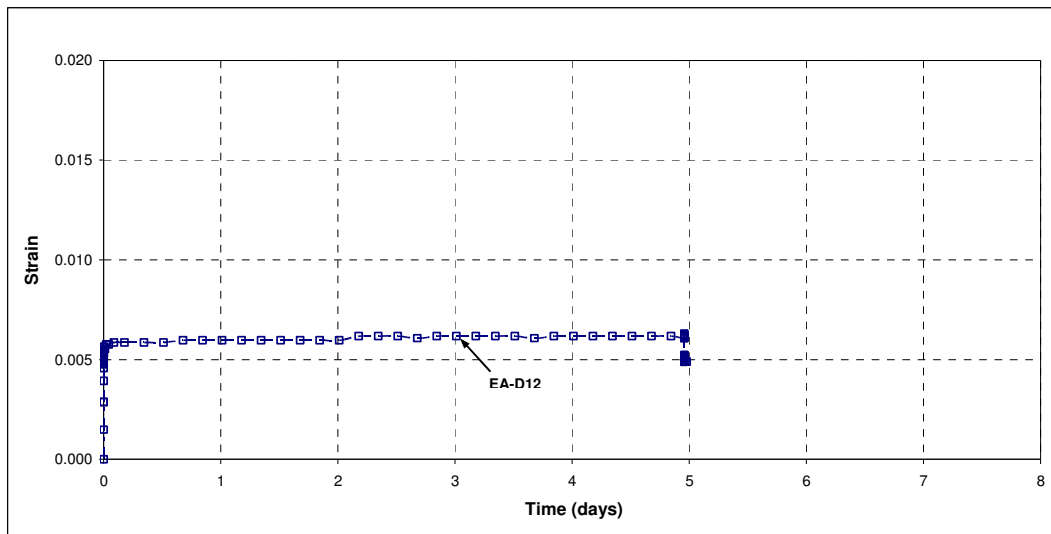
Mix Proportions			
EA	100%	12	lbs

Moisture Content		
Mass of water	0.84	lb

Proctor Max Dry Unit Weight		
At 6.1%	127.82	pcf

Compaction Moisture Content				
Can Number	B31		B32	
	11/1/2005		11/1/2005	
Mass of Can	14.73	g	14.80	g
Mass of Wet Soil & Can	302.76	g	343.47	g
Mass of Dry Soil & Can	283.32	g	321.29	g
Mass of Dry Soil	268.59	g	306.49	g
Mass of Water	19.44	g	22.18	g
w (%)	7.24%		7.24%	
<b>Weighted Average w (%)</b>	<b>7.24%</b>		<b>St Dev = 0.00%</b>	

Density Computations		
Sample 1		
Mass of Mold	14.42	lb
Mass of Mold and Wet Soil	24.50	lb
Vol of Mold	0.075	ft <sup>3</sup>
Mass of Wet Soil	10.080	lb
Unit Weight	134.4	lb/ft <sup>3</sup>
Dry Unit Weight	125.3	lb/ft <sup>3</sup>
Relative Compaction	98.0%	
Ave Relative Compaction	98.0%	
St Deviation		



**DATA SHEET FOR CREEP TESTING**

**Description of Soil:** 100% A-3 Soil at 9% MC

**Sample No.:** A3-E12;A3-F12

**Stress Level:** 12 psi

**Date Mixed:** 10/30/2005

**Number of Samples:** 2

**Date Compacted:** 11/1/2005

**Tested by:** DD

**Compaction:** Modified/D/Mechanical

**Mold Size:** 4.59 in

Mix Proportions			
RAP	0%	0	lbs
A-3 Sand	100%	24	lbs

**Date Start Creep:** 11/1/2005

**Date End Creep:** 11/6/2005

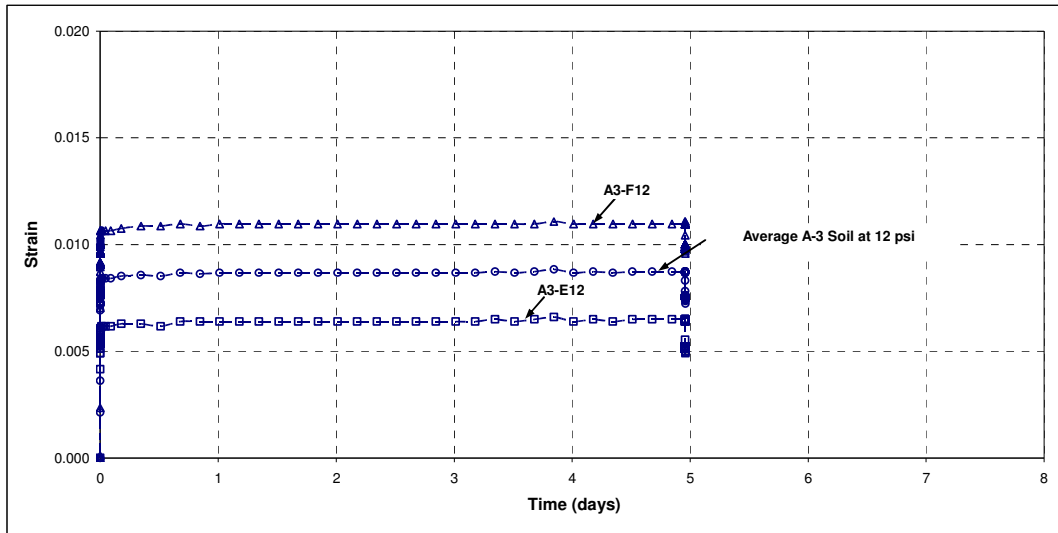
**Load Frame:** E, F

Moisture Content		
Mass of water	2.16	lb

Proctor Max Dry Unit Weight		
At 8.9%	96.33	pcf

Compaction Moisture Content						
Can Number Date	B7		B51		B52	
	11/1/2005		11/1/2005		11/1/2005	
Mass of Can	43.15	g	14.55	g	14.49	g
Mass of Wet Soil & Can	309.63	g	283.24	g	315.16	g
Mass of Dry Soil & Can	287.94	g	261.45	g	290.52	g
Mass of Dry Soil	244.79	g	246.90	g	276.03	g
Mass of Water	21.69	g	21.79	g	24.64	g
w (%)	8.86%		8.83%		8.93%	
<b>Weighted Average w (%)</b>	<b>8.87%</b>		<b>St Dev =</b>	<b>0.05%</b>		

Density Computations				
	Sample1		Sample2	
	Mass of Mold	14.21	lb	14.56
Mass of Mold and Wet Soil	22.17	lb	22.47	lb
Vol of Mold	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>
Mass of Wet Soil	7.960	lb	7.910	lb
Unit Weight	106.1	lb/ft <sup>3</sup>	105.4	lb/ft <sup>3</sup>
<b>Dry Unit Weight</b>	<b>97.5</b>	<b>lb/ft<sup>3</sup></b>	<b>96.8</b>	<b>lb/ft<sup>3</sup></b>
<b>Relative Compaction</b>	<b>101.2%</b>		<b>100.5%</b>	
<b>Ave Relative Compaction</b>	<b>100.9%</b>			
<b>St Deviation</b>	<b>0.45%</b>			









Date_Time mm/dd/yr_h:m	Load Frame D EA			Load Frame E A-3 Soil			Load Frame F A-3 Soil			Ave Strain A-3	
	Time (days)	Deflection (in)	Strain	Time (days)	Deflection (in)	Strain	Time (days)	Deflection (in)	Strain	Ave Strain	St Dev
11/6/2005 13:09	4.959241	0.022972	0.005005	4.959359	0.023460	0.005111	4.959356	0.044966	0.009797	0.007454	0.003313
11/6/2005 13:09	4.959253	0.022483	0.004898	4.959371	0.023460	0.005111	4.959367	0.044966	0.009797	0.007454	0.003313
11/6/2005 13:09	4.959265	0.022972	0.005005	4.959383	0.022972	0.005005	4.959380	0.044966	0.009797	0.007401	0.003388
11/6/2005 13:09	4.959277	0.022483	0.004898	4.959395	0.023460	0.005111	4.959392	0.045455	0.009903	0.007507	0.003388
11/6/2005 13:09	4.959287	0.022483	0.004898	4.959406	0.023460	0.005111	4.959404	0.044966	0.009797	0.007454	0.003313
11/6/2005 13:09	4.959299	0.022483	0.004898	4.959418	0.023460	0.005111	4.959417	0.044966	0.009797	0.007454	0.003313
11/6/2005 13:09	4.959312	0.022483	0.004898	4.959430	0.023460	0.005111	4.959428	0.044966	0.009797	0.007454	0.003313
11/6/2005 13:09	4.959324	0.022483	0.004898	4.959442	0.023460	0.005111	4.959440	0.044966	0.009797	0.007454	0.003313
11/6/2005 13:09	4.959335	0.022483	0.004898	4.959455	0.023460	0.005111	4.959451	0.044966	0.009797	0.007454	0.003313
11/6/2005 13:09	4.959348	0.022483	0.004898	4.959467	0.022972	0.005005	4.959463	0.044966	0.009797	0.007401	0.003388
11/6/2005 13:09	4.959360	0.022483	0.004898	4.959479	0.023460	0.005111	4.959475	0.045455	0.009903	0.007507	0.003388
11/6/2005 13:09	4.959373	0.022483	0.004898	4.959490	0.023460	0.005111	4.959487	0.044966	0.009797	0.007454	0.003313
11/6/2005 13:09	4.959385	0.022483	0.004898	4.959502	0.023460	0.005111	4.959500	0.044966	0.009797	0.007454	0.003313
11/6/2005 13:09	4.959397	0.022972	0.005005	4.959514	0.023460	0.005111	4.959512	0.044966	0.009797	0.007454	0.003313
11/6/2005 13:09	4.959409	0.022483	0.004898	4.959526	0.023460	0.005111	4.959526	0.044966	0.009797	0.007454	0.003313
11/6/2005 13:09	4.959421	0.022483	0.004898	4.959538	0.023460	0.005111	4.959538	0.044966	0.009797	0.007454	0.003313
11/6/2005 13:10	4.959434	0.022483	0.004898	4.959550	0.023460	0.005111	4.959550	0.044966	0.009797	0.007454	0.003313
11/6/2005 13:10	4.959446	0.022483	0.004898	4.959562	0.023460	0.005111	4.959562	0.044966	0.009797	0.007454	0.003313
11/6/2005 13:10	4.959458	0.022483	0.004898	4.959575	0.023460	0.005111	4.959575	0.044966	0.009797	0.007454	0.003313
11/6/2005 13:10	4.959470	0.022483	0.004898	4.960573	0.023460	0.005111	4.960510	0.044966	0.009797	0.007454	0.003313
11/6/2005 13:10	4.959481	0.022483	0.004898	4.963374	0.022972	0.005005	4.963311	0.044966	0.009797	0.007401	0.003388
11/6/2005 13:10	4.959493	0.022483	0.004898	4.968976	0.022972	0.005005	4.968913	0.044477	0.009690	0.007347	0.003313
11/6/2005 13:10	4.959505	0.022483	0.004898	4.980179	0.022483	0.004898	4.980117	0.043988	0.009583	0.007241	0.003313
11/6/2005 13:10	4.959517	0.022483	0.004898								
11/6/2005 13:10	4.959529	0.022483	0.004898								
11/6/2005 13:10	4.959542	0.022483	0.004898								
11/6/2005 13:10	4.959554	0.022483	0.004898								
11/6/2005 13:10	4.959565	0.022483	0.004898								
11/6/2005 13:10	4.959577	0.022483	0.004898								
11/6/2005 13:10	4.959588	0.022483	0.004898								
11/6/2005 13:10	4.959601	0.022483	0.004898								
11/6/2005 13:10	4.959613	0.022483	0.004898								
11/6/2005 13:10	4.959625	0.022483	0.004898								
11/6/2005 13:10	4.959637	0.022483	0.004898								
11/6/2005 13:10	4.959650	0.022483	0.004898								
11/6/2005 13:10	4.960648	0.022483	0.004898								
11/6/2005 13:10	4.963449	0.022483	0.004898								
11/6/2005 13:24	4.969051	0.022483	0.004898								
11/6/2005 13:40	4.980254	0.022483	0.004898								

**DATA SHEET FOR CREEP TESTING**

**Description of Soil:** EA at 7% MC  
**Stress Level:** 18 psi  
**Number of Samples:** 1  
**Tested by:** DD

**Sample No.:** EA-D18  
**Date Mixed:** 10/17/2005  
**Date Compacted:** 10/19/2005  
**Compaction:** Modified/D/Mechanical  
**Mold Size:** 4.59 in  
**Date Start Creep:** 10/19/2005  
**Date End Creep:** 10/26/2005  
**Load Frame:** D

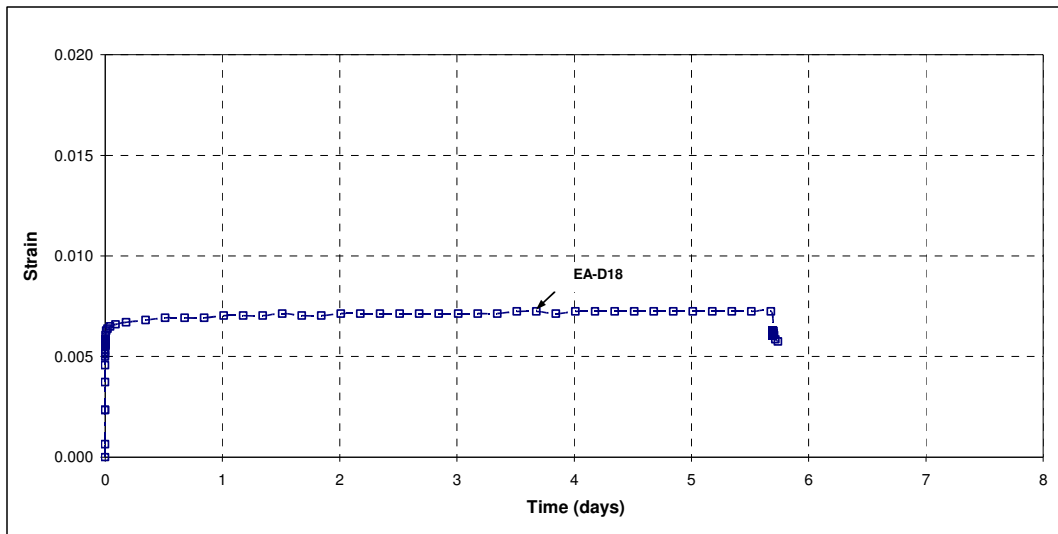
Mix Proportions			
EA	100%	12	lbs

Moisture Content		
Mass of water	0.84	lb

Proctor Max Dry Unit Weight		
At 6.1%	127.82	pcf

Compaction Moisture Content				
Can Number	B31		B32	
	Date	10/19/2005		10/19/2005
Mass of Can	14.88	g	14.82	g
Mass of Wet Soil & Can	320.14	g	283.47	g
Mass of Dry Soil & Can	300.36	g	265.69	g
Mass of Dry Soil	285.48	g	250.87	g
Mass of Water	19.78	g	17.78	g
w (%)	6.93%		7.09%	
<b>Weighted Average w (%)</b>	<b>7.01%</b>		<b>St Dev = 0.11%</b>	

Density Computations		
Sample 1		
Mass of Mold	14.42	lb
Mass of Mold and Wet Soil	24.27	lb
Vol of Mold	0.075	ft <sup>3</sup>
Mass of Wet Soil	9.850	lb
Unit Weight	131.3	lb/ft <sup>3</sup>
<b>Dry Unit Weight</b>	<b>122.7</b>	<b>lb/ft<sup>3</sup></b>
<b>Relative Compaction</b>	<b>96.0%</b>	
<b>Ave Relative Compaction</b>	<b>96.0%</b>	
<b>St Deviation</b>		





**DATA SHEET FOR CREEP TESTING**

Description of Soil: 100% A-3 Soil at 9% MC

Sample No.: A3-E18;A3-F18

Stress Level: 18 psi

Date Mixed: 10/17/2005

Number of Samples: 2

Date Compacted: 10/19/2005

Tested by: DD

Compaction: Modified/D/Mechanical

Mold Size: 4.59 in

Mix Proportions			
RAP	0%	0	lbs
A-3 Sand	100%	24	lbs

Date Start Creep: 10/19/2005

Date End Creep: 10/26/2005

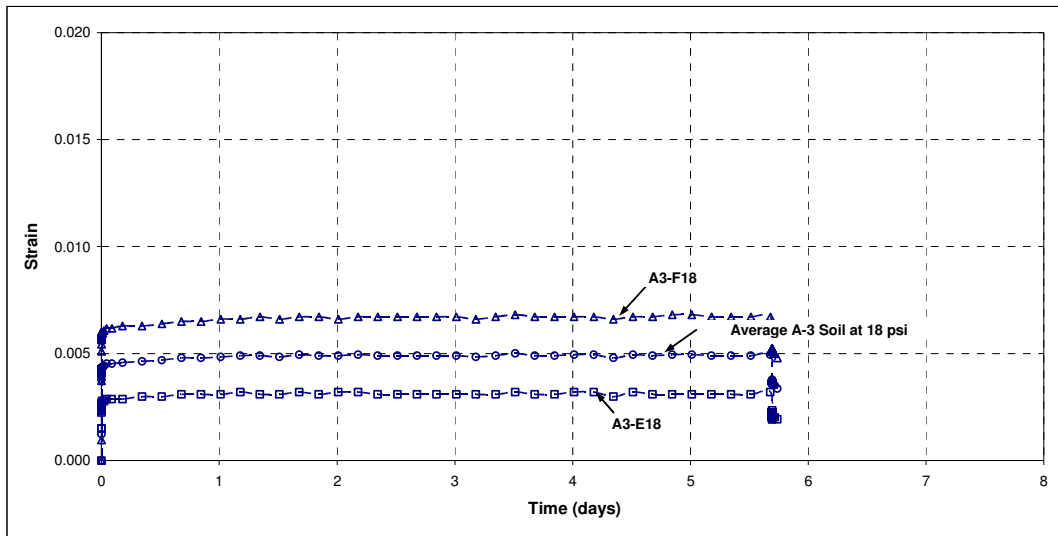
Load Frame: E, F

Moisture Content		
Mass of water	2.16	lb

Proctor Max Dry Unit Weight		
At 8.9%	96.33	pcf

Compaction Moisture Content								
Can Number Date	B6		B7		B8		B10	
	10/19/2005		10/19/2005		10/19/2005		10/19/2005	
Mass of Can	42.47	g	43.15	g	56.61	g	60.11	g
Mass of Wet Soil & Can	265.69	g	284.45	g	297.94	g	326.96	g
Mass of Dry Soil & Can	247.42	g	264.35	g	278.32	g	305.84	g
Mass of Dry Soil	204.95	g	221.20	g	221.71	g	245.73	g
Mass of Water	18.27	g	20.10	g	19.62	g	21.12	g
w (%)	8.91%		9.09%		8.85%		8.59%	
Weighted Average w (%)	8.86%		St Dev = 0.20%					

Density Computations				
	Sample 1		Sample 2	
	Mass of Mold	14.22	lb	14.56
Mass of Mold and Wet Soil	22.01	lb	22.27	lb
Vol of Mold	0.075	ft <sup>3</sup>	0.075	ft <sup>3</sup>
Mass of Wet Soil	7.790	lb	7.710	lb
Unit Weight	103.8	lb/ft <sup>3</sup>	102.8	lb/ft <sup>3</sup>
Dry Unit Weight	95.4	lb/ft <sup>3</sup>	94.4	lb/ft <sup>3</sup>
Relative Compaction	99.0%		98.0%	
Ave Relative Compaction	98.5%			
St Deviation	0.72%			









### **Appendix A-3: Summary of Creep Data**

**Table A-3.1 Summary of Creep Strains for RAP, RAP-soil mixtures, A-3 soil,  
and EA, loaded at stress levels of 6, 12, and 18 psi**

	6 psi			12 psi			18 psi		
	RC	Strain	Deviation	RC	Strain	Deviation	RC	Strain	Deviation
100-A	97.1%	0.0326	0.0035	98.6%	0.0349	0.0036	97.5%	0.0518	0.0065
100-B	97.9%	0.0247	-0.0044	98.9%	0.0278	-0.0035	98.9%	0.0399	-0.0053
100-C	97.2%	0.0299	0.0009	98.8%	0.0313	0.0000	98.4%	0.0441	-0.0012
<b>Ave 100</b>	<b>97%</b>	<b>0.0291</b>		<b>99%</b>	<b>0.0313</b>		<b>98%</b>	<b>0.0453</b>	
<b>St Dev 100</b>	<b>0.4%</b>	<b>0.0040</b>		<b>0.2%</b>	<b>0.0036</b>		<b>0.7%</b>	<b>0.0060</b>	
<b>CV</b>		<b>14%</b>			<b>11%</b>			<b>13%</b>	
80-D	96.9%	0.0150	-0.0030	97.6%	0.0216	0.0025	98.1%	0.0254	-0.0002
80-E	96.9%	0.0185	0.0005	97.9%	0.0184	-0.0007	98.2%	0.0256	-0.0001
80-F	96.7%	0.0206	0.0025	98.3%	0.0174	-0.0018	97.3%	0.0261	0.0004
<b>Ave 80</b>	<b>97%</b>	<b>0.0180</b>		<b>98%</b>	<b>0.0191</b>		<b>98%</b>	<b>0.0257</b>	
<b>St Dev 80</b>	<b>0.1%</b>	<b>0.0028</b>		<b>0.4%</b>	<b>0.0022</b>		<b>0.5%</b>	<b>0.0003</b>	
<b>CV</b>		<b>16%</b>			<b>12%</b>			<b>1%</b>	
60-A	97.6%	0.0084	0.0002	98.0%	0.0094	-0.0025	97.1%	0.0159	0.0002
60-B	98.9%	0.0080	-0.0002	97.8%	0.0104	-0.0015	97.0%	0.0165	0.0008
60-C	98.8%	0.0082	0.0000	98.6%	0.0159	0.0040	98.2%	0.0147	-0.0010
<b>Ave 60</b>	<b>98%</b>	<b>0.0082</b>		<b>98%</b>	<b>0.0119</b>		<b>97%</b>	<b>0.0157</b>	
<b>St Dev 60</b>	<b>0.7%</b>	<b>0.0002</b>		<b>0.4%</b>	<b>0.0035</b>		<b>0.7%</b>	<b>0.0009</b>	
<b>CV</b>		<b>3%</b>			<b>29%</b>			<b>6%</b>	
40-D	99.3%	0.0059	0.0004	98.5%	0.0085	-0.0005	99.4%	0.0139	0.0035
40-E	98.4%	0.0044	-0.0011	99.1%	0.0088	-0.0001	98.9%	0.0126	0.0021
40-F	97.5%	0.0062	0.0007	98.7%	0.0096	0.0006	98.2%	0.0048	-0.0056
<b>Ave 40</b>	<b>98%</b>	<b>0.0055</b>		<b>99%</b>	<b>0.0090</b>		<b>99%</b>	<b>0.0104</b>	
<b>St Dev 40</b>	<b>0.9%</b>	<b>0.0010</b>		<b>0.3%</b>	<b>0.0005</b>		<b>0.6%</b>	<b>0.0049</b>	
<b>CV</b>		<b>18%</b>			<b>6%</b>			<b>47%</b>	
20-A	99.0%	0.0061	-0.0020	98.4%	0.0067	-0.0019	97.3%	0.0061	-0.0032
20-B	97.9%	0.0059	-0.0022	98.1%	0.0085	-0.0001	97.1%	0.0079	-0.0013
20-C	98.7%	0.0124	0.0043	98.5%	0.0106	0.0020	96.8%	0.0137	0.0045
<b>Ave 20</b>	<b>99%</b>	<b>0.0081</b>		<b>98%</b>	<b>0.0086</b>		<b>97%</b>	<b>0.0092</b>	
<b>St Dev 20</b>	<b>0.6%</b>	<b>0.0037</b>		<b>0.2%</b>	<b>0.0020</b>		<b>0.2%</b>	<b>0.0040</b>	
<b>CV</b>		<b>46%</b>			<b>23%</b>			<b>43%</b>	
A3-E	100.4%	0.0055	0.0003	101.2%	0.0066	-0.0022	99.0%	0.0032	-0.0018
A3-F	99.5%	0.0049	-0.0003	100.5%	0.0111	0.0022	98.0%	0.0068	0.0018
<b>Ave A3</b>	<b>100%</b>	<b>0.0052</b>		<b>101%</b>	<b>0.0088</b>		<b>99%</b>	<b>0.0050</b>	
<b>St Dev A3</b>	<b>0.6%</b>	<b>0.0005</b>		<b>0.4%</b>	<b>0.0032</b>		<b>0.7%</b>	<b>0.0026</b>	
<b>CV</b>		<b>9%</b>			<b>36%</b>			<b>51%</b>	
EA-D	96.5%	0.0059		98.0%	0.0063		96.0%	0.0072	

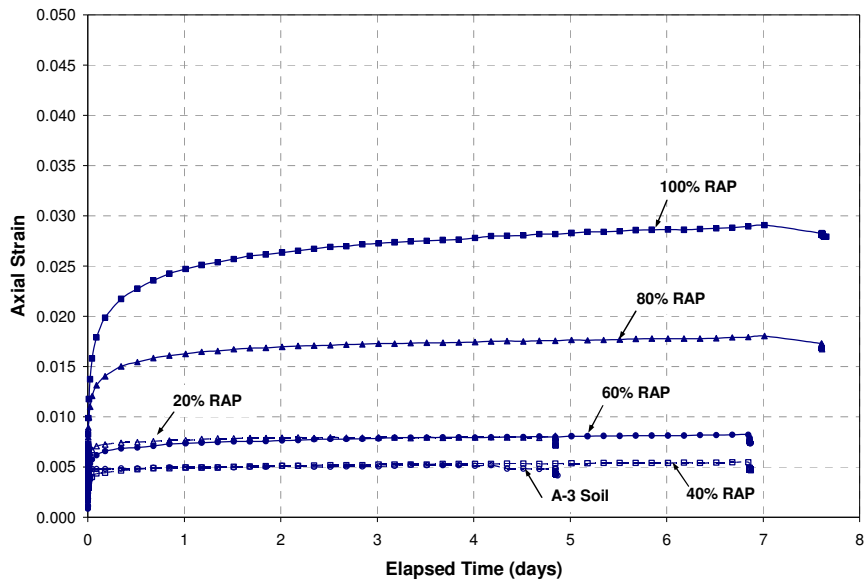
RC - Relative Compaction  
CV - Coefficient of Variation

**Table A-3.2 Summary of irrecoverable creep strains in RAP, RAP-soil mixtures, A-3 soil, and EA, after loading at stress levels of 6, 12, and 18 psi**

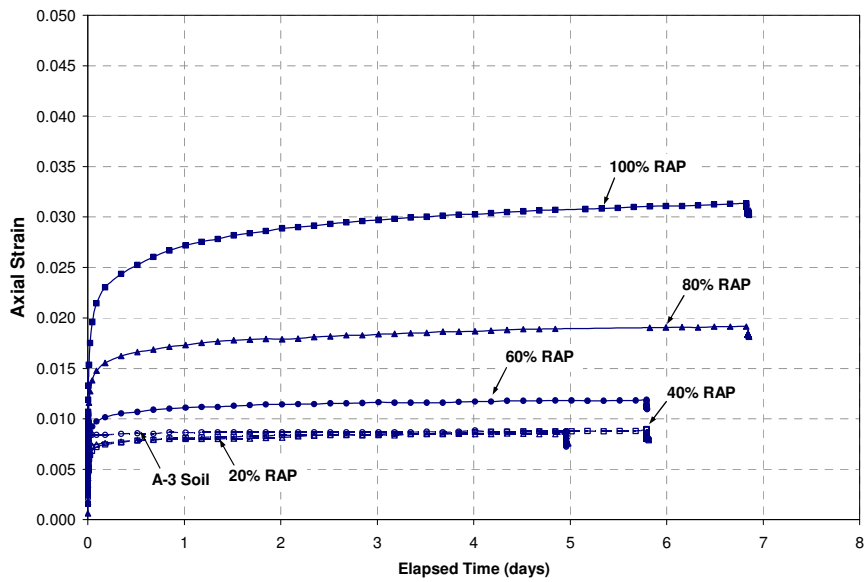
	Loading Strain		Unloading Strain		Residual Strain
	Mean	St Dev	Mean	St Dev	
<b>6 psi</b>					
100% RAP	0.0291	0.0040	0.0279	0.0040	96%
80% RAP	0.0180	0.0028	0.0167	0.0030	93%
60% RAP	0.0082	0.0002	0.0074	0.0004	90%
40% RAP	0.0055	0.0010	0.0047	0.0009	86%
20% RAP	0.0081	0.0037	0.0071	0.0036	88%
0% RAP	0.0052	0.0005	0.0042	0.0006	80%
EA	0.0059		0.0047		80%
<b>12 psi</b>					
100% RAP	0.0313	0.0036	0.0302	0.0036	96%
80% RAP	0.0191	0.0022	0.0181	0.0022	95%
60% RAP	0.0119	0.0035	0.0110	0.0034	92%
40% RAP	0.0090	0.0005	0.0079	0.0006	88%
20% RAP	0.0086	0.0020	0.0076	0.0018	88%
0% RAP	0.0088	0.0032	0.0072	0.0033	82%
EA	0.0063		0.0049		78%
<b>18 psi</b>					
100% RAP	0.0453	0.0060	0.0444	0.0058	98%
80% RAP	0.0257	0.0003	0.0245	0.0001	95%
60% RAP	0.0157	0.0009	0.0147	0.0008	93%
40% RAP	0.0104	0.0049	0.0089	0.0231	85%
20% RAP	0.0092	0.0040	0.0078	0.0039	85%
0% RAP	0.0050	0.0026	0.0034	0.0020	67%
EA	0.0072		0.0058		79%

## **Appendix A-4: Strain-Time Relationships from Creep Testing**

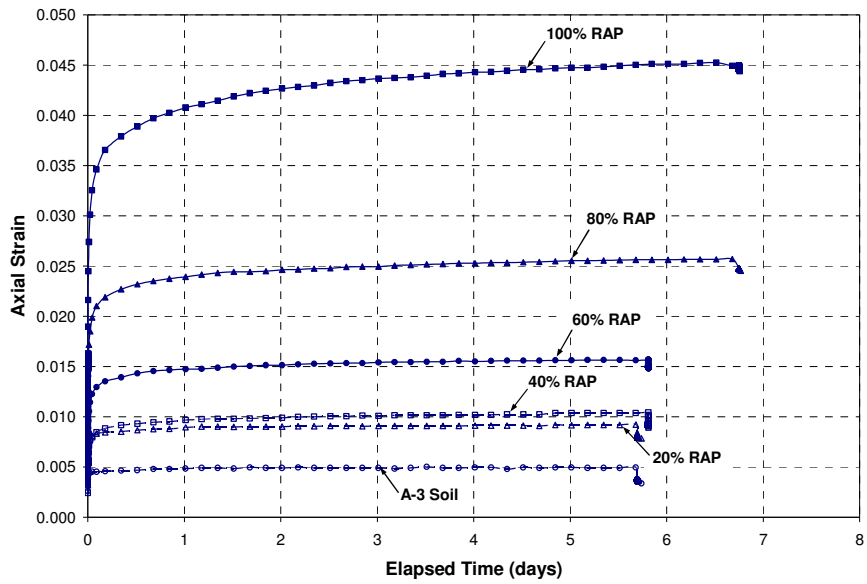




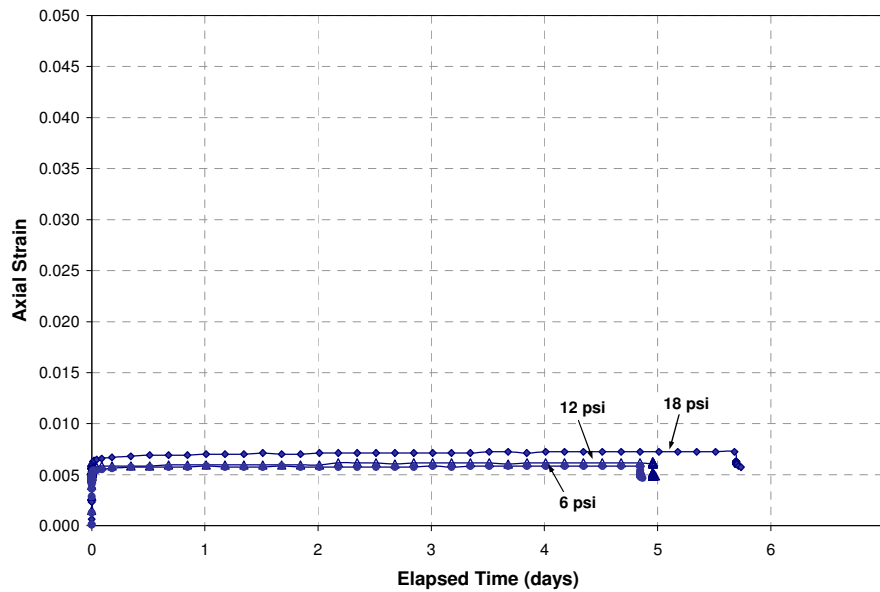
**Figure A-4.1 Axial Strain vs. time relationship during creep of RAP, RAP-soil mixtures, and A-3 soil at 6 psi stress level**



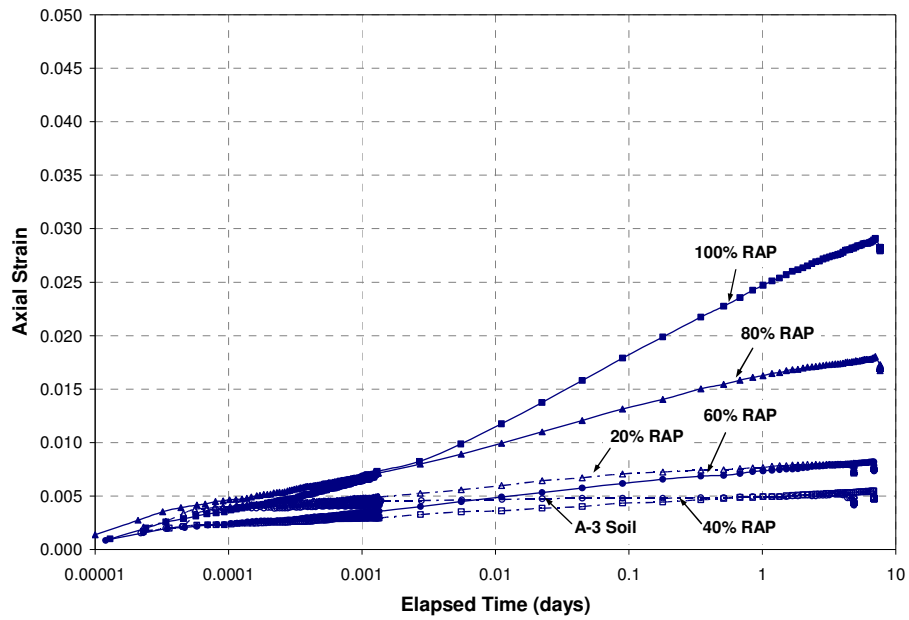
**Figure A-4.2 Axial Strain vs. time relationship during creep of RAP, RAP-soil mixtures, and A-3 soil at 12 psi stress level**



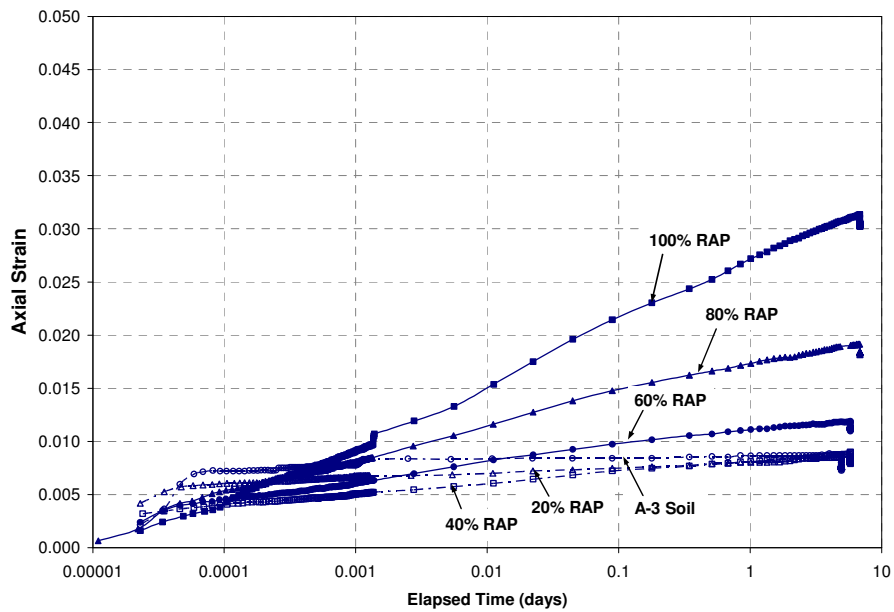
**Figure A-4.3 Axial Strain vs. time relationship during creep of RAP, RAP-soil mixtures, and A-3 soil at 18 psi stress level**



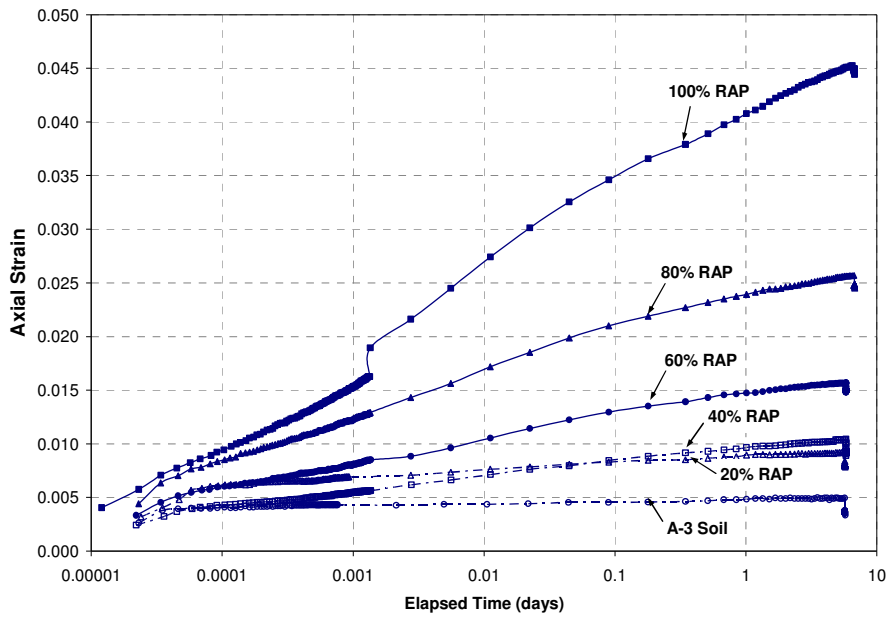
**Figure A-4.4 Axial strain vs. time relationship during creep of EA at stress levels of 6, 12, and 18 psi**



**Figure A-4.5 Axial strain vs. log time relationship during creep of RAP, RAP-soil mixtures, and A-3 soil at 6 psi stress level**



**Figure A-4.6 Axial strain vs. log time relationship during creep of RAP, RAP-soil mixtures, and A-3 soil at 12 psi stress level**



**Figure A-4.7 Axial strain vs. log time relationship during creep of RAP, RAP-soil mixtures, and A-3 soil at 18 psi stress level**

## **Appendix A-5: Strain Rate Calculations**

**DATA SHEET FOR STRAIN RATE CALCULATION - MITCHELL METHOD**

Soil: 100% RAP  
 Compaction: Modified Proctor

Wall Height (ft):	8	20	30
Stress Level (psi):	6	12	18

**Strain Rate**

Formula:  $\dot{\epsilon} = Ae^{\alpha D} \left(\frac{t_1}{t}\right)^m$

Unit time  $t_1$ : 1 day

**Figure D.1**

Equation:  $y = 0.0013 \cdot (\text{EXP}(0.0407 \cdot x))$   
 Coefficients:  
 Intercept: A = 0.0013 %/min  
 Slope:  $\alpha = 0.0407$  1/psi

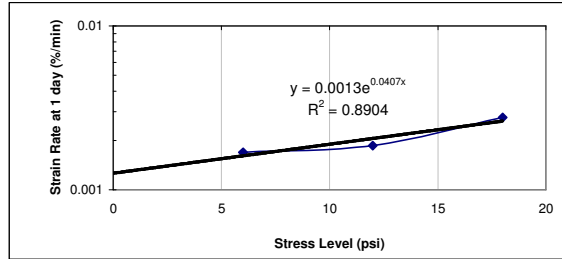


Figure 1 Logarithm of strain rate vs. stress level for 100% RAP

**Figure D.2**

Coefficient Average m = 0.9302

Stress Level D (psi)	Slope m
6	0.9209
12	0.9273
18	0.9423

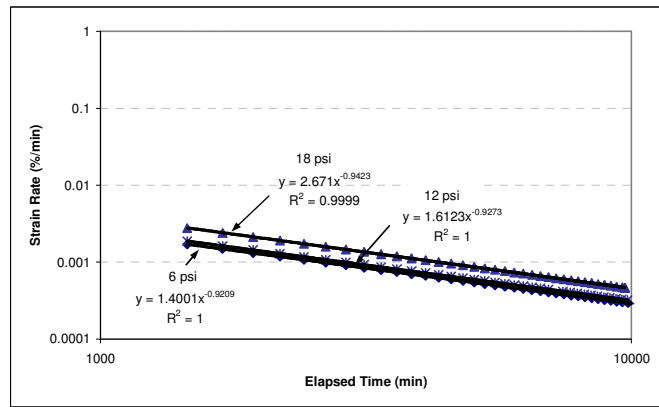


Figure 2 Logarithm of strain rate vs. logarithm of time for 100% RAP

**Settlement**

Experimental Data:

Stress Level D (psi)	1 Day		7 Day	
	Strain (in/in)	Strain Rate (%/min)	Strain (in/in)	Strain Rate (%/min)
6	0.02470	0.00169	0.02907	0.00029
12	0.02719	0.00186	0.03134	0.00032
18	0.04025	0.00276	0.04526	0.00046

Calculation Applying Formula:

Wall Height H (ft)	Stress Level D (psi)	1 Day			7 Day		
		Strain Rate (%/min)	Strain (in/in)	Settlement (in)	Strain Rate (%/min)	Strain (in/in)	Settlement (in)
8	6	0.00166	0.02390	2.3	0.00027	0.02738	2.6
20	12	0.00212	0.03051	7.3	0.00035	0.03495	8.4
30	18	0.00270	0.03895	14.0	0.00044	0.04462	16.1

Wall Height H (ft)	Stress Level D (psi)	1 Year			50 Year		
		Strain Rate (%/min)	Strain (in/in)	Settlement (in)	Strain Rate (%/min)	Strain (in/in)	Settlement (in)
8	6	6.86E-06	0.03608	3.5	1.80E-07	0.04742	4.6
20	12	8.76E-06	0.04606	11.1	2.30E-07	0.06053	14.5
30	18	1.12E-05	0.05880	21.2	2.94E-07	0.07728	27.8

**DATA SHEET FOR STRAIN RATE CALCULATION - MITCHELL METHOD**

Soil: 80% RAP  
 Compaction: Modified Proctor

Wall Height (ft):	8	20	30
Stress Level (psi):	6	12	18

**Strain Rate**

Formula:  $\dot{\epsilon} = Ae^{\alpha D} \left(\frac{t_1}{t}\right)^m$

Unit time  $t_1$ : 1 day

**Figure D.1**

Equation:  $y = 0.0009 \cdot (\text{EXP}(0.0322 \cdot x))$   
 Coefficients:  
 Intercept: A = 0.0009 %/min  
 Slope:  $\alpha = 0.0322$  1/psi

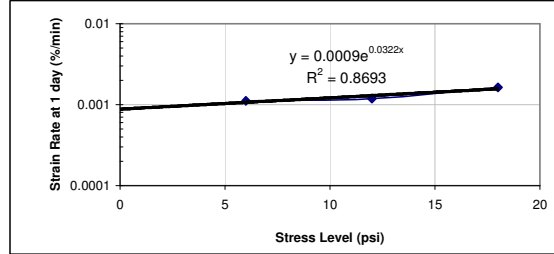


Figure 1 Logarithm of strain rate vs. stress level for 80% RAP

**Figure D.2**

Coefficient Average:  $m = 0.9544$

Stress level D (psi)	Slope m
6	0.9537
12	0.9472
18	0.9624

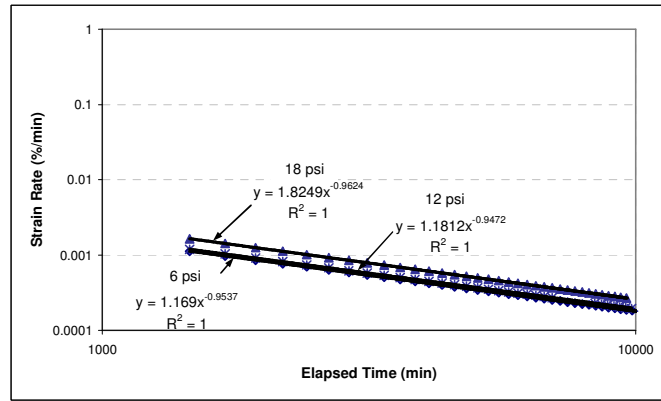


Figure 2 Logarithm of strain rate vs. logarithm of time for 80% RAP

**Settlement**

Experimental Data:

Stress Level D (psi)	1 Day		7 Day	
	Strain (in/in)	Strain Rate (%/min)	Strain (in/in)	Strain Rate (%/min)
6	0.01626	0.00111	0.01803	0.00018
12	0.01732	0.00119	0.01913	0.00019
18	0.02392	0.00164	0.02570	0.00027

Calculation Applying Formula:

Wall Height H (ft)	Stress Level D (psi)	1 Day			7 Day		
		Strain Rate (%/min)	Strain (in/in)	Settlement (in)	Strain Rate (%/min)	Strain (in/in)	Settlement (in)
8	6	0.00109	0.01572	1.5	0.00017	0.01718	1.6
20	12	0.00132	0.01907	4.6	0.00021	0.02084	5.0
30	18	0.00161	0.02314	8.3	0.00025	0.02528	9.1

Wall Height H (ft)	Stress Level D (psi)	1 Year			50 Year		
		Strain Rate (%/min)	Strain (in/in)	Settlement (in)	Strain Rate (%/min)	Strain (in/in)	Settlement (in)
8	6	3.91E-06	0.02057	2.0	9.36E-08	0.02459	2.4
20	12	4.75E-06	0.02496	6.0	1.13E-07	0.02983	7.2
30	18	5.76E-06	0.03027	10.9	1.38E-07	0.03618	13.0

**DATA SHEET FOR STRAIN RATE CALCULATION - MITCHELL METHOD**

Soil: 60% RAP  
 Compaction: Modified Proctor

Wall Height (ft):	8	20	30
Stress Level (psi):	6	12	18

**Strain Rate**

Formula:  $\dot{\epsilon} = Ae^{\alpha D} \left(\frac{t_1}{t}\right)^m$

Unit time  $t_1$ : 1 day

**Figure D.1**

Equation:  $y = 0.0004 * (\text{EXP}(0.058 * x))$   
 Coefficients:  
 Intercept: A = 0.0004 %/min  
 Slope:  $\alpha = 0.0580$  1/psi

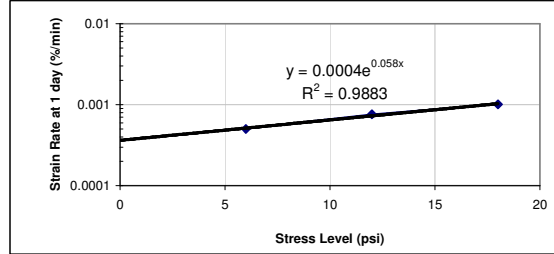


Figure 1 Logarithm of strain rate vs. stress level for 60% RAP

**Figure D.2**

Coefficient Average:  $m = 0.9570$

Stress level D (psi)	Slope m
6	0.9426
12	0.9632
18	0.9651

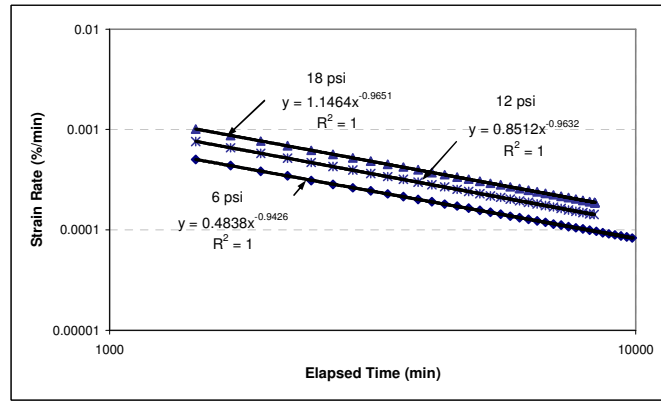


Figure 2 Logarithm of strain rate vs. logarithm of time for 60% RAP

**Settlement**

Experimental Data:

Stress Level D (psi)	1 Day		7 Day	
	Strain (in/in)	Strain Rate (%/min)	Strain (in/in)	Strain Rate (%/min)
6	0.00735	0.00050	0.00820	0.00008
12	0.01111	0.00076	0.01189	0.00014
18	0.01473	0.00101	0.01569	0.00019

Calculation Applying Formula:

Wall Height H (ft)	Stress Level D (psi)	1 Day			7 Day		
		Strain Rate (%/min)	Strain (in/in)	Settlement (in)	Strain Rate (%/min)	Strain (in/in)	Settlement (in)
8	6	0.00057	0.00816	0.8	0.00009	0.00887	0.9
20	12	0.00080	0.01155	2.8	0.00012	0.01256	3.0
30	18	0.00114	0.01636	5.9	0.00018	0.01779	6.4

Wall Height H (ft)	Stress Level D (psi)	1 Year			50 Year		
		Strain Rate (%/min)	Strain (in/in)	Settlement (in)	Strain Rate (%/min)	Strain (in/in)	Settlement (in)
8	6	2.00E-06	0.01052	1.0	4.73E-08	0.01244	1.2
20	12	2.83E-06	0.01489	3.6	6.71E-08	0.01762	4.2
30	18	4.01E-06	0.02109	7.6	9.50E-08	0.02496	9.0



**DATA SHEET FOR STRAIN RATE CALCULATION - MITCHELL METHOD**

Soil: 40% RAP  
 Compaction: Modified Proctor

Wall Height (ft):	8	20	30
Stress Level (psi):	6	12	18

**Strain Rate**

Formula:  $\dot{\epsilon} = Ae^{mD} \left(\frac{t_1}{t}\right)^m$

Unit time  $t_1$ : 1 day

**Figure D.1**

Equation:  $y = 0.0003 \cdot (\text{EXP}(0.0559 \cdot x))$   
 Coefficients:  
 Intercept: A = 0.0003 %/min  
 Slope: m = 0.0559 1/psi

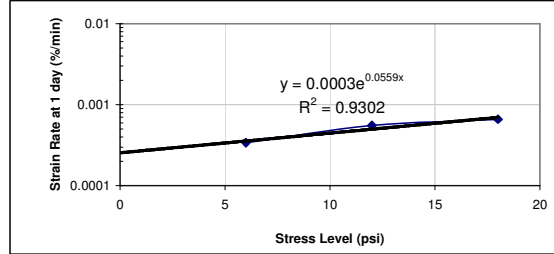


Figure 1 Logarithm of strain rate vs. stress level for 40% RAP

**Figure D.2**

Coefficient Average: m = 0.9493

Stress level D (psi)	Slope m
6	0.9450
12	0.9444
18	0.9585

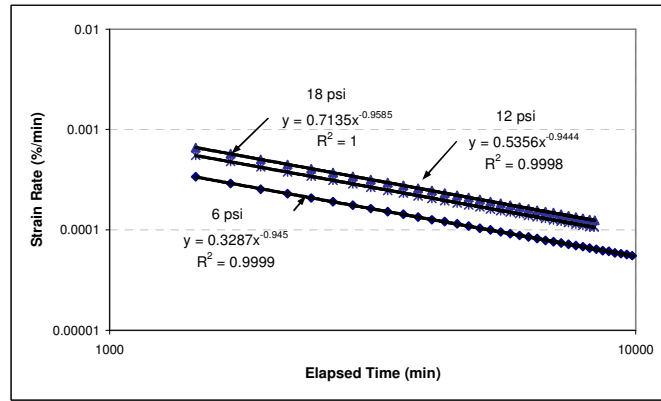


Figure 2 Logarithm of strain rate vs. logarithm of time for 40% RAP

**Settlement**

Experimental Data:

Stress Level D (psi)	1 Day		7 Day	
	Strain (in/in)	Strain Rate (%/min)	Strain (in/in)	Strain Rate (%/min)
6	0.00493	0.00034	0.00546	0.00006
12	0.00809	0.00056	0.00898	0.00011
18	0.00965	0.00066	0.01044	0.00012

Calculation Applying Formula:

Wall Height H (ft)	Stress Level D (psi)	1 Day			7 Day		
		Strain Rate (%/min)	Strain (in/in)	Settlement (in)	Strain Rate (%/min)	Strain (in/in)	Settlement (in)
8	6	0.00042	0.00604	0.6	0.00007	0.00667	0.6
20	12	0.00059	0.00845	2.0	0.00009	0.00933	2.2
30	18	0.00082	0.01182	4.3	0.00013	0.01304	4.7

Wall Height H (ft)	Stress Level D (psi)	1 Year			50 Year		
		Strain Rate (%/min)	Strain (in/in)	Settlement (in)	Strain Rate (%/min)	Strain (in/in)	Settlement (in)
8	6	1.55E-06	0.00815	0.8	3.78E-08	0.00994	1.0
20	12	2.17E-06	0.01140	2.7	5.29E-08	0.01389	3.3
30	18	3.03E-06	0.01594	5.7	7.39E-08	0.01943	7.0

**DATA SHEET FOR STRAIN RATE CALCULATION - MITCHELL METHOD**

Soil: 20% RAP  
 Compaction: Modified Proctor

Wall Height (ft):	8	20	30
Stress Level (psi):	6	12	18

**Strain Rate**

Formula:  $\dot{\epsilon} = Ae^{\alpha D} \left(\frac{t_1}{t}\right)^m$

Unit time  $t_1$ : 1 day

**Figure D.1**

Equation:  $y = 0.0005 * (\text{EXP}(0.0128 * x))$   
 Coefficients:  
 Intercept: A = 0.0005 %/min  
 Slope:  $\alpha = 0.0128$  1/psi

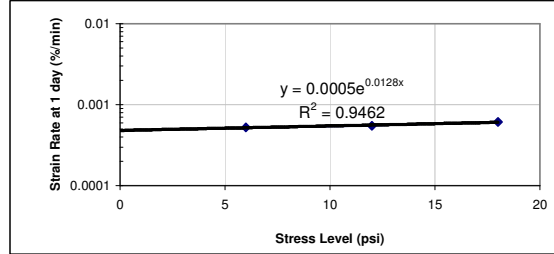


Figure 1 Logarithm of strain rate vs. stress level for 20% RAP

**Figure D.2**

Coefficient Average:  $m = 0.9737$

Stress level D (psi)	Slope m
6	0.9821
12	0.9534
18	0.9857

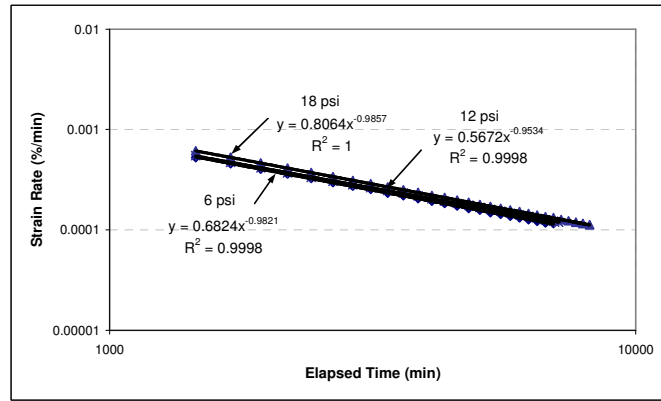


Figure 2 Logarithm of strain rate vs. logarithm of time for 20% RAP

**Settlement**

Experimental Data:

Stress Level D (psi)	1 Day		7 Day	
	Strain (in/in)	Strain Rate (%/min)	Strain (in/in)	Strain Rate (%/min)
6	0.00767	0.00053	0.00809	0.00012
12	0.00802	0.00055	0.00863	0.00012
18	0.00894	0.00061	0.00923	0.00011

Calculation Applying Formula:

Wall Height H (ft)	Stress Level D (psi)	1 Day			7 Day		
		Strain Rate (%/min)	Strain (in/in)	Settlement (in)	Strain Rate (%/min)	Strain (in/in)	Settlement (in)
8	6	0.00054	0.00777	0.7	0.00008	0.00818	0.8
20	12	0.00058	0.00840	2.0	0.00009	0.00884	2.1
30	18	0.00063	0.00907	3.3	0.00009	0.00954	3.4

Wall Height H (ft)	Stress Level D (psi)	1 Year			50 Year		
		Strain Rate (%/min)	Strain (in/in)	Settlement (in)	Strain Rate (%/min)	Strain (in/in)	Settlement (in)
8	6	1.73E-06	0.00908	0.9	3.83E-08	0.01006	1.0
20	12	1.87E-06	0.00980	2.4	4.13E-08	0.01086	2.6
30	18	2.01E-06	0.01059	3.8	4.46E-08	0.01173	4.2

**DATA SHEET FOR STRAIN RATE CALCULATION - MITCHELL METHOD**

Soil: *A-3 Soil*  
 Compaction: *Modified Proctor*

Wall Height (ft):	8	20	30
Stress Level (psi):	6	12	18

**Strain Rate**

Formula:  $\dot{\epsilon} = Ae^{\alpha D} \left(\frac{t_1}{t}\right)^m$

Unit time  $t_1$ : 1 day

**Figure D.1**

Equation:  $y = 0.0004 * (\text{EXP}(0.0018 * x))$   
 Coefficients:  
 Intercept:  $A = 0.0004$  %/min  
 Slope:  $\alpha = 0.0018$  1/psi

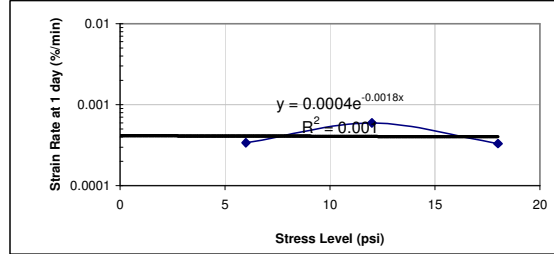


Figure 1 Logarithm of strain rate vs. stress level for A-3 Soil

**Figure D.2**

Coefficient Average  $m = 0.9859$

Stress level D (psi)	Slope m
6	0.9681
12	0.9946
18	0.9949

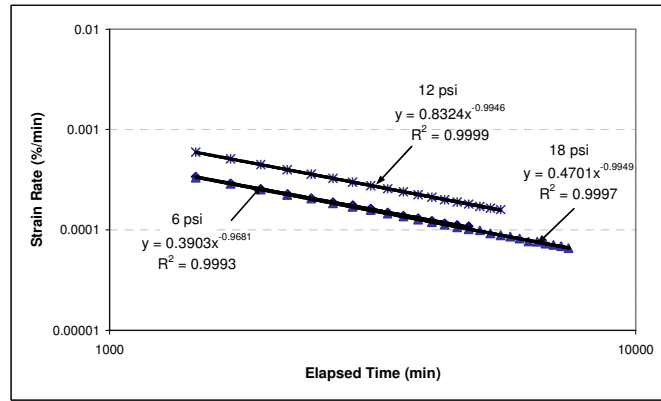


Figure 2 Logarithm of strain rate vs. logarithm of time for A-3 Soil

**Settlement**

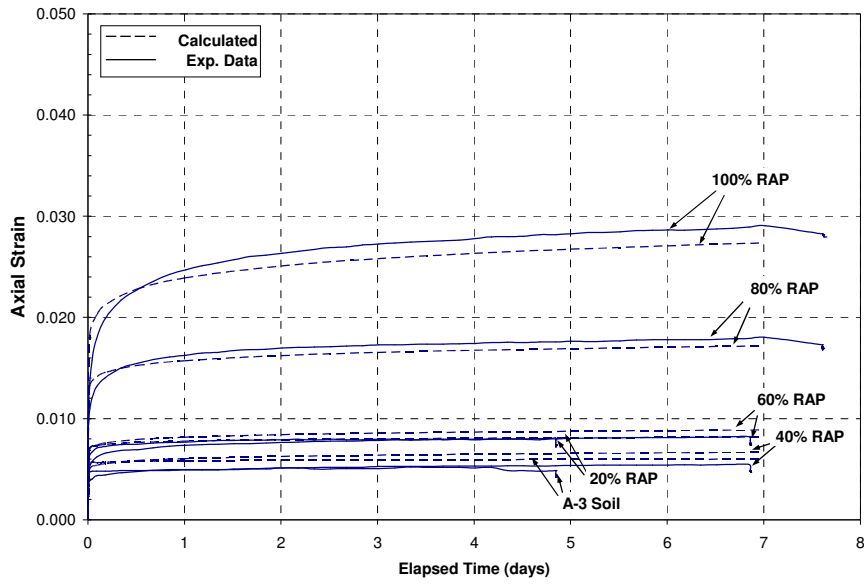
Experimental Data:

Stress Level D (psi)	1 Day		7 Day	
	Strain (in/in)	Strain Rate (%/min)	Strain (in/in)	Strain Rate (%/min)
6	0.00495	0.00034	0.00522	0.00011
12	0.00868	0.00060	0.00884	0.00012
18	0.00484	0.00033	0.00500	0.00007

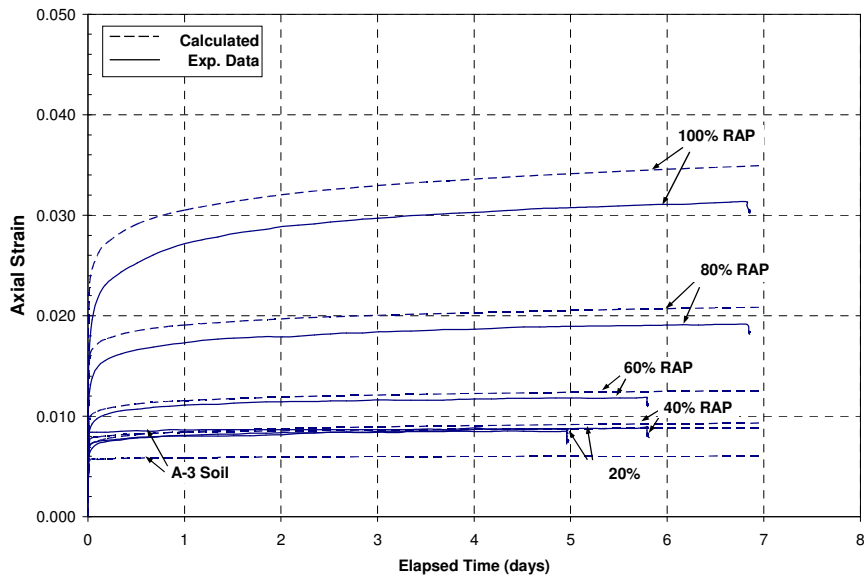
Calculation Applying Formula:

Wall Height H (ft)	Stress Level D (psi)	1 Day			7 Day		
		Strain Rate (%/min)	Strain (in/in)	Settlement (in)	Strain Rate (%/min)	Strain (in/in)	Settlement (in)
8	6	0.00040	0.00582	0.6	0.00006	0.00598	0.6
20	12	0.00041	0.00589	1.4	0.00006	0.00605	1.5
30	18	0.00041	0.00595	2.1	0.00006	0.00612	2.2

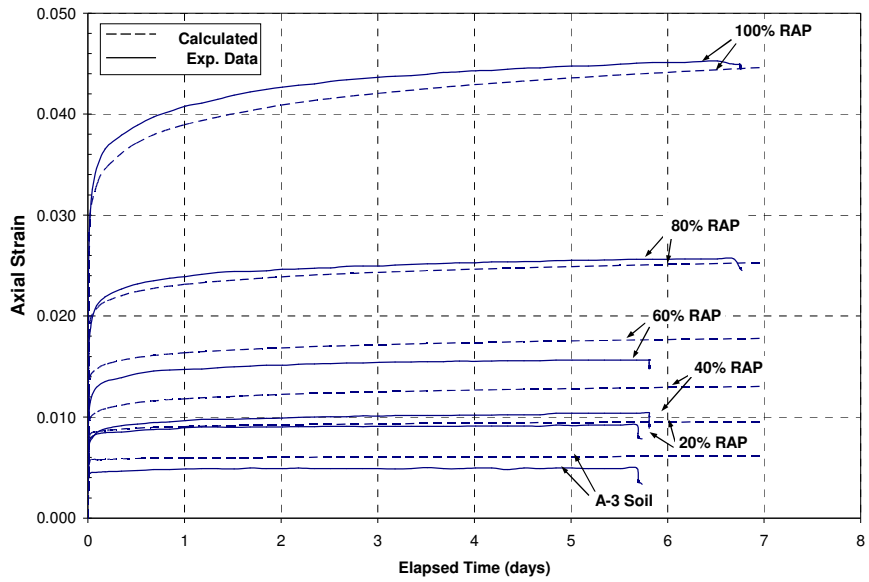
Wall Height H (ft)	Stress Level D (psi)	1 Year			50 Year		
		Strain Rate (%/min)	Strain (in/in)	Settlement (in)	Strain Rate (%/min)	Strain (in/in)	Settlement (in)
8	6	1.20E-06	0.00633	0.6	2.55E-08	0.00669	0.6
20	12	1.22E-06	0.00640	1.5	2.57E-08	0.00676	1.6
30	18	1.23E-06	0.00647	2.3	2.60E-08	0.00683	2.5



**Figure A-5.1 Comparison between experimentally collected and calculated strains in RAP, RAP-soil mixtures, and A-3 soil tested in creep at stress level of 6 psi for a period of 5-7 days**



**Figure A-5.2 Comparison between experimentally collected and calculated strains in RAP, RAP-soil mixtures, and A-3 soil tested in creep at stress level of 12 psi for a period of 5-7 days**



**Figure A-5.3 Comparison between experimentally collected and calculated strains in RAP, RAP-soil mixtures, and A-3 soil tested in creep at stress level of 18 psi for a period of 5-7 days**

**Appendix A-6: Regression Analysis on Creep Data**

**REGRESSION ANALYSIS ON 100% RAP**

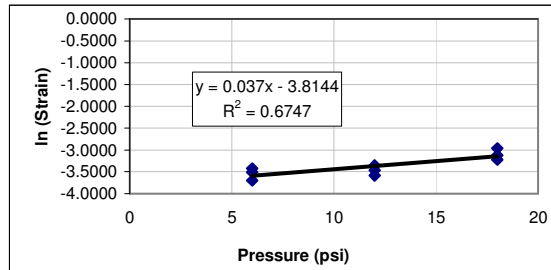
Correlation Table			
<b>6 psi</b>	<i>RC</i>	<i>Strain</i>	<i>LBR</i>
<i>RC</i>	1		
<i>Strain</i>	-0.975	1	
<i>LBR</i>	-0.992	0.995	1
<b>12 psi</b>	<i>RC</i>	<i>Strain</i>	<i>LBR</i>
<i>RC</i>	1		
<i>Strain</i>	-0.984	1	
<i>LBR</i>	0.306	-0.130	1
<b>18 psi</b>	<i>RC</i>	<i>Strain</i>	<i>LBR</i>
<i>RC</i>	1		
<i>Strain</i>	-1.000	1	
<i>LBR</i>	0.444	-0.471	1

**Final Regression Equation**

$$\ln(\text{Strain}) = 0.05 * p + (-19.26) * RC + 14.93$$

**Identification of the Trendline Function**

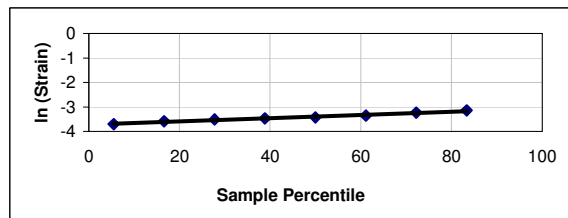
Parameters			
Strain	ln (Strain)	Pressure	RC
0.0326	-3.4239	6	97.1%
0.0247	-3.7008	6	97.9%
0.0299	-3.5092	6	97.2%
0.0349	-3.3545	12	98.6%
0.0278	-3.5830	12	98.9%
0.0313	-3.4639	12	98.8%
0.0518	-2.9613	18	97.5%
0.0399	-3.2206	18	98.9%
0.0441	-3.1217	18	98.4%



**Regression Analysis**

Regression Statistics	
Multiple R	0.9699
R Square	0.9408
Adjusted R Square	0.9210
Standard Error	0.0650
Observations	9

Coefficients	
Intercept	14.927
Pressure	0.050
RC	-19.261



**REGRESSION ANALYSIS ON 80% RAP**

Correlation Table			
<b>6 psi</b>	<i>RC</i>	<i>Strain</i>	<i>LBR</i>
<i>RC</i>	1		
<i>Strain</i>	-0.779	1	
<i>LBR</i>	-0.246	-0.416	1
<b>12 psi</b>	<i>RC</i>	<i>Strain</i>	<i>LBR</i>
<i>RC</i>	1		
<i>Strain</i>	-0.935	1	
<i>LBR</i>	0.807	-0.545	1
<b>18 psi</b>	<i>RC</i>	<i>Strain</i>	<i>LBR</i>
<i>RC</i>	1		
<i>Strain</i>	-1.000	1	
<i>LBR</i>	-1.000	1.000	1

**Final Regression Equation**

$$\ln(\text{Strain}) = 0.045 * p + (-17.59) * RC + 12.73$$

Identification of the Trendline Function			
Parameters			
Strain	ln (Strain)	Pressure	RC
0.0150	-4.1988	6	96.9%
0.0185	-3.9885	6	96.9%
0.0206	-3.8848	6	96.7%
0.0216	-3.8343	12	97.6%
0.0184	-3.9942	12	97.9%
0.0174	-4.0538	12	98.3%
0.0254	-3.6711	18	98.1%
0.0256	-3.6669	18	98.2%
0.0261	-3.6463	18	97.3%

Regression Statistics		Coefficients	
Multiple R	0.8942	Intercept	12.731
R Square	0.7996	Pressure	0.045
Adjusted R Square	0.7328	RC	-17.589
Standard Error	0.1005		
Observations	9		



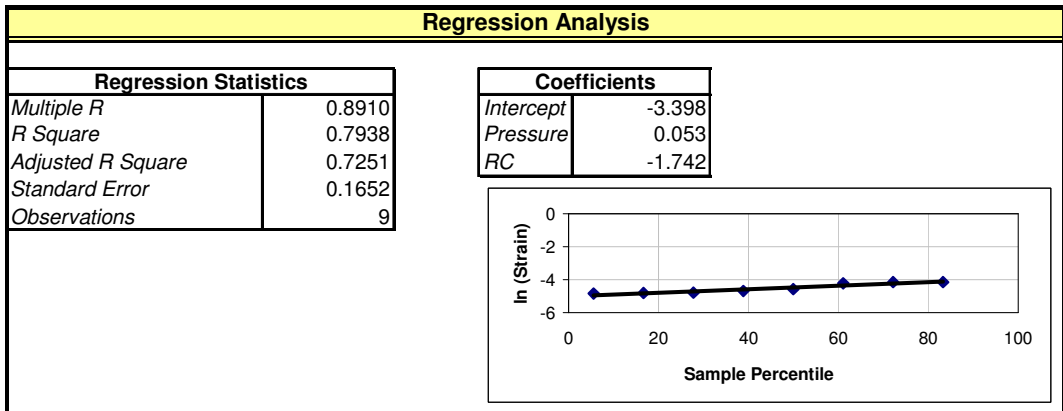
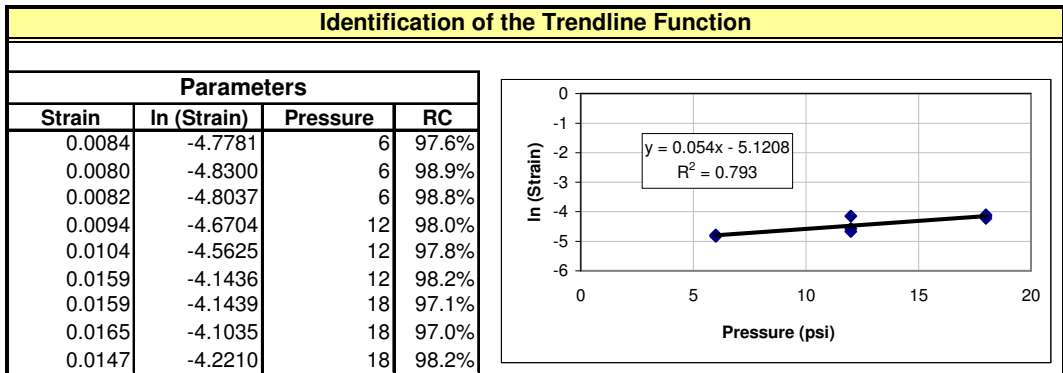
**DATA ANALYSIS ON 60% RAP**

Correlation Table			
<b>6 psi</b>	<i>RC</i>	<i>Strain</i>	<i>LBR</i>
<i>RC</i>	1		
<i>Strain</i>	-0.907	1	
<i>LBR</i>	-0.050	0.466	1
<b>12 psi</b>	<i>RC</i>	<i>Strain</i>	<i>LBR</i>
<i>RC</i>	1		
<i>Strain</i>	0.913	1	
<i>LBR</i>	0.943	0.725	1
<b>18 psi</b>	<i>RC</i>	<i>Strain</i>	<i>LBR</i>
<i>RC</i>	1		
<i>Strain</i>	-0.962	1	
<i>LBR</i>	0.986	-0.901	1

**Correction Status:** Before correction

**Final Regression Equation**

$$\ln(\text{Strain}) = 0.053 * p + (-1.74) * RC - 3.40$$



**DATA ANALYSIS ON 40% RAP**

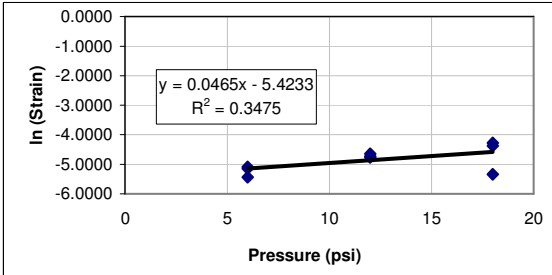
Correlation Table			
<b>6 psi</b>	<i>RC</i>	<i>Strain</i>	<i>LBR</i>
<i>RC</i>	1		
<i>Strain</i>	-0.122	1	
<i>LBR</i>	0.993	-0.239	1
<b>12 psi</b>	<i>RC</i>	<i>Strain</i>	<i>LBR</i>
<i>RC</i>	1		
<i>Strain</i>	0.180	1	
<i>LBR</i>	0.122	-0.954	1
<b>18 psi</b>	<i>RC</i>	<i>Strain</i>	<i>LBR</i>
<i>RC</i>	1		
<i>Strain</i>	0.954	1	
<i>LBR</i>	0.978	0.870	1

**Correction Status:** Before correction

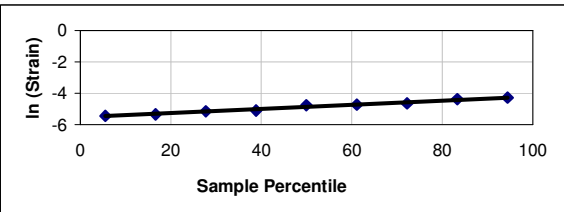
**Final Regression Equation**

$$\ln(\text{Strain}) = 0.037 * p + 28.51 * RC - 33.43$$

Identification of the Trendline Function			
<b>Parameters</b>			
<b>Strain</b>	<b>ln (Strain)</b>	<b>Pressure</b>	<b>RC</b>
0.0059	-5.1395	6	97.6%
0.0044	-5.4340	6	98.9%
0.0062	-5.0888	6	98.8%
0.0085	-4.7655	12	98.0%
0.0088	-4.7279	12	97.8%
0.0096	-4.6474	12	98.2%
0.0139	-4.2728	18	97.1%
0.0126	-4.3764	18	97.0%
0.0048	-5.3409	18	98.2%



Regression Statistics		Coefficients	
Multiple R	0.7032	Intercept	-33.433
R Square	0.4944	Pressure	0.037
Adjusted R Square	0.3259	RC	28.510
Standard Error	0.3362		
Observations	9		



**REGRESSION ANALYSIS ON 20% RAP**

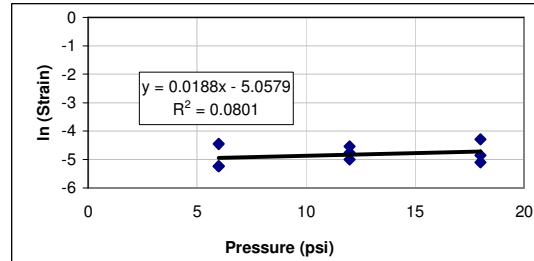
Correlation Table			
<b>6 psi</b>	<i>RC</i>	<i>Strain</i>	<i>LBR</i>
<i>RC</i>	1		
<i>Strain</i>	0.326	1	
<i>LBR</i>	0.933	-0.035	1
<b>12 psi</b>	<i>RC</i>	<i>Strain</i>	<i>LBR</i>
<i>RC</i>	1		
<i>Strain</i>	0.371	1	
<i>LBR</i>	0.982	0.540	1
<b>18 psi</b>	<i>RC</i>	<i>Strain</i>	<i>LBR</i>
<i>RC</i>	1		
<i>Strain</i>	-0.960	1	
<i>LBR</i>	0.720	-0.496	1

**Final Regression Equation**

$$\ln(\text{Strain}) = 0.025 * p + 5.43 * RC - 10.45$$

**Identification of the Trendline Function**

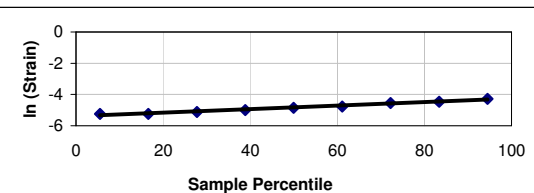
Parameters			
Strain	ln (Strain)	Pressure	RC
0.0061	-5.1045	6	99.0%
0.0059	-5.1402	6	97.9%
0.0124	-4.3939	6	98.7%
0.0067	-5.0041	12	98.4%
0.0085	-4.7655	12	98.1%
0.0106	-4.5423	12	98.5%
0.0061	-5.1045	18	97.3%
0.0079	-4.8435	18	97.1%
0.0137	-4.2877	18	96.8%



**Regression Analysis**

Regression Statistics	
Multiple R	0.2906
R Square	0.0845
Adjusted R Square	-0.2207
Standard Error	0.3819
Observations	9

Coefficients	
Intercept	-10.451
Pressure	0.025
RC	5.425

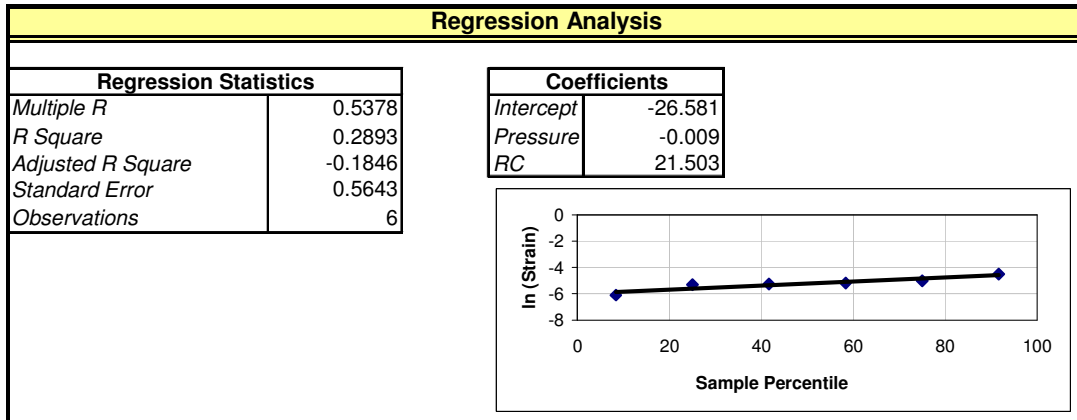
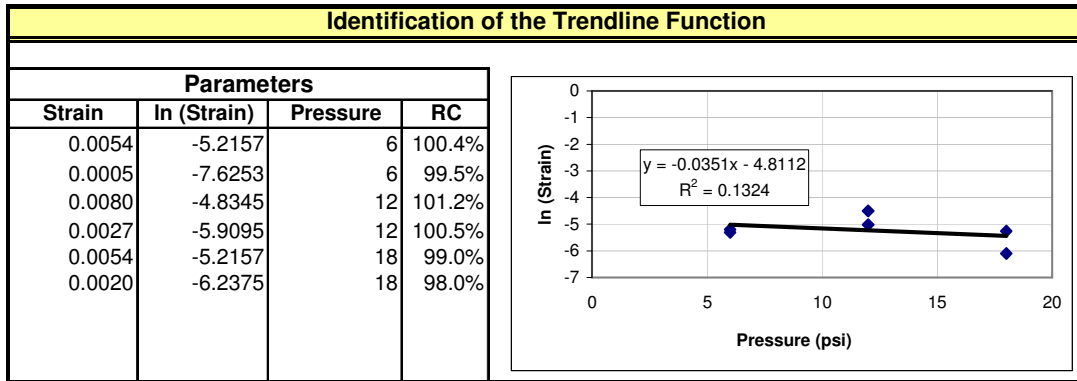


**REGRESSION ANALYSIS ON A-3 SOIL**

Correlation Table			
<b>6 psi</b>	<i>RC</i>	<i>Strain</i>	<i>LBR</i>
<i>RC</i>	1		
<i>Strain</i>	1	1	
<i>LBR</i>	-1	-1	1
<b>12 psi</b>	<i>RC</i>	<i>Strain</i>	<i>LBR</i>
<i>RC</i>	1		
<i>Strain</i>	-1	1	
<i>LBR</i>	-1	1	1
<b>18 psi</b>	<i>RC</i>	<i>Strain</i>	<i>LBR</i>
<i>RC</i>	1		
<i>Strain</i>	-1	1	
<i>LBR</i>			1

**Final Regression Equation**

$$\ln(\text{Strain}) = -0.01 * p + 21.50 * RC - 26.58$$

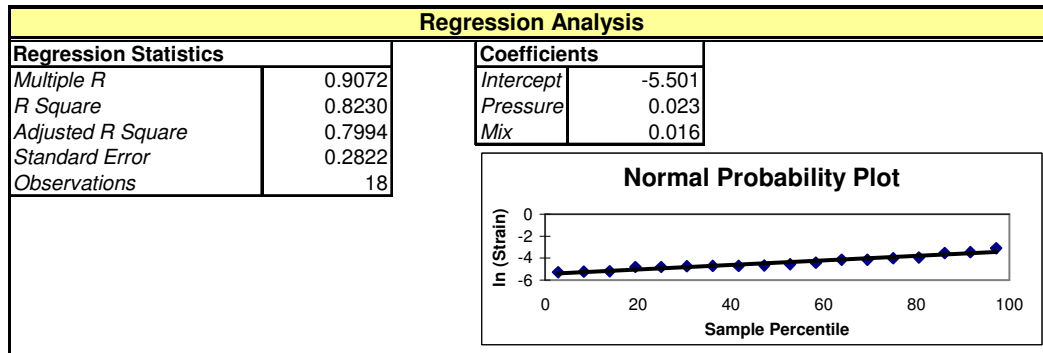
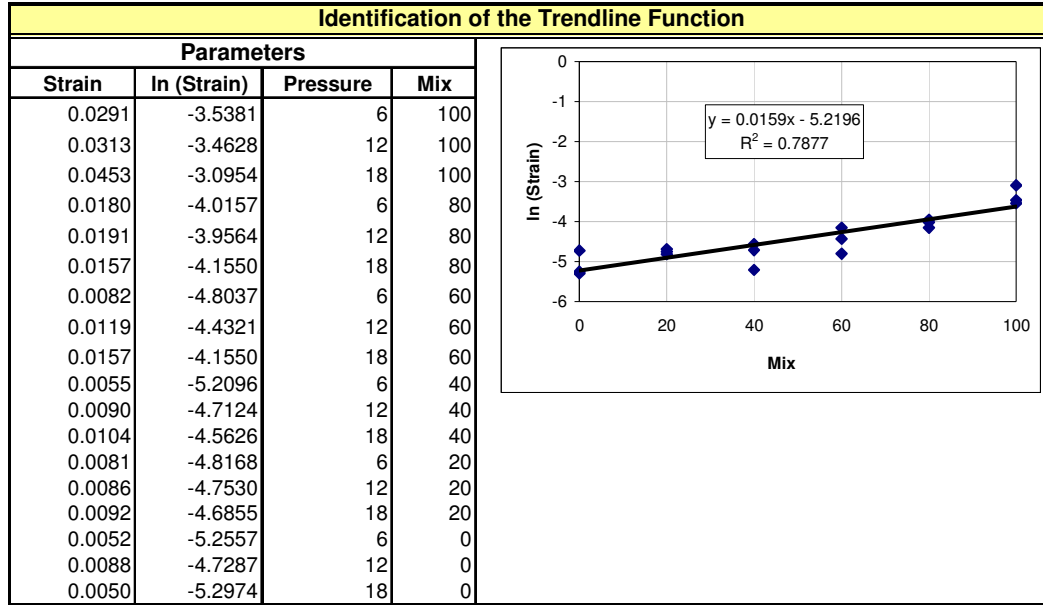


**REGRESSION ANALYSIS ON ALL MIXTURES**

Correlation Table			
	Strain	Pressure	Mix
Strain	1		
Pressure	0.177	1	
Mix	0.817	0	1

**Final Regression Equation**

$$\ln(\text{Strain}) = 0.023 * p + (-0.02) * \text{Mix} - 5.50$$



**REGRESSION ANALYSIS ON ALL MIXTURES**

**Correlation Table**

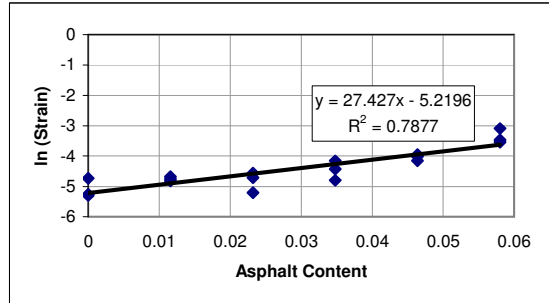
	Strain	Pressure	AC
Strain	1		
Pressure	0.177	1	
AC	0.817	0	1

**Final Regression Equation**

$$\ln(\text{Strain}) = 0.023 * p + 27.43 * \text{Mix} - 5.50$$

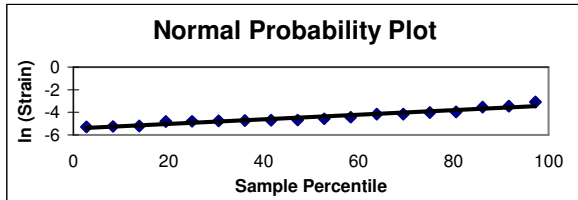
**Identification of the Trendline Function**

Parameters			
Strain	ln (Strain)	Pressure	AC
0.0291	-3.5381	6	0.058
0.0313	-3.4628	12	0.058
0.0453	-3.0954	18	0.058
0.0180	-4.0157	6	0.046
0.0191	-3.9564	12	0.046
0.0157	-4.1550	18	0.046
0.0082	-4.8037	6	0.035
0.0119	-4.4321	12	0.035
0.0157	-4.1550	18	0.035
0.0055	-5.2096	6	0.023
0.0090	-4.7124	12	0.023
0.0104	-4.5626	18	0.023
0.0081	-4.8168	6	0.012
0.0086	-4.7530	12	0.012
0.0092	-4.6855	18	0.012
0.0052	-5.2557	6	0
0.0088	-4.7287	12	0
0.0050	-5.2974	18	0



**Regression Analysis**

Regression Statistics		Coefficients	
Multiple R	0.9072	Intercept	-5.501
R Square	0.8230	Pressure	0.023
Adjusted R Square	0.7994	AC	27.427
Standard Error	0.2822		
Observations	18		



## **Appendix A-7: Settlement Calculations**

**Table A-7.1 Summary of creep settlements in RAP, RAP-soil mixtures, and A-3 soil at 6 psi stress level**

Mixture	1 Day Strain	1 Day Settlement (in)	7 Day Strain	7 Day Settlement (in)	1 Year Strain	1 Year Settlement (in)	50 Year Strain	50 Year Settlement (in)	% of the 50 Year Settlement Occurring in 1 Day	% of the 50 Year Settlement Occurring in 7 Days	% of the 50 Year Settlement Occurring in 1 Year
100% RAP	0.0239	2.3	0.0274	2.6	0.0361	3.5	0.0474	4.6	50%	58%	76%
80% RAP	0.0157	1.5	0.0172	1.6	0.0206	2.0	0.0246	2.4	64%	70%	84%
60% RAP	0.0082	0.8	0.0089	0.9	0.0105	1.0	0.0124	1.2	66%	71%	85%
40% RAP	0.0060	0.6	0.0067	0.6	0.0081	0.8	0.0099	1.0	61%	67%	82%
20% RAP	0.0078	0.7	0.0082	0.8	0.0091	0.9	0.0101	1.0	77%	81%	90%
A-3 Soil	0.0058	0.6	0.0060	0.6	0.0063	0.6	0.0067	0.6	87%	89%	95%

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**Table A-7.2 Summary of creep settlements in RAP, RAP-soil mixtures, and A-3 soil at 12 psi stress level**

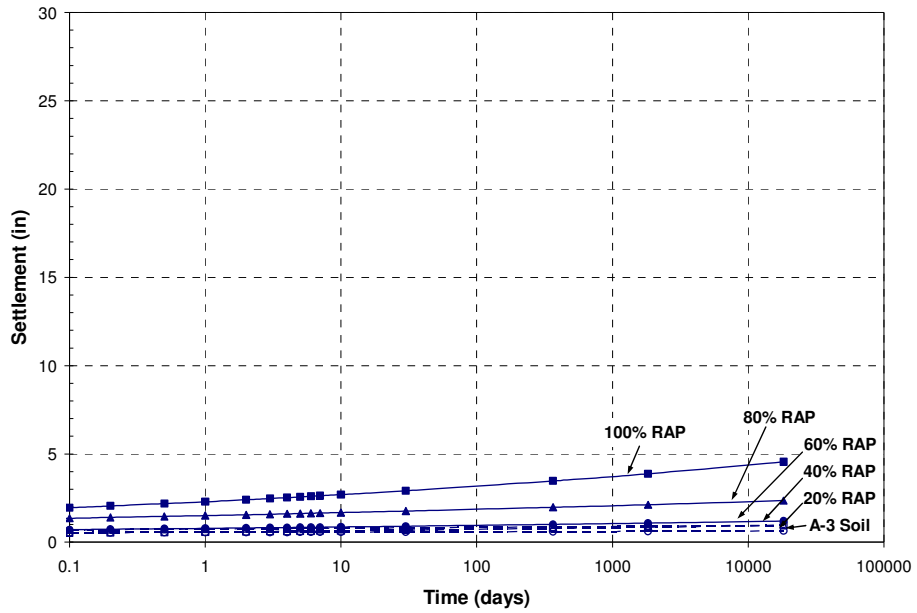
Mixture	1 Day Strain	1 Day Settlement (in)	7 Day Strain	7 Day Settlement (in)	1 Year Strain	1 Year Settlement (in)	50 Year Strain	50 Year Settlement (in)	% of the 50 Year Settlement Occurring in 1 Day	% of the 50 Year Settlement Occurring in 7 Days	% of the 50 Year Settlement Occurring in 1 Year
100% RAP	0.0305	7.3	0.0349	8.4	0.0461	11.1	0.0605	14.5	50%	58%	76%
80% RAP	0.0191	4.6	0.0208	5.0	0.0250	6.0	0.0298	7.2	64%	70%	84%
60% RAP	0.0116	2.8	0.0126	3.0	0.0149	3.6	0.0176	4.2	66%	71%	85%
40% RAP	0.0084	2.0	0.0093	2.2	0.0114	2.7	0.0139	3.3	61%	67%	82%
20% RAP	0.0084	2.0	0.0088	2.1	0.0098	2.4	0.0109	2.6	77%	81%	90%
A-3 Soil	0.0059	1.4	0.0060	1.5	0.0064	1.5	0.0068	1.6	87%	89%	95%

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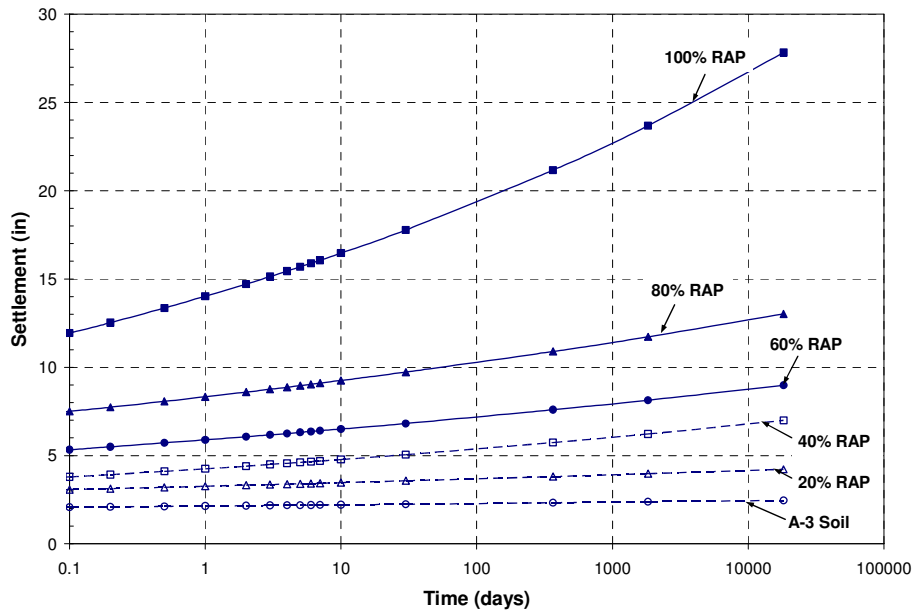
**Table A-7.3 Summary of creep settlements in RAP, RAP-soil mixtures, and A-3 soil at 18 psi stress level**

<b>Mixture</b>	<b>1 Day Strain</b>	<b>1 Day Settlement (in)</b>	<b>7 Day Strain</b>	<b>7 Day Settlement (in)</b>	<b>1 Year Strain</b>	<b>1 Year Settlement (in)</b>	<b>50 Year Strain</b>	<b>50 Year Settlement (in)</b>	<b>% of the 50 Year Settlement Occurring in 1 Day</b>	<b>% of the 50 Year Settlement Occurring in 7 Days</b>	<b>% of the 50 Year Settlement Occurring in 1 Year</b>
100% RAP	0.0389	14.0	0.0446	16.1	0.0588	21.2	0.0773	27.8	50%	58%	76%
80% RAP	0.0231	8.3	0.0253	9.1	0.0303	10.9	0.0362	13.0	64%	70%	84%
60% RAP	0.0164	5.9	0.0178	6.4	0.0211	7.6	0.0250	9.0	66%	71%	85%
40% RAP	0.0118	4.3	0.0130	4.7	0.0159	5.7	0.0194	7.0	61%	67%	82%
20% RAP	0.0091	3.3	0.0095	3.4	0.0106	3.8	0.0117	4.2	77%	81%	90%
A-3 Soil	0.0059	2.1	0.0061	2.2	0.0065	2.3	0.0068	2.5	87%	89%	95%

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**Figure A-7.1 Backfill settlement with time for RAP, RAP-soil mixtures, and A-3 soil behind 8-ft high wall**



**Figure A-7.2 Backfill settlement with time for RAP, RAP-soil mixtures, and A-3 soil behind 30-ft high wall**

## **Appendix A-8: Stress Level Verification**

**DATA SHEET FOR STRESS LEVEL VERIFICATION**

**Soil:** 100% RAP

**Stresses at failure:**

$\sigma_{1f} = 50$  psi

$\sigma_{3f} = 5$  psi

**Shear strength parameters:**

$c = 4.9$  psi

$\phi = 44^\circ$

**Equations:**

Eq.1  $K_0 = 1 - \sin \phi$

Eq.4  $\alpha_f = 45^\circ + \frac{\phi}{2}$

Eq.2  $\frac{\sigma_3}{\sigma_1} = K_0$

Eq.5  $\tan \alpha_f = \frac{\tau_{ff}}{\sigma_{ff} - \sigma_{3f}}$

Eq.3  $\sigma_n = \frac{\sigma_{1f} + \sigma_{3f}}{2}$

Eq.6  $\tan \phi = \frac{\tau_{ff}}{\sigma_{ff} + c * \cot \phi}$

From Eq.1  $K_0 = 0.305$

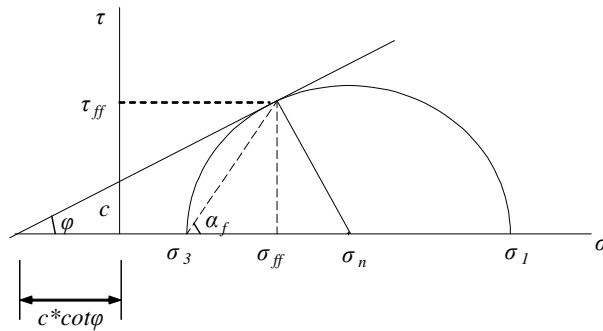
From Eq.2  $\sigma_3 / \sigma_1 = 0.305$

From Eq.4  $\alpha_f = 67^\circ$

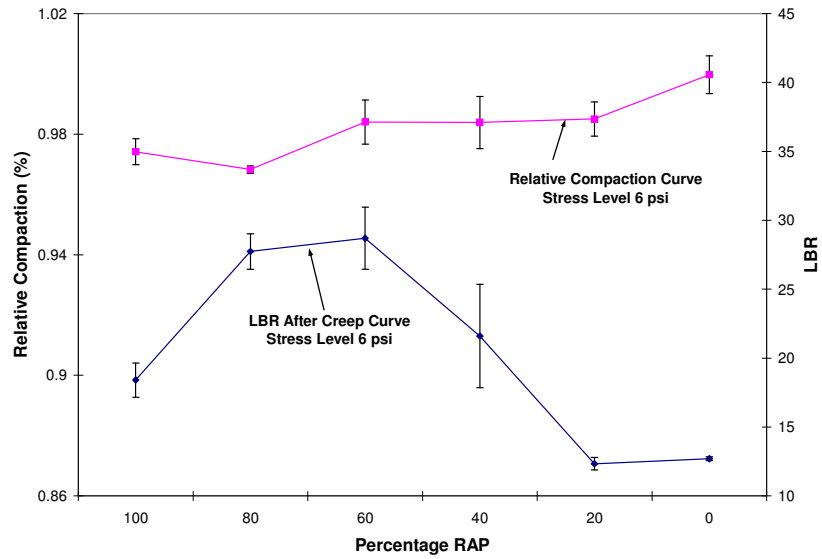
From Eq.5 and 6  $\sigma_{ff} = 12.0$  psi

$\tau_{ff} = 16.5$  psi

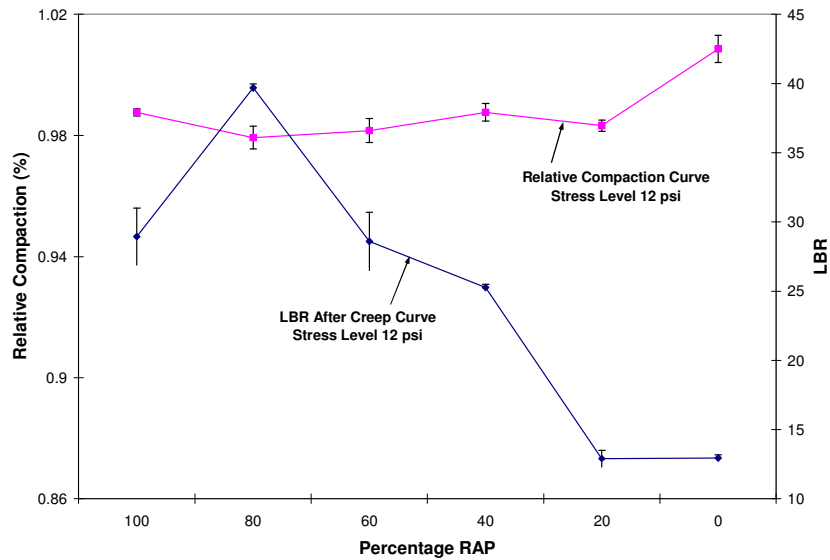
$\sigma_1$ (psi)	$\sigma_3 = K_0 * \sigma_1$ (psi)	$(\sigma_1 - \sigma_3)$ (psi)	$(\sigma_1 - \sigma_3) / \tau_{ff}$ (%)
6	1.83	4.17	25%
12	3.66	8.34	51%
18	5.50	12.50	76%



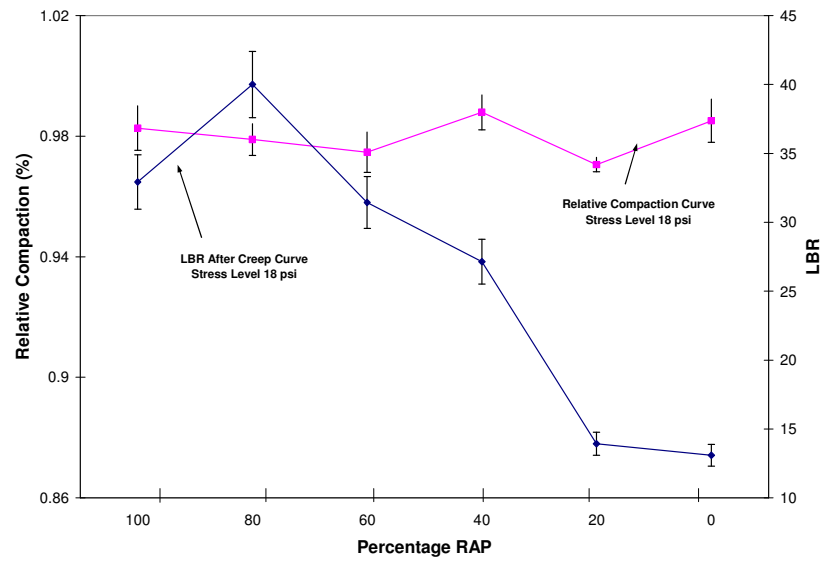
**Appendix A-9: LBR after Creep in Relation to Relative Compaction**



**Figure A-9.1 Comparison between the standard deviations in Relative Compaction and LBR for samples of RAP, RAP-soil mixtures, and A-3 soil tested in creep at 6 psi stress level**



**Figure A-9.2 Comparison between the standard deviations in Relative Compaction and LBR for samples of RAP, RAP-soil mixtures, and A-3 soil tested in creep at 12 psi stress level**



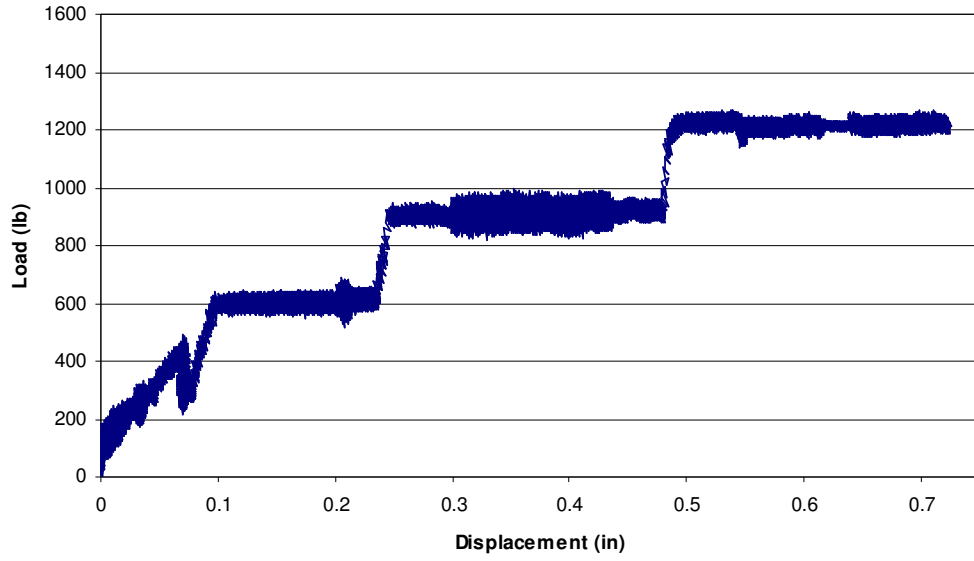
**Figure A-9.3 Comparison between the standard deviations in Relative Compaction and LBR for samples of RAP, RAP-soil mixtures, and A-3 soil tested in creep at 18 psi stress level**



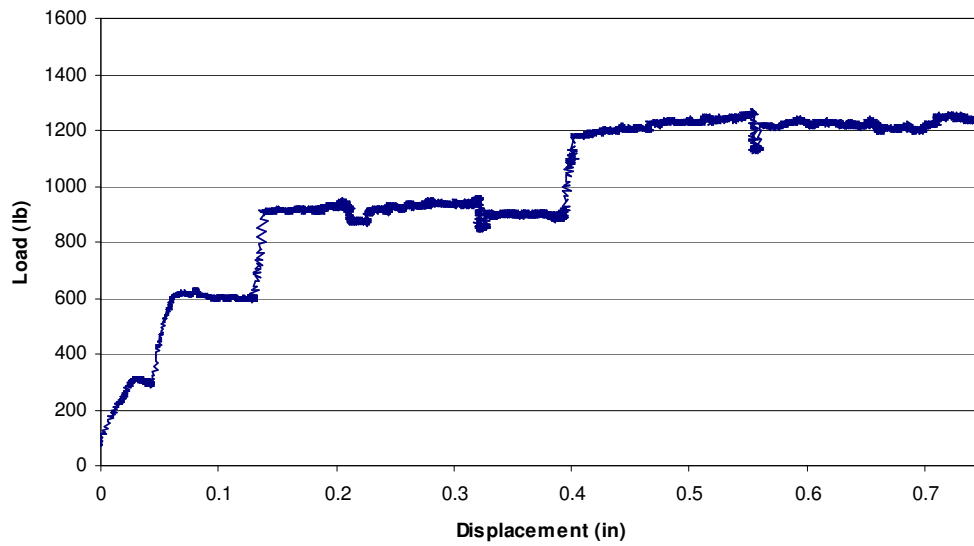
**Appendix B-1: Incremental Pullout Test Data - Load vs. Displacement Plots**

# 100% RAP

6 psi Strip 2

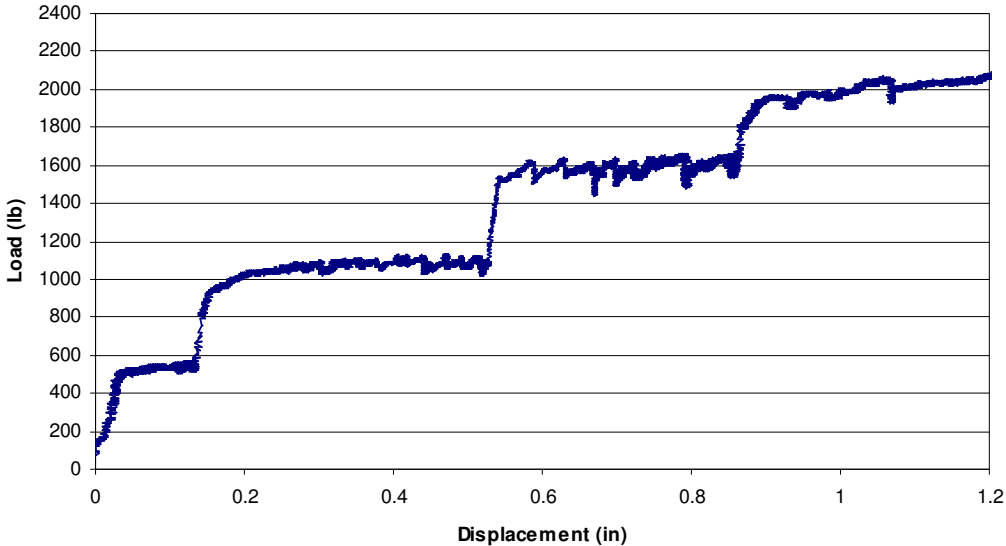


6 psi Strip 3



# 100% RAP

12 psi Strip 2

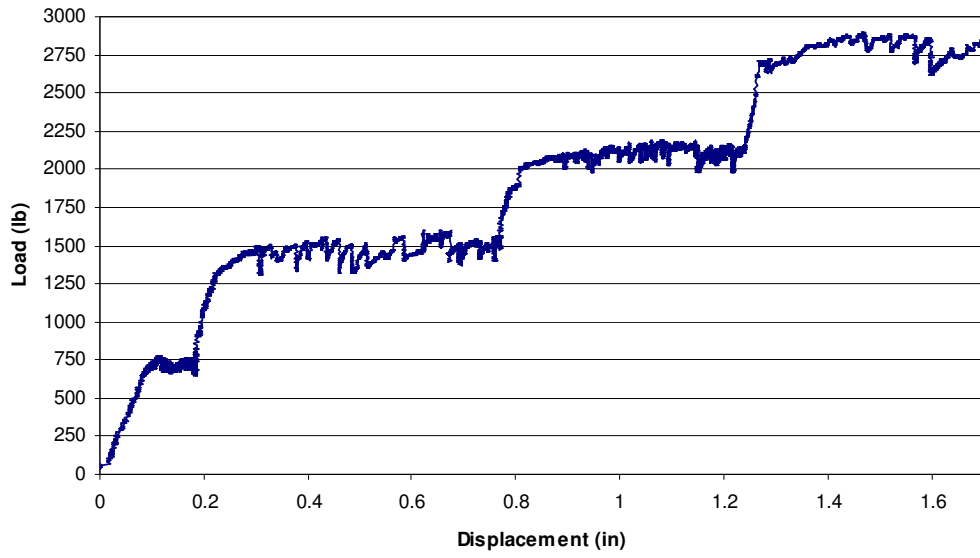


12 psi Strip 3

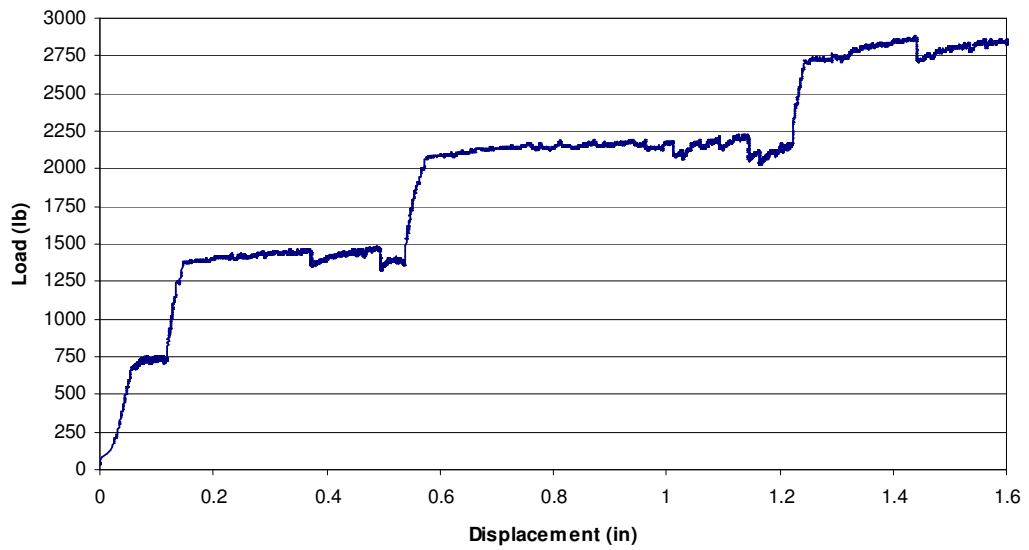


# 100% RAP

18 psi Strip 2



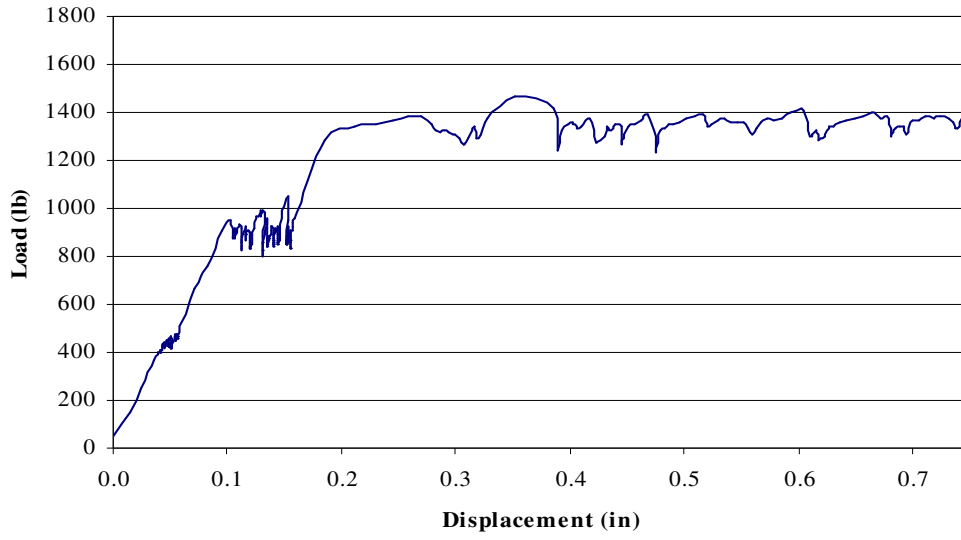
18 psi Strip 3



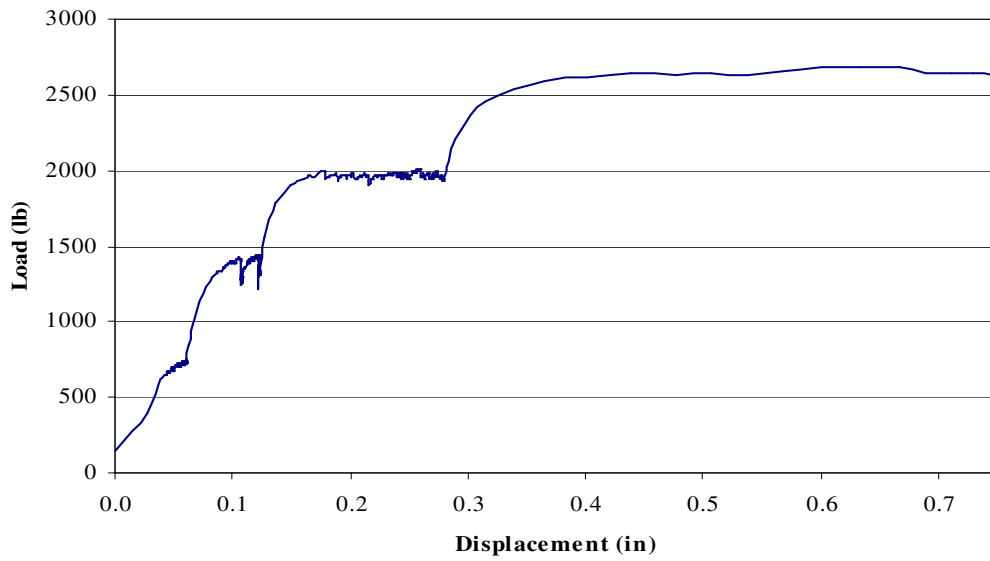


# 50/50% Blend

6 psi - Strip 2

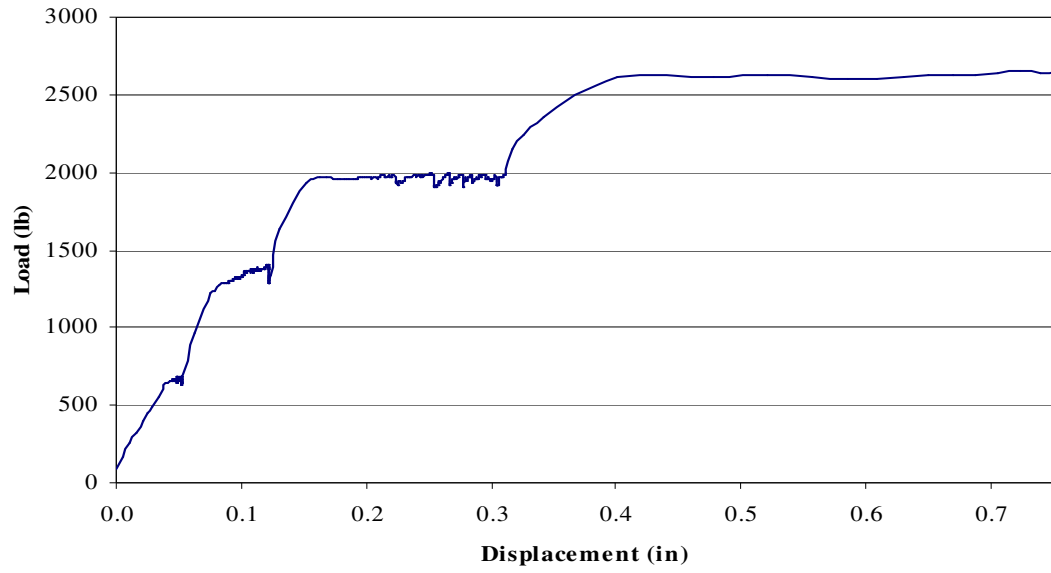


12 psi - Strip 2

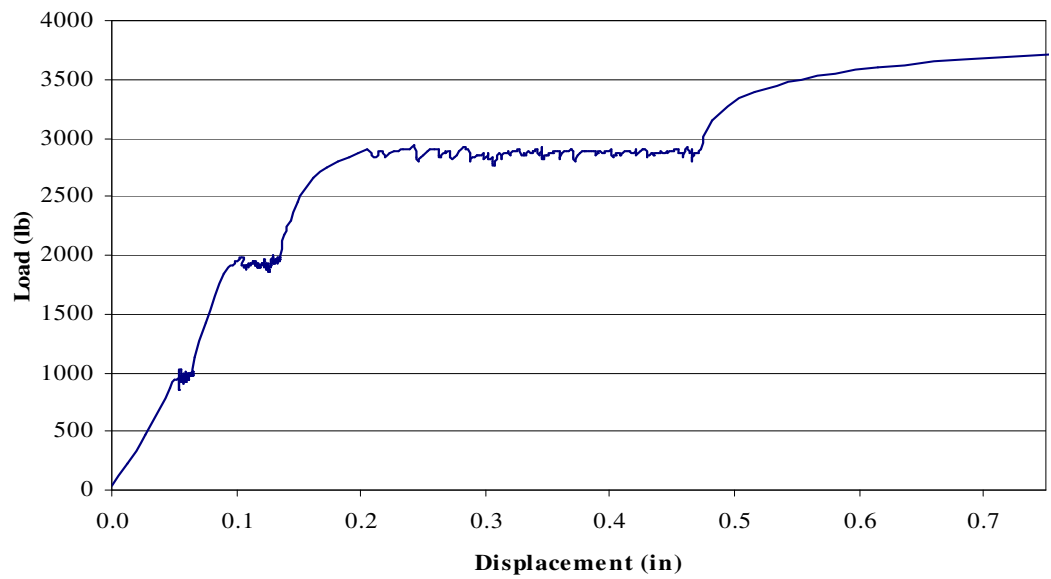


# 50/50% Blend

12 psi - Strip 3

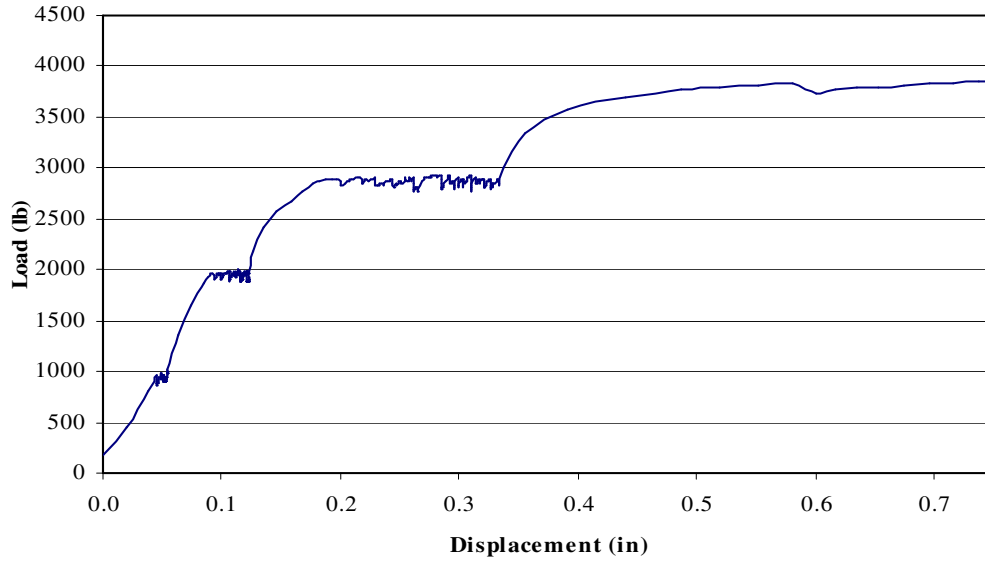


18 psi - Strip 2



# 50/50% Blend

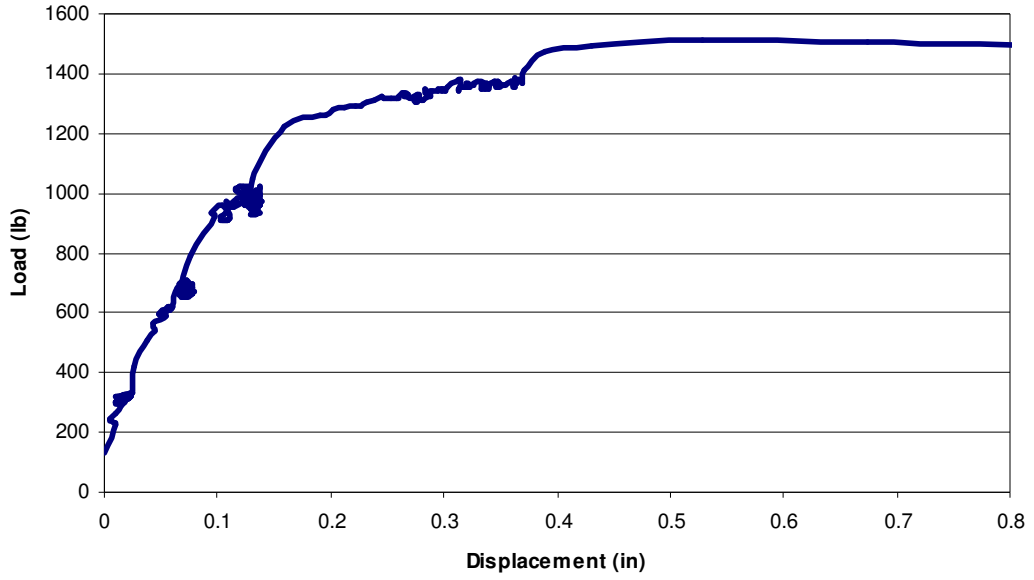
18 psi - Strip 3



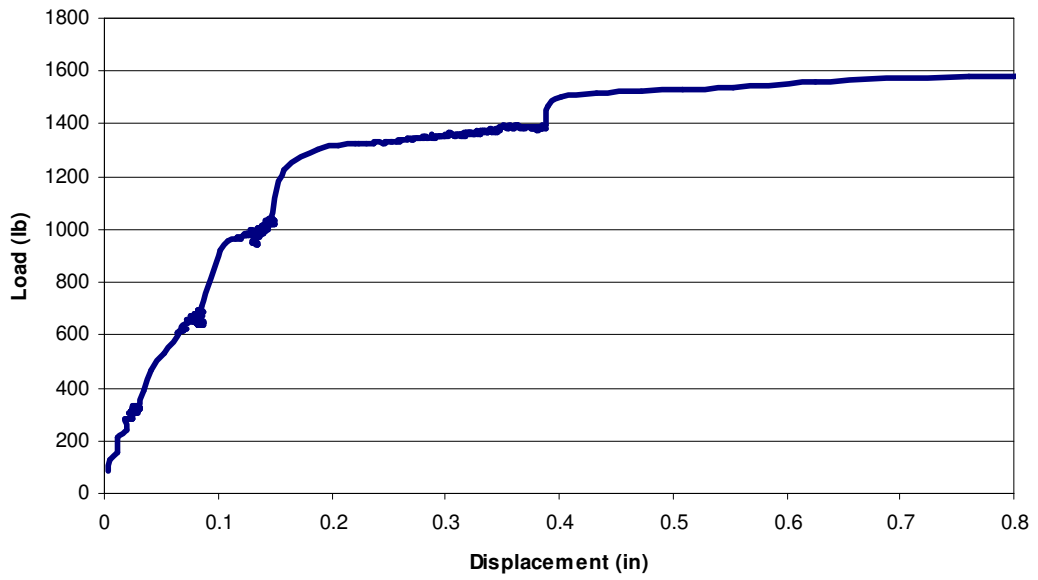


# A-3 Sand

6 psi - Strip 2



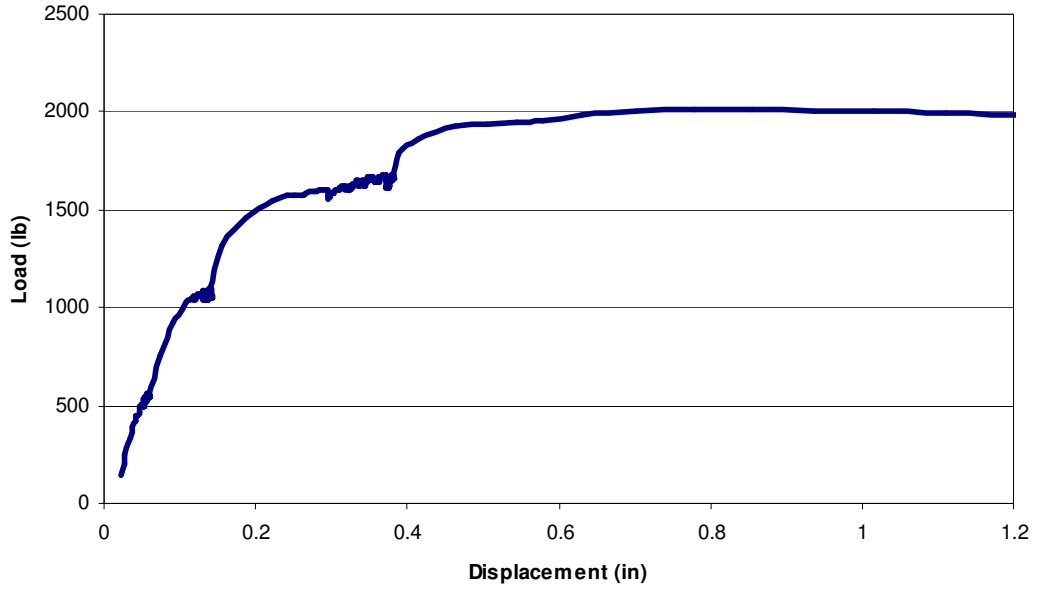
6 psi - Strip 3



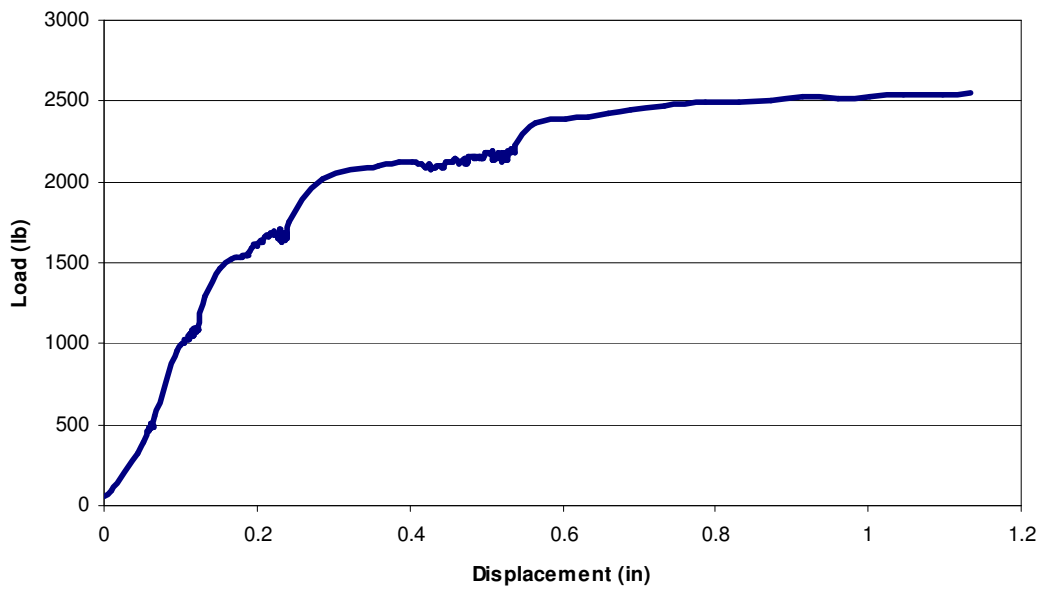
B-9

# A-3 Sand

12 psi - Strip 2



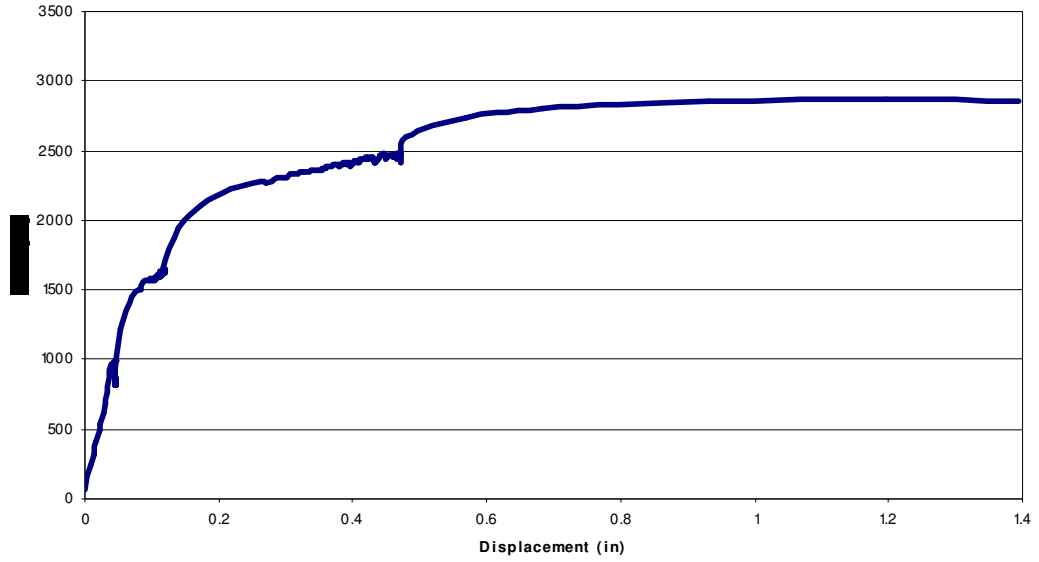
12 psi - Strip 3



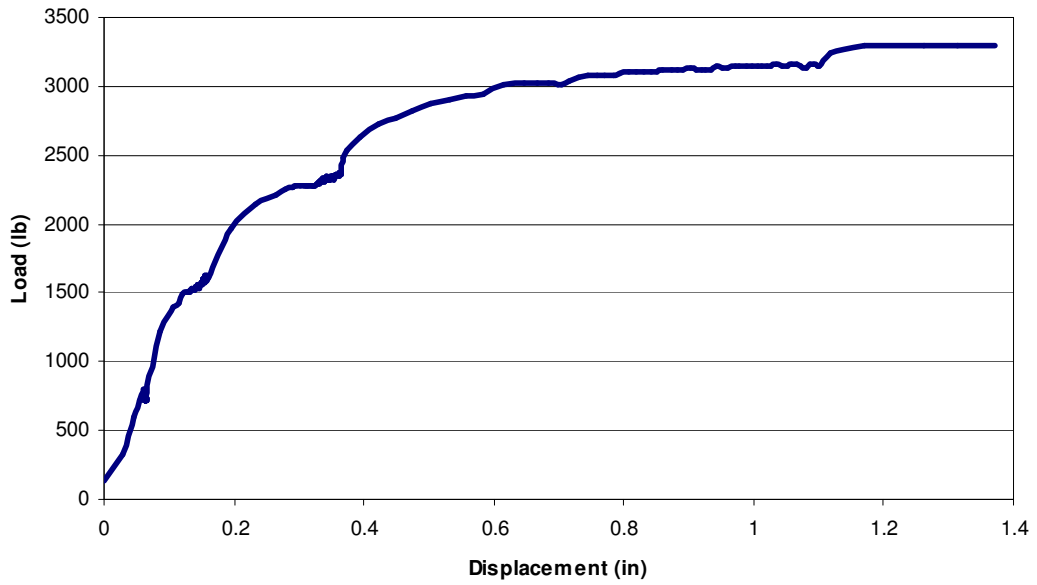
B-10

# A-3 Sand

18 psi - Strip 2



18 psi - Strip 3



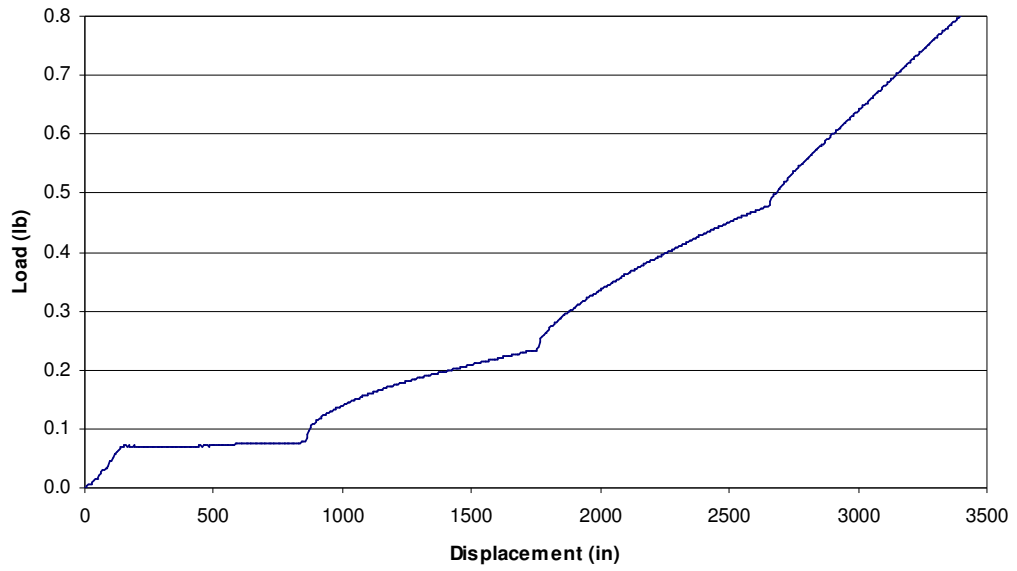
B-11



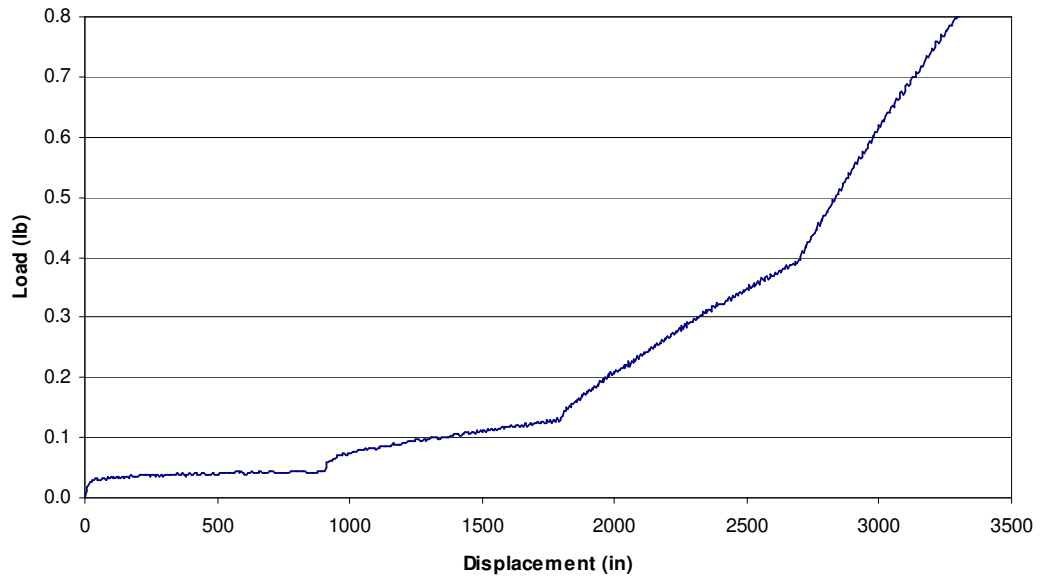
**Appendix B-2: Incremental Pullout Test Data - Displacement vs. Time Plots**

# 100% RAP

6 psi Strip 2

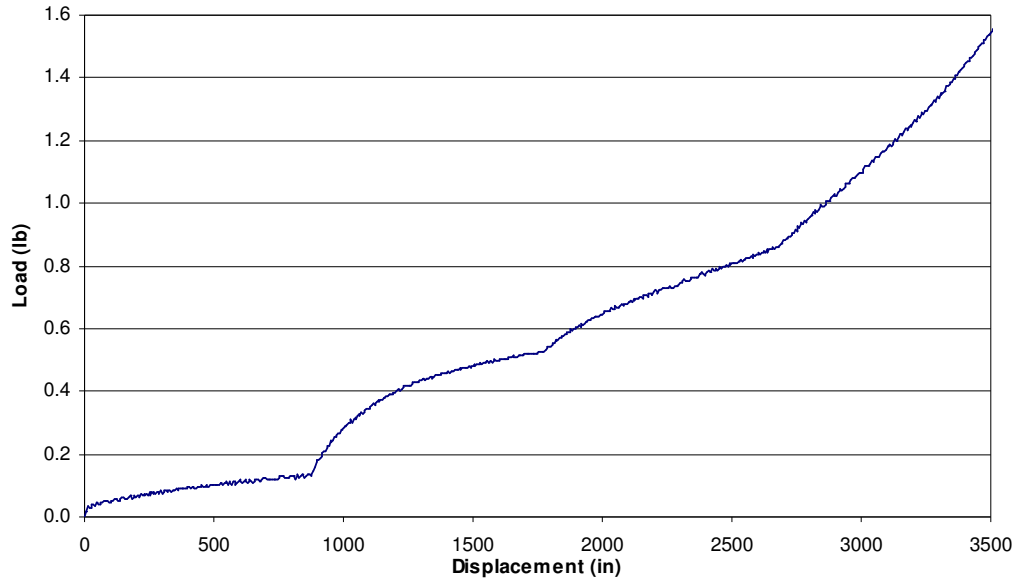


6 psi Strip 3

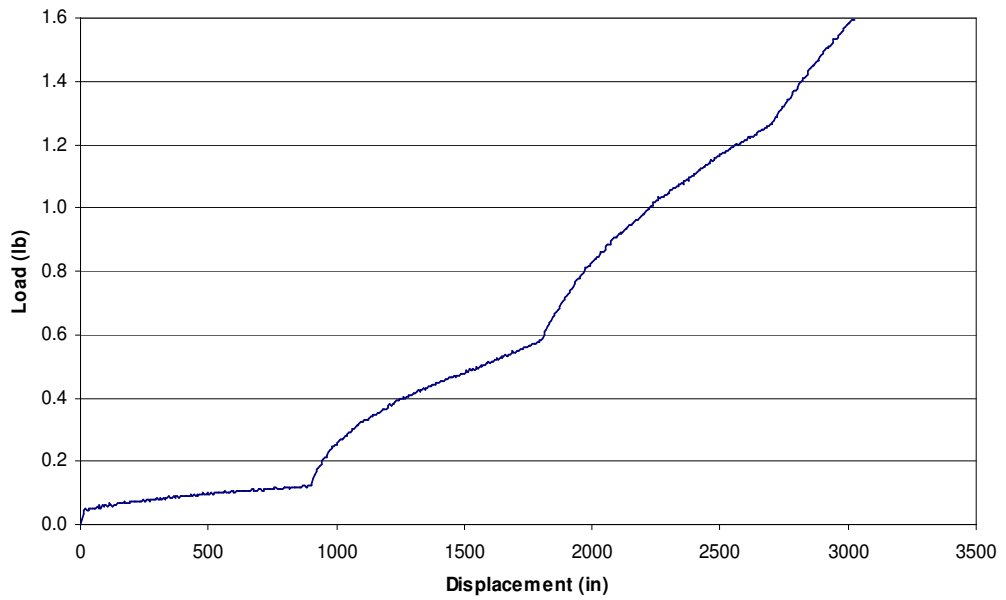


# 100% RAP

12 psi Strip 2

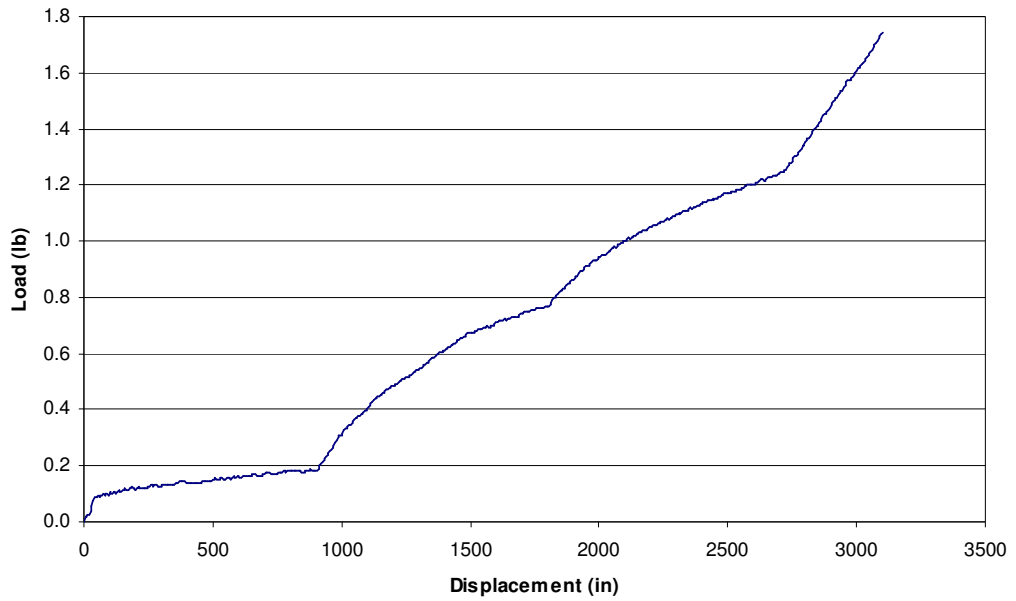


12 psi Strip3

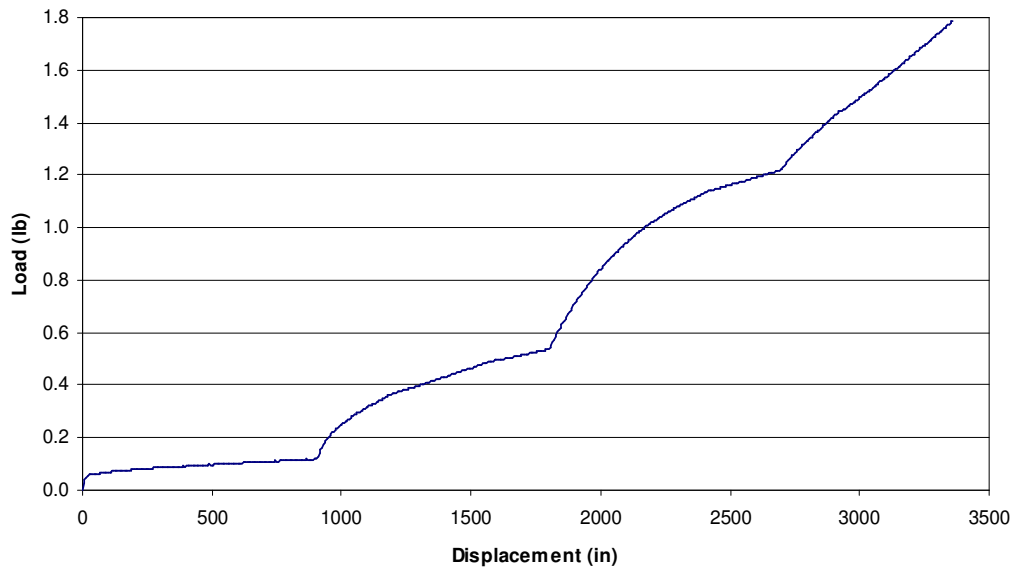


# 100% RAP

18 psi Strip 2



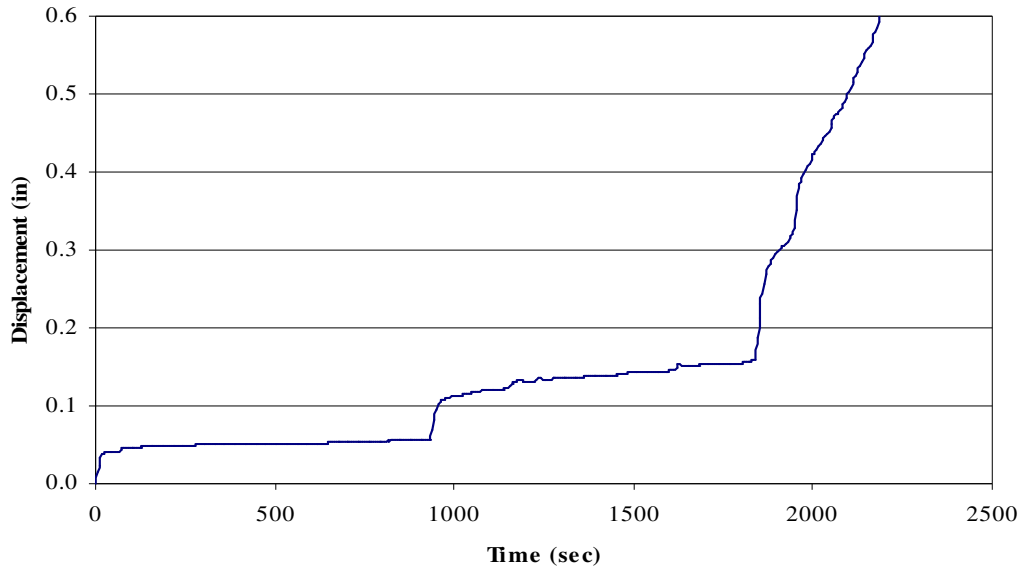
18 psi Strip 3



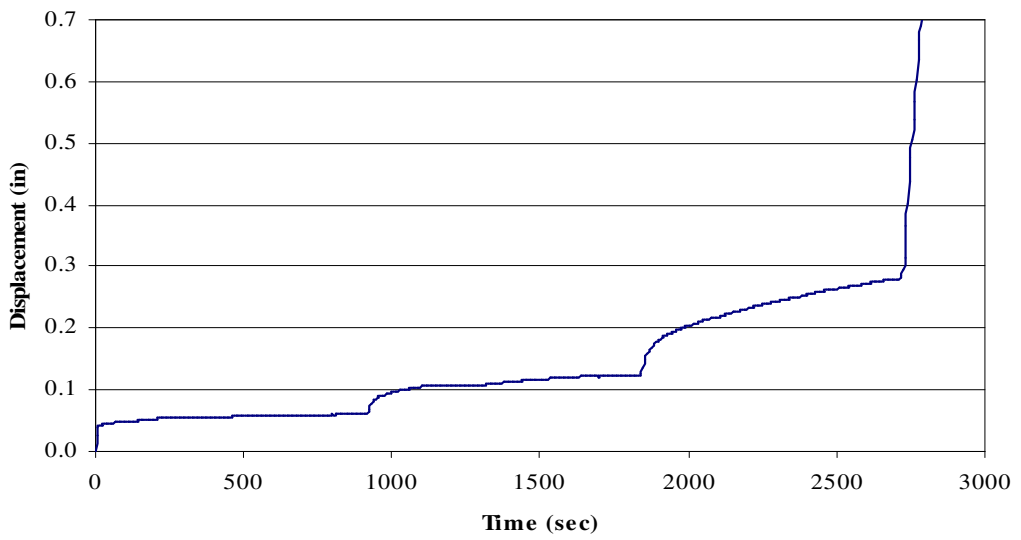


# 50/50% Blend

6 psi - Strip 2

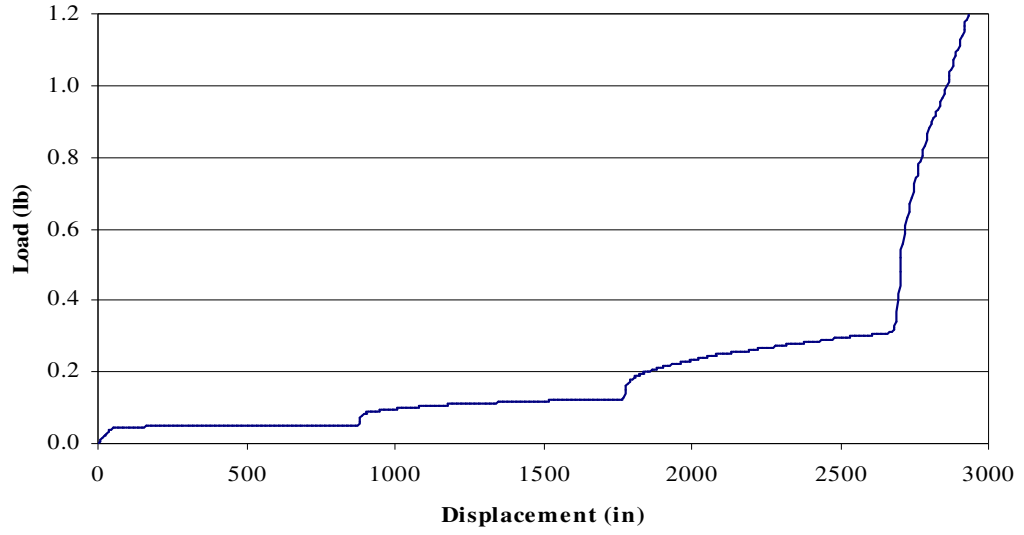


12 psi - Strip 2

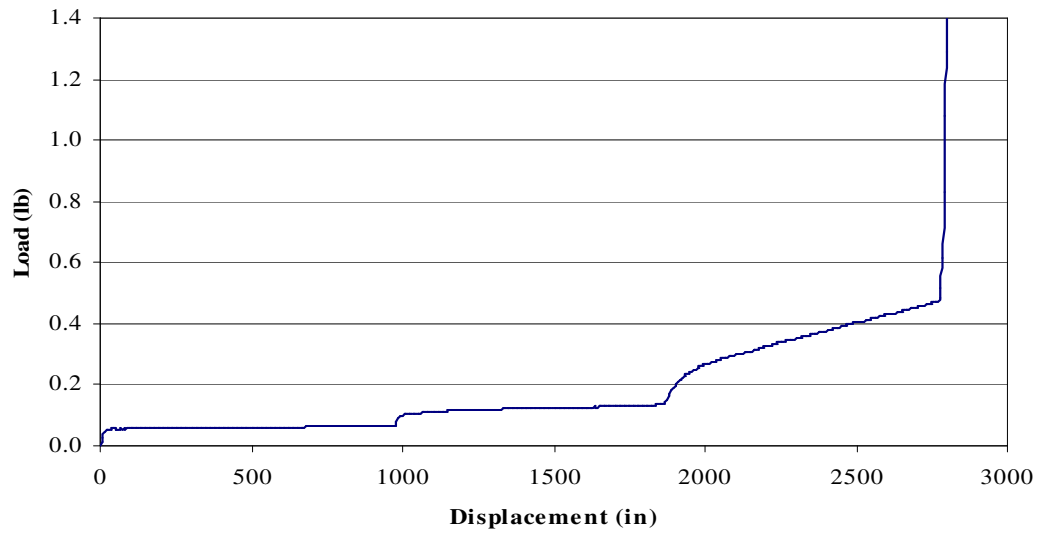


# 50/50% Blend

12 psi - Strip 3



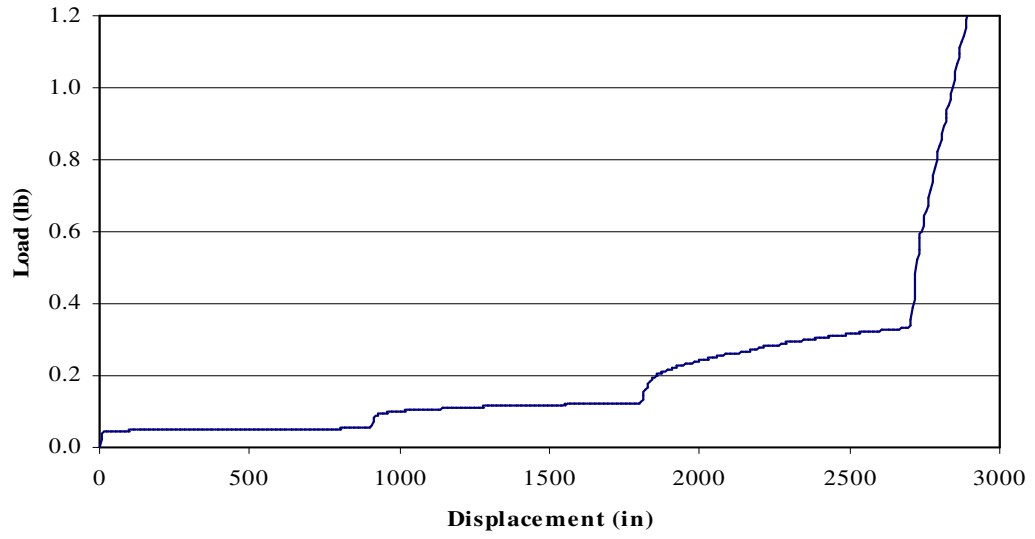
18 psi - Strip 2



B-18

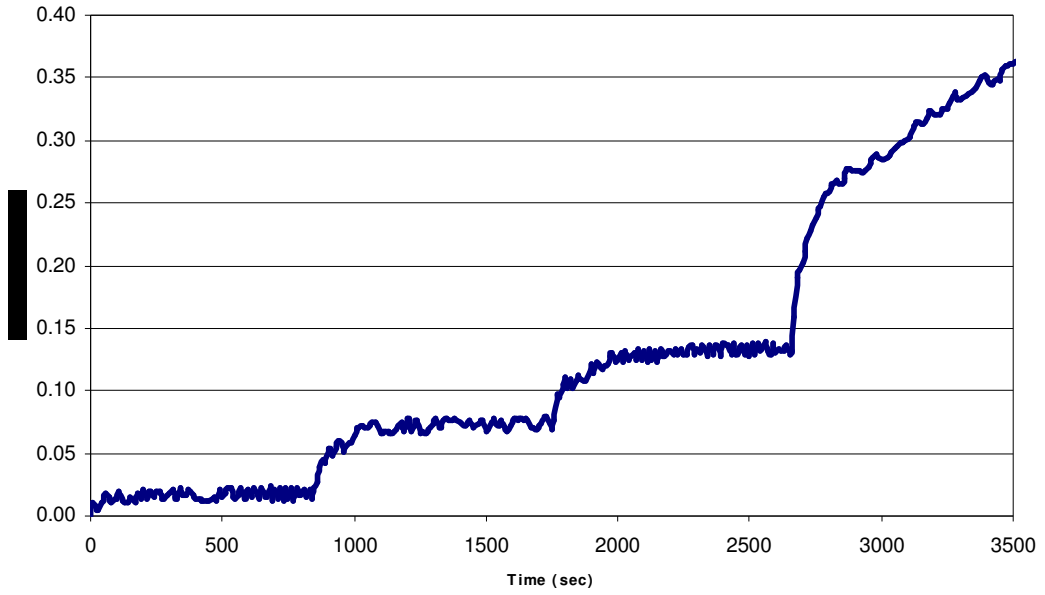
# 50/50% Blend

18 psi - Strip 3

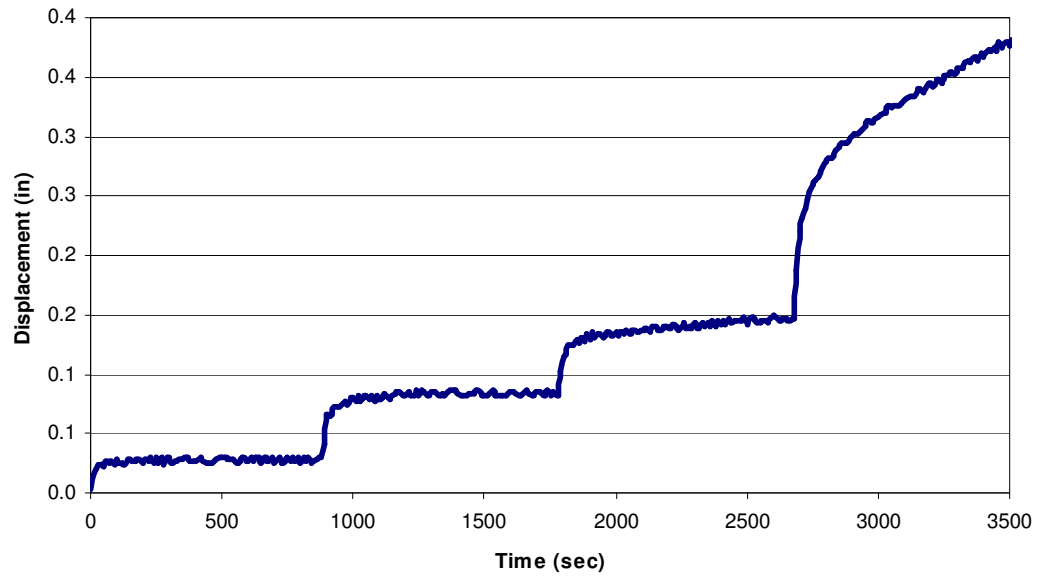


# A-3 Sand

6 psi - Strip 2



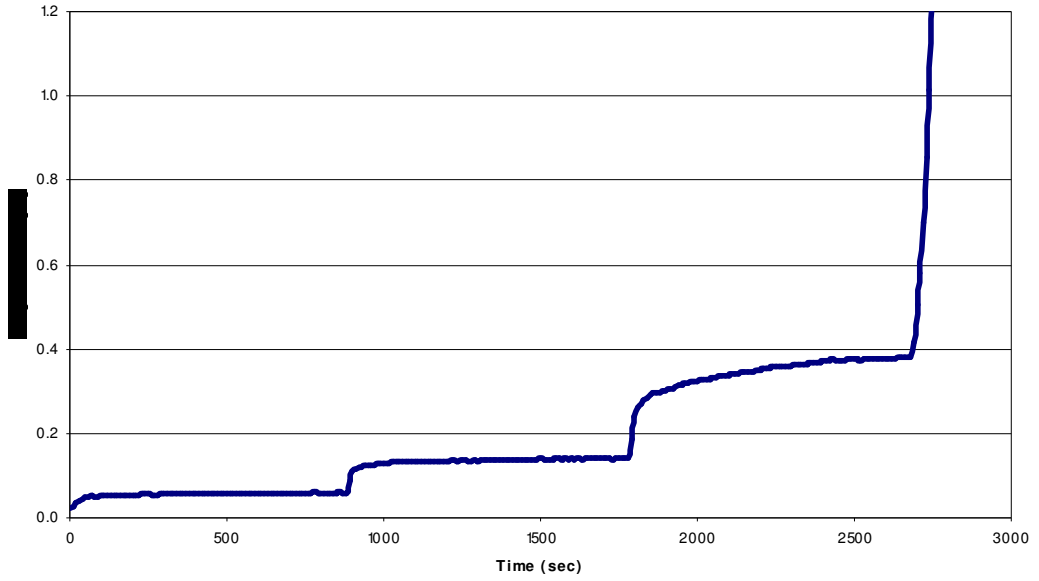
6 psi - Strip 3



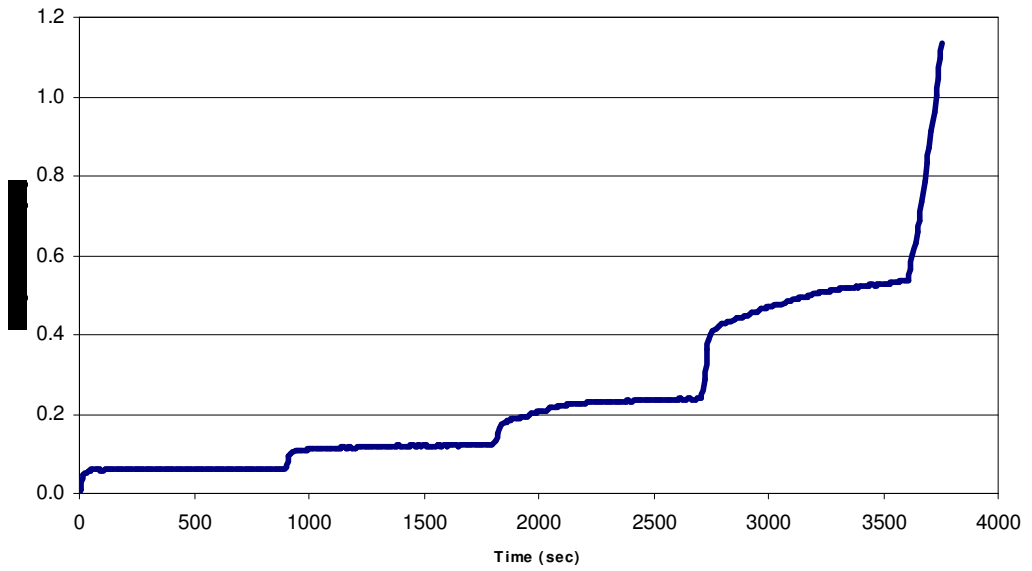
B-20

# A-3 Sand

12 psi - Strip 2



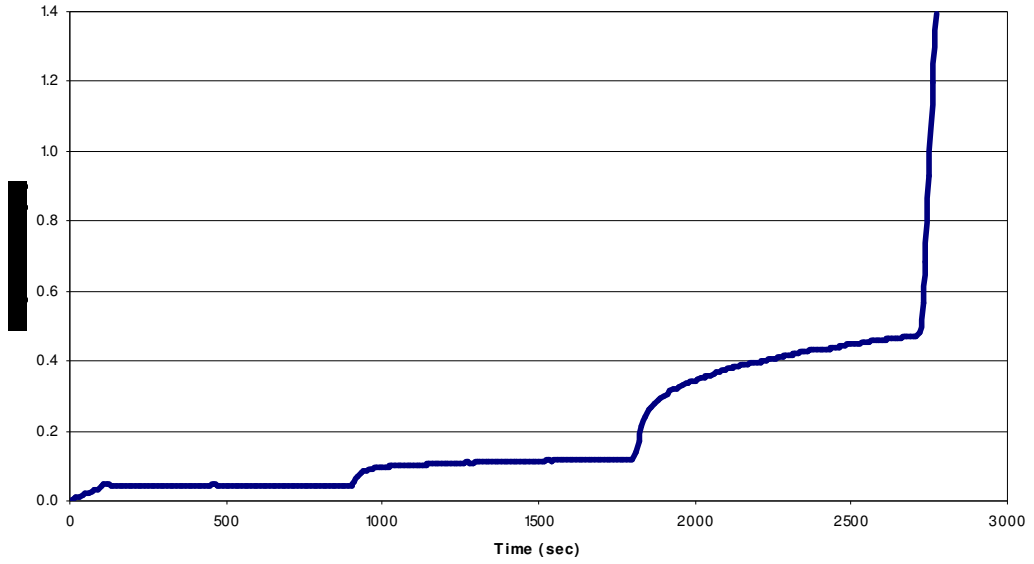
12 psi - Strip 3



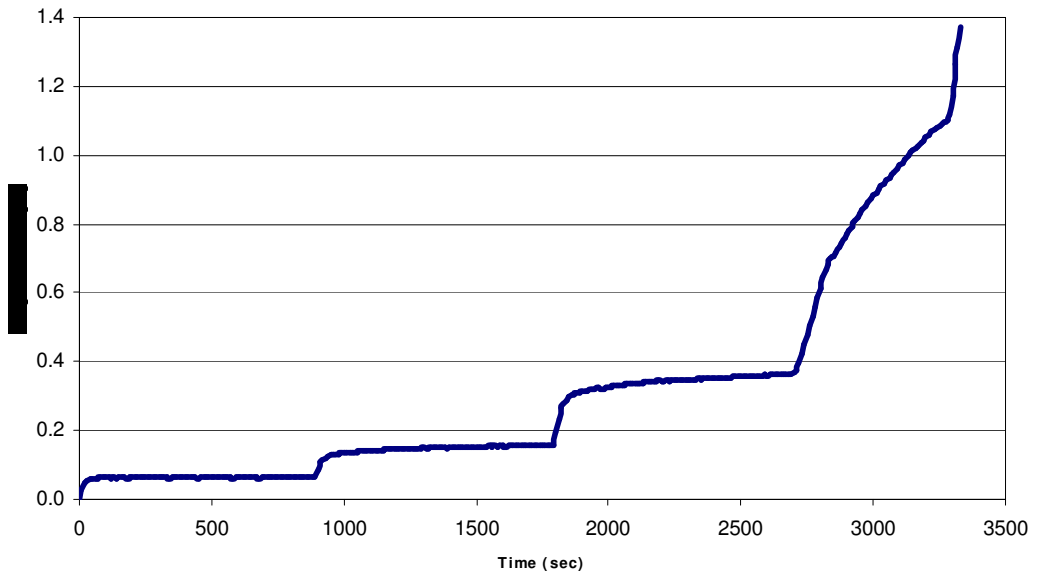
B-21

# A-3 Sand

18 psi - Strip 2



18 psi - Strip 3



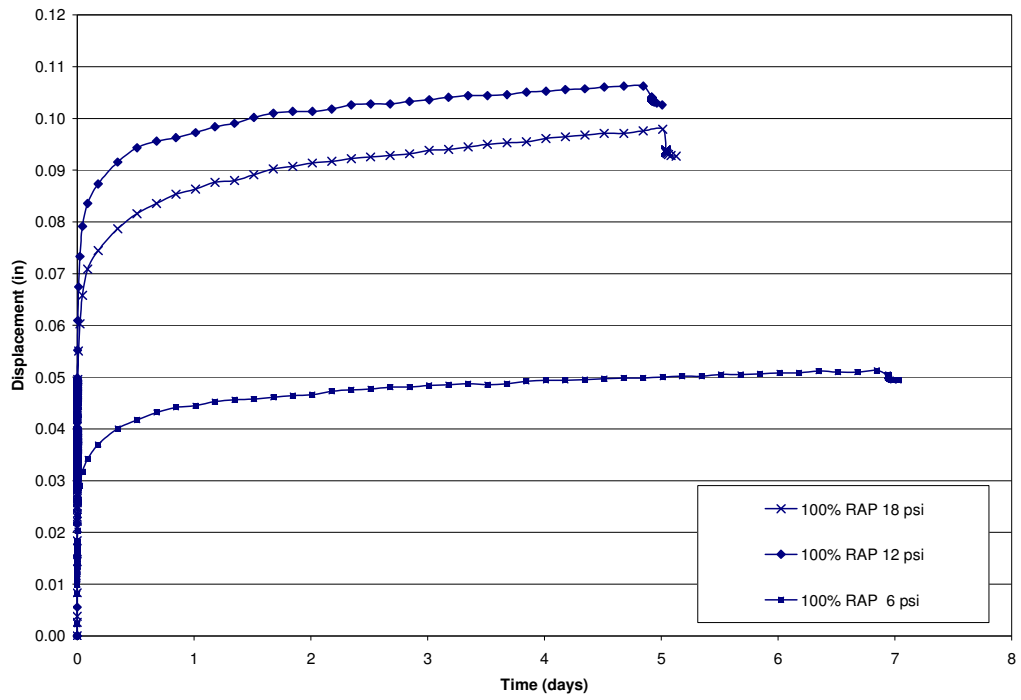
B-22



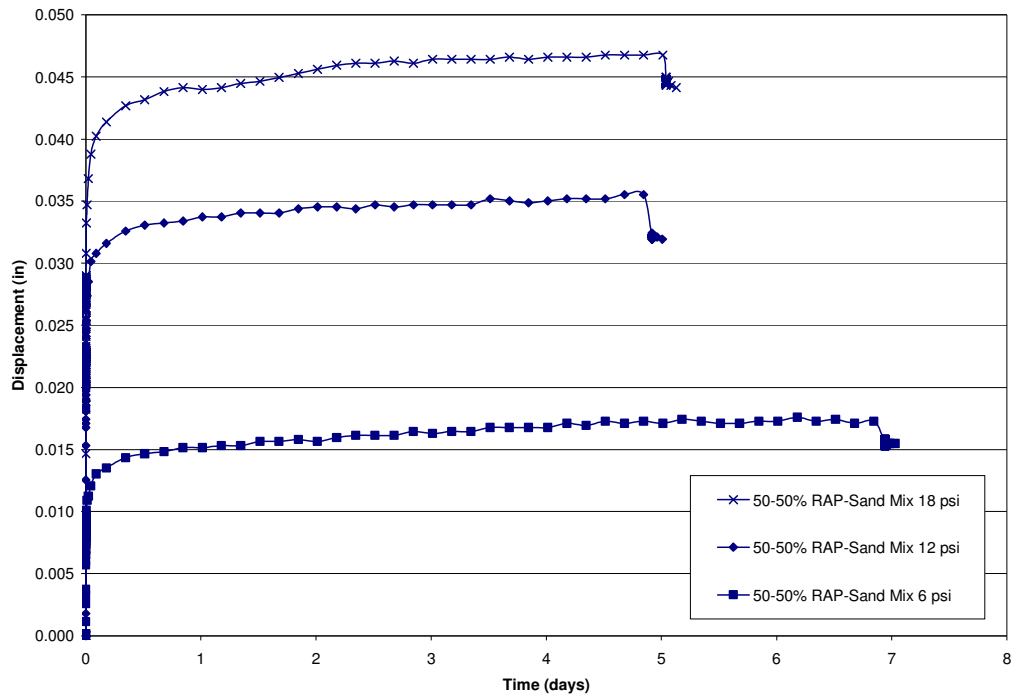
**Appendix B-3: FIT Small-Scale Vertical Creep Test Data - Displacement vs.  
Time Plots**



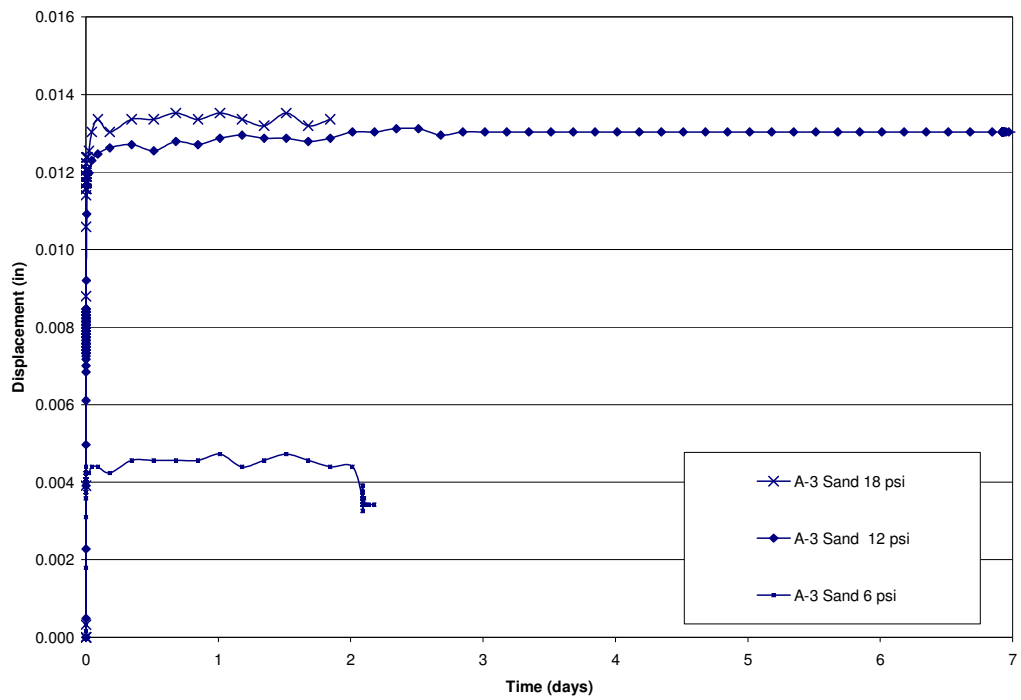
# 100% RAP



# 50/50% Blend

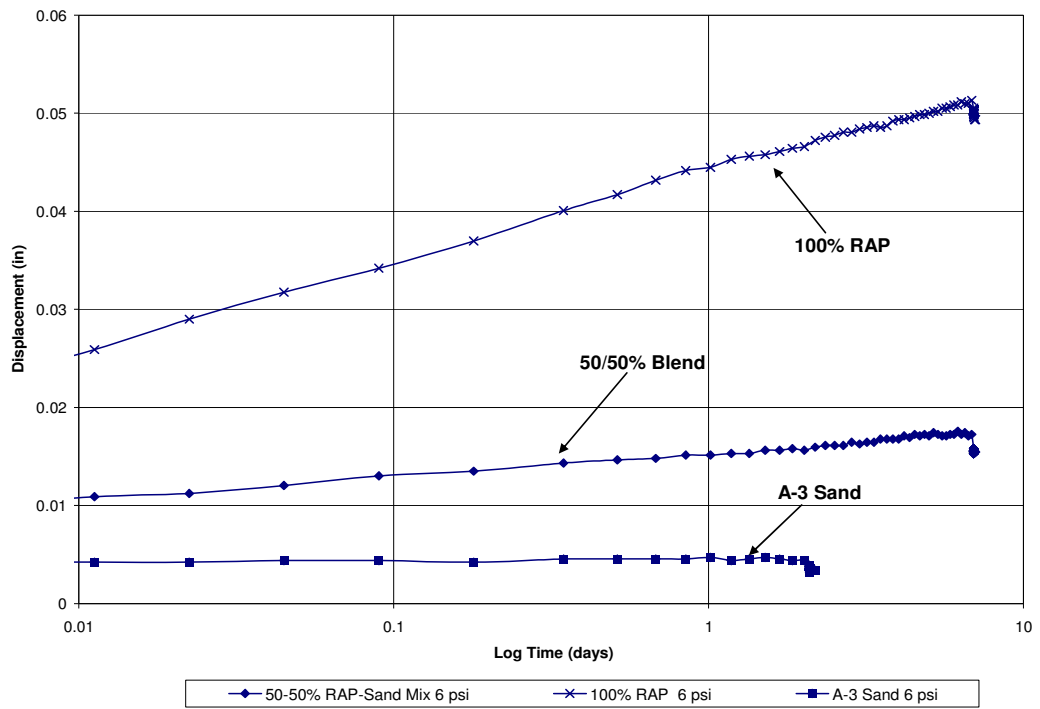


# A-3 Sand

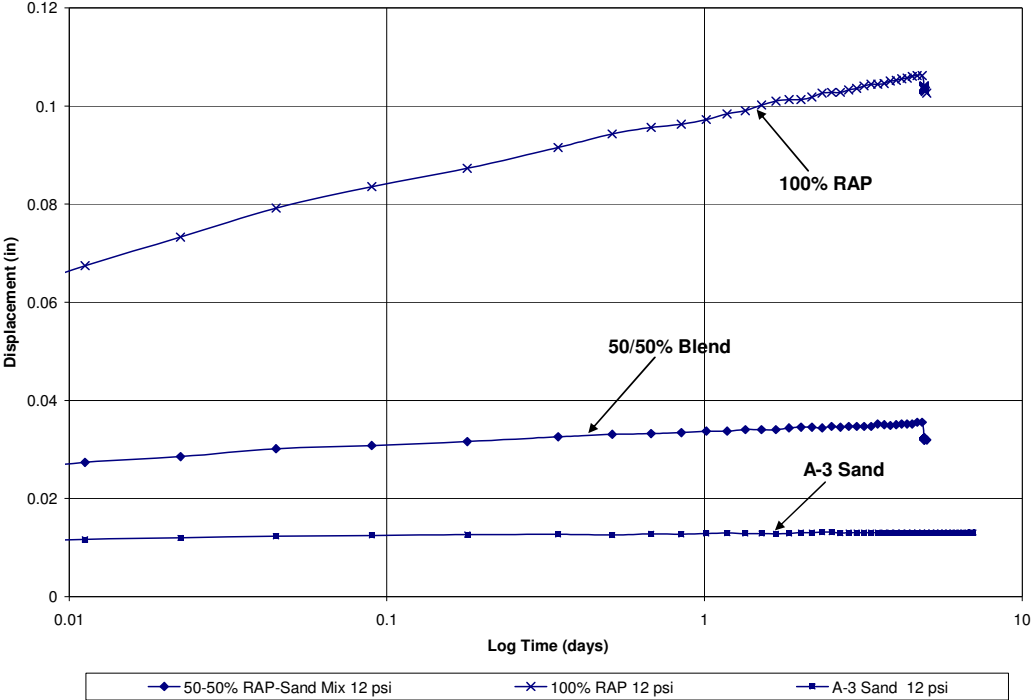


**Appendix B-4: FIT Small Scale Vertical Creep Test Data - Displacement vs.  
Logarithm Time Plots**

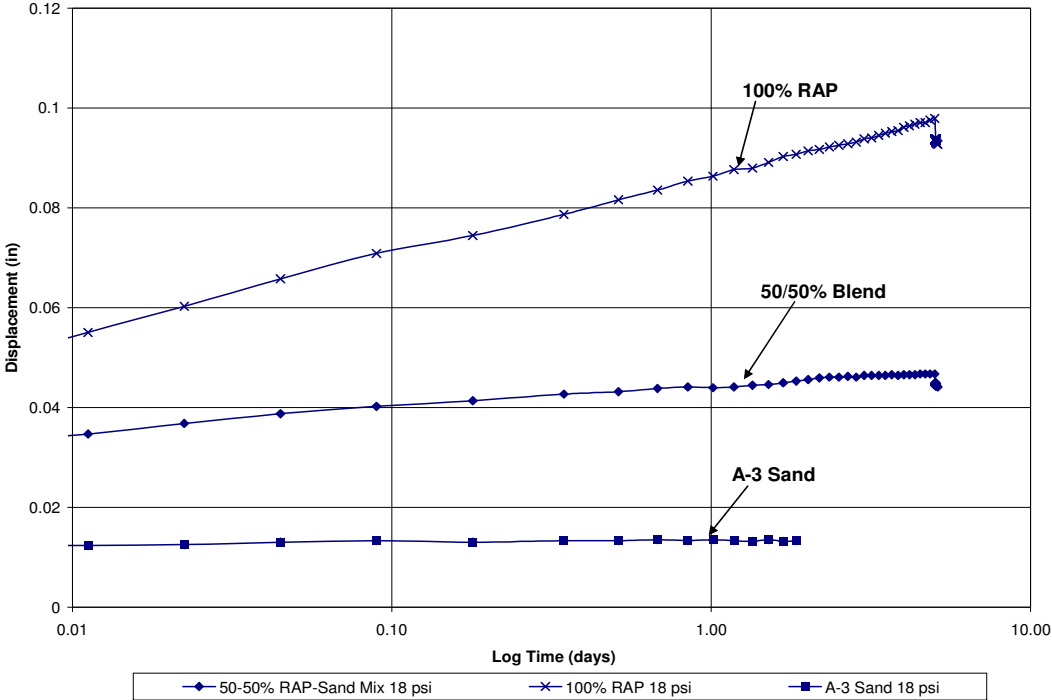
## 6 psi Applied Vertical Stress



# 12 psi Applied Vertical Stress



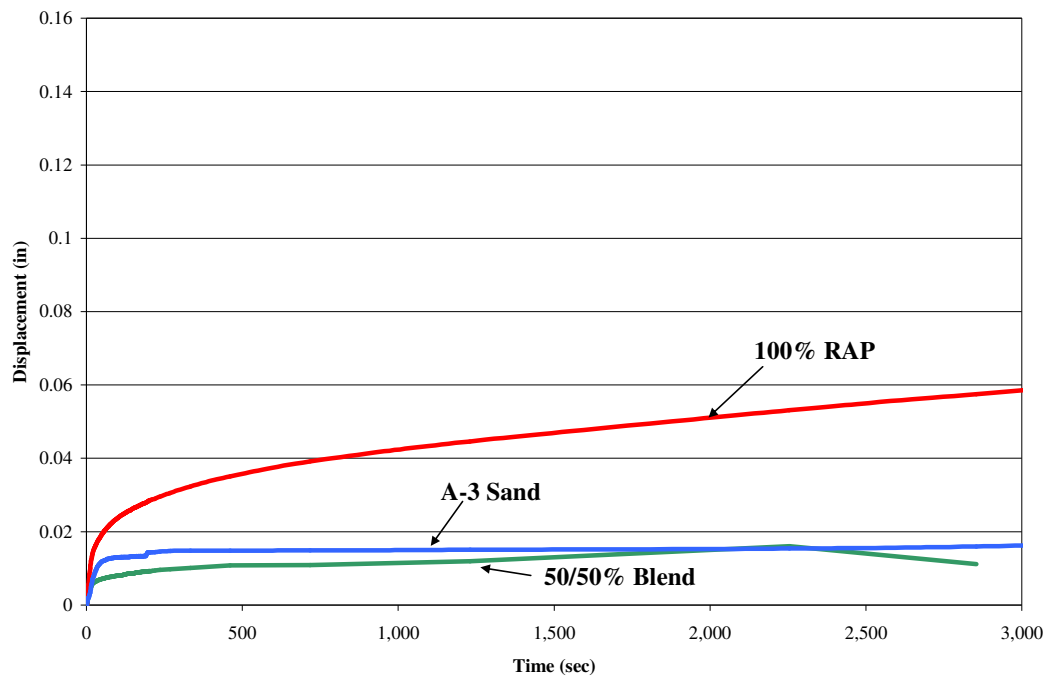
# 18 psi Applied Vertical Stress



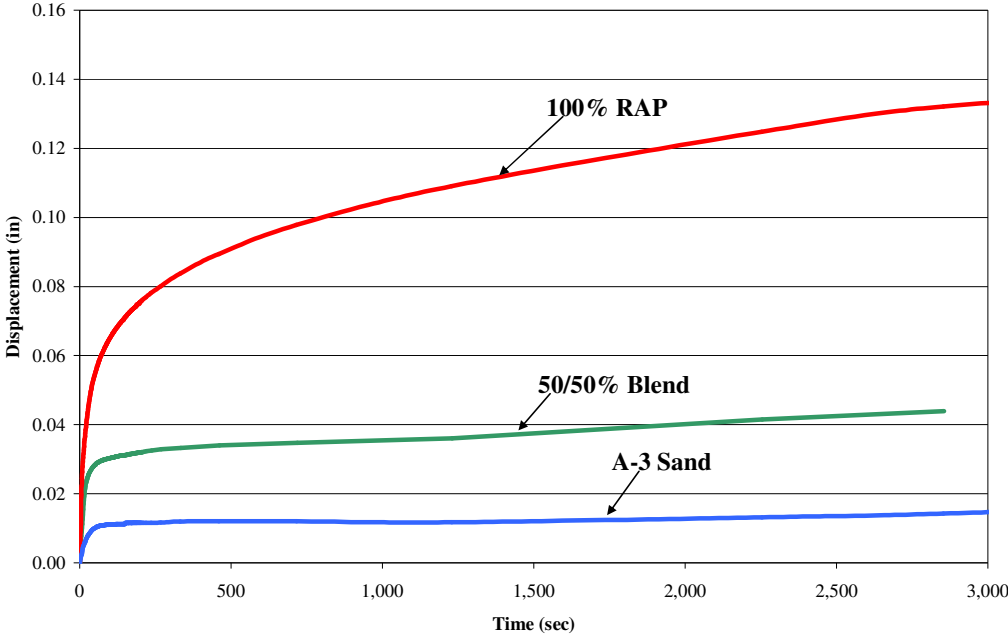
**Appendix B-5: 45 Minute Vertical Creep Test Data - Displacement vs. Time  
Plots**



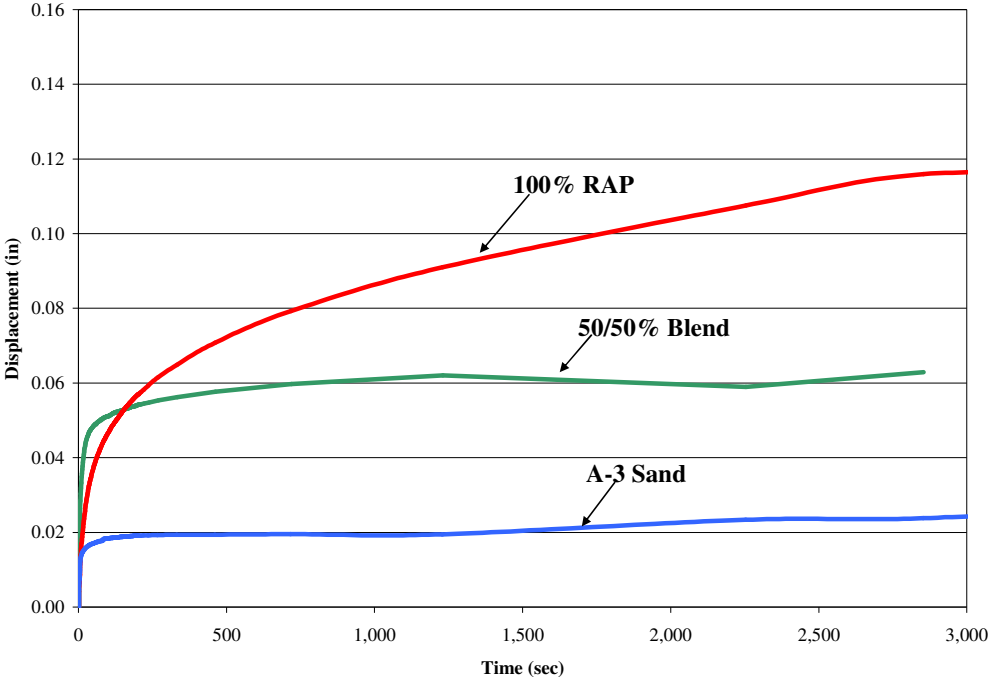
## 6 psi Applied Vertical Stress



# 12 psi Applied Vertical Stress



# 18 psi Applied Vertical Stress



**Appendix C-1: Statewide Pavement Composition Summary**

Region	County Group (Major Cities) Counties		Pavement Composition Reports				Milled RAP Laboratory Samples			Crushed RAP Laboratory		
			Number of Projects	Weighted Average Asphalt Content	Weighted Standard Deviation	Range	Number of Samples	Average Asphalt Content	Range	Number of Samples	Average Asphalt Content	Range
<b>NW</b>			<b>47</b>	<b>6.66</b>	<b>1.10</b>	<b>5.2 - 10.0</b>	<b>8</b>	<b>5.8</b>	<b>5.2 - 7.8</b>	<b>7</b>	<b>6.4</b>	<b>4.8 - 6.0</b>
	Ft. Walton Beach	Okaloosa, Walton, Holmes, Washington	8	6.07	0.10	5.9 - 6.3	5	6.5	6 - 7.3	2	5.3	5.2 - 5.5
	Marianna	Jackson	13	6.75	1.15	5.6 - 9.8	1	5.2	N/A	1	5.7	N/A
	Panama City	Bay, Gulf, Calhoun	9	6.90	1.59	5.2 - 10	1	7.8	N/A	2	5.4	4.8 - 6.0
	Pensacola	Escambia, Santa Rosa	17	6.69	0.62	5.6 - 7.9	1	6.5	N/A	2	5.2	4.8 - 5.5
<b>Tal/BB</b>	<b>BB</b>		<b>25</b>	<b>7.44</b>	<b>0.63</b>	<b>5.8 - 8.5</b>	<b>2</b>	<b>7.8</b>	<b>7.7 - 7.9</b>	<b>2</b>	<b>6.2</b>	<b>5.7 - 6.6</b>
	Perry	Taylor, Lafayette, Madison	16	7.38	0.52	6.5 - 7.9	0	N/A	N/A	0	N/A	N/A
	Tallahassee	Leon, Gadsden, Wakulla, Jefferson, Liberty, Franklin	9	7.55	0.80	5.8 - 8.5	2	7.8	7.7 - 7.9	2	6.2	5.7 - 6.6
<b>NE</b>	<b>NE</b>		<b>131</b>	<b>6.58</b>	<b>0.54</b>	<b>4.6 - 8.2</b>	<b>3</b>	<b>6.6</b>	<b>6.3 - 6.9</b>	<b>5</b>	<b>5.6</b>	<b>4.9 - 6.5</b>
	Fernandina Beach	Nassau	10	6.38	0.51	6.3 - 7.3	0	N/A	N/A	0	N/A	N/A
	Jacksonville	Duval	52	6.55	0.50	5.1 - 7.7	2	6.4	6.3 - 6.4	4	5.4	4.9 - 5.6
	Lake City	Columbia, Suwannee, Hamilton, Baker, Union	26	6.45	0.57	5.4 - 7.4	1	6.9	N/A	1	6.5	N/A
	Orange Park	Bradford, Clay	16	6.72	0.61	6.1 - 8.2	0	N/A	N/A	0	N/A	N/A
	Palatka	Putnam	12	6.85	0.56	5.4 - 7.5	0	N/A	N/A	0	N/A	N/A
	St Augustine	St. Johns, Flagler	15	7.32	0.46	4.6 - 7.3	0	N/A	N/A	0	N/A	N/A
<b>NC</b>	<b>NC</b>		<b>44</b>	<b>7.14</b>	<b>0.62</b>	<b>6.3 - 8.3</b>	<b>6</b>	<b>6.6</b>	<b>5.8 - 8.2</b>	<b>4</b>	<b>5.7</b>	<b>4.5 - 6.8</b>
	Chiefland	Levy, Dixie, Gilchrist	11	6.98	0.66	6.3 - 8	0	N/A	N/A	0	N/A	N/A
	Gainesville	Alachua	19	7.55	0.52	6.5 - 8.3	1	6.7	N/A	2	5.9	5.0 - 6.8
	Ocala	Marion	14	6.92	0.52	6.5 - 8	5	6.6	5.8 - 8.2	2	5.4	4.5 - 6.4
<b>WC</b>	<b>WC</b>		<b>52</b>	<b>6.67</b>	<b>0.43</b>	<b>5.9 - 8.2</b>	<b>7</b>	<b>6.4</b>	<b>5.6 - 6.8</b>	<b>7</b>	<b>5.5</b>	<b>5.5 - 5.8</b>
	Brooksville	Pasco, Citrus, Sumter, Hernando	12	6.34	0.49	5.9 - 7.8	1.5	6.1	N/A	1	5.6	N/A
	Tampa Bay	Hillsborough, Pinellas	15	6.78	0.31	6.2 - 7.2	1.5	6.7	N/A	1	4.7	N/A
	Sarasota	Sarasota, Manatee, Hardee, DeSoto	15	6.79	0.34	6.2 - 7.3	2	6.4	4.9 - 8	3	5.7	5.7 - 5.8
	Lakeland	Polk	10	6.76	0.56	6.2 - 8.2	2	6.3	6 - 6.7	2	5.5	5.5 - 5.6
<b>EC</b>	<b>EC</b>		<b>77</b>	<b>6.54</b>	<b>0.38</b>	<b>5.1 - 7.7</b>	<b>12</b>	<b>6.5</b>	<b>5.5 - 7.3</b>	<b>6</b>	<b>5.6</b>	<b>4.8 - 5.9</b>
	Daytona Beach	Volusia	19	6.63	0.32	6 - 7.1	6	6.7	6.2 - 7.3	0	N/A	N/A
	Melbourne	Brevard	15	6.33	0.46	5.1 - 7.4	2	5.9	5.5 - 6.3	2	5.7	5.5 - 5.9
	Leesburg	Lake	10	6.46	0.52	6 - 7.7	1	6.9	N/A	1	6.0	N/A
	Orlando	Orange, Seminole	23	6.65	0.23	5.9 - 7	3	6.3	5.9 - 6.7	3	5.4	4.8 - 5.9
	Kissimmee	Osceola	10	6.51	0.35	5.9 - 6.1	0	N/A	N/A	0	N/A	N/A
<b>SW</b>	<b>SW</b>		<b>26</b>	<b>7.40</b>	<b>0.56</b>	<b>6.8 - 9.1</b>	<b>4</b>	<b>7.1</b>	<b>5.8 - 8.5</b>	<b>5</b>	<b>10.2</b>	<b>6.4 - 7.4</b>
	Ft. Myers	Charlotte, Lee, Collier	18	7.59	0.54	6.8 - 9.1	4	7.1	5.8 - 8.5	5	6.7	6.4 - 7.4
	Sebring	Highlands, Glades, Hendry	8	7.32	0.46	6.9 - 8.1	0	N/A	N/A	0	N/A	N/A
<b>SE</b>	<b>SE</b>		<b>94</b>	<b>6.57</b>	<b>0.33</b>	<b>5.6 - 8.1</b>	<b>10</b>	<b>6.3</b>	<b>6.0 - 6.8</b>	<b>11</b>	<b>5.9</b>	<b>5.0 - 6.9</b>
	Ft. Lauderdale	Broward	28	6.67	0.28	6.1 - 7.6	1	n/a	N/A	0	N/A	N/A
	Ft. Pierce	Indian River, St Lucie, Okeechobee	15	6.37	0.27	6.1 - 7.3	4	6.2	6 - 6.4	3	5.3	5.0 - 5.5
	Stuart	Martin	10	6.46	0.35	5.8 - 7.1	0	N/A	N/A	0	N/A	N/A
	Miami	Miami-Dade, Monroe	22	6.79	0.32	6.1 - 8.1	3	6.5	6.1 - 6.8	7	6.1	5.4 - 6.9
	West Palm Beach	Palm Beach	19	6.45	0.29	5.6 - 7.2	3	6.4	6 - 6.6	1	6.00	N/A
<b>Total</b>			<b>496</b>	<b>6.78</b>	<b>0.65</b>	<b>4.9 - 10.0</b>	<b>52</b>	<b>6.6</b>	<b>4.9 - 8.5</b>	<b>47</b>	<b>5.8</b>	<b>4.5 - 7.4</b>

**Appendix C-2: Asphalt Content Summary Data - Crushed and Milled**

9	Land Lakeland #9	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	Crushed	100.00	99.67	95.79	76.75	57.02	46.01	38.15	27.93	15.54	8.70	5.75
11	APAC (WH) #11	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	Crushed	98.91	97.83	85.12	70.88	61.38	53.73	38.99	18.21	11.88	10.18	5.47
13	Ajax Odessa #13	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	Crushed	100.00	98.88	95.55	76.57	60.01	49.04	41.29	32.46	18.77	10.31	5.55
15	APAC Tampa #15	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	Crushed	100.00	97.68	93.43	75.42	62.21	52.43	44.28	33.99	16.39	10.46	4.70
18	APAC Sarasota #18	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	Crushed	100.00	98.67	93.48	71.70	54.67	43.17	35.67	26.71	13.35	5.61	5.62
20	Ajax Sarasota FDOT #20	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	Crushed	100.00	99.59	97.66	84.20	68.38	57.75	48.64	35.86	17.24	6.66	5.75
21	Ajax Sarasota Com #21	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	Crushed	100.00	99.57	97.18	76.08	58.89	49.50	41.73	31.03	14.19	5.43	5.69
22	APAC (WS) #22	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	Crushed	100.00	94.85	88.25	68.86	55.11	46.69	40.50	32.10	16.87	9.58	4.80
24	OPC Oviedo #24	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	Crushed	100.00	100.00	97.65	78.27	59.87	49.47	42.78	34.26	19.68	11.93	5.91
26	OPC Taft #26	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	Crushed	100.00	96.36	92.86	69.83	52.47	42.81	36.14	28.61	16.40	9.95	5.63
28	Ranger Ft. Pierce #28	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	Crushed	100.00	98.81	96.02	80.16	62.90	51.86	43.75	31.43	16.41	7.79	5.22

30	Dickerson Ft.Pierce #30	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	Crushed	100.00	98.85	96.92	80.27	65.78	55.20	47.48	33.58	18.56	10.03	5.50
32	Community Vero Beach #32	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	Crushed	100.00	99.39	95.07	74.92	61.68	52.16	45.40	32.08	16.23	8.33	5.15
35	Whitehurst Crushed Comm #35	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
		99.50	97.11	92.17	74.25	56.86	44.97	37.52	24.91	11.72	6.52	5.00
36	Whitehurst Crushed-DOT #36	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
		100.00	99.59	97.56	84.03	70.55	60.10	48.40	31.10	16.05	10.72	6.81
39	AC-LC Crushed #39	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
		100.00	99.21	96.02	77.79	59.44	50.48	43.64	33.18	18.95	10.12	6.48
43	APAC-Jax Crushed A #43	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
		100.00	98.00	93.27	77.63	60.96	49.31	41.08	33.24	19.28	10.46	5.47
44	APAC-Jax Crushed B #44	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
		100.00	98.31	94.54	81.14	64.90	52.75	43.94	35.13	19.72	10.76	5.60
45	ACA Crushed #45	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
		100.00	98.57	91.86	70.17	52.47	41.21	33.87	27.81	16.74	8.33	5.61
48	Duval Crushed #48	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
		100.00	97.73	94.58	78.97	62.90	52.09	44.74	37.52	19.35	7.57	4.86
49	SCI Crushed #49	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
		100.00	99.04	83.16	67.19	56.23	46.66	31.91	18.59	11.95	10.25	6.36
51	AC Ocala Crushed #51	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
		100.00	95.55	87.44	66.25	49.90	37.76	30.58	22.79	12.36	7.05	4.28
56	Middlesex Crushed #56	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
		100.00	99.82	97.70	78.17	59.76	49.84	42.79	32.31	17.71	10.56	5.98



4	P&S (A) #4	Percent Passing Sieves									
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
	Milled	100.00	97.11	88.68	46.18	31.74	26.74	23.03	18.62	11.65	6.75
5	P&S (A) #5	Percent Passing Sieves									
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
	Milled	100.00	92.06	81.06	46.05	27.99	21.56	18.01	14.49	9.84	6.62
6	P&S (A) #6	Percent Passing Sieves									
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
	Milled	100.00	99.76	99.46	91.01	79.75	67.76	49.84	32.71	13.75	7.15
7	Ranger Debary #7	Percent Passing Sieves									
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
	Milled	100.00	99.39	95.95	70.90	52.83	43.87	37.60	29.88	17.45	10.80
8	Land Lakeland #8	Percent Passing Sieves									
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
	Milled	100.00	99.61	96.23	75.19	55.86	46.92	40.01	28.75	13.89	9.07
10	APAC (WH) #10	Percent Passing Sieves									
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
	Milled	100.00	99.12	96.64	81.65	67.28	57.81	50.45	37.33	18.96	11.03
12	Ajax Odessa #12	Percent Passing Sieves									
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
	Milled	100.00	98.99	97.71	78.83	61.11	50.45	42.37	31.42	16.42	8.71
14	APAC Tampa #14	Percent Passing Sieves									
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
	Milled	100.00	99.70	98.12	88.93	76.22	62.95	48.25	32.66	18.27	12.15
16	DAB #16	Percent Passing Sieves									
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
	Milled	100.00	94.60	91.32	73.47	60.00	51.50	44.18	32.77	18.95	13.13
17	APAC Sarasota #17	Percent Passing Sieves									
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
	Milled	100.00	99.54	97.61	80.71	64.74	53.29	42.97	31.93	17.61	8.50
19	Ajax Sarasota #19	Percent Passing Sieves									
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
	Milled	100.00	99.63	98.46	86.44	73.08	62.80	53.18	39.00	17.59	7.51

23	APAC (WS) #23	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	Milled	100.00	98.36	95.80	78.22	62.54	53.00	45.67	34.53	17.24	9.93	6.72
25	OPC Oviedo #25	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	Milled	100.00	99.80	96.96	78.94	60.82	50.95	43.87	34.95	20.00	11.79	5.89
27	OPC Taft #27	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	Milled	100.00	98.14	93.91	80.65	63.59	50.42	40.92	29.97	18.16	12.95	6.29
29	Ranger Ft.Pierce #29	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	Milled	100.00	100.00	98.97	76.67	58.55	48.21	41.41	31.57	18.21	9.92	6.35
31	Dickerson M-Ft.Pierce #31	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	#31	100.00	99.21	96.12	72.86	54.62	47.11	41.90	30.85	16.72	8.89	6.44
33	Community Vero Beach #33	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	Milled-1	100.00	99.11	97.44	81.28	64.59	56.04	50.24	38.12	23.12	12.05	6.03
34	Community VB M-B #34	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	#34	100.00	99.65	96.40	68.19	48.67	41.54	37.19	26.10	13.25	8.36	6.13
37	APAC-Melb Milled 2 #37	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	#37	98.78	96.33	93.72	74.53	55.52	44.04	36.99	26.66	14.59	7.86	6.28
38	USF Milled #38 *	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	#38 *	100.00	97.87	95.60	79.55	63.70	52.06	42.81	31.76	15.95	7.64	6.56
40	AC-LC Milled #40	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	#40	100.00	99.84	98.46	82.25	66.41	55.53	47.15	35.28	20.27	11.26	6.94

41	Halifax Milled 95	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	#41	100.00	99.06	97.59	73.26	51.62	42.03	36.43	28.61	14.53	8.76	6.74

42	Halifax Milled A1A	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	#42	100.00	99.05	97.20	84.74	70.80	59.64	46.99	31.23	14.65	8.55	7.34

46	ACA Milled A	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	#46	100.00	98.54	96.34	71.74	51.96	40.81	34.06	28.49	16.21	7.85	6.21

47	ACA Milled B	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	#47	100.00	99.38	97.39	80.50	64.45	54.08	46.25	38.62	21.19	9.63	6.42

50	SCI Milled	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	#50	100.00	99.21	98.46	83.75	67.92	57.39	48.75	33.96	19.09	11.85	6.60

52	AC Ocala Milled 492	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	#52	100.00	98.47	90.39	78.20	65.37	51.34	31.88	17.91	12.63	10.80	6.54

53	AC Ocala Milled 40	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	#53	100.00	99.23	96.33	77.61	62.39	52.12	42.46	31.26	17.36	11.11	6.13

54	AC Ocala Milled 75	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	#54	100.00	99.82	94.86	45.86	30.14	24.85	21.06	16.13	10.13	6.89	5.75

55	ACA Ocala Milled 441	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	#55	100.00	99.33	98.06	78.00	59.48	45.81	36.23	28.25	18.04	10.99	8.21

57	Middlesex Milled	Percent Passing Sieves										
		3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	% AC
	#57	100.00	99.38	98.11	80.40	63.72	54.10	45.90	34.73	19.62	11.69	6.90

**Appendix C-3: Sieve Analysis Summary Data – Crushed and Milled**

Sieve #	Sieve Opening (mm)	APAC Melbourne crushed stockpile Average Percent Finer
1.5"	37.5	100%
0.75"	19	100%
0.25"	6.3	81%
4	4.75	72%
8	2.36	53%
10	2	49%
16	1.18	39%
20	0.85	33%
30	0.6	26%
40	0.425	19%
50	0.30	13%
100	0.15	3%
200	0.075	1%
Pan	-	0%

Sieve #	Sieve Opening (mm)	Goodson AJAX-Sarasota Paving, Cocoa, FL	
		Average Percent Finer	Average Percent Finer
1.5"	37.5	100%	100%
1"	25	100%	100%
3/4"	19	100%	100%
1/2"	12.5	98%	99%
3/8"	9.5	85%	94%
1/4"	6.3	59%	79%
#4	4.75	44%	70%
#8	2.36	24%	51%
#10	2	22%	47%
#16	1.18	16%	36%
#20	0.85	14%	29%
#30	0.6	12%	22%
#40	0.425	9%	15%
#50	0.3	7%	10%
#100	0.15	2%	2%
#140	0.106	1%	1%
#200	0.075	0%	0%
Pan	-	0%	0%





















		APAC Melbourne	APAC Melb.	P&S Paving #1	P&S Paving #2	P&S Paving #3	Ranger-Debary
Sieve #	Sieve Opening (mm)	Average Percent Finer	Average Percent Finer	Average Percent Finer	Average Percent Finer	Average Percent Finer	Average Percent Finer
1.5"	37.5	100.0%	100.0%	100.0%	100.0%	95.2%	100.0%
1"	25	97.6%	96.9%	99.4%	99.2%	89.5%	98.2%
3/4"	19	90.1%	90.9%	94.7%	98.6%	80.6%	93.6%
1/2"	12.5	80.9%	82.1%	84.9%	85.7%	68.5%	87.8%
3/8"	9.5	71.3%	71.7%	68.8%	71.0%	57.0%	76.1%
1/4"	6.3	56.0%	55.3%	40.9%	46.2%	40.3%	54.9%
#4	4.75	46.0%	45.0%	29.4%	33.9%	32.1%	42.5%
#8	2.36	28.5%	27.2%	15.6%	16.2%	18.9%	25.3%
#10	2.00	25.9%	24.5%	14.2%	14.1%	17.3%	23.0%
#16	1.18	18.1%	16.6%	10.6%	8.0%	13.5%	17.5%
#20	0.85	14.7%	13.3%	9.1%	6.9%	11.7%	14.9%
#30	0.60	10.7%	9.8%	7.6%	5.6%	9.4%	12.0%
#40	0.425	7.1%	6.7%	6.1%	4.4%	7.1%	9.0%
#50	0.300	4.5%	4.5%	5.0%	3.7%	5.1%	6.7%
#100	0.150	1.1%	1.2%	2.2%	2.0%	1.5%	2.2%
#140	0.106	0.6%	0.7%	1.3%	1.3%	0.8%	1.1%
#200	0.075	0.3%	0.4%	0.9%	1.1%	0.6%	0.7%
Pan2	-	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%

		Ajax-Sarasota	APAC-Sarasota	APAC- Tampa	APAC-WS	DAB
Sieve #	Sieve Opening (mm)	Average Percent Finer	Average Percent Finer	Average Percent Finer	Average Percent Finer	Average Percent Finer
2"	50	100.0%	100.0%	95.7%	100.0%	100.0%
1"	25	94.5%	92.9%	89.6%	93.8%	94.1%
3/4"	19	91.9%	89.9%	80.5%	85.2%	90.1%
1/2"	12.5	87.0%	83.6%	69.2%	71.8%	84.0%
3/8"	9.5	81.8%	76.3%	61.0%	60.9%	75.2%
1/4"	6.3	70.7%	61.1%	49.9%	46.1%	60.5%
#4	4.75	62.6%	51.8%	43.3%	36.6%	51.3%
#8	2.36	45.8%	34.7%	30.0%	20.9%	34.7%
#10	2.00	43.1%	31.7%	27.7%	18.6%	31.9%
Pan1	2.00	43.1%	31.7%	27.7%	18.6%	31.9%
#16	1.18	34.6%	23.0%	20.9%	13.1%	24.4%
#20	0.85	30.5%	18.7%	17.3%	10.9%	20.4%
#30	0.60	25.6%	14.1%	13.1%	8.9%	15.9%
#40	0.425	20.2%	9.8%	9.2%	6.3%	11.0%
#50	0.300	14.0%	6.3%	6.1%	4.0%	7.1%
#100	0.150	3.1%	1.8%	2.1%	1.3%	2.5%
#140	0.106	1.1%	1.0%	1.3%	0.8%	1.6%
#200	0.075	0.4%	0.6%	0.8%	0.5%	1.1%
Pan2	-	0.0%	0.0%	0.0%	0.0%	0.0%

		Ajax-Odessa	APAC WH
Sieve #	Sieve Opening (mm)	Average Percent Finer	Average Percent Finer
1.5"	37.5	100.0%	99.0%
1"	25	98.1%	88.7%
3/4"	19	93.2%	79.0%
1/2"	12.5	83.5%	66.9%
3/8"	9.5	74.8%	56.4%
1/4"	6.3	59.9%	41.5%
#4	4.75	49.6%	33.0%
#8	2.36	30.8%	18.4%
#10	2.00	27.9%	16.4%
Pan1	2.00	27.9%	16.4%
#16	1.18	20.3%	11.8%
#20	0.85	17.0%	9.9%
#30	0.60	13.3%	8.1%
#40	0.425	9.6%	6.5%
#50	0.300	6.7%	5.2%
#100	0.150	2.0%	2.4%
#140	0.106	1.0%	1.6%
#200	0.075	0.8%	1.4%
Pan	-	0.0%	0.0%



**Appendix C-4: Dry Rodded Unit Weight Summary Data – Crushed and Milled**

Site No.	Location	Crushed	Milled
		Average Bulk Density (pcf)	Average Bulk Density (pcf)
1	ACA JAX #1	99.0	87.2
2	ACA JAX #2	---	93.7
3	AC-Holt 10	---	90.9
4	AC-Holt 85	---	86.7
5	AC-Holt 90	---	87.4
6	AC-Mar	97.1	---
7	Anderson Columbia - Lake City	98.8	87.6
8	AC-OC 75	---	93.6
9	AC-OC 492	100.5	91.1
10	AC-Ocala 40	---	94.4
11	AC-Ocala 441	---	82.6
12	Anderson Columbia - Panama City	99.6	92.0
13	Ajax - DOT PG	82.1	---
14	AJAX - Odessa	92.8	94.8
15	Ajax - PG	---	82.9
16	AJAX - Sarasota	97.0	100.8
17	AJAX - Sarasota Commercial	83.3	---
18	Ajax- Comm PG	80.4	---
19	Ajax- Ft. Myers	85.2	---
20	APAC - DFS	97.4	90.9
21	Apac - Miami	99.5	88.3
22	APAC - Naples	86.1	87.7
23	APAC - Pen	99.9	93.0
24	APAC - Sarasota	97.5	92.9
25	APAC - Tampa	---	95.1
26	APAC - Winter Haven	98.8	89.0
27	APAC - Winter Springs	91.3	91.9
28	APAC JAX	98.8	---
29	APAC JAX #2	95.6	---
30	APAC MELB CrushedRAP 100%	96.5	---
31	Better Road-PG	82.8	---
32	Better Roads - PG	87.7	---
33	Comm Miami Private	90.3	---

Site No.	Location	Crushed	Milled
		Average Bulk Density (pcf)	Average Bulk Density (pcf)
34	Comm. - Miami	92.1	86.1
35	Community Asphalt - Vero Beach #1	99.0	100.7
36	Community Asphalt - Vero Beach #2	---	92.2
37	CRW - Freeport	102.1	99.3
38	CWR - PC	99.8	---
39	CWR - Tally	92.0	75.4
40	DAB	---	96.0
41	Dickerson - Ft. Pierce	97.1	93.5
42	Duval	100.2	---
43	General Miami 1	89.9	91.4
44	General Miami 2 (0.5)	92.7	---
45	General Miami 2 (0.75)	88.9	---
46	General Miami 2 (1.25)	92.5	---
47	Halifax 95	---	89.5
48	Halifax A1A	---	90.9
49	Hipp	---	85.1
50	Lane - Lakeland	91.3	---
51	MB - Tally	94.3	85.0
52	Middlesex - Leesburg	98.2	92.7
53	OPC - Oviedo	91.3	91.3
54	OPC - Taft	97.4	93.4
55	P&S #1	---	85.8
56	P&S #2	---	81.1
57	Ranger South	90.4	---
58	Ranger-Ft.Pierce	98.4	89.4
59	Ranger WPB 710	---	87.2
60	Ranger WPB LWR	---	92.4
61	Santa Rosa	94.3	---
62	SCI-Ocala	95.2	95.3
63	USF Largo	---	94.1
64	Whitehurst - Gainesville	94.5	95.7
65	Whitehurst-Gainesville	97.3	---

**Appendix C-5: Moisture Density Summary Data – Crushed and Milled**

Moisture Content Interval			0%			2%			4%			6%			8%			10%			12%		
Sample Description			Moisture Content	Wet Density (lb/ft <sup>3</sup> )	Dry Density (lb/ft <sup>3</sup> )	Moisture Content	Wet Density (lb/ft <sup>3</sup> )	Dry Density (lb/ft <sup>3</sup> )	Moisture Content	Wet Density (lb/ft <sup>3</sup> )	Dry Density (lb/ft <sup>3</sup> )	Moisture Content	Wet Density (lb/ft <sup>3</sup> )	Dry Density (lb/ft <sup>3</sup> )	Moisture Content	Wet Density (lb/ft <sup>3</sup> )	Dry Density (lb/ft <sup>3</sup> )	Moisture Content	Wet Density (lb/ft <sup>3</sup> )	Dry Density (lb/ft <sup>3</sup> )	Moisture Content	Wet Density (lb/ft <sup>3</sup> )	Dry Density (lb/ft <sup>3</sup> )
Sample Number	Facility Name & Location	Type & Stockpile																					
4	P&S	Milled #1	0.12%	106.47	106.34				3.86%	114.20	109.95	6.24%	119.33	112.32									
8	Lane - Lakeland	Milled	0.16%	109.07	108.90	3.05%	116.16	112.72	3.87%	117.87	113.48	5.64%	120.20	113.78	7.52%	121.60	113.09	9.95%	130.01	118.25			
10	APAC-WH	Milled	0.20%	107.47	107.25				4.04%	118.73	114.12	5.94%	123.20	116.30	7.74%	126.29	117.22	10.07%	127.75	116.06			
12	Ajax - Odessa	Milled	0.15%	109.67	109.51	2.38%	116.20	113.50	3.52%	118.13	114.12	5.94%	120.80	114.03	7.88%	123.33	114.32	9.57%	127.67	116.52			
14	APAC - Tampa	Milled	0.16%	107.49	107.32	2.14%	114.58	112.17	3.97%	119.67	115.10	6.01%	123.01	116.04	8.23%	124.19	114.75	10.00%	129.24	117.49			
16	DAB	Milled	0.10%	113.35	113.24				3.90%	122.41	117.82	6.01%	127.16	119.95	7.10%	129.53	120.94	9.88%	132.49	120.58			
17	APAC - Sarasota	Milled	0.19%	100.89	100.70	1.92%	107.58	105.56	3.96%	113.69	109.37	5.80%	114.54	108.19	7.91%	117.97	109.32						
19	Ajax - Sarasota	Milled	0.16%	111.13	110.95	0.86%	114.71	113.72	4.10%	123.17	118.32	5.68%	125.52	118.77	7.85%	127.52	118.24	10.15%	130.09	118.10			
23	APAC - WS	Milled	0.11%	103.52	103.41				3.73%	113.68	109.59	6.37%	114.76	107.89	8.62%	118.08	108.70						
25	OPC - Oviedo	Milled	0.10%	108.99	108.88				3.73%	120.16	115.84	5.97%	124.68	117.66	8.05%	126.04	116.65						
27	OPC - Taft	Milled	0.20%	111.96	111.74				3.18%	119.71	116.02	6.07%	125.53	118.35	7.66%	125.43	116.50						
29	Ranger - Ft.P	Milled	0.24%	104.15	103.90				3.95%	108.37	104.26	4.81%	110.21	105.15	7.47%	118.96	110.69	8.65%	117.77	108.39			
31	Dickerson	Milled	0.04%	105.80	105.75				3.94%	118.29	113.81	5.94%	122.43	115.57	7.45%	122.57	114.07	10.10%	128.60	116.80			
33	CommAsph - VB (CAVB)	Milled #1	0.37%	110.60	110.19				3.46%	122.44	118.35	5.70%	125.87	119.08	8.11%	129.27	119.57	9.93%	132.49	120.53			
35	Whitehurst	Crushed	0.12%	107.68	107.55				3.92%	120.61	116.07	5.98%	124.64	117.60	7.86%	126.97	117.72						
36	Whitehurst	Crushed - DOT	0.06%	113.01	112.94				3.72%	125.28	120.79	6.06%	127.20	119.93	7.94%	128.92	119.44	11.00%	133.03	119.84			
40	AC - LC	Milled	0.02%	103.99	103.96				4.56%	114.80	109.80	6.02%	121.79	114.87	7.84%	122.01	113.15						
41	Halifax	Milled - 95	0.14%	105.59	105.44				4.01%	113.64	109.26	6.36%	114.81	107.95	7.94%	116.04	107.50						
46	ACA - Jax	Milled #1	0.18%	109.16	108.97				4.23%	117.52	112.75	5.90%	121.47	114.70	7.82%	123.59	114.62	9.42%	122.96	112.37			
47	ACA - Jax	Milled #2	0.22%	106.56	106.33				4.40%	112.47	107.73	6.40%	117.51	110.44	8.40%	119.31	110.06						
50	SCI	Milled	0.16%	108.28	108.11				4.00%	118.92	114.34	5.95%	124.15	117.18	7.99%	125.40	116.12						
55	AC - Ocala	Milled - 441	0.08%	102.73	102.65				4.08%	108.68	104.42	6.40%	112.80	106.01	8.10%	109.65	101.43						
57	Middlesex - Leesburg	Milled	0.11%	107.35	107.22				3.93%	118.80	114.31	5.92%	122.24	115.41	8.46%	125.40	115.62				12.00%	130.24	116.29
59	General - Miami	Milled	0.11%	107.04	106.93				4.28%	114.71	110.00	6.16%	116.52	109.76	7.99%	120.68	111.75	10.30%	122.88	111.41			
62	CommAsph - Miami	Milled	0.16%	106.16	105.99				3.99%	117.36	112.85	5.89%	119.63	112.97	7.44%	120.41	112.08	9.47%	123.85	113.14			
63	Whitehurst - Pit	Milled	0.20%	108.37	108.16				4.61%	118.04	112.84	5.79%	120.85	114.24	7.00%	121.37	113.43	10.73%	125.87	113.67			
66	Ajax - PG	Milled	0.02%	102.53	102.51				3.37%	112.67	109.00	5.42%	115.47	109.53	7.20%	114.59	106.89						
69	Better Rd - PG	Milled	0.08%	102.68	102.60				3.97%	112.59	108.29	5.74%	114.48	108.26	7.31%	116.92	108.95	8.71%	117.11	107.72			
72	APAC - Naples	Milled	0.63%	105.09	104.43				4.32%	112.57	107.91	6.47%	115.63	108.60	8.41%	119.53	110.26	10.09%	125.47	113.97	12.18%	125.47	111.84
74	APAC - Miami	Milled	0.12%	104.60	104.47				3.89%	115.52	111.20	5.74%	120.71	114.15	7.59%	123.05	114.37	11.28%	129.13	116.05			
79	APAC - Pen.	Milled	0.17%	105.64	105.46				4.17%	116.60	111.94	6.06%	119.64	112.80	8.03%	121.80	112.74	9.50%	124.13	113.36			
81	AC - Holt	Milled - 10	0.07%	109.29	109.21	2.49%	115.75	112.94	4.05%	123.36	118.56	6.00%	122.98	116.02	8.44%	126.61	116.76						
82	AC - Holt	Milled - 90	0.28%	107.44	107.14				4.32%	119.04	114.11	6.24%	120.89	113.80	7.89%	122.23	113.29	9.24%	123.56	113.11			
85	APAC - DFS	Milled	0.10%	106.95	106.84				4.00%	116.09	111.63	6.22%	121.05	113.97	7.27%	119.92	111.79	9.93%	123.73	112.56			
87	CWR - Freeport	Milled	0.22%	106.25	106.01				4.24%	121.12	116.19	5.74%	125.28	118.48	7.75%	127.12	117.98	10.56%	130.72	118.24			
90	AC - PC	Milled	0.22%	101.08	100.86	2.35%	110.25	107.72	4.19%	116.20	111.53	6.23%	118.88	111.91	8.25%	119.72	110.59						
92	AC - Marianna	Milled	0.18%	114.16	113.95				3.94%	129.61	124.69	5.89%	133.40	125.97	7.83%	135.80	125.94	10.21%	138.31	125.49			
94	CWR - Tally	Milled	0.12%	101.71	101.59				3.96%	107.52	103.42	5.70%	110.87	104.89	7.36%	113.65	105.86						
96	MB - Tally	Milled	0.16%	103.65	103.49				4.19%	113.85	109.27	5.88%	115.84	109.41	8.18%	119.45	110.42	10.21%	125.04	113.45			
97	Hipp	Milled							4.14%	114.45	109.90	6.22%	118.12	111.20	8.76%	120.23	110.54						
98	Ranger - WPB	Milled - LWR							3.98%	118.77	114.23	5.77%	123.57	116.83	7.72%	125.45	116.47						

**Appendix C-6: LBR Summary Data – Crushed and Milled**

		0%	2%	4%	6%	8%	10%
		LBR	LBR	LBR	LBR	LBR	LBR
<u>Number</u>	<u>Name &amp; Location</u>	Avg.	Avg.	Avg.	Avg.	Avg.	Avg.
1	Ranger-DeBary	6	-	6	5	5	-
2	Lane - Lakeland	13	18	18	17	17	35
3	APAC-WH	18	-	25	25	44	34
4	Ajax - Odessa	13	20	15	14	14	24
5	APAC - Tampa	13	23	25	23	25	21
6	DAB	20	-	31	31	27	20
7	APAC - Sarasota	10	12	15	17	19	-
8	Ajax - Sarasota	8	16	27	30	21	35
9	APAC - WS	9	-	11	9	12	-
10	OPC - Oviedo	10	-	19	23	19	-
11	OPC - Taft	12	-	23	28	26	-
12	Ranger - Ft.P	-	-	-	-	-	-
13	Dickerson	10	-	20	19	17	23
14	CommAsph - VB (CAVB)	11	-	18	17	20	21
15	Whitehurst	8	-	13	12	7	-
16	Whitehurst	12	-	14	16	16	-
17	AC - LC	7	-	10	14	10	-
18	Halifax	11	-	13	16	13	-
19	ACA - Jax	11	-	13	15	13	11
20	ACA - Jax	14	-	12	16	14	-
21	SCI	12	-	20	26	23	-
22	AC - Ocala	13	-	10	10	10	-
23	Middlesex - Leesburg	11	-	18	17	15	16
24	General - Miami	11	-	17	17	20	15
25	CommAsph - Miami	9	-	10	13	10	10
26	Whitehurst - Pit	14	-	22	24	16	15
27	Ajax - PG	11	-	12	11	10	-
28	Better Rd - PG	11	-	10	9	9	-
29	APAC - Naples	10	-	11	12	13	18
30	APAC - Miami	8	-	16	22	21	22
31	APAC - Pen.	-	-	16	16	16	-
32	AC - Holt	6	8	10	9	10	-
33	AC - Holt	9	-	12	12	10	10
34	APAC - DFS	9	-	9	12	8	8
35	CWR - Freeport	8	-	19	20	17	20
36	AC - PC	12	10	21	22	19	-
37	AC - Marianna	9	-	26	28	27	21
38	CWR - Tally	7	-	5	7	-	-
39	MB - Tally	7	-	8	8	9	11
40	Hipp	-	-	14	17	15	-
41	Ranger - WPB	-	-	19	20	23	-