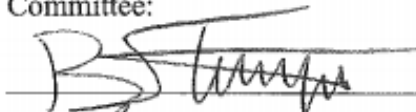


EFFECTS OF PARTICLE SHAPE ON PERFORMANCE AND DURABILITY OF
AGGREGATES USED IN ROAD CONSTRUCTION

by

Clayton Cook
A Thesis
Submitted to the
Graduate Faculty
of
George Mason University
in Partial Fulfillment of
The Requirements for the Degree
of
Master of Science
Civil and Infrastructure Engineering

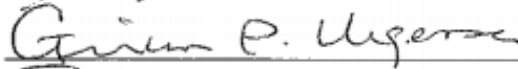
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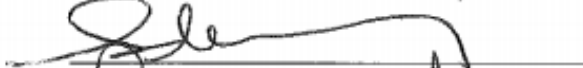


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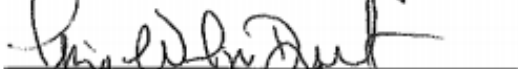
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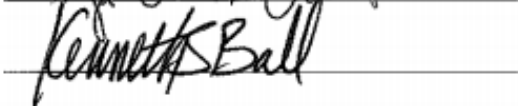
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Summer Semester 2015
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Effects of Particle Shape on Performance and Durability of Aggregates Used in Road
Construction

A Thesis submitted in partial fulfillment of the requirements for the degree of Master of
Science at George Mason University

by

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Bachelor of Science
Western Kentucky University, 2013

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DEDICATION

This is dedicated to my family and loving wife for their support.

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ABSTRACT

EFFECTS OF PARTICLE SHAPE ON PERFORMANCE AND DURABILITY OF AGGREGATES USED IN ROAD CONSTRUCTION

Clayton Cook, M.S.

George Mason University, 2015

Thesis Director: Dr. Burak F. Tanyu

Based on a comprehensive review of current United States transportation agency specifications, it was found that there is a large variation in current limits set on the shape of aggregate particles that are considered as suitable to be used in an unbound base layer. Previous studies found in the literature indicated that the shape of the aggregate may affect the overall performance of the base course, but the extent of that effect has not been quantitatively documented. Therefore, a well-defined limit on particle shape for unbound aggregate base does not exist. This research is providing a detailed quantitative comparison of the effect of aggregate shape on unbound aggregate base as it relates to performance within an unbound base layer.

The experimental program for this study included particle shape analyses, compaction, matrix interlocking characterization (Bailey method), resilient modulus, and permanent deformation testing. Four different types of aggregates including diabase, hornfels, dolostone, and slate were included in the study and aggregates from each rock

type were separated and sieved to create varying particle shapes within the same mineralogy and open- and dense-graded aggregate gradations used in road construction. Samples with consistent mineralogy were compared with each other to understand the effect of particle shape of aggregates with a specific mineralogy and samples with different mineralogy (but with same gradation) were compared with each other to understand the variation of the findings with differences in mineralogy.

The findings of the study showed that the durability of the aggregate was dominated by the mineralogy, but marked variations in durability were observed because of particle shape. Additionally, matrix interlocking characterization by the Bailey method showed that the effect of particle shape was more pronounced in the open-graded materials since the coarse particle made up the load carrying skeleton of the structure. The overall effect of particle shape on the performance, as it was defined in this study, of typically designed flexible pavement structures appear to be minor. However, particle shape was noted to significantly affect non-typical flexible pavement structures, specifically an open-graded base layer supporting a thin asphalt surface.

1: INTRODUCTION

The crushing of aggregates with varying mineralogical origins can result in variations in how the aggregate breaks, which have an impact on the angularity (i.e. angular vs. rounded particles) and shape (i.e. cubical vs. flat and/or elongated particles) of the individual aggregate particles (Prowell, Zhang, & Brown, 2005). Based on previous studies, typically when aggregate is considered to be used for base layer as an unbound material, it is preferred that the particles of the aggregate be as angular and cubical as possible (as opposed to rounded and flat/elongated) (Tutumluer, 2013). Laboratory studies with varying unbound aggregate base (UAB) angularity have shown that as the angularity increases, the resilient modulus and the resistance to permanent deformation increases (important improvements as it relates to performance), and the Poisson's ratio decreases (Allen & Thompson, 1974; Barksdale & Itani, 1989; Bilodeau & Doré, 2012; Hicks & Monismith, 1971; Janoo, Bayer Jr., & Benda, 2004; Saeed, Hall, & Barker, 2001; Thom, 1988; Tutumluer & Pan, 2008). These studies show that crushed aggregates in UAB applications perform better mechanically than uncrushed aggregate (natural aggregates).

Although the previous literature supports the importance of the angularity of the aggregates clearly, the effects of varying shape of the aggregates are not as conclusive. Less research has been conducted on the effects of specifically aggregate shape on UAB,

hence the published literature is limited and not conclusive as different researchers have different findings. Four different laboratory studies that could be found in the literature are described below in chronological order to present the discrepancies. Additional data is presented by Xiao, Tutumluer and Siekmier (2011), who studied the importance of aggregate properties in the Minnesota base aggregate database and a summary of current shape specifications in the United States and Internationally is discussed.

Barksdale and Itani (1989) performed laboratory repeated load tests to evaluate the suitability of five different types of crushed rock (gneiss (with granitic origin), uncrushed gravel, shale, quartzite, and limestone) to be used as UAB. The repeated load tests included resilient modulus (M_R) tests according to AASHTO T-274-82 and permanent deformation (PD) tests using a confining pressure of 41.4 kPa (6 psi), principal stress ratio of both 4 and 6, and applying 70,000 cycles. The findings showed that uncrushed gravel with rounded particles were two times more susceptible to rutting than the crushed aggregates with angular particles. The study also made an attempt to compare the shape of the crushed aggregates, which varied from one source to another. For example, limestone, shale, and quartzite all resulted in somewhat blade-shaped particles, but these particles were not specifically sorted to create a specific category and were not specifically called as elongated or flat. Among all of the blade-shaped particles, the study concluded that only the quartzite particles appeared to be 30 percent more susceptible to rutting than the other crushed aggregates. However, the authors stated that this observation was not conclusive and could be due to the scatter in their test data. The study did not evaluate the difference in particle shape within a given mineralogy (e.g.,

blade-shaped vs. angular or rounded quartzite particles). In conclusion, the results of Barksdale and Itani (1989) showed definitive increase in resistance to permanent deformation and increased M_R values as particle angularity increased, but no definitive trends on the effects of particle shape on UAB mechanical performance.

Rismantojo (2002) conducted a study on five different types of coarse aggregates with different mineralogy (dolostone, limestone, uncrushed gravel, granite, and trap rock) to evaluate their performance as it relates to asphalt mixtures as well as UAB. The study included several different tests, but the ones relevant to UAB were identified aggregate loss tests as determined by Los Angeles (LA) abrasion, magnesium sulfate (MS), and Micro-deval (MD) tests. The five aggregates tested were also classified based on the percentages of both flat and elongated (F&E) and flat or elongated (FOE) particles above the No. 4 sieve at the 3:1 and 5:1 ratios. The range of percentages classified as F&E were between 13.2% and 28.0% for the 3:1 ratio and 1.8% to 8.1% at the 5:1 ratio. The FOE ranges recorded for the five aggregate types were between 2.6% to 6.0% at the 3:1 ratio and 0.0% to 1.3% at the 5:1 ratio. Linear correlations were also investigated between the LA abrasion, MS, and MD tests and the percentages of F&E and FOE above the No. 4 sieve at both ratios.

The results by Rismantojo (2002) showed that there was a good correlation between increase in particle shape and increase in LA abrasion loss, a low to moderate correlation between particle shape and increase in MD loss, and no correlation between particle shape and MS loss. This indicates that the importance of particle shape in coarse aggregate degradation tests could be important but could be dependent on the type of

durability test conducted (i.e. MD or LA abrasion). In a similar study, Fowler, Allen, Lange and Range (2006) could not find any correlation between the aggregate loss as it related to the shape of coarse aggregates in the MD , MS, or LA abrasion tests. The researchers tested significant amount of aggregates with different mineralogy (71 sedimentary, 22 igneous, and 16 metamorphic in origin) in the MD test.

The short-comings of both Rismantojo (2002) and Fowler et al. (2006) studies are that shape could not be singled-out independently from the mineralogy of the samples because varying particle shapes were not produced from the same aggregates samples. Additionally, the amounts of F&E or FOE particles investigated in the Rismantojo (2002) study had small variation in particle shape and Fowler et al. (2006) used a unique method for particle shape comparison, which is difficult to compare to standard methods of particle shape classification .

In a more recent study, Uthus et al. (2007) investigated the influence of particle shape, angularity, and surface texture on the elastic and permanent deformation properties of UAB by only using gneiss as the rock source for their aggregate type. The study only used a coarse, single-sized material to represent a UAB. They crushed the rock samples following different processes to produce four sample variations; cubical angular, cubical rounded, flakey angular, and flakey rounded. The idea was that the variation in UAB performance due to grain shape, angularity, and surface texture would be more pronounced if only a coarse, single-sized material was used. No proctor curves were produced for the individual sample variations. Instead, all laboratory samples were compacted using a gyratory mold with the same compaction energy. This methodology

lead to variable compaction throughout the four samples tested, with the highest densities coming from the cubical rounded samples and the lowest coming from the flaky aggregates. This indicated that the more angular and less cubical the aggregates are, the lower the achieved dry density from compaction. All four samples showed similar M_R values, showing no major change in value due to the variation in dry density or material shape. The permanent deformation accumulation was the highest in the rounded cubical aggregate samples, but showed smaller differences in the other three aggregate types. The importance of fines and gradation were not considered due to the coarse single-sized gradation used. This approach may not capture the correct particle interaction characteristics for UAB.

Xiao et al. (2011) used the Minnesota Department of Transportation (Mn/DOT) aggregate property database to develop correlations between aggregate physical properties and resilient modulus characterization model parameters to be used in mechanistic pavement design. Two aggregate databases were used to predict M_R values and correlations were developed using a Monte Carlo analysis. The first database used 376 aggregate samples that were classified based on gradation, fines content, moisture content and dry density. The second database used 135 aggregate samples from the previous 376 samples that were additionally classified based on aggregate particle shape properties as quantified based on F&E ratio, angularity index, and surface texture. The shape properties were determined using the Illinois Aggregate Image Analyzer (UIAIA). The results of the study found that significantly higher R^2 values were obtained in the second database because of the inclusion of aggregate shape properties. Results showed

that higher F&E ratios and lower angularity decreased predicted M_R values, but no comment was made on the extent of these changes could have on the M_R value or their importance in the design process.

The inconclusive information in the literature is also reflected in the design specifications developed by department of transportation (DOT) agencies throughout the U.S. A detailed literature search showed that only seven DOT agencies in the U.S. have specifications as it relates to particle shape for UAB and the specific requirements widely vary between different States.

The study described in this thesis is performed to evaluate these gaps in the literature and to develop guidelines as it relates to the effect of particle shape to the performance of the UAB. Four different aggregates with different mineralogy compositions were evaluated. For each aggregate type, two different particle gradations were developed (one with 6% percent fines and another with no fines) and each gradation was created with aggregates of varying particle shape. Samples with consistent mineralogy were compared with each other to understand the effect of particle shape of aggregates with a specific mineralogy and samples with different mineralogy were compared with each other to understand the variation of the findings with differences in mineralogy. All samples were tested to evaluate the performance properties as they are defined as M_R , PD, and durability (as defined from MD tests) in this study.

2: BACKGROUND

2.1: Behavior of Aggregate Under Repeated Loading

UAB materials are subjected to repeated loading imposed from traffic. During these loading cycles UAB undergoes both resilient and permanent strains, as illustrated in **Figure 1**. The resilient strains are used to determine the resilient modulus and permanent strains are used to determine the permanent deformation. In a typical UAB layer the accumulation of permanent deformation for each load repetition gradually decreases as the number of load repetitions increases. With a sufficiently large application of cycles, the permanent deformation of the material reduces to near zero and all movement due to the load application becomes a function of resilient deflection, given that the load application is not near the strength of the material. A discussion of resilient and permanent deformation of UAB and the most prevalent models associated with predicting their behavior is presented below.

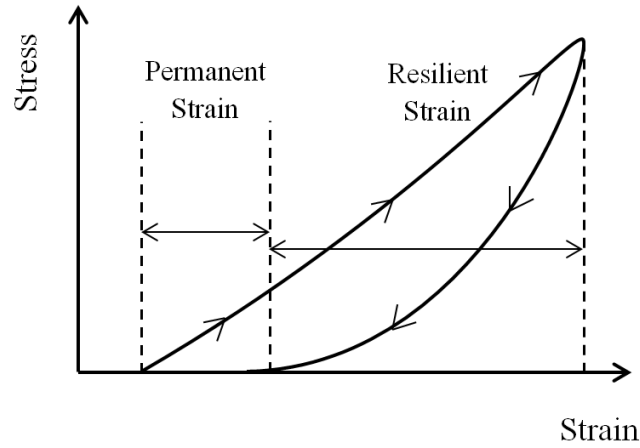


Figure 1. Strains in UAB During One Load Cycle

2.1.1: Resilient Modulus and Associated Models

The resilient response of UAB materials is characterized by the resilient modulus (M_R) in pavement design. For repeated load triaxial tests with constant confining stress, the resilient modulus is defined as the ratio of the peak axial repeated deviator stress to the peak resilient axial strain (ϵ_r) of the specimen. The resilient modulus of a material under constant confining pressure is expressed as:

$$M_R = \frac{(\sigma_1 - \sigma_3)}{\epsilon_r} = \frac{\sigma_d}{\epsilon_r} \quad (1)$$

where;

M_R is the resilient modulus;

σ_1 is the major principal or axial stress;

σ_3 is the minor principal or confining stress;

σ_d is the deviator/shear stress; and

ϵ_r is the axial resilient strain.

Many different material characterization models have been proposed by researchers for UAB materials. Three of the most common characterization models are summarized below:

K-Theta Model

Proposed by Hicks and Monismith (1971), the K-Theta model is generally used for unbound granular materials. This model represents the resilient modulus values by the bulk stress as:

$$M_R = K \theta^n \quad (2)$$

where;

θ is the bulk stress ($\sigma_1 + \sigma_2 + \sigma_3$) and

K, n are model parameters from test data.

Generally, as the K values increases there is a reduction in the n value. The simplicity of the equation makes it widely accepted and useful, but it does have some drawbacks. The K-theta model assumes a constant Poisson's ratio, the effect of stress on M_R is accounted for only by the sum of principal stress, and there is no term for the magnitude of difference between major and minor principal stresses (Lekarp, Isacsson, & Dawson, 2000b).

Uzan Model

Proposed by Uzan (1985), the model goes beyond the K-Theta model and incorporates the shear stress, which is the deviator stress during constant confining triaxial testing.

$$M_R = k_1 p_a \left(\frac{\theta}{p_a} \right)^{k_2} \left(\frac{\sigma_d}{p_a} \right)^{k_3} \quad (3)$$

where;

p_a is atmospheric pressure (101.3 kPa or 14.7 psi);

θ is the bulk stress;

σ_d is the deviator stress; and

k_1, k_2, k_3 are model parameters from test data.

The shear stress term in the Uzan model is the σ_d and this term improved the correlation between observed M_R values obtained from laboratory data and predicted M_R values. This was especially important when the confining pressure was larger than the deviator pressure applied to the sample.

MEPDG Model

The Mechanistic Empirical Pavement Design Guide (MEPDG) model is used in the Mechanistic-Empirical Pavement Design Guide (NCHRP 1-37A, 2004) and accounts for both the stress hardening and stress-softening behavior of unbound pavement materials.

$$M_R = k_1 p_a \left(\frac{\theta}{p_a} \right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1 \right)^{k_3} \quad (4)$$

where;

p_a is atmospheric pressure (101.3 kPa or 14.7 psi);

θ is the bulk stress;

τ_{oct} is the octahedral shear stress

$(\tau_{oct} = \frac{1}{3} \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_1 - \sigma_3)^2})$; and

k_1 , k_2 , and k_3 are model parameters from test data.

The stress hardening behavior of unbound materials is captured by the k_2 coefficient, which should be positive, since increasing the bulk stress produces a stiffer material response. Inversely, the k_3 coefficient shows the softening of unbound materials since the value is usually negative.

Of the resilient modulus models presented, the MEPDG model was selected to be used for this study since the model has terms to quantify both the stress hardening and softening of unbound materials. Additionally, the MEPDG model was used since it is specified for base materials by American Association State Highway Transportation Officials (AASHTO) to be used in the recent AASHTOWare Pavement ME Design software for road design (AASHTO, 2008). This road design software was used in the practical implications chapter of this thesis.

2.1.2: Permanent Deformation and Associated Models

The permanent deformation in UAB applications is the amount of strain that is not recovered after a load cycle. If the load applied to the base layer is considerably smaller than the shear strength of the material, the permanent strain should substantially reduce as the number of load applications increases. If the load approaches the shear strength of the material, there will be incremental increase in permanent strain that may lead to sudden failure.

Werkmeister (2003) concluded that the reduction of permanent strain rate with increasing load applications in granular materials at low stress levels can be attributed to strain hardening, while increase in strain rate at higher stress levels can be attributed to strain softening. This behavior is illustrated by Werkmeister (2003) in **Figure 2**.

Werkmeister (2003) explains as the granular material is being loaded at lower stress levels the particles are being reoriented. This reorientation of particles increases the density of the material, increases particle interlocking, and increasing the strength of the material. When the stress level approaches the shear strength of the material, there is a strain softening of the material since particle interlocking is decreasing, which will result in failure of the material as the number of load applications increases.

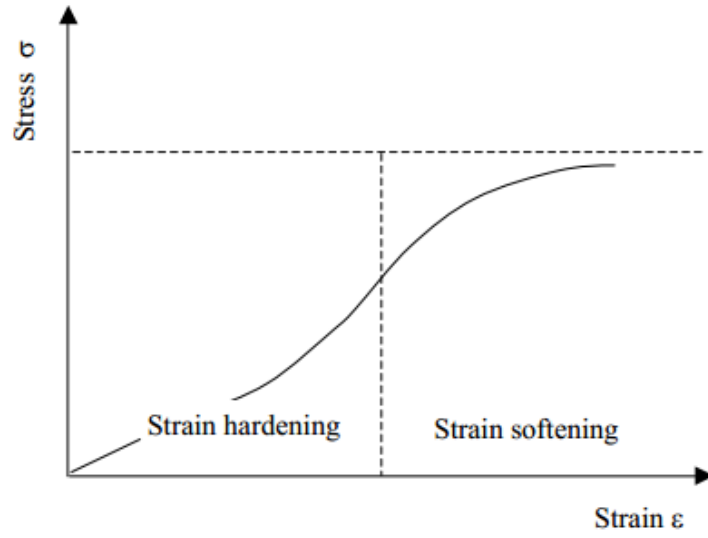


Figure 2. Stress-Strain Behaviour of Granular Materials after (Werkmeister, 2003)

The permanent deformation of UAB material is defined as the accumulated irrecoverable (permanent) strain through the pavement service life. This accumulated is calculated based on Equation 5 below:

$$\varepsilon_p = \frac{\Delta H}{H_0} \times 100\% \quad (5)$$

where;

ε_p is the permanent deformation in percent (strain);

ΔH is the change in specimen height after a certain number of load applications;

H_0 is the original specimen height.

The measurement of permanent deformation accumulation is simple, but the prediction of this accumulation is complex due to the varying behavior types dependent on the stress and strain levels. One of the main approaches for describing this behavior is by the shakedown concept proposed by Werkmeister et al. (2001). This approach involves studying the development of permanent deformation in cyclic load triaxial tests and classifying the results in one of three shakedown ranges; A, B, or C (**Figure 3**).

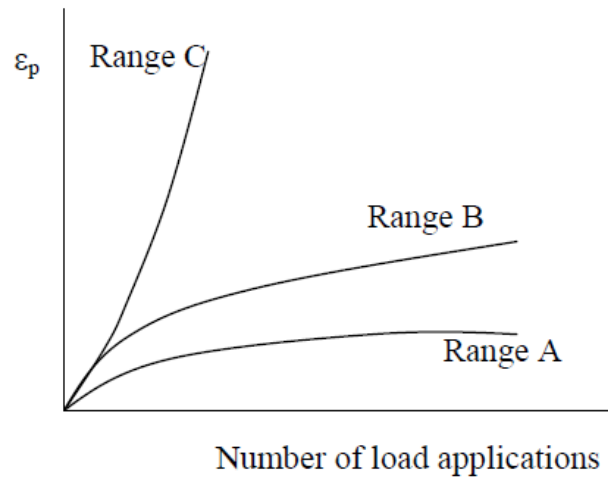


Figure 3. Different Types of Permanent Deformation Behavior after (Araya, 2011)

Range A in **Figure 3** refers to the plastic shakedown range, where the material experiences a finite post-compaction period of permanent deformation, then an entirely resilient response with no further permanent deformation accumulation. Range B in **Figure 3** is the intermediate response, or plastic creep, where there is post-compaction period of permanent deformation followed by an incremental increase in plastic strain.

Range C in **Figure 3** is defined as incremental collapse, where the plastic strain continually increases with load application until the material reaches failure.

Predictive models for permanent deformation accumulation have been proposed by past researchers. The models developed predict the permanent deformation accumulation through one or more factors such as: number of load applications, applied stress states, and aggregate shear strength. Four common permanent deformation models are presented below.

Barksdale Model

Barksdale (1972) proposed a linear relationship between permanent deformation accumulation and the logarithm of load applications:

$$\varepsilon_p = a + b \log(N) \quad (6)$$

where;

ε_p is the axial permanent strain;

N is the number of load applications;

a and b are model parameters obtained from laboratory data.

This relationship shows that with an increasing number of load cycles the percentage of permanent deformation decreases.

Phenomenological Model

Monismith et al. (1975) presented a log-log relationship between the number of load applications and the permanent strain:

$$\varepsilon_p = aN^b \quad (7)$$

where:

ε_p is the axial permanent strain;

N is the number of load applications;

a and b are model parameters obtained from laboratory data.

Tseng and Lytton Model

Tseng and Lytton (1989) introduced a rutting model that was based on unbound material testing that then could be used to predict accumulation of permanent deformation. The model incorporated three material parameters that could be adjusted based on the physical properties, moisture content and bulk stress of the laboratory tested material.

$$\varepsilon_p = \varepsilon_0 e^{-\left(\frac{\rho}{N}\right)^\beta} \quad (8)$$

where:

ε_p is the axial permanent strain;

N is the number of load applications;

ε_0 , β , and ρ are material parameters obtained from laboratory data.

MEPDG Model

The MEPDG model modified the Tseng and Lytton (1989) model for permanent deformation accumulation for granular materials. This model is used for rutting prediction in base and subgrade layers in the new MEPDG design guide (Chow, 2014) and is shown below:

$$\frac{\varepsilon_p(N)}{\varepsilon_v} = \beta_1 \left(\frac{\varepsilon_0}{\varepsilon_r} \right) e^{-\left(\frac{\rho}{N} \right)^\beta} \quad (9)$$

where;

ε_p is the permanent strain in the unbound pavement layer at N loads applied of a typical equivalent standard axle;

ε_v is the vertical resilient strain determined for the sublayer;

β_1 is a constant, which is 1.673 for base layers;

N is the number of load cycles from traffic;

ε_r is the resilient strain imparted in the laboratory to determine material properties

ε_0 is a material property

β and ρ are materials properties which are computed from Equation 10 through Equation 13.

$$\log \beta = -0.6119 - 0.017638W_c \quad (10)$$

$$\log \rho = 0.622685 + 0.541524 W_c \quad (11)$$

$$W_c = 51.712 \times CBR^{-0.3586 GWT^{0.1192}} \quad (12)$$

$$CBR = \left(\frac{M_R}{2555} \right)^{\frac{1}{0.64}} \quad (13)$$

where;

W_c is the water content

GWT is the depth of the ground water table

CBR is the California Bearing Ratio of the base layer

M_R is the resilient modulus of the base layer

Both variables β and ρ are currently dependent only on the water content, which is ultimately determined from the CBR or M_R value of the layer. So, the determination of permanent strain accumulation in the MEPDG model relies heavy on the CBR or M_R value.

2.1.3: Typical Stresses Induced in the UAB layer From Traffic Loading

The behavior of a UAB material in the pavement structure, as shown in the two previous sections, is highly dependent on the amount of stress that is applied to the material. The higher the amount of stress that is applied to the UAB layer the more strain that is induced. The stresses induced on the UAB layer is directly related to the thickness of the asphalt or concrete surface layer. Generally, as the traffic level increases the thickness of the asphalt concrete increases and the stresses that is induced on the UAB layer reduces. A correct estimation of stresses induced in the base layer is imperative for

laboratory testing to ensure that the measured response of the material is representative of that same material in the field.

Barksdale and Itani (1989) investigated the ranges of stresses that could be induced on a UAB layer from traffic loading for varying pavement structures specifically to ensure that accurate stresses were induced during permanent deformation testing of dense-graded UAB material. First, stresses that develop in a base layers for light, medium, and heavy types of pavement structures with both poor and good subgrades were investigated. A *light pavement* structure was a pavement structure with a 5.1 cm (2 in.) asphalt layer and a base layer thickness of 15.2 or 25.4 cm (6 or 10 in.). *Medium pavement* structure stress was determined from a 10.2 cm (4 in.) asphalt layer and 25.4 cm (10 in.) base layer. Lastly, a *heavy pavement* layer was defined as a pavement structure with a 10.2 or 20.3 cm (4 or 8 in.) asphalt pavement layer with 15.2 or 50.8 cm (6 or 20 in.) of base. The subgrades were modeled as either poor or good based on a piecewise linear variation of the subgrade resilient modulus value.

A nonlinear finite element program (GAPPS7) was used to determine the boundary stresses that develop in the base layer, depending on the location in the layer, for the three types of pavement structures analyzed. The use of a single confining pressure, 41.4 kPa (6 psi) for the light and medium pavements and 31.0 kPa (4.5 psi) for the heavy pavement, was stated to be sufficiently accurate for testing purposes. The base material was modeled using a simplified contour model proposed by Paute and Martinez (1982). The poor subgrade was modeled as having a resilient modulus value of 110 MPa (16 ksi) if the deviator pressure was less than 41.4 kPa (6 psi) and a resilient modulus

value of 34.5 MPa (5 ksi) if the deviator pressure was greater than 41.4 kPa (6 psi). Good subgrade was modeled by assigning a resilient modulus value to the subgrade of 206.8 MPa (30 ksi) when the deviator pressure was between 0 and 34.5 kPa (0 and 5 psi), 103.4 MPa (15 ksi) when the deviator pressure was between 34.5 and 172.4 kPa (5 and 25 psi), and 106.9 MPa (15.5 ksi) when the deviator pressure was higher than 172.4 kPa (25 psi). The asphalt layer was modeled as having a resilient modulus value of 275.8 MPa (400 ksi) and a Poisson ratio of 0.2.

From this analysis a range of stresses, presented in **Figure 4**, were found for the varying pavement types and the location in the base layer. As can be seen in the **Figure 4**, as the type of pavement goes from light to heavy the principal stress ratio decreases. This means that the ability of the UAB layer to resist permanent deformation and provide adequate support for the asphalt layer increases.

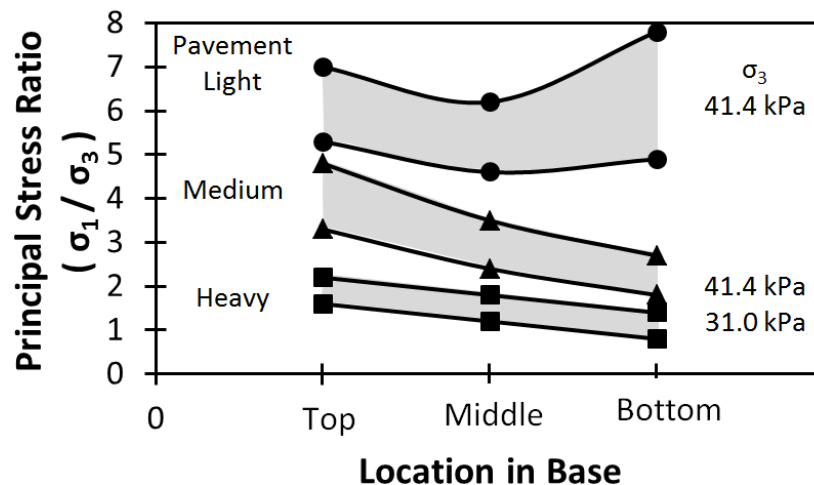


Figure 4. Typical Stress Developed in Bases Layers in Flexible Pavements (Barksdale and Itani, 1989)

A more recent investigation into stresses induced in the UAB layer in a pavement structure was conducted by Chow (2014) so that adequate stresses could be used in permanent deformation estimation for flexible pavements. The pavement structure used for the investigation was based on typical flexible pavement structure used in the state of North Carolina. Flexible pavement structure representing low, moderate, and high traffic volumes had asphalt thicknesses of 7.6 cm (3 in.), 15.2 cm (6 in.), and 22.9 cm (9 in.), with corresponding UAB thicknesses of 20.3 cm (8 in.), 20.3 cm (8 in.), and 25.4 cm (10 in.). The stresses induced in the base layer were estimated using ILLI-PAVE, a finite element analysis program. The stresses induced in middle of the base layer for the low, moderate, and high traffic volume pavement structures are presented in **Table 1**. The σ_3 values were increased to 5 psi by Chow (2014) from the calculated σ_3 by the ILLI-PAVE program to account for residual stresses induced in the UAB layer from compaction, which is supported by work conducted by Stewart et al. (1985) and Uzan (1985).

Table 1. Pavement Stress in the Middle of the Base Layer (Chow, 2014)

Traffic Volume	Pavement Profile	Principal Stress		
		σ_1 (kPa)	σ_3 (kPa)	Ratio (σ_1 / σ_3)
Low	7.6 cm HMA – 20.3 cm Base	204.1	34.5	5.9
Moderate	15.2 cm HMA – 20.3 cm Base	84.1	34.5	2.4
High	22.9 cm HMA – 25.4 cm Base	57.9	34.5	1.7

As can be seen when comparing **Figure 4** and **Table 1**, the principal stress ratios that are induced in the middle of the base layer for the two studies are very similar for the

range of pavement structures investigated. These two studies found very similar results, therefore because of this it is reasonable to assume that these two studies represent generally the possible expected stresses in the middle of a base layer in a wide range of road structures.

2.2: Road Design Process: Mechanistic-Empirical Pavement Design

Typical flexible pavement structures are composed of three layers, a top asphalt concrete layer, an unbound aggregate base course, and a natural subgrade (Huang, 2004). Design of these structures in the late 1880's started with reliance on just the experience of the engineer, moved on to the development of purely empirically based designs with the AASHTO pavement design guides from the 1970's to the 1990's, and is currently transitioning to a mechanistic-empirical design approach spearheaded by NCHRP Project 1-37A (2004) (Christopher, Schwartz, & Boudreau, 2010).

The current design software offered by AASHTO for mechanistic-empirical design, which will be used in this thesis, is the AASHTOWare Pavement ME Design Version 2.1. This section will be looking into the overall design process when using a mechanistic-empirical design approach and highlighting specifically materials inputs and the effect of the base layer in the overall design process.

The design of pavements by the AASHTOWare Pavement ME Design involves two aspects. The first is the calculation of the mechanistic response (stress and strain) based on either a finite element method or elastic layer theory of the varying materials used in the pavement structure due to the traffic loading and environmental conditions. Then, the mechanistic response of the material is then related to empirical distress

predictions (i.e. asphalt concrete fatigue cracking, asphalt concrete and granular layer plastic deformation, transverse thermal cracking, rutting, and roughness). The ability of the mechanistic-empirical design approach to predict actual performance of the pavement structure is directly related to the quality of the inputs (traffic, climate data, and materials properties) and the calibration of the empirical distress models to observed field performance. The overall design flow chart for the AASHTOWare Pavement ME Design is given in **Figure 5**.

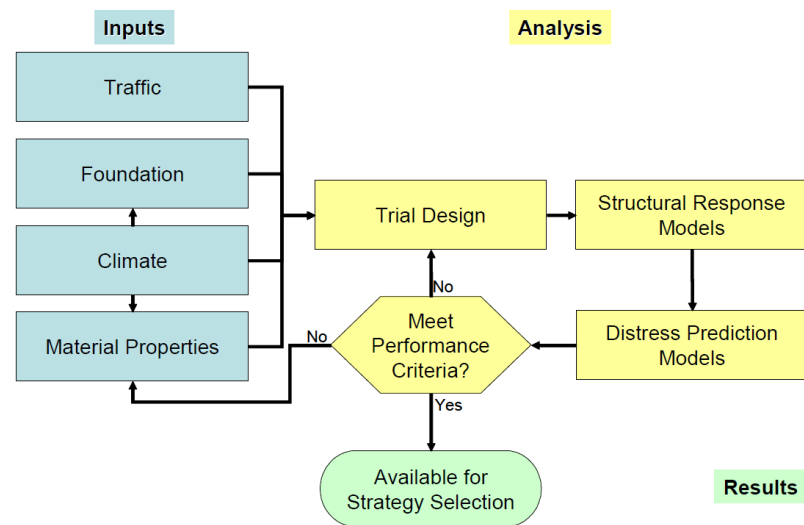


Figure 5. Mechanistic-Empirical Pavement Design Flow Chart after (Schwartz & Carvalho, 2007)

The first step in the design is to characterize the traffic and materials that will be incorporated into the design. The MEPDG design inputs have a hierarchical approach with level 1, 2, and 3 inputs. The level 1 inputs are measured laboratory material properties or site specific traffic data, level 2 are empirical correlations and level 3 are

inputs selected from default values obtained from national or regional values. Generally, as the level of input increases there is a higher accuracy of the data and higher cost for collecting the data. A list of inputs for specifically unbound materials is presented in **Table 2** with the possible input level for each input.

Table 2. Inputs Required for Unbound Material in AASHTOWare Pavement ME Design

Material Type	Required Inputs
Unbound Material	• Resilient Modulus (at max density and optimum moisture content) ○ Level 1: k_1, k_2, and k_3 parameters in the MEPDG model from testing procedure AASHTO T-307 or NCHRP 1-28A ○ Level 2: Empirical correlation from CBR, R-value, layer coefficient, DCP Penetration, or PI and Gradation ○ Level 3: Default values for given soil type • Poisson's Ratio (only Level 3) • Gradation and Engineering Properties (Level 1 or Level 3) ○ Dry Density and Optimum Moisture ○ Plasticity Index and Liquid Limit ○ Sieve Size ○ Specific Gravity ○ Saturated Hydraulic Conductivity

With the inputs selected, a trail design is implemented and run with estimated traffic loading and the mechanistic response is estimated. The trail design is then either rejected or accepted based on the estimated performance of the pavement system in terms of “alligator” or bottom-up fatigue cracking in the asphalt layer, longitudinal or top down

fatigue cracking in the asphalt layer, thermal cracking in the asphalt layer, rutting in the asphalt and unbound layers and roughness based on the international roughness index (IRI). The performance criteria for the pavement structure that are directly related to the UAB layer is the amount of “alligator” or bottom-up fatigue cracking in the asphalt layer, rutting development in the UAB layer, and the IRI (Schwartz & Carvalho, 2007).

The performance of the asphalt layer in terms of “alligator” or bottom-up fatigue cracking is directly related to the “support” given by the base layer since cracking of this kind develops due to tensile strains at the bottom of the asphalt layer. UAB layers with low resilient modulus values have reduced “support” of the asphalt layer, increasing the tensile strain magnitude at the bottom of the asphalt concrete and increases the probability of fatigue cracking developing (Schwartz & Carvalho, 2007). The overall rutting of the pavement is also directly related to the permanent deformation resistance of the UAB, since the UAB constitutes a component of the pavement structure permanent deformation or rutting. Lastly, the equation used to calculate IRI is directly related to the type of granular base that is used in the design (ie. granular base, asphalt-treated base, and cement-stabilized base) (Schwartz & Carvalho, 2007) These three performance criteria of the pavement structure in the AASHTOWare Pavement ME Design are directly related to the stiffness and resistance to permanent deformation of the UAB layer.

2.3: Important Properties of Unbound Granular Materials

The UAB layer purpose in the overall pavement structure is to provide structural support to the asphalt concrete pavement surface by providing adequate stiffness (M_R)

and permanent deformation resistance and must not deteriorate from environmental influences (Christopher et al., 2010). The most important factors that effect the ability of the base layer to provide structural support is the stresses and number of load applications that are applied to the UAB (Lekarp, Isacsson, & Dawson, 2000a; Lekarp et al., 2000b). The stresses and number of load applications that are applied to the UAB layer are directly related to the design of the structure and the type and amount of traffic.

Secondary factors that influence the structural support of the UAB layer is the moisture content, degree of compaction, grading, and aggregate type (Lekarp et al., 2000a; Lekarp et al., 2000b). A background of these supporting influences on the performance of UAB will be discussed through the lens of the matrix structure, gradation, permeability, aggregate breakdown, and aggregate form.

2.3.1: Matrix Structure

When a load is applied to an unbound aggregate structure the load is carried and distributed through the soil structure. The nature of the load carrying structure is dependent on the primary structure (PS) and secondary structure (SS) of the material as proposed by Yideti, Birgisson, Jelagin, and Guarin (2013). The PS is composed of material in a certain size range that forms the load-carrying skeleton. The SS is composed of materials that fill in the gaps between the PS. There are three proposed aggregate mixture states by Yiedti et al. (2013) as show in **Figure 6**: Low SS, Optimum SS, and High SS (disruptive).

The Low SS structure represents a material where all of the PS particles are in full contact with each other and there is not enough SS to fill in the gaps. The Optimum SS

structure forms when the PS particles are in full contact with each other and the SS particles fill in all available gaps that are present in between the PS. The High SS structure represents when there is enough SS particles that they disrupt the interaction of the PS, where every particle in the PS is no longer in contact with each other and the load carrying structure is now primarily the SS particles.

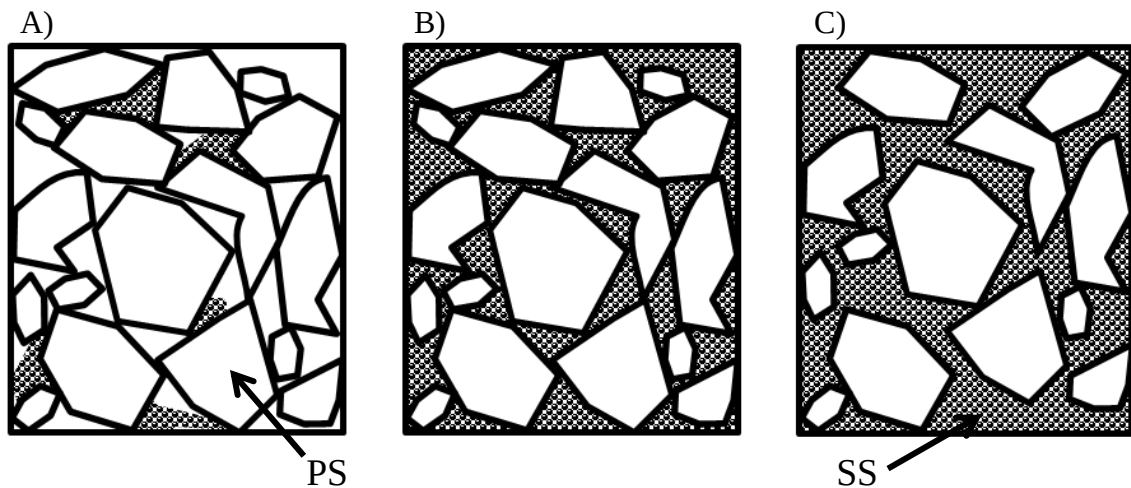


Figure 6. Aggregate Matrix Structure: A) Low SS B) Optimum SS C) High SS (after Yideti et al., 2013)

2.3.2: Gradation

The nature of the matrix structure is directly related to the gradation of the material. The two general types of gradations used in a base layer of a pavement structure are dense- and open-graded gradations. Dense-graded gradations have a larger amount of finer particles (passing the 0.075 mm sieve) than open-graded materials. The increase in

finer particles in a dense-graded UAB material increases the amount of SS particles when compared to an open-graded gradation.

Thom and Brown (1988) investigated varying gradations of a crushed limestone material used for road base in England. They found that the stiffness of the material decreased with adage of fines content and that large fines contents can lead to a sudden failure during permanent deformation testing. These results supports the idea that high fines content can cause disruption in the PS's load carry capacity in both the elastic and permanent response.

Yideti et al. (2013) used published permanent deformation data on unbound materials and found that the resistance to permanent deformation was maximized when the structure of the aggregate was in the optimum SS state. **Figure 7** shows that there is an optimum volume of SS that fills the voids between the PS, which results in the least amount of permanent deformation accumulation. Additionally, Yideti et al. (2014) found that the M_R of unbound aggregate decreased with an increase in porosity specifically associated with the particles that compose the PS, meaning that as the PS become closer packed the M_R value increases. This would indicate that if the amount of interlocking of the PS particles increases the M_R of an unbound material increases (as shown in **Figure 8**).

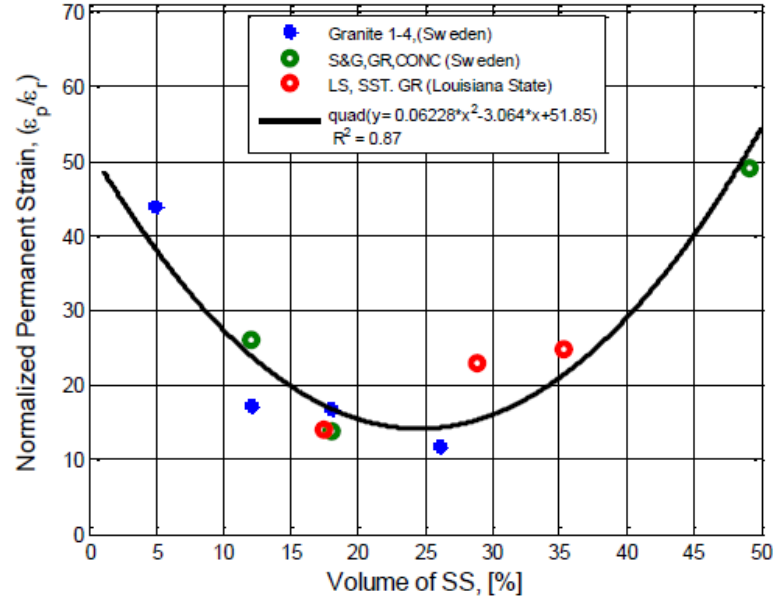


Figure 7. Permanent Strain vs. Volume of SS (Yideti et al., 2013)

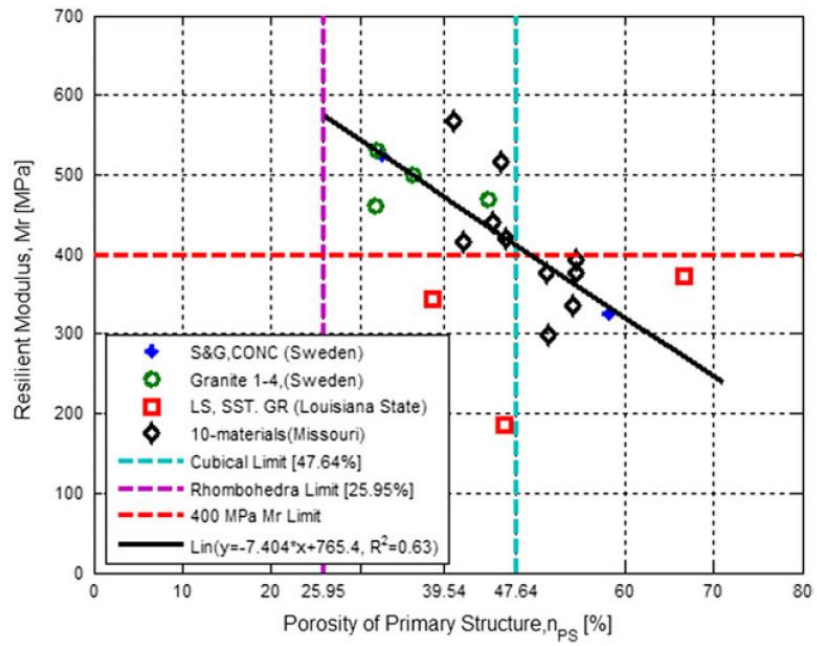


Figure 8. Resilient Modulus vs. Porosity of PS (%) (Yideti et al., 2014)

Similar reduction in M_R values with an increase in fines content from 4% to 12% was found by Tien et al. (1998) for a limestone and sandstone UAB material. Richardson and Lusher (2009) based on a literature review for performance of unbound aggregate materials in relation to fines content indicated that an optimum fines content is present, which reinforces the idea of an optimum SS. Typical optimum fines content ranges for unbound aggregate given by Richardson and Lusher (2009) based on summary of research by Yoder and Witczak (1975) was 6 to 9% in the CBR test, research conducted by the National Crushed Stone Association (Gray, 1962) was 8 to 12% for triaxial testing, and research conducted by Jorenby and Hicks (1986) was 5% for M_R testing results.

2.3.3: Permeability

An optimum amount of fines content is important for the maximum M_R and maximum resistance to PD, but the presence of fines will reduce the ability of the UAB layer to remove excess water. The ranges of possible fines content in a UAB layer and the reduction in permeability is shown in **Figure 9**. It has been well established that that the M_R and PD is reduced when saturation is approached in UAB materials (Barksdale, 1972; Lekarp et al., 2000a, 2000b; Thom & Brown, 1987; Yoder & Witczak, 1975). Open-graded materials are more resistant to sustained saturation because of the increased permeability which makes them less likely to induce damage because of water inflow than dense-graded materials (Tian et al., 1998). Because of this fact, FHWA (1992) recommends use of an open-graded base layer in a rigid pavement structure (portland cement riding surface). Open-graded base layers in flexible pavement structures (asphalt cement riding surface) are also desirable, but use in practice is uncommon.

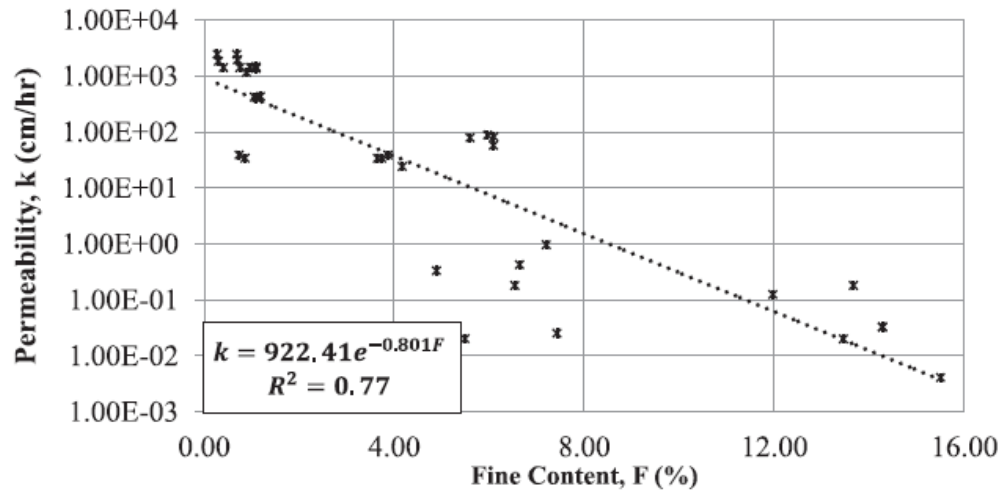


Figure 9. Variation in Permeability of UAB with Fines Content (Ghabchi, Zaman, Kazmee, & Singh, 2014)

2.3.4: Aggregate Breakdown

For an aggregate to be suitable in construction it must be able to withstand the stresses that are induced during production, transportation, placement, and environmental degradation throughout the service life of the material (Hossain, Lane, & Schmidt, 2008). In order to gauge the ability of an aggregate to resist the stresses induced during construction process, varying index tests are conducted by DOT's to screen construction aggregate. For UAB applications, typical index tests conducted in the United States to measure the suitability of aggregate to resist breakdown are the Magnesium/Sodium Sulfate Soundness test (MS) (American Society of Testing Materials (ASTM) C88), Los Angeles Abrasion test (LA) (ASTM C131), and more recently, Micro-deval (MD) test (ASTM D6928) (Tutumluuer, 2013).

The MS test is conducted to evaluate the resistance of aggregate to weathering action. The test is conducted by repeatedly submerging aggregate in solutions of sodium or magnesium sulfate followed by oven drying the particles. The process of submersion and oven drying simulates the expansion of water upon freezing which can cause breakdown of the individual particle by weathering. As stated by ASTM C88, the MS tests are helpful in judging the soundness of aggregates, but the precision of the method is poor and should not be used for outright rejection of an aggregate.

The LA test is an index test that measures the impact resistance of an aggregate (Erichsen, Ulvik, & Sævik, 2011). The method involves rotating a steel drum containing steel balls and the aggregate sample. Inside the drum there is a shelf that pickups up the aggregate and steel balls and drops them on the other side of the drum. This action creates an impact that results in breakdown of the material. After a set number of revolutions, the contents are then removed and sieved over the 1.70-mm sieve. A loss is then calculated as the difference between the original mass and the final mass as a percentage of the original mass of the samples. Research conducted by Senior and Rodgers (1991) indicated little correlations with field performance of granular base for the LA tests when the percent loss was less than 50%.

Testing of samples in the MD test is an indicator of an aggregates resistance to wear (Erichsen et al., 2011). A sample is initially soaked in 2 L of water for an hour before the test, then the soaked sample is placed in a jar and 5,000 grams of abrasive charges are added. The sample, water, and charges are then rotated for 12,000 revolutions, which results in breakdown of the aggregate by abrasion and grinding after

being soaked in water. After the number of rotations are complete, the sample is then washed over the 1.18-mm sieve and dried. The percent loss is then calculated the same way as for the LA test. The MD test has been found to be able to distinguish between good and poor quality aggregates used for base in road construction, with losses of greater than 40 percent generally able to identify poor slakey material (Senior & Rogers, 1991). Additionally, the precision of the micro-deval test is excellent, with low coefficient of variations being reported (Fowler et al., 2006).

Based on review of the three index tests in this section, the MD test was selected to measure the resistance to particle breakdown in the testing methods due to the excellent repeatability of the test (Fowler et al., 2006) and test results have been shown to correlate well with the performance of UAB materials in the field (Senior & Rogers, 1991). The MD test was also quicker and required less material than the Los Angeles Abrasion test.

2.3.5: Aggregate Form

The form of an aggregate particle is broken down into shape, angularity, and texture (Masad, Al-Rousan, Button, Little, & Tutumluer, 2007). These three aggregate form characteristics of coarse aggregate in a UAB mixture have been shown to effect the performance of the material (Janoo, 1998). These three aspects of form is affected by the mineralogical origins and the crushing processes used during production of the material (Prowell et al., 2005). If the material is a naturally occurring aggregate then the angularity is substantially lower than crushed materials since the material was broken and typically transported to some distance by natural processes. The importance of angularity

and surface texture on the performance of unbound mixtures has been well established (Allen & Thompson, 1974; Barksdale & Itani, 1989; Bilodeau & Doré, 2012; Hicks & Monismith, 1971; V. C. Janoo et al., 2004; Saeed et al., 2001; Thom, 1988; Tutumluer & Pan, 2008). These studies have shown that as the angularity and surface texture of the particles increases there is an increase in M_R and resistance to PD.

The effect of shape on the performance of the aggregate is less well understood than angularity and surface texture. Generally it is thought for better UAB performance the aggregate particles need to be cubical as opposed to flat or elongated (Tutumluer, 2013), but little data has conclusively supported this hypothesis. Barksdale and Itani (1989) investigated the effect of particle shape on the permanent deformation of UAB materials and found that blade-shaped aggregates were 30 percent more susceptible to rutting than the other crushed aggregates. However, the authors stated that this observation was not conclusive and could be due to the scatter in their test data. No trend was observed by Chow (2014) in the amount of flat and elongated particles and the amount of PD accumulation or the friction angle of 16 UAB materials investigated. Uthus et al. (2007) found that as the particle became less cubical in a uniformly graded gneiss sample the ease of compaction decreases, but found no change in M_R or PD properties because of the change in particle shape.

Increase in particle breakdown in the LA and MD test was noted by Rismantojo (2002) as the particle became less cubical, but no trend was observed in MD testing conducted by Fowler et al. (2006). As seen in the varying results, particle shape either does not have a major impact on the performance of unbound materials or has not been

investigated thoroughly enough to understand the potential effects. As is the intent of this thesis, particle shape is investigated to better understand if there is an effect on performance. The next section describes the major ways that particle shape is classified.

2.4: Defining Shape of Coarse Aggregate Particles

The defining of particle shape can be broken into two broad testing methods. The first method of testing defines a particle by varying ratios between three different particle extents (length, width, and thickness). This method can be accomplished by either a manual method using some sort of caliper or by image analysis. The United States uses this type of analysis of coarse aggregate shape in UAB mixtures because the two testing standards institutes relevant for road construction in the United States, American Society for Testing and Materials (ASTM) and American Association of State Highway and Transportation Officials (AASHTO), both use this type of analysis. ASTM D4791 (2010) uses a manual method of analysis using a caliper. AASHTO on the other hand has discontinued their method on a manual method of analysis of particle shape and their current standards for particle shape classification, AASHTO PP64 (2012) and AASHTO TP81 (2012), use only image analysis processes.

The second method involves the use of standard gages to define the shape of coarse aggregate. There are two types of gages to separate both elongated and flaky particles based on the standard sieves that the particles pass and retain on. This method type is standardized by the British Standards Institute in BS 812 Section 105 and is used for classification of coarse aggregate particle shape in construction. The specifics of each method will now be discussed.

2.4.1: ASTM D4791

The ASTM D4791 *Standard Test Method for Flat Particles, Elongated Particles, or Flat and Elongated in Coarse Aggregate* is based on ratios between the length, width, and thickness. These three extents of a particle are shown in **Figure 10**. The length is the maximum dimension of the particle, thickness is the minimum dimension of the particle perpendicular to the length, and width is the intermediate dimension in a plane perpendicular to the length and thickness.

The particles above the 4.75 mm or 9.5 mm sieve are then classified into particle shape groups manually based on the ratio between the three particle extents and the method defined. The use of a caliper device, similar to the one shown in **Figure 11**, is used in this standard. The ratios specified in the ASTM are 2:1, 3:1, and 5:1. The ratio used is based on the application, as the ratios get larger they are less restrictive on the particle shape (ex. 2:1 ratio is more restrictive than the 5:1). The particle shape groups are dependent on whether Method A or B is used.

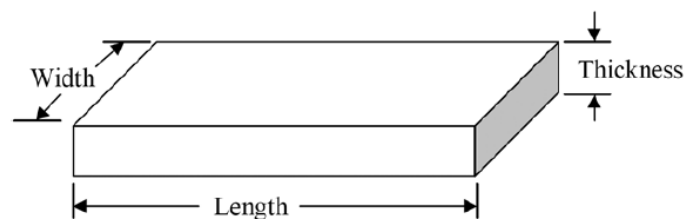


Figure 10. Coarse Particle Extents (ASTM D4791, 2010)



Figure 11. Example of Caliper Used in ASTM D4791

In Method A, the coarse particles are separated into four groups:

1. Flat (those particles of aggregate having a ratio of width to thickness greater than a specified value)
2. Elongated (those particles of aggregate having a ratio of length to width greater than a specified value)
3. Both completely flat and elongated (meeting both the criteria of both flat and an elongated particles), or
4. Not meeting the criteria of flat or elongated particles

In Method B, the coarse particles are separated into two groups:

1. Flat and Elongated (those particles having a ratio of length to thickness greater than a specified value), or
2. Not Flat and Elongated

Once the particle are separated into groups, based on the method, the percent of particles fitting into the different groups are calculated based on either the percentage by weight or particle count. The ratios used, method, and percentage of allowed particles in each particle classification is based on the agency implementing them.

2.4.2: AASHTO PP64 and TP81

As was mentioned before, the AASHTO PP64 (2012) *Standard Practice for Determining Aggregate Source Shape Values from Digital Image Analysis Shape Properties* and AASHTO TP81 (2012) *Standard Method for Test for Determining Aggregate Shape Properties by Means of Digital Image Analysis* define the particle shape by means of image analysis and still use ratios to define the shape of the particle. Additional information on the form of the coarse aggregate particles is also collected, such as angularity and surface texture is defined in these standards.

The ability to use digital image analysis to determined particle form characteristics has stemmed from development of such image analysis systems as the University of Illinois Aggregate Image Analyzer (UIAIA) (Pan & Tutumluer, 2006) and Aggregate Image Measurement System 2 (AIMS) (Gates, Masad, Pyle, & Bushee, 2011). The use of such image analysis systems allows for more accurate and repeatable measurements of particle shapes, but requires the purchase or development of expensive image analysis systems. For this reason, these procedures are mostly used for research purposes.

For analysis of the shape of coarse aggregate particle, the particles can be classified based on:

1. Flatness Ratio (average particle shortest dimension divided by the particle intermediate dimension)
2. Elongation Ratio (average particle intermediate dimension divided by the particle longest dimension)

3. Flat and Elongated Ratio (average particle longest dimension divided by the particle shortest dimension)

Since the image analysis procedure measures the aggregate extents directly, the aggregate particles can then be plotted based on the exact particle ratios that were measured. A plot that is produced by the AIMS2 image analysis system is shown in

Figure 12.

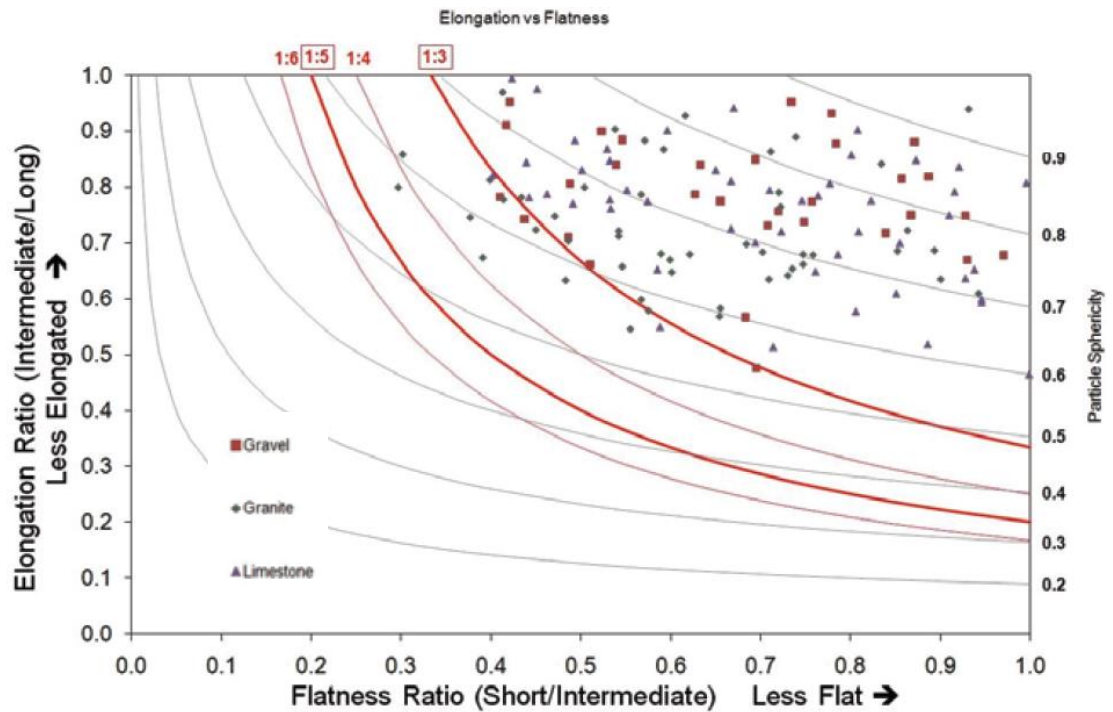


Figure 12. AIMS2 Particle Shape Analysis Output (Gates et al., 2011)

2.4.3: BS 812 Section 105

The *British Standard Testing Aggregate Section 105: Method for Determination of Particle Shape* is the method specified for particle shape classification of coarse aggregate particles in unbound materials for road applications, and is not used by agencies in the U.S.. There are two classifications procedures given in Section 105. The first method is *British Standard 812 Section 105.1 “Flakiness Index”* (1989). In this method an aggregate particle is classified as flaky if the particle has a thickness of less than 0.6 of their mean sieve size. The flakiness index of an aggregate sample is found by separating the flaky particles by a special sample divider and expressing their mass as a percentage of the mass of the sample tested. The other classification procedure given in Part 105 is *British Standard 812 Section 105.2 “Elongated Index of Coarse Aggregate”* (1990). An aggregate particle is classified as elongated if the particle has a length of more than 1.8 of its mean sieve size. The elongation index is found by separating the elongated particles by means of a length gage and expressing their mass as a percentage of the mass of sample tested. The two gages used by the British standard are shown in **Figure 13**.

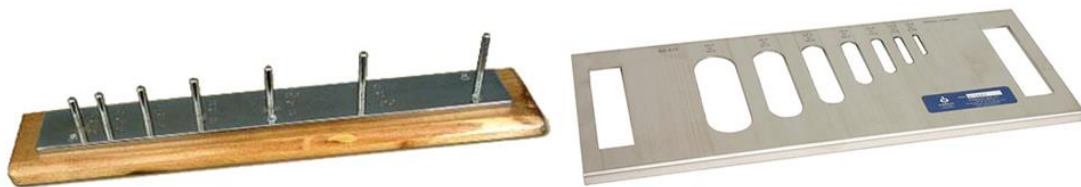


Figure 13. Elongated Index Gage (left), and Flakiness Index Gage (right)

2.5: State and International Specifications of Shape for Aggregates Used in Road Construction

An extensive review of UAB specifications developed by different department of transportation (DOT) agencies in the United States revealed that, seven of the fifty states have restrictions on aggregate shape in base applications. These seven states are Alabama, Maryland, New Jersey, New York, Pennsylvania, Vermont, and Virginia. The specified ratios, percentages, and method are presented in **Table 3**.

Table 3. Summary of DOT Specification on UAB Shape in U.S

States UAB Shape Specifications		
State	Percentages and Ratio	Method
Alabama	10% 5:1 or 20% 3:1	ASTM D4791 Method A
Maryland	15% 5:1	ASTM D4791 Method B
New Jersey	10% 5:1	ASTM D4791 Method B
New York	30% 3:1	ASTM D4791 (A or B not specified)
Pennsylvania	20% or 15% 5:1*	ASTM D4791 Method B
Vermont	30% 5:1	Vermont Test Procedure AOT-22 (Similar to Method B)
Virginia	30% 5:1	ASTM D4791 Method B

Note: *percentage is dependent on material type

There is a large variation in shape restriction stated by these specifications. The strictest specification is defined by Alabama, restricting flat and elongated particles to 20% by the 3:1 ratio ASTM D4791 Method A. While, Virginia and Vermont have less

stringent specifications limiting 30% of the particles shaped at the 5:1 ratio being classified as flat and elongated using ASTM D4791 Method B.

Also, in the United States varying standard agencies have recommended shape specification for UAB. AASHTO M 283 (1983) *Standard Specification for Coarse Aggregate for Highway and Airport Construction* specified a shape specification for base aggregate. The specification suggested that “the portion of aggregate retained on the 9.5 mm (3/8 in.) sieve shall not contain more than 15 percent of particles by weight so flat or elongated, or both that the ratio between the maximum and the minimum dimensions of a circumscribing rectangular prism exceeds 5:1.” This specification was discontinued by AASHTO due to lack of use.

ASTM also had a specification for base aggregate shape; ASTM D 693 (2008) *Standard Specification for Crushed Aggregate for Macadam Pavements*. This specification suggested that “the portion retained on the 9.5 mm (3/8 in.) sieve shall not contain more than 15% by mass, of particles so flat and elongated that the ratio between the maximum (length) and the minimum (thickness) dimensions exceeds 5:1”. This specification was also discontinued, without replacement, due to lack of use.

Although the standard was discontinued, the Federal Aviation Administration (FAA) in their Advisory Circular on Airport and Pavement Design Evaluation (2011) specified in Item P-209 Crushed Aggregate for UAB that “coarse aggregate portion, defined as the material retained on the No. 4 (4.75 mm) sieve and larger, shall contain no more than 15 percent, by weight, of flat or elongated pieces as defined in ASTM D 693.”

Internationally, countries have specified shape specification for base aggregate as well based on the British standard for coarse aggregate shape in UAB applications. In Australia there is a limit of 35 on flakiness index for base applications (Queensland, 2011) and Malaysia had a limit of 30 on flakiness index for base applications (Arshad & Rahman, 2008), with both methods being by BS 812 Section 105.1. As can be seen, the specifications vary in method and shape restriction for both the U.S. and Internationally.

3: MATERIALS

3.1: Aggregate Selected for the Study

Four different aggregate materials including diabase (igneous rock), dolostone (sedimentary rock), hornfels and slate (both metamorphic rocks) were obtained from quarries in the Commonwealth of Virginia. With the exception of slate, all materials were collected from facilities that produce commonly used aggregates in UAB applications in Virginia. The crushed slate was chosen for the study because of the large amount of flat pieces in the material to represent the extreme case in terms of percentages of flat pieces in a given aggregate. This material is not commonly used for UAB, but is sometimes used for unpaved road applications. All of these samples were 100% crushed by the quarries, which allowed for consistent levels of angularity across all four aggregate samples.

All as-received aggregate samples were evaluated for their index properties including the Atterberg Limits of the particles passing 425 μm (No. 40) sieve and specific gravity (both fine and coarse particles), and absorption of the coarse particles (**Table 4**). The grain size distributions of the as-collected materials were not evaluated in detail as the samples were re-sieved to create design gradations for the research as described in the subsequent sections.

Table 4. Index Properties of UAB Materials

Materials	Specific Gravity (G_s)		Atterberg Limits (ASTM D4318)	Absorption (%) (ASTM C127)
	Fine Particles (ASTM C128)	Coarse Particles (ASTM C127)		
Diabase	2.97	2.93	NP	0.6
Hornfels	2.97	2.81	NP	1.0
Slate	2.78	2.83	NP	0.4
Dolostone	2.82	2.84	NP	0.3

Note: NP = Non-Plastic, G_s = Bulk Specific Gravity

3.2: Geological Significance and Mineral Composition of Aggregate Particles Used in the Study

In order to better understand the mineralogical differences of the samples, each of the four aggregate types had x-ray diffraction (XRD) and thin sections tests conducted. Additionally, each aggregate is discussed in terms of how the aggregate forms and common applications for that rock type.

3.2.1: Diabase

Diabase is a fine- to medium-grained intrusive igneous rock that forms in dikes, sills and other shallow bodies. Due to the nature of formation, diabase is also usually classified as a trap rock, which also includes basalt, peridotite, and fine-grained gabbro. Diabase is a good source for aggregates used in road construction because of the high toughness of the material (Blyth & de Freitas, 1984) and is regularly used as UAB along the east coast from Virginia to New York and the west coast from California to Washington (Langer, 1988).

The diabase collected for this study contained roughly 40%, by weight, flat particles at the 3:1 ratio material retained on the 9.5 mm (3/8 inch) sieve. The specific mineral composition of the diabase collected for this study was primarily plagioclase and pyroxene and also contained mica and quartz, as shown by x-ray diffraction (**Figure 14**). This mineral composition is consistent with descriptions given by Blyth and Freitas (1984). The plagioclase minerals present in the rock is lath-shaped and enclosed by pyroxene (augite), as shown in thin section (**Figure 15a**).

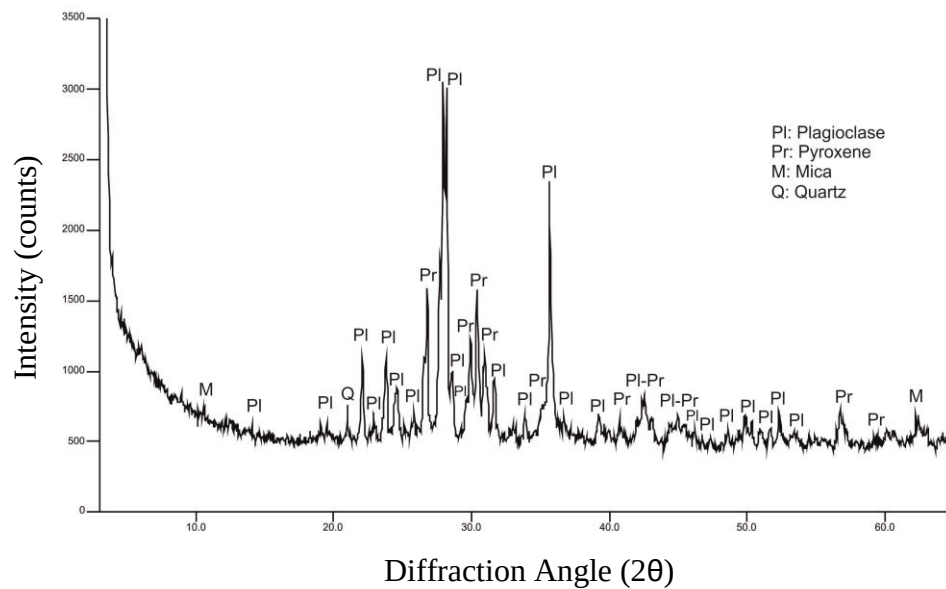


Figure 14. X-ray Diffraction Results of the Diabase Samples Collected For this Study

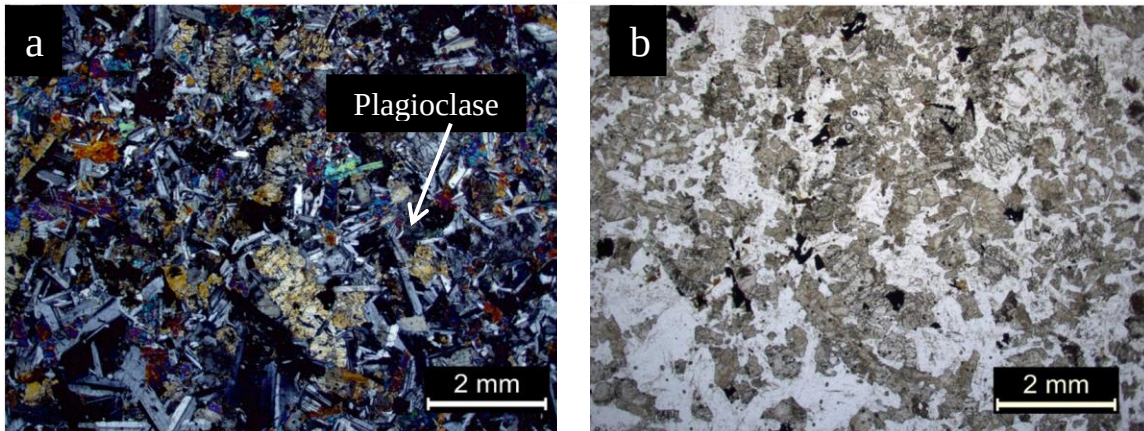


Figure 15. Diabase Thin Section; (a) Cross Polarization, (b) Parallel Polarization

3.2.2: Hornfels

One of the metamorphic rocks in this study is hornfels. Hornfels is classified as a metamorphic rock because it forms from clay or silt that is contact metamorphosed due to the heat produced from an igneous intrusion. Since hornfels is metamorphosed from a clay or silt material, with heat being the main mechanism (with little pressure) causing the transformation, the rock is fine-grained and has no mineral orientation. The main application of hornfels is use as an aggregate, such as UAB, because of its high strength and low abrasion (Waltham, 2009).

The hornfels samples collected for this study had flat particles above the 3:1 ratio at roughly 30% by weight material retained on the 9.5 mm (3/8 inch) sieve. The development of the fairly large amount of flat particles in both the diabase and hornfels samples could be caused by their brittle nature which can produce misshapen fragments by splintering during crushing processes (Waltham, 2009). The minerals present in the

hornfels samples collected for this study had large amounts of quartz, plagioclase, and pyroxene, with smaller amounts of mica and chlorite, as shown in **Figure 16**.

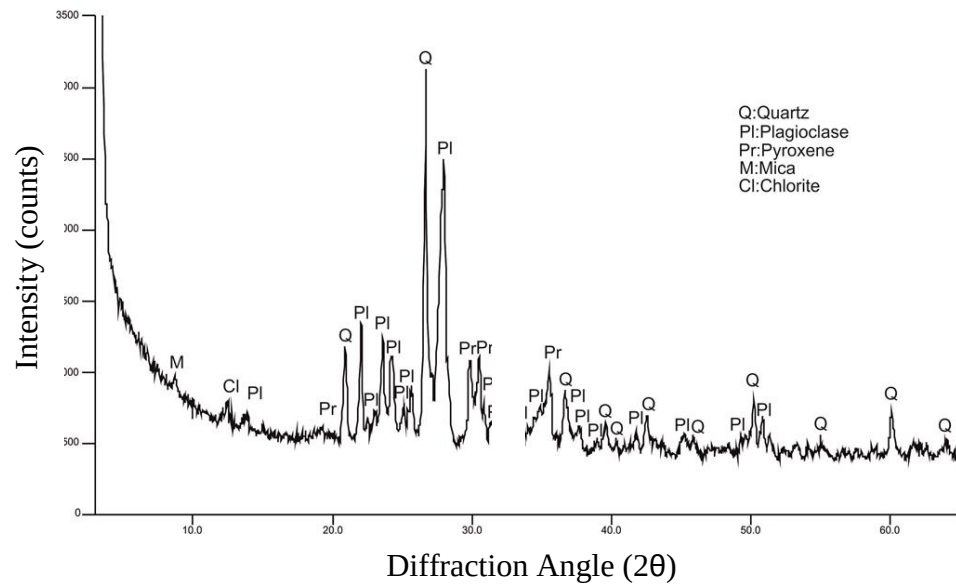


Figure 16. Hornfels X-ray Diffraction

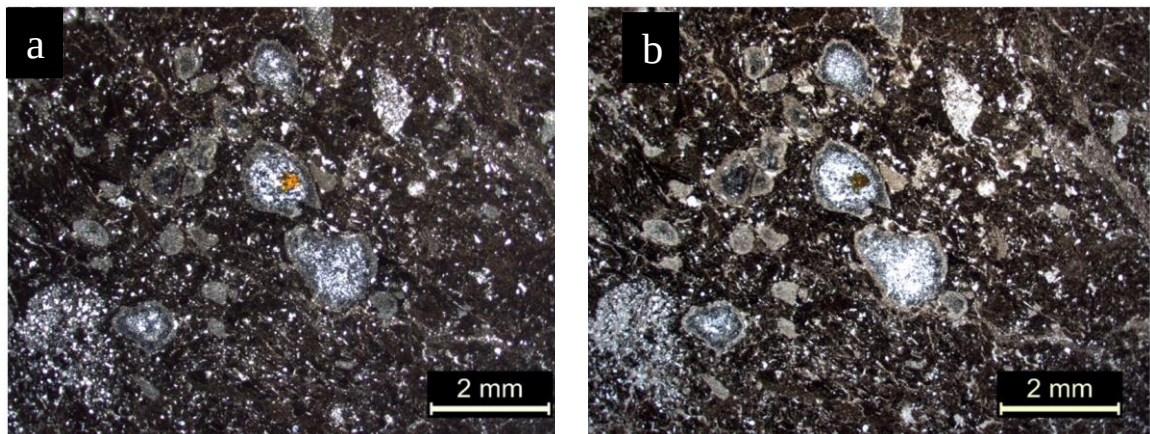


Figure 17. Hornfels Thin Section; (a) Cross Polarization, (b) Parallel Polarization

3.2.3: Dolostone

Dolostone is a sedimentary carbonate rock, which is typically grouped with descriptions of limestone due to the similarities of the two rock types. In the United States, two-thirds of the quarried stone is either dolostone or limestone (McNally, 1998). Use of carbonate rock as UAB materials is prevalent throughout the United States.

The dolostone samples collected from the quarry for this study contained vary little variation in particle shape, with less than 5% of the material retained on the 9.5 mm (3/8 inch) sieve being classified as either flat or elongated at the 3:1 ratio. The dolostone for this study was almost exclusively made of the mineral dolomite, with minor amounts of calcite, quartz, and feldspar (**Figure 18**). The dolomite mineral can be seen in **Figure 19a**, as dark rhomb-shaped crystals. The calcite mineral present in the sample can be seen in **Figure 19b**.

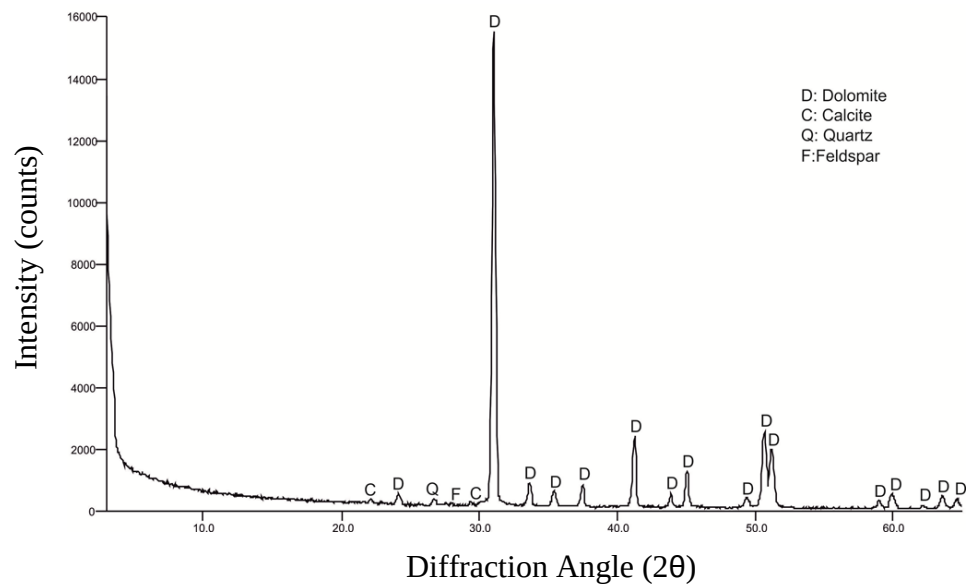


Figure 18. Dolostone X-Ray Diffraction

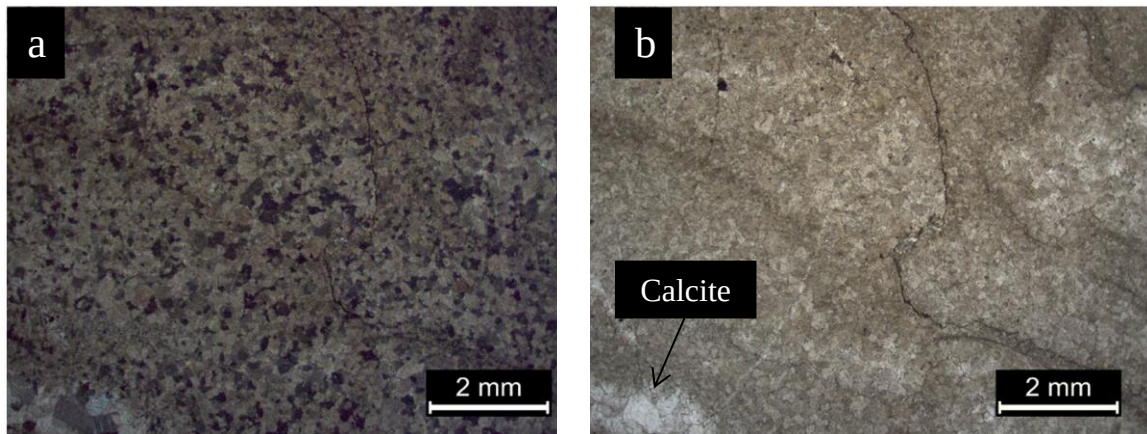


Figure 19. Dolostone Thin Section; (a) Cross Polarization, (b) Parallel Polarization

3.2.4: Slate

The other metamorphic rock collected for this study was slate. Typically, slate is not used as a road base because of the low abrasion resistance and the large amount of flat particles, but could be used if no better rock is available (Waltham, 2009). More

often, slate is used for roofing or ornamental stone (Blyth & de Freitas, 1984). In Virginia, slate is only used for limited unpaved road applications. Formations of slate have preferential direction of splitting, called a slaty cleavage, which results in large amounts of flat particles from crushing processes (Blyth & de Freitas, 1984).

The slate material used in this study had roughly 70% of the particles classifying as flat above the 3:1 ratio and 40% classifying as flat above the 5:1 ratio for material retained on the 9.5 mm (3/8 inch) sieve. The large amount of flat particles present in this material allowed for investigation of effects of flat particles with ratios above the 5:1 ratio. The mineral composition of the slate used in this study was mainly quartz, plagioclase, chlorite, and illite as shown in **Figure 20**. This mineral composition agrees with the main minerals specified by Blyth and Freitas (1984) for slate, with the exception of no sericite being present.

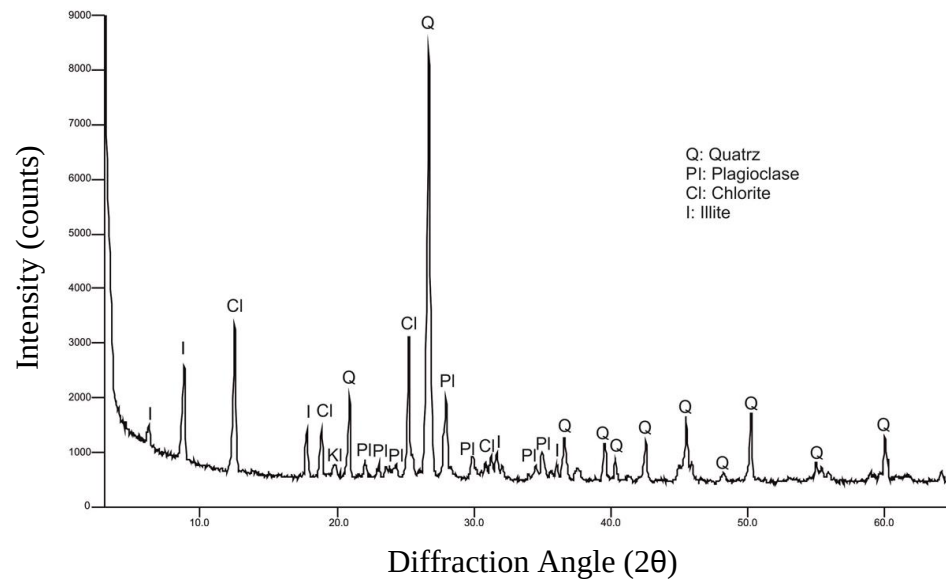


Figure 20. Slate X-Ray Diffraction

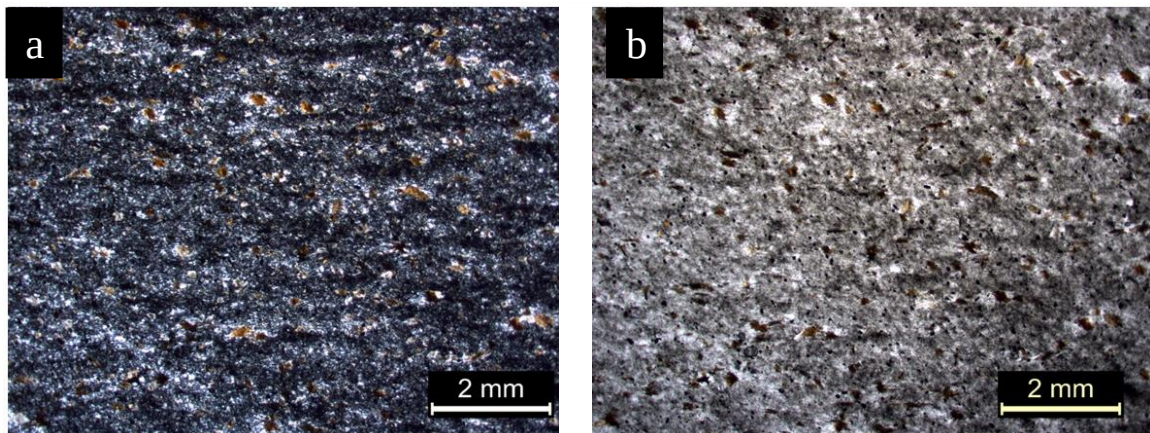


Figure 21. Slate Thin Sections; (a) Cross Polarization, (b) Parallel Polarization

4: METHODS

This chapter is written to present the methods associated with creating the different aggregate mixes used in this study, properly characterizing the UAB samples based on particle shape analyses, compaction, and aggregate matrix characterization, and classifying the material for durability (MD test), M_R , and PD.

4.1: Gradations for Each Aggregate Type

All collected material were first sieved to separate the size of the particles, washed, and then recombined to create two separate gradations as dense-graded (material with sizable fines content) and open-graded (material predominantly with coarse grained particles). These two gradations were chosen to simulate two very different gradations which are used as UAB in practice by different transportation agencies. The dense-graded gradation was modeled from the gradation used in the Commonwealth of Virginia and the open-graded gradation was modeled from a gradation used in the State of Wisconsin. The target gradations established from these sources are presented in **Table 5**.

Table 5. Target Percent Passing Gradation Curve for Each Gradation Type

Sieve (mm)	Dense-Graded Gradation	Open-Graded Gradation
25.0	100	100
9.50	70	60
4.75	60	25
2.00	48	15
0.425	25	0
0.075	6	0

Creating Samples With Different Particle Shape but Same Gradation

All aggregate samples retained above the 9.5 mm (3/8 in.) sieve were hand sorted using a caliper to separate the different particle shapes. The separation was performed with the procedures described in ASTM D4791 and the 9.5 mm (3/8 in.) sieve was chosen for the shape separation not only because ASTM specifies the 9.5 mm (3/8 in.) sieve as an option to separate the material, but also because particles below that sieve were too small to separate consistently by hand. The particle separation process was established based on Method A of ASTM D4791 as this method is particularly provided for the aggregate to be used in UAB applications. Flat particles and elongated particles were separated based on the 3:1 ratio. Additionally, slate flat particles were separated on both the 3:1 and 5:1 ratio because of the abundance of flat particles present in the as-received material.

The four aggregate types used in this study had varying aggregate shapes present in the as-received materials. Therefore, aggregates were separated based on the present particle shapes. The present particle shapes in the as-received material for the four

aggregate types are presented in **Figure 22** through **Figure 25**. The present shapes in the as-received material dictated the shape separation process for each aggregate type.

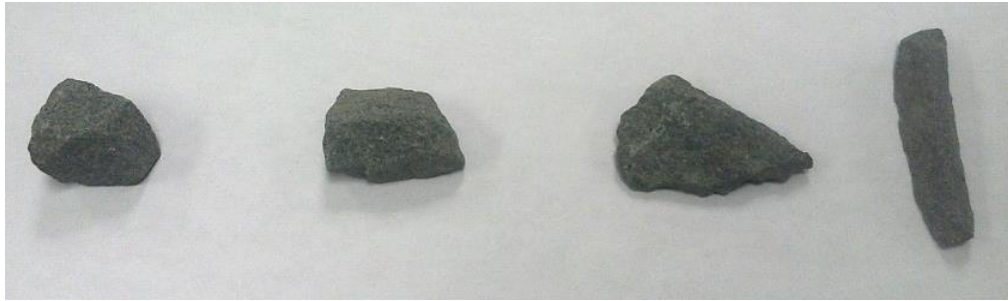


Figure 22. Diabase Representative Particle Shapes (from left to right): Equidimensional, Non-Equidimensional, Flat Above 3:1 Ratio, Elongated Above 3:1 Ratio



Figure 23. Hornfels Representative Particle Shapes (from left to right): Equidimensional, Non-Equidimensional, Flat Above the 3:1 Ratio, Elongated Above the 3:1 Ratio



Figure 24. Dolostone Representative Particle Shapes (from left to right): Equidimensional and Non-Equidimensional



Figure 25. Slate Representative Particle Shapes (from left to right): Non-Flat or Elongated Above the 3:1 Ratio, Flat Above the 3:1 Ratio Below 5:1 Ratio, Flat Above 5:1 Ratio, and Elongated Above the 3:1 Ratio

The separation process is outlined in **Figure 26**. Aggregate particles shapes separated in the diabase and hornfels as-received material were non-flat or elongated above the 3:1 ratio, elongated above the 3:1 ratio, and flat above the 3:1 ratio. Then, the non-flat or elongated particles were visually separated into equidimensional and non-equidimensional shapes. Equidimensional were classified as particles having equal extents in the length, width, and thickness dimensions. Non-equidimensional particles were then classified as particles that were not classified as equidimensional.

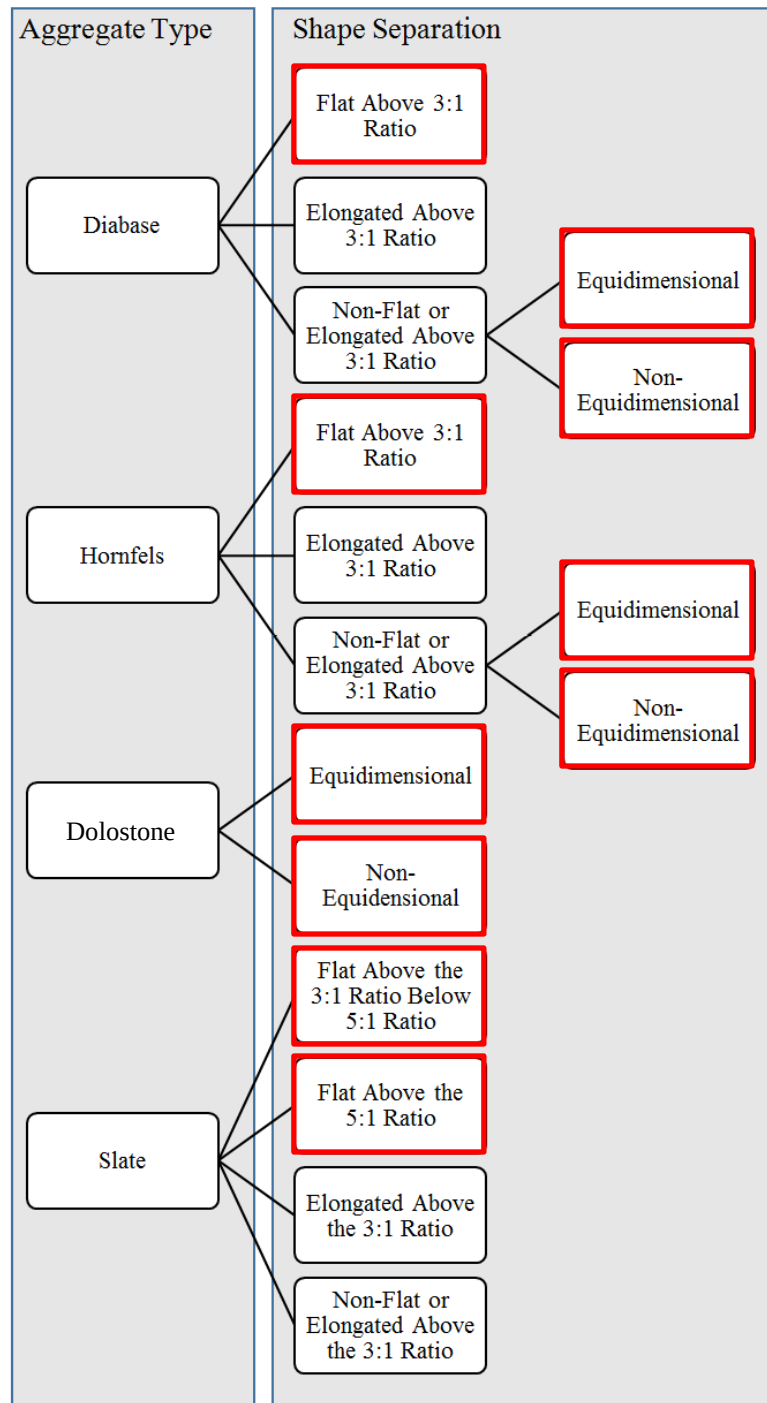


Figure 26. Aggregate Shape Separation Process

Note: Boxes outlined in red are shapes used in this thesis

Aggregate particle shapes separated in the dolostone as-received material was limited because of the very small amount of flat or elongated particles, less than 5% by mass. So, dolostone aggregate particles were separated into two categories visually; equidimensional and non-equidimensional as discussed for the diabase and hornfels samples. Lastly, the aggregate particle shapes separated in the slate as-received material were non-flat or elongated above the 3:1 ratio, elongated above the 3:1 ratio, flat above the 3:1 ratio and below the 5:1 ratio, and flat above the 5:1 ratio. As mentioned before, the flat particles of the slate samples were separated based on two separate flat ratios to develop varying degrees of flat samples and to develop flat samples that were comparable to the flat particles in the diabase and hornfels samples.

Not all of the separated particles were used in the study. The lack of particles elongated above the 3:1 ratio in all aggregate types results in to little material to use in testing. Additionally, not enough material of non-flat or elongated above the 3:1 ratio was present in the slate as-received material.

Once all of the material above the 9.5 mm (3/8 in.) sieve was separated based on particle shape and grain size, they then were combined into specified gradations based on the target percentages. **Table 6** presents the different samples that were created for each material type based on different gradations and particle shapes. All materials were combined based on weight. Maximum variation in particle shape was developed for each aggregate type based on the as-received present particle shapes.

Combining of particle shape types was used in one type of particle shape variation in the diabase and hornfels material. The combining of 40% flat particles above the 3:1

ratio and 60% non-equidimensional particles by weight was used to represent an aggregate type that had a distribution of flat particles. The non-equidimensional particles were mixed in the samples, as opposed to the equidimensional particles, to better represent an aggregate mixture with particle shapes that were less cubical but not 100% flat above the 3:1 ratio.

Table 6. Materials Combined For the Study

Gradation	Aggregate Type	Particle Shape Above 9.5 mm Sieve (% by Weight)	Sample Name
Dense-Graded	Diabase	100 % Equidimensional	DD100%E
		100 % Non-Equidimensional	DD100%NonE
		60 % Non-Equidimensional and 40% Flat Above 3:1 Ratio	DD40%F↑3:1
		100 % Flat Above 3:1 Ratio	DD100%F↑3:1
	Hornfels	100 % Equidimensional	DH100%E
		100 % Non-Equidimensional	DH100%NonE
		60 % non-Equidimensional and 40% Flat Above 3:1 Ratio	DH40%F↑3:1
		100 % Flat Above 3:1 Ratio	DH100%F↑3:1
	Dolostone	100 % Equidimensional	Dd100%E
		100 % Non-Equidimensional	Dd100%NonE
	Slate	100 % Flat Above 3:1 Ratio Below 5:1 Ratio	DS100%F↑3:1↓5:1
		100 % Flat Above 5:1 Ratio	DS100%F↑5:1
Open-Graded	Diabase	100 % Equidimensional	OD100%E
		100 % Non-Equidimensional	OD100%NonE
		60 % Non-Equidimensional and 40% Flat Above 3:1 Ratio	OD40%F↑3:1
		100 % Flat Above 3:1 Ratio	OD100%F↑3:1
	Hornfels	100 % Equidimensional	OH100%E
		100 % Non-Equidimensional	OH100%NonE
		60 % non-Equidimensional and 40% Flat Above 3:1 Ratio	OH40%F↑3:1
		100 % Flat Above 3:1 Ratio	OH100%F↑3:1
	Dolostone	100 % Equidimensional	Od100%E
		100 % Non-Equidimensional	Od100%NonE
	Slate	100 % Flat Above 3:1 Ratio Below 5:1 Ratio	OS100%F↑3:1↓5:1
		100 % Flat Above 5:1 Ratio	OS100%F↑5:1

Sample names noted in **Table 6** were created to describe each sample composition. The first letter of the sample name represents the overall gradation (i.e.,

dense (D) vs. open (O) graded). In the MD testing samples names, the first letter describing the gradation was omitted since the gradation is specified in the test method. The second letter represents the geological classification of the source rock of the aggregate, with an upper case D representing diabase, a lower case d representing dolostone, H representing hornfels, and S representing slate. The percentage and the letters afterwards describe the particle shape composition (e.g., 100%E indicating 100% equidimensional pieces or 40% F3:1 represents 40% of the sample consisting of flat pieces above 3:1 ratio etc.). The arrows represent whether particles used was above or below a specified flatness ratio. This distinction is important in comparing the 100% flat samples. For examples, 100% flat particles above the 3:1 ratio for the diabase and hornfels samples are the same shape as the slate 100% flat particles above the 3:1 ratio and below the 5:1 ratio, but not similar to the 100% flat particles above the 5:1 ratio.

The sample compositions depicted in **Table 6** are particularly developed to create a matrix to study not only the effects of particle shape in a given mineralogical composition (e.g., changes in particle shape from equidimensional to flat pieces) but also to compare the effects of particle shape based on differences of mineralogical composition (e.g., differences between samples of DD100%E, DH100%E, Dd100%E, etc.).

4.2: Particle Shape Analysis

Quantifying the particle shape was determined by first creating batches of particles that were going to be used for a particular group of tests. Overall in this study five test batches were quantified for particle shape. The five batches corresponded to

particles above the 9.5 mm (3/8 in.) sieve that were going to be used in compaction of dense-graded materials, compaction of open-graded materials, MD testing, M_R and PD testing of dense-graded materials, and M_R and PD testing of open-graded materials.

From each test batch 100 individual particles were then selected. Previous studies indicate that at least 50 to 60 particles are needed for each classification before a confident characterization of a given aggregate sample can be achieved (Fowler et al., 2006). Each aggregate particle was then evaluated to determine the length to width, width to thickness, and length to thickness ratios to the nearest one-quarter ratio. The weight of each individual particle was also noted. These classification batches were then used to create the desired samples depicted in **Table 6** for the particular test that was being conducted.

Determination of each particle dimension was performed following a method similar to the one used by Fowler et al. (2006). A four-station proportional caliper device equipped with two small light-gauge steel angles was used by Fowler et al. (2006) which allowed for measurement of ratios of width to thickness, length to thickness, and length to width to the nearest one half of a ratio for an individual particle. Similar equipment was used in this study, but the aluminum plates were modified to determine the ratios to the nearest one-quarter ratio (more precise measurement). The caliper that was used in this study is shown in **Figure 27**.

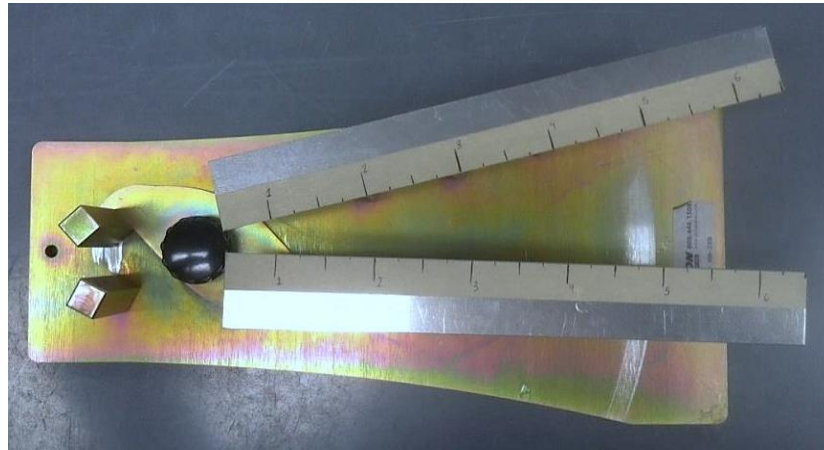


Figure 27. Caliper Device Used in This Study

During particle shape classification, measurements for each aggregate particle were recorded into a spreadsheet as well as the weight of that particle. The average ratio between the length to width from each sample batch was then used to classify the elongation index and the average ratio between the width to thickness was used to classify the flatness index of a given sample batch. These two indices were used to define the overall shape of the sample (Lees, 1964).

In this study, the average elongated and flatness indices for each batch were both used to classify the samples as well as to determine the average particle shape ratio. A graph similar to the one developed by Barksdale and Itani (1989) was created to achieve this comparison (**Figure 28**). The elongated and flatness indices determined from the average ratio of 100 particles were then related to the particle shape ratio by also marking the flat ratio and elongated ratios that are defined by ASTM D4791 Method A.

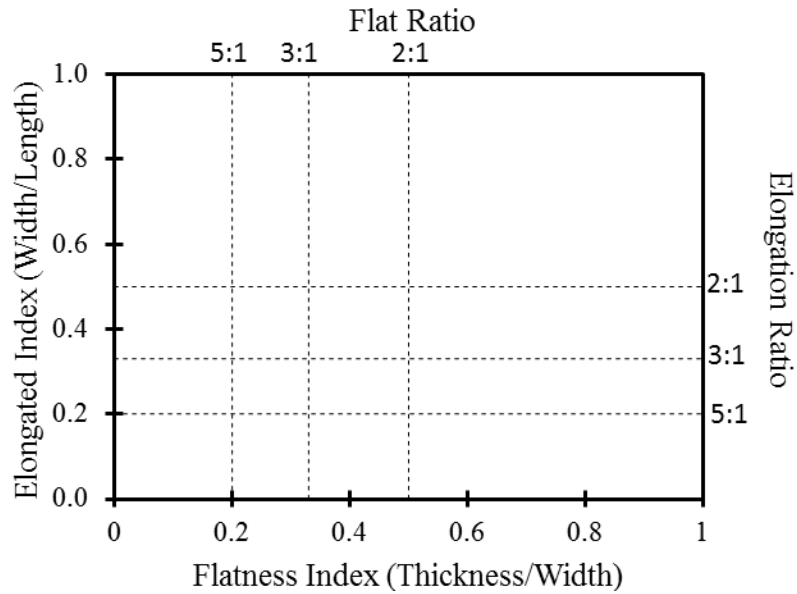


Figure 28. Graphical Representation of Particle Shape after (Barksdale and Itani, 1989)

Additionally, the percent by weight of particles classified as flat at the 2:1, 3:1, and 5:1 ratio, elongated at the 2:1, 3:1, and 5:1 ratio, and flat and elongated at the 2:1, 3:1, and 5:1 ratio per ASTM D4791 Method A and B were determined. For example, in ASTM D4791 Method A a particle is classified as flat at the 3:1 ratio when the width of the particle is three times the thickness and elongated at the same ratio when the length is three times the thickness. In ASTM D4791 Method B, a particle is classified as flat and elongated at the 3:1 ratio when the length of the particle is three times the thickness. Since state specifications in the United States dictating particle shape of UAB use both method A and B of ASTM D4791 (as shown in **Table 3**) it was important to know how the samples used in this study related to both Method A and Method B of ASTM D4791.

These method of classify particle shape was chosen due to the simplicity of the equipment and the ability to distinguish particle shapes present in the samples. The goal

of the classification process was to ensure that similar mineralogical samples actually do have varying particle shape and that varying mineralogical samples could be grouped together base on particle shape.

4.3: Compaction Procedure

Target compaction densities of aggregates used in this study were determined based on the maximum dry density ($\rho_{d \text{ max}}$) and optimum moisture contents (w_{opt}) determined from laboratory testing. However, two different compaction procedures were followed for the dense- and open-graded aggregates based on the differences in grain size distributions and the relevancy of the compaction methods.

For the samples created with particle size distribution consistent with a dense-graded UAB gradation, samples were compacted following the Proctor compaction method. The method for Proctor compaction was based on ASTM D698 (2012). Minimum of four points were compacted at specific moisture contents to determine the w_{opt} and $\rho_{d \text{ max}}$.

For samples created with the open-graded UAB gradation, a variation of ASTM D698 was followed based on research conducted by Richardson and Lusher (2009). The problem observed by Richardson and Lusher (2009) with compaction of open-graded base materials by the standard proctor was the lack of a defined density-moisture curve. Open-graded materials are highly porous and do not hold large amounts of moisture due to the lack of fines particles, so no optimum moisture content for compaction is present for open-graded materials. Richard and Lusher (2009), based on review of state highway

specifications, used an effective moisture content of 2% for compaction of open-graded materials as an “optimum moisture content”.

The effective moisture content is defined as the total moisture, amount of moisture added to the mixture, minus the absorption of the coarse aggregate as determined by ASTM C 127 (2015). The effective moisture is then the amount of surface moisture present in an open-graded mixture. An effective moisture content of 2% was used to allow surface moisture to hold smaller particles to the coarser particles and prevent segregation.

Based on the effective moisture content concept, a one-point proctor using standard effort at an effective moisture content of 2% was conducted for each open-graded aggregate sample and the maximum dry density was determined. **Table 7** shows the effective moisture used for each aggregate type for open-graded compaction testing.

Table 7. Effective Moisture Content Used for Each Aggregate Type in Open-Graded Compaction Testing

Aggregate Type	Absorption (%)	Total Moisture (%)	Effective Moisture (%)
Diabase	0.6	2.6	2.0
Hornfels	1.0	3.0	2.0
Slate	0.4	2.4	2.0
Dolostone	0.3	2.3	2.0

No material was scalped before compaction and a 15.24 cm (6 in.) diameter mold was used for compaction of open-graded samples. In order to understand the variability

of the test and better define if a variation was present due to particle shape; three one point proctors were conducted for each open-graded sample.

4.4: Aggregate Matrix Characterization

The method used to quantify the nature of the matrix structure of the UAB samples was the Bailey method of gradation selection (Vavrik, Huber, Pine, Carpenter, & Bailey, 2002). The method was originally developed for studying asphalt concrete mixes, but has been successfully used by Bilodeau and Dore (2012) to study the matrix and interlocking characteristics of unbound aggregate material. As discussed in section 2.3.1, there are two components that make up the matrix structure of unbound aggregate mixtures; PS and SS. The PS is composed of the coarse aggregate that could be in contact with one another and the SS is composed of fine aggregate that fills in the voids between the PS. In the Bailey method only the terms coarse and fine aggregate are used, but for clarity of the terms previously defined in section 2.3.1 the coarse aggregate will be called the PS and the fine aggregate will be called the SS.

The Bailey method distinguishes between the PS and SS by means of a primary control sieve (PCS). Particles retained above the PCS are considered the PS and materials passing the PCS are the SS. The PCS is determined from equation 14.

$$PCS = 0.22 \cdot NMPS \quad (14)$$

where;

PCS is the Primary Control Sieve; and

NMPS is the Nominal Max Particle Size defined as one sieve larger than the first sieve that retains more than 10% by weight.

The 0.22 represents an average value found from both 2-D and 3-D analysis of the packing potential of various particles (Vavrik et al., 2002). The PCS is then rounded to the nearest sieve size that is used during grain size distribution determination. The NMPS was determined for both the dense- and open-graded materials used in this study as the 19.0 mm (3/4 in.) sieve, which makes the PCS the 4.75 mm (No. 4) sieve.

With the PS and SS determined, the Bailey method specifies determining the loose unit weight (LUW) of the PS and rodded unit weight (RUW) of the SS by AASHTO T 19 (2009). The LUW determination involves placing material in a measure of known volume and determining the unit weight. The RUW tests are conducted by placing material in a measure in three lifts and densifying each layer by means of agitation by a rod.

The LUW of the PS can then be compared to the density of the PS in the compacted material as defined by equation 15.

$$\%LUW_{PS} = 100 \cdot \left(\frac{\%PS \cdot \rho_d}{LUW_{PS}} \right) \quad (15)$$

where;

$\%LUW_{PS}$ is the PS density in the compacted mix compared to its LUW;

$\%PS$ is the percentage by weight of the PS inside the aggregate mix;

ρ_d is the dry density of the total aggregate mixture; and

LUW_{PS} is the LUW of the PS.

The %LUW_{PS} is used to determine if the PS particle are in contact with each other. Based on numbers given in the Bailey method (Vavrik et al., 2002), if the %LUW_{PS} is above 95% then the PS particles are in complete contact and are therefore the load bearing skeleton of the mix, if the %LUW_{PS} is between 95% and 90% then the PS particle are in a transition state between complete contact and a floating state, and if the %LUW_{PS} is below 90% then the PS particles are not in contact and do not make up the main load carrying skeleton.

The RUW is used to determine how the SS fill the voids between the PS, as defined by equation 16.

$$\%RUW_{SS} = 100 \cdot \left(\frac{\%SS \cdot \rho_d}{RUW_{SS} \cdot V_V^{SS}} \right) \quad (16)$$

where;

%RUW_{SS} is the SS density in the compacted mix compared to its RUW;

%SS is the percentage by weight of the SS in the aggregate mix;

ρ_d is the dry density of the total aggregate mixture;

RUW_{SS} is the RUW of the SS; and

V_V^{SS} is the volume of voids between the PS (available voids that the SS can fill)

and is further defined in equation 17.

$$V_V^{SS} = 1 - \left(\frac{\%PS \cdot \rho_d}{G_S^{PS} \cdot \rho_w} \right) \quad (17)$$

where;

G_S^{PS} is the specific gravity of the PS; and

ρ_w is the density of water.

The %RUW_{SS} is used to determine if the SS is filling the voids between the PS. If the %RUW_{SS} is near 100% then the fine aggregate particles are densifying during compaction and could either be adding stability to the PS or disrupting the structure of the PS based on the value of the %LUW_{SS}. If the %RUW_{SS} is not near 100% and the PS is in complete contact, then the fine aggregate particles are not completely filling the voids left between the PS and do not make up the main load carrying skeleton of the aggregate structure.

This method was used to interpret the matrix structure for each aggregate sample of both the dense- and open-graded materials. Each aggregate sample mixture had two LUW_{PS} tests and two RUW_{SS} conducted at the appropriate grain size distributions that were then averaged to determine the LUW_{PP} and RUW_{SS} of the sample. The RUW tests were conducted using a 100-mm diameter proctor mold with approximately a 0.9 liter volume and the LUW tests were conducted using a 150-mm diameter proctor mold with approximately a 2.1 liter volume.

4.5: Micro-Deval Procedure

The MD testing was conducted based on ASTM D6928 (2010). The grading used for testing is specified in Section 8.2 with 375 g between 19.0-mm and 16.0-mm sieves, 375 g between 16.0-mm and 12.5-mm sieves, and 750 g between 12.5-mm and 9.5-mm sieves. This gradation was chosen for testing because it best represented the coarse aggregate fraction in both gradations used for testing. The samples were soaked for the specified hour before testing in 2 L of tap water. Then, the samples were combined with the 5000 g of steel balls and rotated for 12,000 revolutions inside abrasion jars. Two tests

were conducted for each sample type to insure that the coefficient of variation (two standard deviations) was within the 9.6% limit specified by the standard, with four tests conducted for one sample type.

4.6: Resilient Modulus Procedure

The M_R of samples tested were determined based on the procedure outlined in AASTHO T 307 (2007) for Type I materials (granular bases). A Geocomp LoadTrac-II RM with an electric linear actuator was used for testing of samples. A 0.1 second haversine loading pulse followed by a 0.9 second resting period was implemented by the actuator for traffic loading simulation. Two linear variable differential transducers (LVDTs) mounted externally to the load cell were used to determine deformations. Air pressure was used as the confining fluid and was automatically applied and maintained by an electro-pneumatic air pressure regulator. The loading sequence, confining pressure, and data acquisition were controlled by a computer equipped with RM 6.0 software. Resilient modulus testing equipment used in laboratory testing is shown in **Figure 29**.

Specimens used for testing were first combined based on target gradations (Table 5) with at least 600 grams of additional dry materials added for compaction moisture determination after testing. Samples were then mixed with desired moisture and were set aside for a minimum of 30 minutes after they were wrapped with plastic bags (this was a variation from the 16 to 18 hours stated in AASHTO T-307). All dense-graded aggregates were conditioned to optimum moisture content. For open-graded material, an effective moisture content of 2% was used as discussed in the section 4.3. All samples

were prepared based on $\rho_{d \max}$ as determined from compaction testing for each aggregate sample.

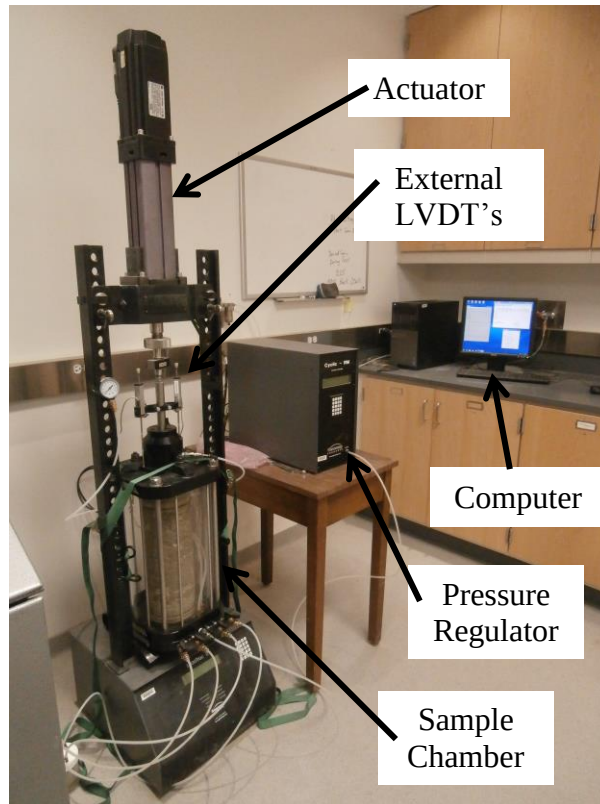


Figure 29. M_R Testing Equipment Used in This Study

When prepared for M_R testing, samples were compacted in 6 lifts using a vibratory hammer in a split mold of 15.2 cm (6 in.) diameter and 30.5 cm (12 in.) height with a 0.305 mm (0.012 in.) thick latex membrane placed inside. The left over material that was not used in the samples was then used to confirm compaction moisture content. The split mold was removed and an additional 0.305 mm (0.012 in.) thick latex

membrane was placed around the outside of the sample. The additional membrane was needed due to punctures that developed in the first membrane from compaction.

Once the split mold was removed, samples were then placed inside the sample chamber. Per AASHTO standard procedures, the specimen were subjected to initial loading, referred to as conditioning, under equal confining and deviator pressures of 103 kPa (15 psi) for 1,000 cycles. The loading sequence followed as outlined in AASHTO T-307 is presented in **Table 8** below. The test was terminated if the vertical permanent deformation reached 5% of the total height of the specimen. The moduli from the last five cycles of each test sequence were found using equation 1 for each LVDT, then the moduli values reported were then the averaged M_R values obtained from the two external LVDT's.

Table 8. AASHTO T-307 Testing Sequence for Base Materials

Sequence No.	Confining Pressure		Max Axial Stress		Deviator Stress		Constant Stress		No of Load Applications
	kPa	psi	kPa	psi	kPa	psi	kPa	psi	
Conditioning	103.4	15	103.4	15	93.1	13.5	10.3	1.5	1,000
1	20.7	3	20.7	3	18.6	2.7	2.1	0.3	100
2	20.7	3	41.4	6	37.3	5.4	4.1	0.6	100
3	20.7	3	62.1	9	55.9	8.1	6.2	0.9	100
4	34.5	5	34.5	5	31.0	4.5	3.5	0.5	100
5	34.5	5	68.9	10	62.0	9.0	6.9	1.0	100
6	34.5	5	103.4	15	93.1	13.5	10.3	1.5	100
7	68.9	10	68.9	10	62.0	9.0	6.9	1.0	100
8	68.9	10	137.9	20	124.1	18.0	13.8	2.0	100
9	68.9	10	206.8	30	186.1	27.0	20.7	3.0	100
10	103.4	15	68.9	10	62.0	9.0	6.9	1.0	100
11	103.4	15	103.4	15	93.1	13.5	10.3	1.5	100
12	103.4	15	206.8	30	186.1	27.0	20.7	3.0	100
13	137.9	20	103.4	15	93.1	13.5	10.3	1.5	100
14	137.9	20	137.9	20	124.1	18.0	13.8	2.0	100
15	137.9	20	275.8	40	248.2	36.0	27.6	4.0	100

The nonlinear behavior of the UAB material was then defined by the MEPDG model (Equation 4). This model was chosen based on the fact that it is the suggested model for estimating UAB M_R response in the new MEPDG design methodology (AASHTO, 2008). After running the M_R test, the whole sample was then oven dried to determine the moisture content after the test.

4.7: Permanent Deformation Procedure

As discussed in section 2.1.3, typical pavement stresses induced in the base layer over a range of pavement structures was presented. Barksdale and Itani (1989), based on their review of typical pavement stresses, suggested stress states for permanent deformation testing as presented in **Table 9**. These stresses are very similar to the stresses

used to simulate permanent deformation stresses in a UAB layer in a study conducted by Chow (2014).

Table 9. Suggested Stress States for Laboratory Permanent Deformation
(Barksdale & Itani, 1989)

Pavement Structure	Stress State		
	σ_3 (kPa)	σ_3 (psi)	σ_1 / σ_3
Light	41.4	6	6
Medium	41.4	6	4
Heavy	31.0	4.5	2

The performance of the base layer is most important in light and medium pavement structures since the base layer is experiencing the highest stresses, so two permanent deformation tests were conducted to represent the typical stresses presented by Barksdale and Itani (1989) for the light and medium pavement structures. The single stage permanent deformation tests conducted in this study are shown in **Table 10** with the representative situations modeled, stresses used, and applied loads. The equipment and sample preparation for permanent deformation was the same as for M_R testing. The single stage permanent deformation tests first consisted of conditioning the sample in the same pressures used in AASHTO T-307 (103.4 kPa confining and 103.4 kPa deviator) for 500 cycles and then applying the specified sequence afterwards. This conditioning sequence was performed to remove the majority of the irregularities at the top and bottom of the tested sample.

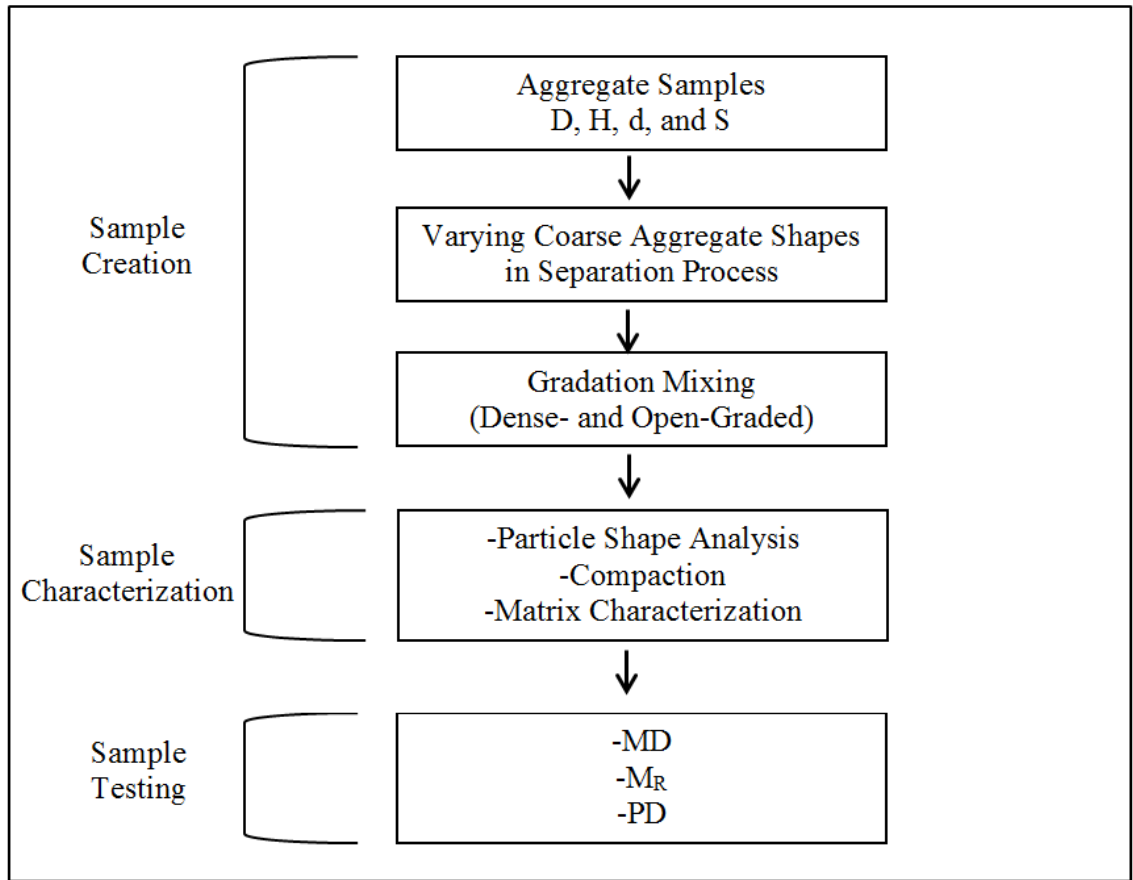
Table 10. Single Stage Permanent Deformation Testing Sequence

Field Condition	σ_3		σ_d		Loads application
	(kPa)	(psi)	(kPa)	(psi)	
Light Pavement	41.4	6	206.8	30	10,000
Medium Pavement	41.4	6	124.1	18	10,000

The selection of 10,000 load applications for the light and medium pavement permanent deformation testing was selected based on similar studies conducted by Kumar et al. (2006), Tao et al. (2010), and Chow (2014). The drawback to this load duration could be insufficient length to define the point when there is a transition from plastic creep to incremental collapse at very high load applications as noted by Werkmeister (2003).

4.8: Overview of Testing Protocol Followed in the Study

The overall process of sample creation, characterization, and testing for this study is presented in **Figure 30**. This process allowed for study of how coarse aggregate particle shape in UAB could affect the durability (MD test), M_R and PD response of both dense- and open-graded materials within a particular mineralogy (ex. comparing diabase samples with varying shape) and across varying mineralogies (ex. comparing diabase and dolostone samples with similar shape) could be compared.



Note: D = Diabase, H = Hornfels, d = Dolostone, S = Slate, MD = Micro-Deval, M_R = Resilient Modulus, PD = Permanent Deformation

Figure 30. Overview of Testing Matrix Followed in This Study

5: RESULTS

The results section is divided into three sections; (1) results for sample characterization testing, (2) summary of experimental testing results, and (3) interpreted results. The sample characterization section will present results for particle shape analysis, compaction of both dense- and open-graded materials and matrix characterization by the Bailey method. The experimental testing section will present the MD results, M_R testing sample preparation and model regression analyses, PD sample testing preparation and model regression analyses, and grain size analysis results of both the M_R and PD test samples. The interpreted results section will present interpreted results of the M_R and the PD tests with regard to the effect of particle shape and mineralogy.

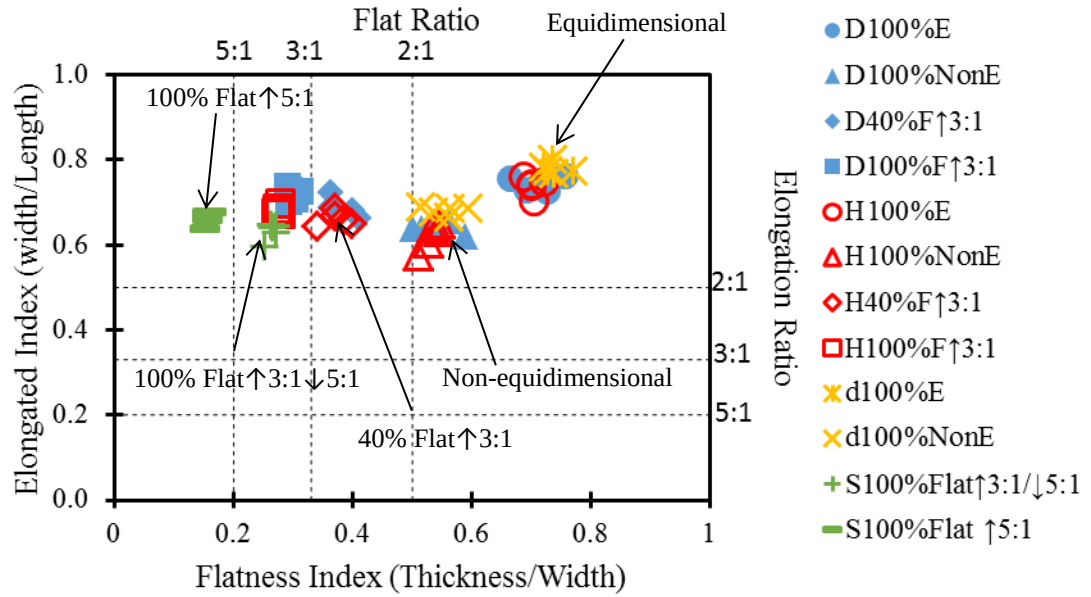
5.1 Characterization of Materials

5.1.1 Particle Shape Analyses

Aggregate samples with four different mineralogical compositions could be segregated into five different batches based on their particle shape as shown in **Figure 31** following the procedures described in Section 4.1. This process allowed the particles to be grouped independent of their mineralogy as:

- Equidimensional shaped particles;
- Non-equidimensional shaped particles;

- 40% of particles classified as flat and above the 3:1 ratio;
- 100% of particles classified as flat above 3:1 ratio and below 5:1; and
- 100% of particles classified as flat above the 5:1 ratio.



Note: Symbols Used in This Figure Have Been Previously Defined in Table 6

Figure 31. Particle Shape Factors Found for Five Testing Batches

Based on the classification shown on **Figure 31**, for example, samples from diabase, hornfels, and dolostone could all be grouped together with equidimensional particle shapes. This grouping also allowed creation of a wide range of particles with different flatness index (i.e., ranging from 0.15 to 0.8), but more similar elongated index (i.e., elongated index ranging from 0.55 to 0.8). **Figure 31** also allows the comparison of samples with the same mineralogical composition, but with varying particle shape (e.g., diabase and hornfels samples showing a large variation in flat particles).

To explain the relevancy and convenience of the classification shown on **Figure 31**, a detailed comparison was made between the “Flatness Index” defined by Barksdale and Itani (1989) and particle shapes defined with ASTM methods D4791A and B. The flatness index values for each particle shape was plotted against the percentage of flat or elongated particles by weight as defined by ASTM D4791 Method A and flat and elongated particles as defined by ASTM D4791 Method B (**Figure 32**). Details of individual particle shape analyses recorded for this comparison are given in **Appendix A**.

When the results obtained from Method A were compared with each other, **Figure 32B** and **C** shows that almost no individual particles were classified as “elongated” for the particles with a ratio of 3:1 and 5:1. Therefore the main variations for these particles were due to the range in flat shapes as shown in **Figure 32 D** and **E**. For the particles with a ratio of 2:1, there was an elongated range of up to 40% (**Figure 32 A**), however the primary variation was still due to the flat pieces as the flat particles range from 0 to 100% by weight (**Figure 32 D, E, and F**). The diabase and hornfels samples containing 40% flat particles at the 3:1 ratio by ASTM Method A reflect that contained percentage since the measured percentage for the samples ranged between 35% and 40% (**Figure 32 E**). The diabase, hornfels, and slate samples with 100% flat particles above the 3:1 ratio by weight when measured ranged between 75% and 100% (**Figure 32 E**).

When the results obtained from Method B were compared, it can be seen that higher percentage of particles were classified as flat and elongated across all three ratios (**Figure 32 G, H, and I**) when compared to the percentage of particles that classified as

flat by Method A (**Figure 32 D, E, and F**). Results obtained from Method B are difficult to interpret without also knowing the results from Method A. For example, **Figure 32 G** shows an initial curvature and then a somewhat linear relationship, which is due to the results shown on **Figure 32 A** (a small range in elongated ratio) and **D** (a wide range of flat ratio). The same relationship is also true for **Figure 32 H and I** where not having any elongated ratios (**Figure 32 B and C**) results in more linear relationships in **Figure 32 H and I**. However, without knowing the Method A results, it would be very difficult to make the above listed statements for the results obtained with Method B.

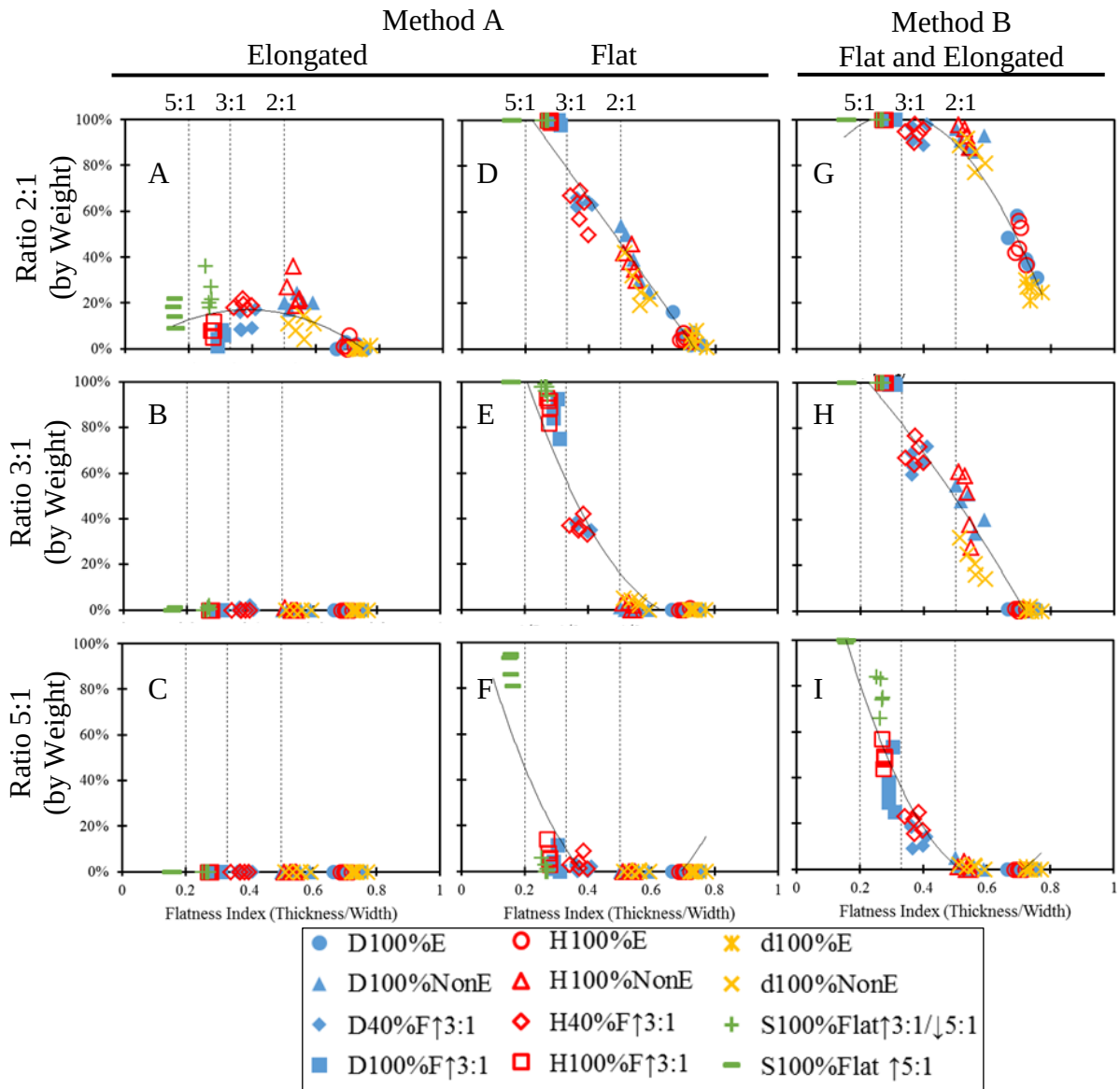


Figure 32. Flatness Index vs. ASTM D4791 Method A and B

As demonstrated from the results described above, the ASTM Method A requires multiple figures to describe a relationship between flat and elongated particles with different particle shape ratios and Method B produces results that are difficult to interpret

in terms of being able to quantify the flat vs. elongated particles. Whereas the method defined by Barksdale and Itani (1989) allows the results to be presented in a way that in a given figure particle shapes can be related to ratios as well as flat and elongation. Therefore, for the remainder of this research, all particle relationships were defined following the relationships shown on **Figure 31**.

5.1.2 Compaction

Compaction curves were created independently for each aggregate with different mineralogy and shape characteristics as defined in **Figure 31**. Below describes the results obtained for each gradation and aggregate.

Dense-Graded Diabase

All compaction curves for diabase dense-graded material are shown in **Figure 33**. It was observed that the compaction curves for diabase dense-graded material are fairly similar, independent of particle shape where the $\rho_{d \max}$ ranged between 2340 and 2310 kg/m³ and w_{opt} ranged between 7 and 8 percent. Based on this observation, a single $\rho_{d \max}$ and optimum moisture content was determined to represent all of the diabase dense-graded material tested in this study. The selected $\rho_{d \max}$ was roughly the average as 2323 kg/m³ (145.0 pcf) and the w_{opt} was 7.5%. These two values were then used for all subsequent engineering tests for the diabase dense-graded material.

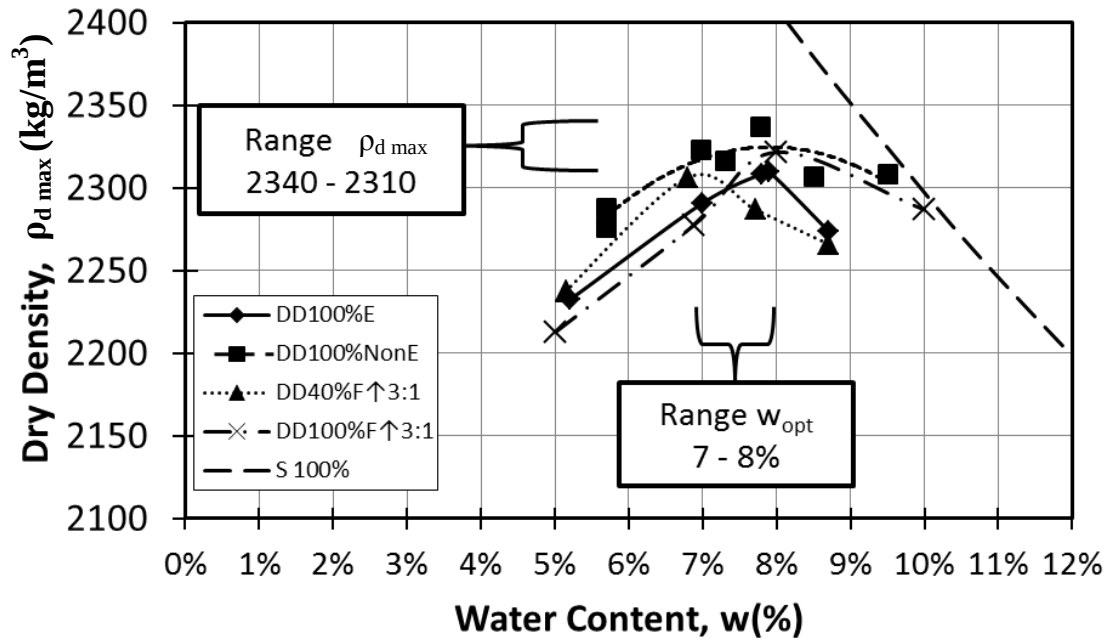


Figure 33. Diabase Dense-Graded Compaction Curves

Dense-Graded Hornfels and Dolostone

The same observation as in diabase was also made with both hornfels and dolostone dense-graded materials, as shown in **Figure 34** and **Figure 35**. The particle shape did not appear to affect the $\rho_{d \max}$ and w_{opt} as determined from the Proctor tests. With this determined, a single $\rho_{d \max}$ and w_{opt} was determined for both hornfels and dolostone dense-graded samples. For the hornfels dense-graded samples, the $\rho_{d \max}$ was selected as 2243 kg/m^3 (140.0 pcf) and the w_{opt} was 7.75% and for dolostone dense-graded samples, $\rho_{d \max}$ was selected as 2291 kg/m^3 (143.0 pcf) with an w_{opt} of 7.7%.

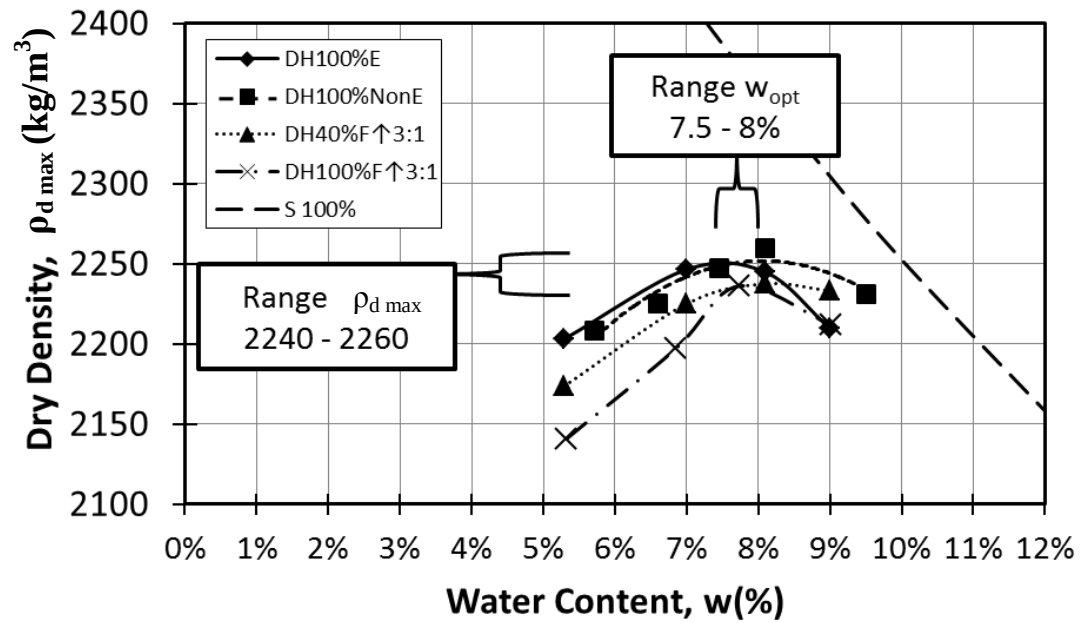


Figure 34. Hornfels Dense-Graded Compaction Curves

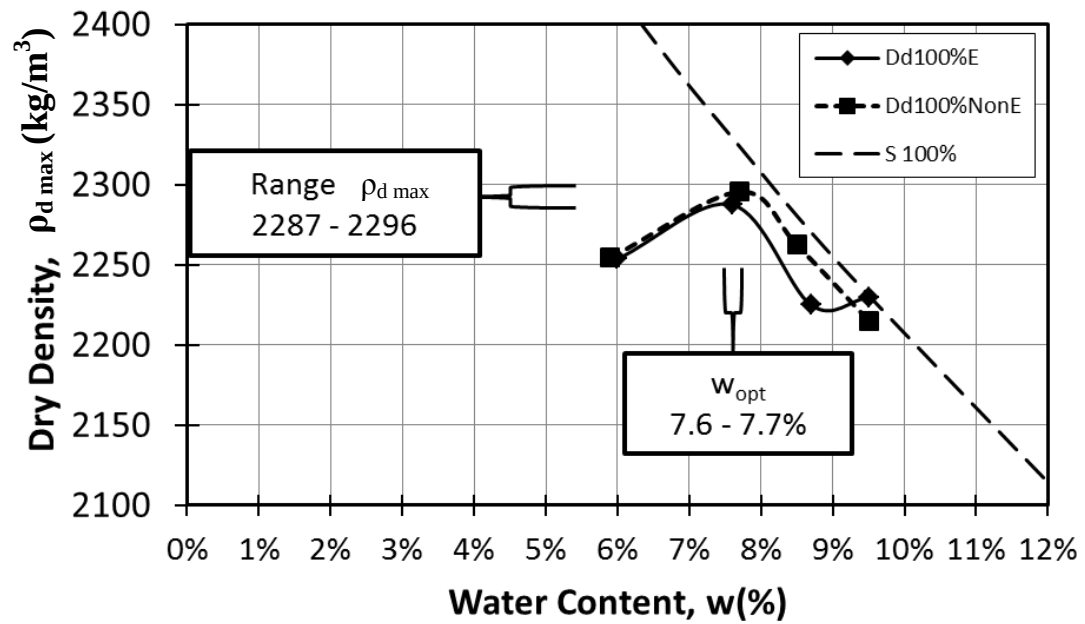


Figure 35. Dolostone Dense-Graded Compaction Curves

Dense-Graded Slate samples

The compaction curves for the slate samples showed variation in compaction. The DS100%F $\uparrow$ 3:1 $\downarrow$ 5:1 sample had a $\rho_{d \max}$ of 2206 kg/m³ (137.7 pcf) and an w_{opt} of 7%, while the DS100%F $\uparrow$ 5:1 sample had a reduced $\rho_{d \max}$ of 2140 kg/m³ (133.6 pcf). This difference in the $\rho_{d \max}$ of 66 kg/m³ (4.1 pcf) indicates that as the flatness index decrease from 0.3 to 0.15 (as shown on **Figure 31**) it can lead to a reduction of unit weight during compaction for a dense graded material. This could be caused by the flat particles at the 5:1 ratio not being able to rotate and move to achieve a dense state as easily as the flat particle at the 3:1 ratio because of the particle shape. A change in w_{opt} was not observed between the two slate dense-graded samples.

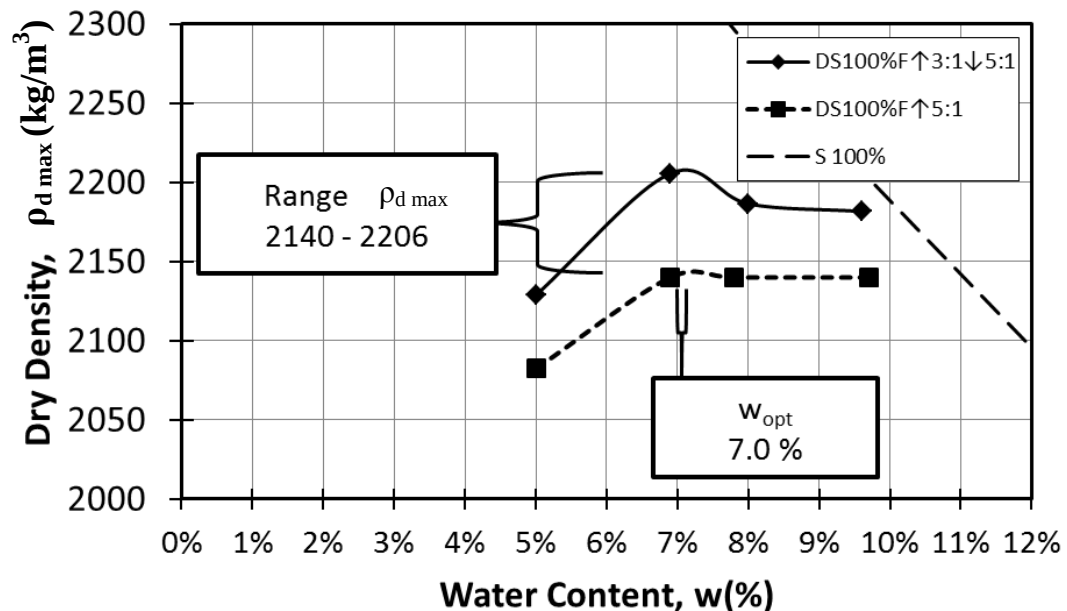


Figure 36. Slate Dense-Graded Compaction Curves

Due to the low durability of the slate materials, particle breakdown was observed after impact compaction of the material, as shown in the grain size distribution curves presented in **Figure 37**. There was a marked increase in fines content from the mixed 6% to 10% after Proctor compaction for both the DS100%F $\uparrow$ 3:1 $\downarrow$ 5:1 and DS100%F $\uparrow$ 5:1 samples. This particle breakdown may lead to higher $\rho_{d \max}$ achieved with impact compaction than could be obtained with vibratory compaction methods. Consequently, the $\rho_{d \max}$ for the two slate samples had to be reduced based on trial and error with $\rho_{d \max}$ that could be obtained through vibratory compaction methods during sample creation for M_R and PD tests.

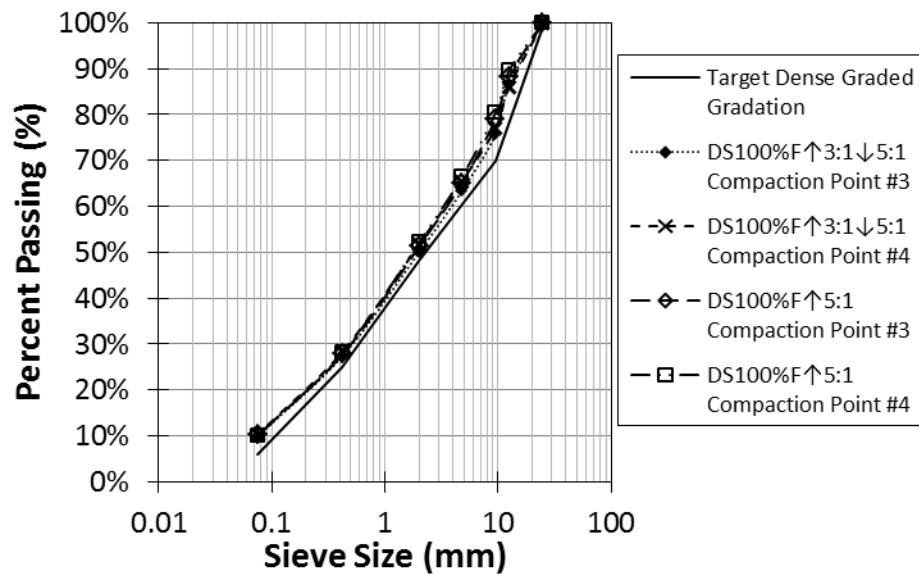


Figure 37. Gradation After Proctor Compaction of DS100%F $\uparrow$ 3:1 $\downarrow$ 5:1 and DS100%F $\uparrow$ 5:1 Points

The achievable $\rho_{d \max}$ during vibratory compaction processes for the DS100%F $\uparrow$ 3:1 $\downarrow$ 5:1 and DS100%F $\uparrow$ 5:1 dense-graded samples were 2118 kg/m³ (132.2 pcf) and 2039 kg/m³ (127.3 pcf) respectfully as opposed to 2206 kg/m³ (137.7 pcf) and 2140 Kg/m³ (133.6 pcf) with standard Proctor tests. There was a greater reduction in the achievable $\rho_{d \max}$ obtained from the compaction tests in the DS100%F $\uparrow$ 5:1 (101 Kg/m³) (88 Kg/m³) than the DS100%F $\uparrow$ 3:1 $\downarrow$ 5:1 (88 Kg/m³) dense-graded sample. This could be cause again by the flat particles at the 5:1 ratio not being able to rotate and move to achieve a dense state as easily as the flat particle at the 3:1 ratio because of the particle shape.

Summary of Dense-Graded Compaction Results

A summary of the target moisture contents and target dry densities for the aggregate samples with different mineralogy that were used in this study for the dense-graded aggregate is presented in **Table 11**. These were the values targeted when preparing samples for M_R and permanent deformation tests for that particular mineralogy.

Table 11. Dense-Graded Target Moisture and Dry Density

Mineralogy	Sample Name	w_{opt} (%)	$\rho_{d\ max}$ (kg/m ³)	(pcf)
Diabase	DD100%E	7.5	2323	145.0
	DD100%NonE			
	DD40%F $\uparrow$ 3:1			
	DD100%F $\uparrow$ 3:1			
Hornfels	DH100%E	7.7	2243	140.0
	DH100%NonE			
	DH40%F $\uparrow$ 3:1			
	DH100%F $\uparrow$ 3:1			
Dolostone	Dd100%E	7.7	2291	143.0
	Dd100%NonE			
Slate	DS100%F $\uparrow$ 3:1 $\downarrow$ 5:1	7	2118	132.2
	DS100%F $\uparrow$ 5:1	7	2039	127.3

Note: Symbols Used in This Table Have Been Previously Defined in Table 6

Open-Graded Diabase, Hornfels, Dolostone, and Slate

The open-graded compaction procedure as mentioned in section 4.3 is based on a one point proctor at an effective moisture content of 2% for each of the samples analyzed. Each mineralogy and particle shape had three repeat tests. The open-graded compaction tests for each sample shape and mineralogy are presented in **Figure 38**. The repeat tests for each aggregate based on mineralogy and particle shape are clustered together consistently which shows the repeatability of the one point proctor method.

In **Figure 38A**, the dry densities achieved for both the equidimensional and non-equidimensional particle shapes overlap, while more separation is observed for the samples with flat particles (ranging from a flatness index of 0.4 to 0.3). A similar trend is observed in **Figure 38B** for the hornfels open-graded samples with the equidimensional and non-equidimensional particle shapes overlapping. More overlapping for the two

hornfels open-graded flat samples was observed than the diabase open-graded flat samples.

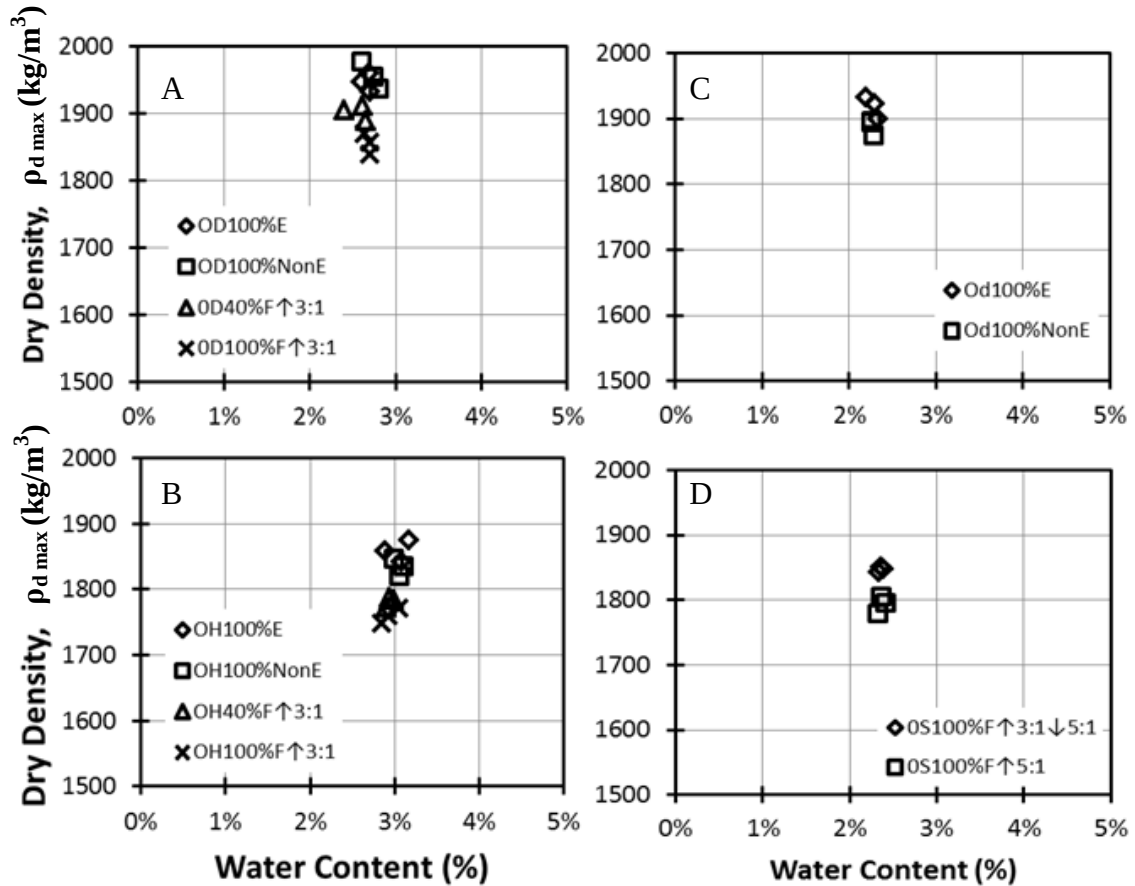


Figure 38. Open-Graded Compaction Points; A) Diabase B) Hornfels c) Dolostone and d) Slate

Overlapping of compaction testing points between samples of equidimensional and non-equidimensional samples was also observed for the dolostone samples as shown in **Figure 38C**. The two varying flat particle shapes in the slate samples show a gap between the achieved dry densities (**Figure 38D**).

In order to determine the target dry density for testing of samples, the average dry density was determined for the three repeat tests for each shape and mineralogy. This average value obtained for the open-graded material was then used as the target $\rho_{d \max}$ for M_R and PD testing for that particular mineralogy and particle shape as shown in **Table 12**.

As seen from **Table 12**, with increased amounts of flat particles in the diabase and hornfels samples the achieved $\rho_{d \max}$ from one-point proctor tests decreased. To investigate if this trend holds across mineralogy, the void ratio was calculated at the average $\rho_{d \max}$ achieved for all four aggregate types and is presented in **Figure 39**. As the flatness index decreases (as the particles become more flat) there is an increase in the void ratio (e). This would indicate that the particles become less compacted with the same compaction energy as the flatness index decreases. Flat particles seem to hinder the compaction of open-graded granular materials.

Table 12. Open Graded Target Moisture and Dry Density

Mineralogy	Sample Name	w_{opt} (%)	$\rho_{d\ max}$ (kg/m ³)	(pcf)
Diabase	OD100%E	2.6	1946	121.5
	OD100%NonE	2.6	1957	122.2
	OD40%F↑3:1	2.6	1903	118.8
	OD100%F↑3:1	2.6	1855	115.8
Hornfels	OH100%E	3.0	1858	116.0
	OH100%NonE	3.0	1834	114.5
	OH40%F↑3:1	3.0	1781	111.2
	OH100%F↑3:1	3.0	1759	109.8
Dolostone	Od100%E	2.3	1919	119.8
	Od100%NonE	2.3	1887	117.9
Slate	OS100%F↑3:1↓5:1	2.4	1847	115.3
	OS100%F↑5:1	2.4	1794	112.0

Note: Symbols Used in This Table Have Been Previously Defined in Table 6

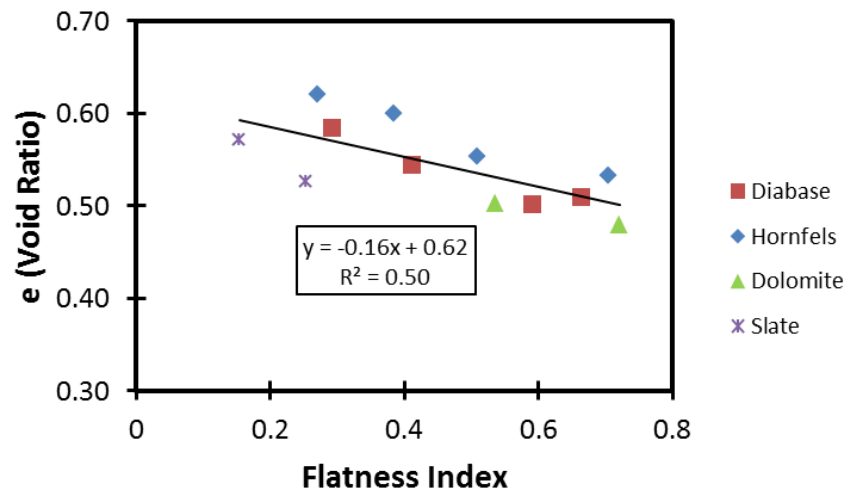


Figure 39. Open Graded Compaction e vs. Flatness Index

Comparison of Dense- and Open-Graded Compaction

In summary, the reduction of $\rho_{d\ max}$ due to an increase in flatness index is much more pronounced in the open-graded material as opposed to the dense-graded. The

variability of compaction density for the dense-graded material was only affected with very large flatness index values (going from particles above the 3:1 ratio and below the 5:1 ratio to particles above the 5:1 ratio). The greater effect of particle shape on the open-graded compaction could be due to the lack of fines content in the open-graded material selected for this study.

5.1.3 Matrix Characterization

The densities determined from the compaction tests for both the dense- and open-graded materials were used for matrix characterization. As mentioned in section 4.4, the NCS was determined to be the 4.75-mm (No. 4) sieve so that any material retained above the 4.75-mm sieve is part of the PS (coarse particles) and the material passing is part of the SS (fine particles). Two tests were conducted and averaged to obtain the LUW_{PS} and the RUW_{SS} . The results for the matrix characterization by the Bailey method are presented in **Table 13**.

Looking at the Dense-Graded materials, the range of the $\%LUW_{PS}$ is between 58 and 69 percent and the $\%RUW_{SS}$ is between 97 and 100 (with one exception). These two values define the structure of the dense-graded materials as a High SS structure as shown in **Figure 6C**. Since the $\%LUW_{PS}$ is noticeably below 100, the PS particles are no longer in contact with one another and are in a floating state. This conclusion is supported by $\%RUW_{SS}$ values being near 100, showing that the SS is densifying and receiving the force of the compaction energy. The only exception to this trend is the DS100%F $\uparrow$ 5:1 which had $\%RUW_{SS}$ at 91. This indicates that the flat particles above the 5:1 are reducing the compaction energy imparted on the SS therefore leading to a reduction in the $\gamma_{d \max}$.

This observation supports the idea stated in section 5.1.2 that the increases in particle flatness between the DS100%F $\uparrow$ 3:1 $\downarrow$ 5:1 and DS100%F $\uparrow$ 5:1 decreases the ability of the PS to densify during compaction processes.

Table 13. Matrix Characterization Results for Dense- and Open-Graded Materials

Sample Name	G_s^{PS}	%PS	%SS	$\rho_{d\ max}^{PS}$ (kg/m ³)	LW_{PS} (kg/m ³)	RW_{SS} (kg/m ³)	V_v^{SS}	% LW_{PS}	% RW_{SS}
Dense-Graded									
DD100%E	2.93	40	60	2323	1595	2043	0.68	58	100
DD100%Non-E	2.93	40	60	2323	1538	2043	0.68	60	100
DD40%F↑3:1	2.93	40	60	2323	1468	2043	0.68	63	100
DD100%F↑3:1	2.93	40	60	2323	1389	2043	0.68	67	100
DH100%E	2.81	40	60	2243	1487	2043	0.68	60	97
DH100%Non-E	2.81	40	60	2243	1430	2043	0.68	63	97
DH40%F↑3:1	2.81	40	60	2243	1468	2043	0.68	61	97
DH100%F↑3:1	2.81	40	60	2243	1297	2043	0.68	69	97
Dd100%E	2.83	40	60	2291	1585	2029	0.68	58	100
Dd100%Non-E	2.83	40	60	2291	1509	2029	0.68	61	100
DS100%F↑3:1↓5:1	2.84	40	60	2118	1327	1876	0.70	64	97
DS100%F↑5:1	2.84	40	60	2039	1205	1876	0.71	68	91
Open-Graded									
OD100%E	2.93	75	25	1946	1574	1670	0.50	93	58
OD100%Non-E	2.93	75	25	1957	1545	1670	0.50	95	59
OD40%F↑3:1	2.93	75	25	1903	1498	1670	0.51	95	56
OD100%F↑3:1	2.93	75	25	1855	1430	1670	0.53	97	53
OH100%E	2.81	75	25	1858	1498	1670	0.50	93	55
OH100%Non-E	2.81	75	25	1834	1454	1670	0.51	95	54
OH40%F↑3:1	2.81	75	25	1781	1405	1670	0.52	95	51
OH100%F↑3:1	2.81	75	25	1759	1328	1670	0.53	99	50
Od100%E	2.83	75	25	1919	1556	1681	0.49	92	58
Od100%Non-E	2.83	75	25	1889	1517	1681	0.50	93	56
OS100%F↑3:1↓5:1	2.84	75	25	1847	1328	1550	0.51	104	58
OS100%F↑5:1	2.84	75	25	1794	1253	1550	0.53	107	55

Note: Symbols Defined in Section 4.4 and Table 6

The open-graded materials show a different matrix structure than the dense-graded materials. The $\%LUW_{PS}$ for all the open-graded materials is between 92 and 107, which covers the transition between PS interlocking and not interlocking ($\%LUW_{PS}$ at 90 to 95) and PS completely interlocking ($\%LUW_{PS} \geq 95$) stated by the Bailey method (Vavrik et al., 2002). So, the PS particles could either be in complete contact with one another or transiting to the High SS. For the $\%RUW_{SS}$ of the open-graded materials, the values found are noticeably below 100 which indicates that the SS is not completely filling the voids between the PS particles. The overall structure of the open-graded materials would then be either in the Transition Zone or Low SS structure depending on the $\%LUW_{PS}$.

The dolostone samples have the lowest $\%LUW_{PS}$ at 92 and 93 and are in the stated transition zone between 90 and 95 where the PS particles could or could not be in contact with one another. The open-graded slate samples show the highest $\%LUW_{PS}$ at 104 and 107 showing that the PS particles are in complete contact with one another. The diabase and hornfels samples $\%LUW_{PS}$ range in and out of the transition zone.

When looking at the effect of particle shape overall on the LUW_{PS} it can be seen that as the particle become more flat across all mineralogies and both gradation types the density decreases. This shows that the flat particles in their loss state have more void space and consequently do not compact as well as the more equidimensional particles. This statement is consistent with the increases in the void ratio show in **Figure 39**.

5.2 Experimental Tests

5.2.1 Micro-Deval Results

MD test results are presented in **Table 14**. As mentioned in section 4.5, replicate tests were conducted for each aggregate and particle shape, with the D100%NonE sample having four tests conducted to ensure the repeatability of the test method. The coefficient of variation (COV) for all tests were under the specified value of 9.6% from ASTM D6928 (2010).

The mean of the MD tests for each aggregate and particle shape were then plotted against the flatness index value, which is presented in **Figure 40**. Looking at **Figure 40**, it is noted that the slate samples had the highest percent loss in the micro-deval test and is an outlier of the other three aggregate types that were tested. This result was expected due to the low abrasion resistance associated with slate (Waltham, 2009). The high loss values for the slate material in the micro-deval test falls above the 25% and 30% loss limits suggested by Rodgers (1998) for base and subbase materials respectively.

The S100%Flat $\uparrow$ 3:1/ $\downarrow$ 5:1 samples had the same flatness index as the D100%F $\uparrow$ 3:1 and H100%F $\uparrow$ 3:1 samples, but it is shown that the mineralogy of the sample had a substantially greater effect on the durability of the sample than the flatness index. The variation of loss between the two particle shapes of the slate samples had a COV of 2.78%, which is under the given 9.6% value specified in the ASTM. This would indicate that the increase in flatness index between the two slate samples did not change the amount of loss during the test.

Table 14. Micro-Deval Results

Aggregate Type	Sample Name	Micro-Deval Loss (%)	Mean (%)	COV (%)
Diabase	D100%E	8.42	8.16	9.01
	D100%E#2	7.90		
	D100%NonE	6.29	6.14	4.20
	D100%NonE#2	6.02		
	D100%NonE#3	6.05		
	D100%NonE#4	6.21		
	D40%F↑3:1	6.94	6.85	3.93
	D40%F↑3:1#2	6.75		
	D100%F↑3:1	5.70	5.70	0.00
	D100%F↑3:1#2	5.70		
Hornfels	H100%E	6.53	6.63	4.27
	H100%E#2	6.73		
	H100%NonE	5.43	5.61	8.83
	H100%NonE#2	5.78		
	H40%F↑3:1	5.36	5.20	8.70
	H40%F↑3:1#2	5.04		
	H100%F↑3:1	4.46	4.62	9.50
	H100%F↑3:1#2	4.77		
Dolostone	d100%E	8.43	8.39	1.52
	d100%E#2	8.34		
	d100%NonE	7.76	8.03	9.34
	d100%NonE#2	8.29		
Slate	S100%Flat↑3:1/↓5:1	32.73	32.99	2.23
	S100%Flat↑3:1/↓5:1#2	33.25		
	S100%Flat ↑5:1	33.84	33.65	1.60
	S100%Flat ↑5:1#2	33.46		

Note: COV = Coefficient of variation (d2s %)

A trendline was fit only to the more durable diabase, hornfels, and dolostone samples, which is presented in **Figure 40**. The three minerologies have low aggregate

loss values after the MD test. This would indicate that the three aggregate mineralogies are very durable.

Regardless of the three mineralogies, as the flatness index value increases (as the particles become more equidimensional) the percent loss in the MD test increases across all three mineralogies. A moderate to low linear correlation is found when the durable (dolostone, hornfels, and diabase) aggregates are grouped together since the R^2 value is 0.53. The opposite trend, meaning that as the particles become more equidimensional there is less loss, was found by Rismantojo (2002) in the MD test but a similar low correlation between particle shape and loss was stated (R^2 value between 0.37 and 0.64 depending on method and ratio used for classification of the rock type). Fowler et al. (2006) found no correlation with particle shape and MD loss, which varies from both this study and Rasmantojo (2002). To be noted is that varying particle shape analysis procedures was conducted between this study, Rasmantojo (2002), and Fowler et al. (2006).

Using the COV of 9.6% stated in ASTM D6928 (two standard deviations) and looking at just the two extremes in flatness index for the diabase and hornfels samples, it was found that the COV for the diabase and hornfels samples were 41.44% and 41.73% respectively. These COV would confirm that the variation in MD loss for the two extremes of particle shape are above observed variations. The COV found for the two dolostone samples was 7.37% which would indicate that the variation in the loss percentages between the two samples is not above observed variations, but this could be due to the reduced flatness index variation present for the dolostone samples.

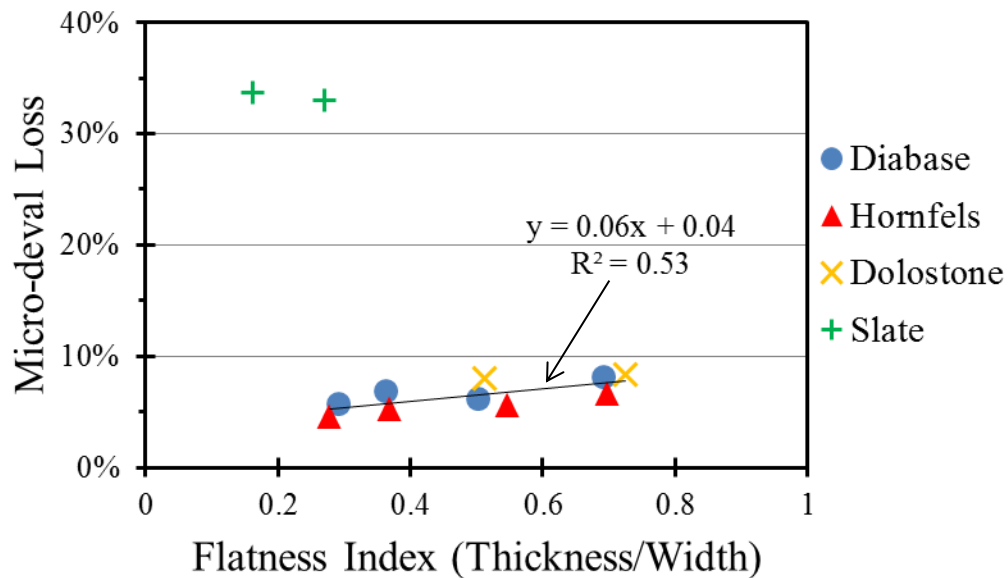


Figure 40. Flatness Index vs. Average Micro-deval Loss

In the MD results, the importance of mineralogy is highlighted for aggregate durability since the slate samples had very high loss values when compared with the other three aggregate types even though the S100%Flat $\uparrow$ 3:1/ $\downarrow$ 5:1 samples had the same flatness index value as the flat diabase and hornfels samples. The variation in loss across the flatness index values created for the diabase and hornfels samples would indicate that less loss is accrued in the MD machine as the flatness index value decreases for the durable aggregate mineralogies tested in this study. The variation found was above variations stated in the ASTM standard, with roughly a 40% increase in loss as the particle became more equidimensional for the diabase and hornfels samples.

5.2.2 Summary of Resilient Modulus Test Results

The results presented in this section were obtained from 28 individual M_R tests performed with both dense- and open-graded aggregate gradations following the AASHTO T-307 guidelines. The individual test results for all M_R are presented in **Appendix B**. The individual tests results were all interpreted using the MEPDG model (Equation 4). This approach resulted in establishing a relationship between the bulk stress, octahedral shear stress, and the measured M_R value. The MEPDG model could then later be used in the interpretation results section to compare the results among different samples (a representative single resilient modulus for each sample) at varying bulk and octahedral shear stresses that were relevant for this study.

Dense-Graded

Details of the dense-graded samples prepared for M_R tests are depicted in **Table 15**. To confirm the repeatability of the M_R tests, two randomly selected samples, which included the DD100%NonE and DD100%F $\uparrow$ 3:1 samples, were prepared twice and tested. These samples are denoted in **Table 15** with a #2 after the original samples name. The actual dry densities achieved for the dense-graded M_R tests (i.e., referred as Percent of Target Density in **Table 15**) were all within $\pm 1\%$ of the target dry density.

Table 15. Dense-Graded M_R Compaction Data

Sample Name	$\rho_{d \max}$ (kg/m ³) (pcf)		Percent of Target Density (%)	Compaction Moisture (%)	Moisture After (%)
DD100%E	2323	145.0	100	7.2	5.9
DD100%NonE			99	7.4	6.0
DD100%NonE #2			99	7.7	6.0
DD40%F $\uparrow$ 3:1			99	7.5	6.0
DD100%F $\uparrow$ 3:1			100	6.7	6.1
DD100%F $\uparrow$ 3:1 #2			100	7.1	6.0
DH100%E	2243	140.0	100	7.2	6.7
DH100%NonE			99	7.2	6.8
DH40%F $\uparrow$ 3:1			100	7.4	6.6
DH100%F $\uparrow$ 3:1			99	7.3	6.5
Dd100%E	2291	143.0	100	7.5	5.4
Dd100%NonE			101	7.1	5.3
DS100%F $\uparrow$ 3:1 $\downarrow$ 5:1	2118	132.2	99	6.5	6.1
DS100%F $\uparrow$ 5:1	2039	127.3	100	6.7	6.1

The k_1 , k_2 , and k_3 model parameter for the MEPDG model was then fitted to the obtained resilient modulus values from the M_R tests conducted on the dense-graded materials by means of regression analysis and is present in **Table 16**. The k values obtained follow trends stated by Tutumluer (2013), with k_2 values being positive, showing an increasing stiffness with increasing bulk stress, and k_3 values being negative showing a softening effect. Also, R^2 obtained from the model fitting is above 0.90 for all but one test, which shows good correlation between observed and predicted M_R values. The only exception to this is the Dd100%NonE sample, but the model parameters still fall within the trends stated by Tutumluer (2013) and a R^2 value of 0.813 still shows a good correlation with observed results.

Table 16. Dense-Graded M_R Test MEPDG Model Parameters

Sample Name	MEPDG Model			
	k ₁	k ₂	k ₃	R ²
DD100%E	918.9	0.760	-0.279	0.981
DD100%NonE	907.5	0.818	-0.283	0.993
DD100%NonE #2	1017.1	0.780	-0.377	0.967
DD40%F↑3:1	887.7	0.799	-0.366	0.983
DD100%F↑3:1	775.4	0.784	-0.275	0.979
DD100%F↑3:1 #2	796.7	0.833	-0.341	0.988
DH100%E	820.4	0.759	-0.223	0.968
DH100%NonE	1002.3	0.767	-0.484	0.920
DH40%F↑3:1	861.3	0.818	-0.470	0.976
DH100%F↑3:1	826.8	0.720	-0.222	0.986
Dd100%E	1069.7	0.764	-0.244	0.975
Dd100%NonE	1476.9	0.824	-0.526	0.813
DS100%F↑3:1↓5:1	950.9	0.729	-0.793	0.941
DS100%F↑5:1	828.2	0.869	-1.028	0.974

Open-Graded

Details of the open-graded samples prepared for resilient modulus tests are presented in **Table 17**. The actual dry density achieved for the open-graded M_R tests (i.e., referred as Percent of Target Density in **Table 17**) were all within ±1% of the target dry unit weight.

Table 17. Open-Graded M_R Compaction Data

Sample Name	$\rho_{d \max}$		Percent of Target Density	Compaction Moisture	Moisture After
	(kg/m ³)	(pcf)	(%)	(%)	(%)
OD100%E	1946	121.5	99	2.4	2.4
OD100%NonE	1957	122.2	100	3.0	2.4
OD40%F↑3:1	1903	118.8	99	2.8	2.4
OD100%F↑3:1	1855	115.8	99	2.6	2.4
OH100%E	1858	116.0	100	3.1	2.6
OH100%E#2	1834	116.0	99	3.4	2.6
OH100%NonE	1781	114.5	99	3.4	2.7
OH40%F↑3:1	1759	111.2	99	3.4	2.7
OH100%F↑3:1	1919	109.8	99	3.0	2.8
OH100%F↑3:1#2	1887	109.8	100	3.1	2.7
Od100%E	1847	119.8	100	2.4	2.0
Od100%NonE	1794	117.9	99	2.7	2.0
OS100%F↑3:1↓5:1	1946	115.3	100	2.3	2.2
OS100%F↑5:1	1957	112.0	99	2.4	2.2

The data obtained from the M_R tests were processed following the same steps as for the dense-graded aggregates utilizing the MEPDG model with corresponding k parameters and R² values (**Table 18**). The results show that k values obtained for open-graded aggregate follow the same trend as for the dense-graded materials, with the k₂ values being positive and the k₃ values being negative. Values obtained for the R² values were lower for the open-graded material than the dense-graded materials, with a few of the R² values in the seventies and lower eighties.

Table 18. Open-Graded M_R Test MEPDG Model Parameters

Sample Name	MEPDG Model			
	k ₁	k ₂	k ₃	R ²
OD100%E	1293.6	0.726	-0.615	0.947
OD100%NonE	1135.6	0.872	-0.719	0.941
OD40%F↑3:1	1086.8	0.713	-0.450	0.975
OD100%F↑3:1	1069.7	0.753	-0.759	0.893
OH100%E	1217.7	0.669	-0.521	0.744
OH100%E#2	1088.9	0.658	-0.371	0.880
OH100%NonE	1148.9	0.781	-0.619	0.964
OH40%F↑3:1	1034.1	0.674	-0.471	0.919
OH100%F↑3:1	986.5	0.791	-0.816	0.913
OH100%F↑3:1#2	1086.9	0.841	-0.968	0.911
Od100%E	1622.6	1.017	-1.336	0.897
Od100%NonE	1386.8	1.228	-1.567	0.760
OS100%F↑3:1↓5:1	1116.8	0.821	-1.041	0.823
OS100%F↑5:1	892.3	0.734	-0.835	0.894

5.2.3 Summary of Permanent Deformation Test Results

A total of 48 single stage PD tests were conducted on both dense- and open-graded materials. The PD tests first involved subjecting the specimen to a conditioning phase of 500 cycles (confining pressure of 103.4 kPa and deviator pressure of 103.4 kPa), then followed by 10,000 cycles at either a pavement stress representing a light pavement structure (confining pressure of 41.4 kPa and deviator pressure of 206.8 kPa) or a medium pavement structure (confining pressure of 41.4 kPa and deviator pressure of 124.1 kPa). All complete test results for both open- and dense-graded materials are presented in **Appendix C**. It is noted that for all samples, both dense and open-graded, that there is an increases in PD with increases in stress.

The PD tests for both dense and open-graded materials were evaluated based on the final PD accumulation at the end of the 10,000 cycle loading sequence, as well as the shakedown range (whether the material classifies as in range A, B, or C as discussed in Section 2.1.2). The shakedown range classification was based on recommendations given by the European standard for PD testing of UAB, standard BS EN 13286-7 Annex C (2004). This European standard classifies the UAB materials in range A, B, or C based on the accumulated permanent strain at 5,000 cycles minus the accumulated permanent strain at 3,000 cycles for samples subjected to 10,000 load cycles. The values given to distinguish the three range types are presented in **Table 19**. For example, if the permanent strain at 5,000 cycles minus the permanent strain at 3,000 cycles is below 0.45×10^{-5} then the material is in Range A, plastic shakedown, which is a stable UAB behavior. An example of this procedure is given in **Appendix C**.

Table 19. Permanent Deformation Behavior Ranges Defined by BS EN 13286-7 Annex C

Range	Name	$\epsilon_p @ 5,000 - \epsilon_p @ 3,000$
A	Plastic Shakedown	$< 0.45 \times 10^{-5}$
B	Plastic Creep	$\geq 0.45 \times 10^{-5}$ and $< 0.4 \times 10^{-3}$
C	Incremental Collapse	$\geq 0.4 \times 10^{-3}$

Note: $\epsilon_p @ 3,000/5,000$ = permanent strain at either 5,000 or 3,000 cycles

Additionally, M_R values of the last five cycles of all dense- and open-graded materials during the PD tests were determined and averaged. This information was collected to distinguish if the same trends present in the elastic response found during the

M_R tests would be held during the PD tests, where the applied loading conditions were different. While the magnitude of the elastic response may not be the same between the M_R test and the measured M_R response at the end of PD tests due to different stress histories, it will allow for verification of the trends observed for the dense and open-graded M_R tests. This discussion is covered in the interpreted results section 5.3.3.

Dense-Graded

Table 20 summarizes the details of the procedure followed to prepare samples for the PD tests, the final ϵ_p , the behavior range, as well as the parameters used for regression analyses following the Barksdale model as outlined in equation 6. The compaction data shows that final compaction was within $\pm 1\%$ of the target dry unit weight. The R^2 values for the regression analyses show very good agreement with the laboratory test results, with values all in the nineties. The interpretation of dense-graded PD results will be further discussed in the section 5.3.2.

Table 20. Dense-Graded Permanent Deformation Test Results

Deviator Pressure (kPa)	Sample Name	$\rho_{d \max}$ (kg/m ³) (pcf)		Percent of Target	Compaction	Moisture	Final	$\epsilon_p@$ 5,000 -	Behavior Range	Barksdale Model		
				Density (%)	Moisture (%)	After (%)	ϵ_p (%)	$\epsilon_p@$ 3,000		$\epsilon_p = a + b \log(N)$ a	b	R ²
124.1 (Medium Pavement Stress)	DD100%E	2323	145.0	101	7.2	6.1	0.44	4.2E-4	C	-0.0027	0.0008	0.991
	DD100%NonE			100	7.5	5.7	0.63	5.1E-4	C	-0.0036	0.0011	0.992
	DD40%F↑3:1			100	7.5	5.7	0.52	4.6E-4	C	-0.0036	0.0010	0.988
	DD100%F↑3:1			99	7.8	6.1	0.24	1.9E-4	B	-0.0012	0.0004	0.996
	DH100%E	2243	140.0	100	7.8	6.1	0.35	2.6E-4	B	-0.0015	0.0006	0.973
	DH100%NonE			100	7.5	6.1	0.42	3.1E-4	B	-0.0021	0.0007	0.988
	DH40%F↑3:1			100	8.0	6.3	0.32	2.6E-4	B	-0.0018	0.0005	0.990
	DH100%F↑3:1			99	8.0	6.4	0.32	2.8E-4	B	-0.0018	0.0006	0.997
	Dd100%E	2291	143.0	100	7.7	4.8	0.54	3.4E-4	B	-0.0026	0.0009	0.975
	Dd100%NonE			101	7.7	4.4	0.36	2.3E-4	B	-0.0013	0.0005	0.958
	DS100%F↑3:1↓5:1	2118	132.2	100	7.2	6.5	0.46	2.4E-4	B	-0.0012	0.0006	0.971
	DS100%F↑5:1	2039	127.3	100	7.0	6.6	0.35	1.6E-4	B	-0.0010	0.0005	0.976
206.8 (Light Pavement Stress)	DD100%E	2323	145.0	100	7.3	5.7	1.24	6.7E-4	C	-0.0013	0.0015	0.969
	DD100%NonE			100	7.6	5.6	1.41	1.1E-3	C	-0.0057	0.0022	0.989
	DD40%F↑3:1			100	7.5	5.5	1.11	7.5E-4	C	-0.0034	0.0016	0.994
	DD100%F↑3:1			100	7.5	5.7	0.71	4.6E-4	C	-0.0010	0.0009	0.997
	DH100%E	2243	140.0	101	7.5	6.0	1.36	8.3E-4	C	-0.0027	0.0018	0.991
	DH100%NonE			101	7.3	6.0	1.26	9.4E-4	C	-0.0045	0.0019	0.992
	DH40%F↑3:1			101	7.9	5.9	0.94	5.4E-4	C	-0.0017	0.0012	0.994
	DH100%F↑3:1			100	7.6	6.1	0.86	4.4E-4	C	-0.0010	0.0010	0.993
	Dd100%E	2291	143.0	101	7.6	4.4	1.49	1.4E-2	C	-0.0006	0.0017	0.942
	Dd100%NonE			101	7.7	4.6	1.30	1.2E-2	C	-0.0011	0.0016	0.966
	DS100%F↑3:1↓5:1	2118	132.2	100	6.9	6.5	1.29	3.5E-4	B	0.0032	0.0011	0.910
	DS100%F↑5:1	2039	127.3	100	7.2	6.3	1.62	5.3E-4	C	0.0026	0.0015	0.929

Open-Graded

Details of the open-graded aggregates prepared for PD tests are presented in **Table 21**, as well the final ϵ_p , the behavior range, and the parameters associated with regression analyses to estimate PD. As can be seen from the compaction data, all of the samples were compacted within $\pm 1\%$ of the target dry density. The only exception was OS100%F $\uparrow$ 3:1 $\downarrow$ 5:1, which was compacted to relative compaction of 98. The R^2 values for the Barksdale model parameters were in very good agreement with the laboratory test results, with all values in the nineties. The interpretation of open-graded PD results will be further discussed in section 5.3.2.

Table 21. Open-Graded Permanent Deformation Test Results

Deviator Pressure (kPa)	Sample Name	ρ_d max		Percent of		Moisture	Final	$\epsilon_p@$ 5,000 - 3,000		Barksdale Model $\epsilon_p = a + b \log(N)$		
		(kg/m ³)	(pcf)	Target Density (%)	Moisture (%)			After (%)	ϵ_p (%)	Behavior Range	a	b
124.1 (Medium Pavement Stress)	OD100%E	1946	121.5	99	2.8	2.2	0.24	1.6E-4	B	-0.00060	0.00030	0.989
	OD100%NonE	1957	122.2	99	2.4	2.1	0.21	1.4E-4	B	-0.00070	0.00030	0.990
	OD40%F↑3:1	1903	118.8	99	2.7	2.3	0.20	1.3E-4	B	-0.00110	0.00030	0.996
	OD100%F↑3:1	1855	115.8	99	2.6	2.3	0.26	1.2E-4	B	-0.00060	0.00030	0.986
	OH100%E	1858	116.0	99	3.5	2.5	0.21	1.6E-4	B	-0.00040	0.00030	0.993
	OH100%NonE	1834	114.5	99	3.3	2.3	0.24	1.4E-4	B	-0.00050	0.00030	0.986
	OH40%F↑3:1	1781	111.2	99	3.2	2.6	0.21	1.1E-4	B	-0.00080	0.00030	0.986
	OH100%F↑3:1	1759	109.8	99	3.5	2.6	0.26	1.7E-4	B	-0.00090	0.00040	0.985
	Od100%E	1919	119.8	99	2.2	1.7	0.19	1.3E-4	B	-0.00100	0.00030	0.994
	Od100%NonE	1887	117.9	99	2.3	1.8	0.35	2.7E-4	B	-0.00150	0.00060	0.993
206.8 (Light Pavement Stress)	OS100%F↑3:1↓5:1	1847	115.3	98	2.6	2.1	0.12	8.6E-5	B	-0.00020	0.00020	0.975
	OS100%F↑5:1	1794	112.0	99	2.5	2.2	0.07	2.6E-5	A	-0.00005	0.00008	0.900
	OD100%E	1946	121.5	99	2.9	2.1	1.52	6.4E-4	C	0.00040	0.00170	0.968
	OD100%NonE	1957	122.2	100	2.6	2.1	0.59	2.9E-4	B	0.00030	0.00060	0.983
	OD40%F↑3:1	1903	118.8	99	2.7	2.1	0.77	4.7E-4	C	-0.00080	0.00090	0.990
	OD100%F↑3:1	1855	115.8	99	2.7	2.2	1.06	7.3E-4	C	-0.00080	0.00130	0.997
	OH100%E	1858	116.0	100	3.0	2.4	0.71	3.4E-4	B	-0.00010	0.00080	0.986
	OH100%NonE	1834	114.5	99	2.9	2.4	0.80	3.8E-4	B	0.00010	0.00090	0.986
	OH40%F↑3:1	1781	111.2	100	3.0	2.5	0.82	4.3E-4	C	-0.00060	0.00100	0.996
	OH100%F↑3:1	1759	109.8	99	2.9	2.6	1.04	4.8E-4	C	0.00040	0.00110	0.983
	Od100%E	1919	119.8	99	2.4	1.7	1.51	9.9E-4	C	-0.00410	0.00210	0.998
	Od100%NonE	1887	117.9	101	2.1	1.8	1.04	7.4E-4	C	-0.00220	0.00140	0.998
	OS100%F↑3:1↓5:1	1847	115.3	100	2.3	2.1	0.57	1.6E-4	B	0.00140	0.00050	0.940
	OS100%F↑5:1	1794	112.0	99	2.4	2.1	0.58	2.0E-4	B	0.00120	0.00050	0.938

5.2.4 Grain Size Analyses After Resilient Modulus and Permanent Deformation Tests

Samples were created based on target gradations for each both dense- and open-graded gradations given in **Table 5**. For testing preparation, samples were moisture conditioned and mixed and then subjected to M_R or PD testing. After the tests, samples were taken out of the test chamber and dried between each of the tests. The possibility of particle breakdown due to compaction and testing procedures was considered minimal for the diabase, hornfels, and dolostone materials due to the very high durability of the materials as shown in the MD test. As mentioned in section 5.1.2 and as the MD tests showed, the slate material was prone to particle breakdown. To counteract this particle breakdown and ensure uniformity of sample gradations, a new slate sample was mixed for the dense-graded M_R test and the two PD tests as well as the open-graded M_R test and the two PD tests. This resulted in three samples for the dense- and open-graded slate performance tests.

The grain size analysis data after testing for the dense-graded materials is presented in **Figure 41**. The gradations found after testing of the dense-graded materials is close to the mixed gradations with small increases in percent passing the 0.075-mm sieve. The variation in materials passing the 0.075-mm sieve varied between 6% and 8% after testing with only two samples at the 8% fines content (DH100%NonE and the average of the DS100%F $\uparrow$ 3:1 $\downarrow$ 5:1). The variation of 2% in fines content across all dense-graded UAB materials tested should cause little variation in matrix strength since the fines are non-plastic for all samples tested (Mishra, Tutumluer, & Butt, 2010).

The grain size analysis of the open-graded materials after testing is presented in **Figure 42**. Less particle breakdown is noted in the open-graded materials than in the dense-graded materials. The mixed gradation for the open-graded materials was the same as the gradation after testing.

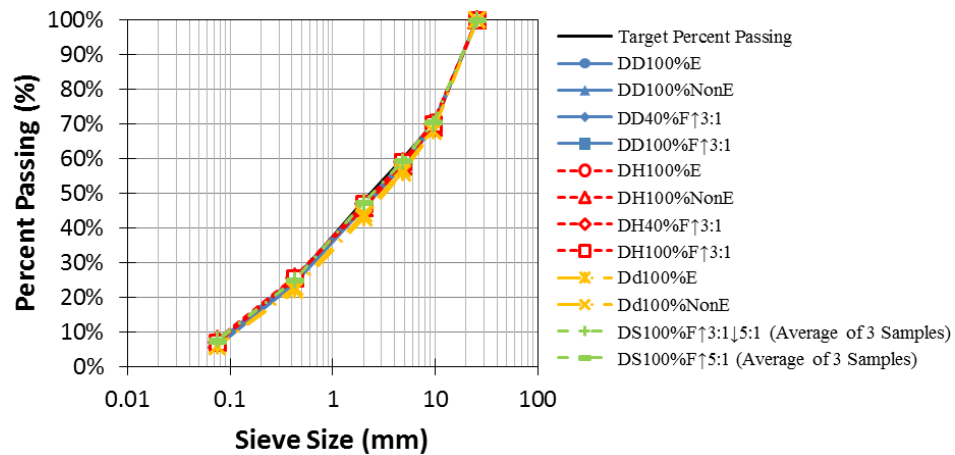


Figure 41. Grain Size Analysis of Dense-Graded Materials After M_R and PD Testing

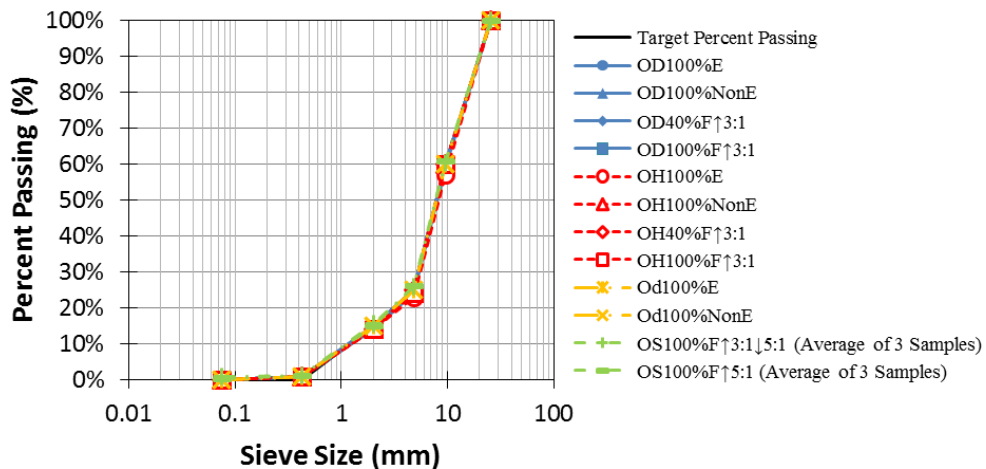


Figure 42. Grain Size Analysis of Dense Graded Materials After M_R and PD Testing

5.3 Interpretation and Discussion of the Results

5.3.1 Interpretation of Resilient Modulus Results

In order to compare the results among the different samples, a representative M_R value for each sample needed to be obtained. This could be obtained if an associated bulk and octahedral shear stress was known that could then be input into the MEPDG model. For this comparison, the three stress states proposed by Barksdale and Itani (1989) representing light, medium, and heavy pavement structure conditions (as presented in **Table 9**) were utilized. Corresponding bulk and octahedral shear stresses were used to obtain associated M_R values for each sample. The light pavement structure has the highest stress in the middle of the base layer because the top asphalt concrete layer is the thinnest, while the heavy pavement structure has the lowest stresses in the middle of the base layer because the asphalt concrete layer is the thickest. An example of how the values in **Table 9** were used to calculate the estimated M_R values are presented in **Appendix B**. This approach allowed each sample to be compared with each other over a range of possible stresses in the middle of the base layer that are common in different pavement structures.

Dense-Graded

The MEPDG model parameters presented in **Table 16** for the dense-graded M_R tests were used to estimate the M_R values for the three stress states for a light, medium, and heavy pavement as presented in **Table 9**. The result of this analysis is presented in **Figure 43**. The M_R values across all samples increased with increasing bulk stress (the

light pavement stress having the highest bulk stress) which is consistent with previous literature (Hicks, 1970; Uzan, 1985).

Looking at the individual aggregate samples with the same mineralogy in **Figure 43**, the dolostone samples show the highest M_R values across all pavement stresses. The slate samples are on the other end of the spectrum with the lowest predicted M_R values and the smallest increase in M_R as the pavement stresses increased from the heavy to the light pavement structure. Variation between all samples in estimated M_R decreases as the pavement stresses decrease, with smaller M_R value variations between mineralogy in the heavy pavement stresses than the light and medium pavement stresses.

The repeat tests of both the DD100%NonE and DD100%F $\uparrow$ 3:1 samples showed good repeatability. As can be seen in **Figure 43**, the values across all three stress levels are similar between the original and repeat tests since plotted points are very similar.

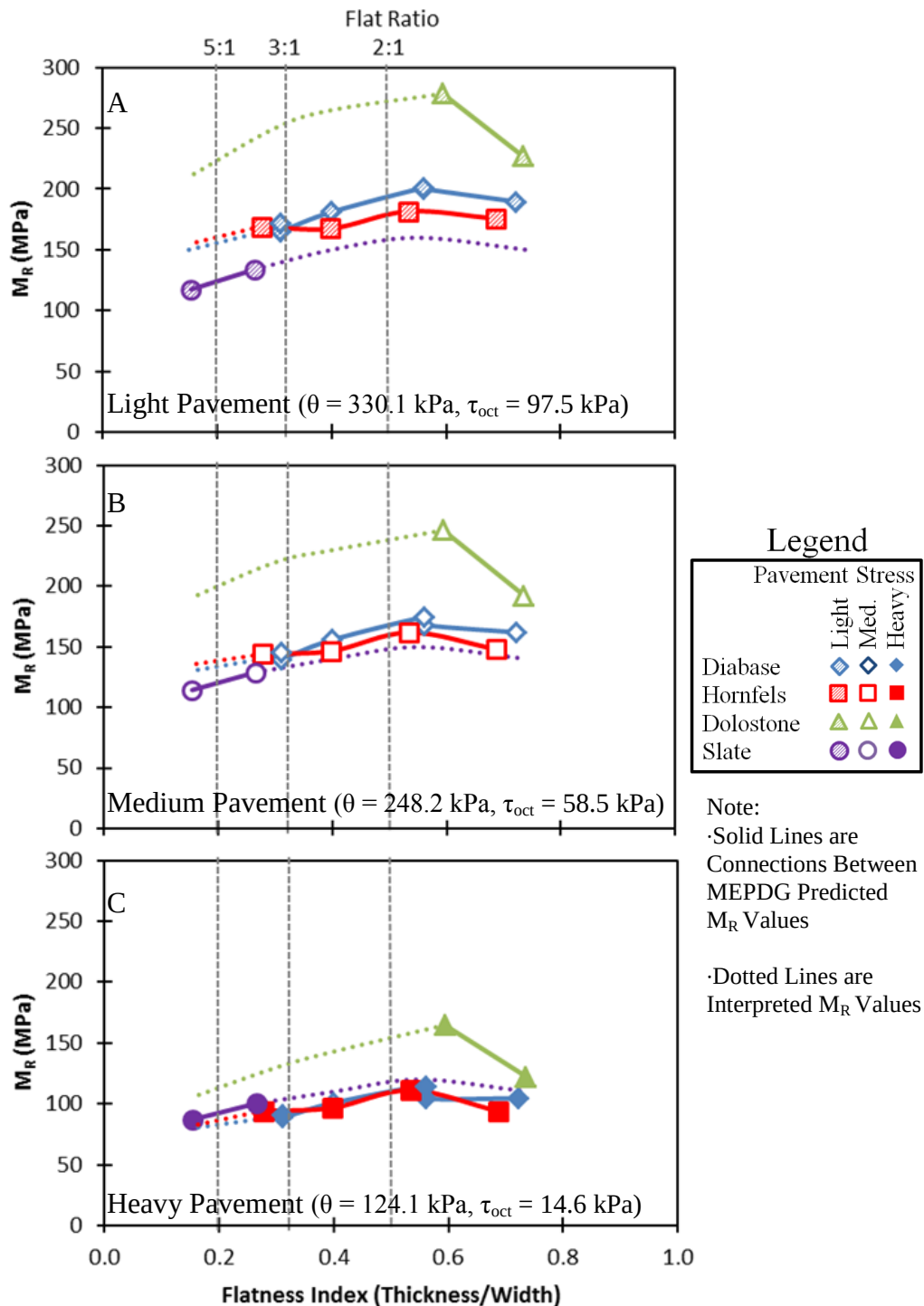


Figure 43. M_R vs. Flatness Index for Dense-Graded Samples for A) Light Pavement Structure, B) Medium Pavement Structure, and C) Heavy Pavement Structure

When **Figure 43** is evaluated based on particle shape (independent of mineralogy), all samples appear to agree on a trend that as the particle shapes become flatter the M_R values decrease, with the max M_R value peaking and then decrease again. That peak appears to correspond to an approximately flatness index value of 0.6 regardless of the stress conditions applied due to different pavement conditions. This observation in M_R value could be due to less elastic movement of PS aggregate particles in the SS structure because of its specific shape.

The matrix structure of the dense-graded materials was classified as in the High SS state by the Bailey method. Based on this, the elastic movement of the PS particles would be dependent on how the particles move and reorient while floating in the SS since they are not locked in with other PS particles. The observed peak at a flatness index value of ~ 0.6 could be because the non-equidimensional shaped particles elastically move the smallest amount while floating in the SS.

Based on this observation, interpreted lines were drawn as pure hypothetical changes in M_R value if all four mineralogies used in the study had particle shapes that varied across the complete range of flatness index values observed. For example, in **Figure 43A** the slate samples M_R values were interpreted (the dotted purple lines) if the slate samples had coarse particles that extended to a flatness index value of ~ 0.75 corresponding to equidimensional shape coarse particles. Slate samples obtained for this study did not have that shape, but results were interpreted to include these to

hypothetically see the effect of particle shape across the complete range of flatness index values observed in this study.

This process allowed for comparison of the effect of mineralogy on the variation of M_R value across the complete range of particle shape for all four mineralogies. In **Figure 43A**, it can be observed that there is a separation in the M_R values across the four aggregate mineralogies with the dolostone samples showing the highest M_R values, the diabase and hornfels samples showing similar M_R responses, and the slate material showing the lowest M_R . This variation across the samples is attributed to the effect of the load carrying characteristics of the SS structure. Since the matrix structure is defined as in the High SS state, the SS particles are significantly affecting the ability of the overall matrix structure to take dynamic loading associated with the M_R test. The SS particles characteristics of the dense-graded materials was not quantified, but based on **Figure 43A**, the interaction between the PS and SS particles play a major role in the M_R response of the material. The effect of the SS particles on the M_R seems to decrease as the stresses decrease, with **Figure 43C** showing the smallest variation in M_R across the four aggregate mineralogies.

For dense-graded aggregates, regardless of the mineralogy differences among the samples tested in this research, there appears to be a trend between particle shapes and corresponding M_R values. However, the difference in magnitude under each stress conditions appear to vary based on the difference in given mineralogy. For example, diabase sample under light pavement structure condition in **Figure 43A** show difference in M_R from ~160 kPa and 200 kPa (difference of 40 kPa) when the flatness index ranged

between 0.3 and 0.6. Along the same range, the hornfels sample only shows difference of ~20 kPa (~160 kPa vs. 180 kPa). The practical implication of these magnitude differences are discussed in the section 6.

Open-Graded

The MEPDG model parameters presented in **Table 18** for the open-graded M_R tests were used to estimate the M_R values for the three stress states for a light, medium, and heavy pavement as presented in **Table 9**. The result of this analysis is presented in **Figure 44**. Again the observed trend was that the M_R values increased with increases in pavement stress. Again the repeat tests for the OH100%E and OH100%F $\uparrow$ 3:1 showed good repeatability with results

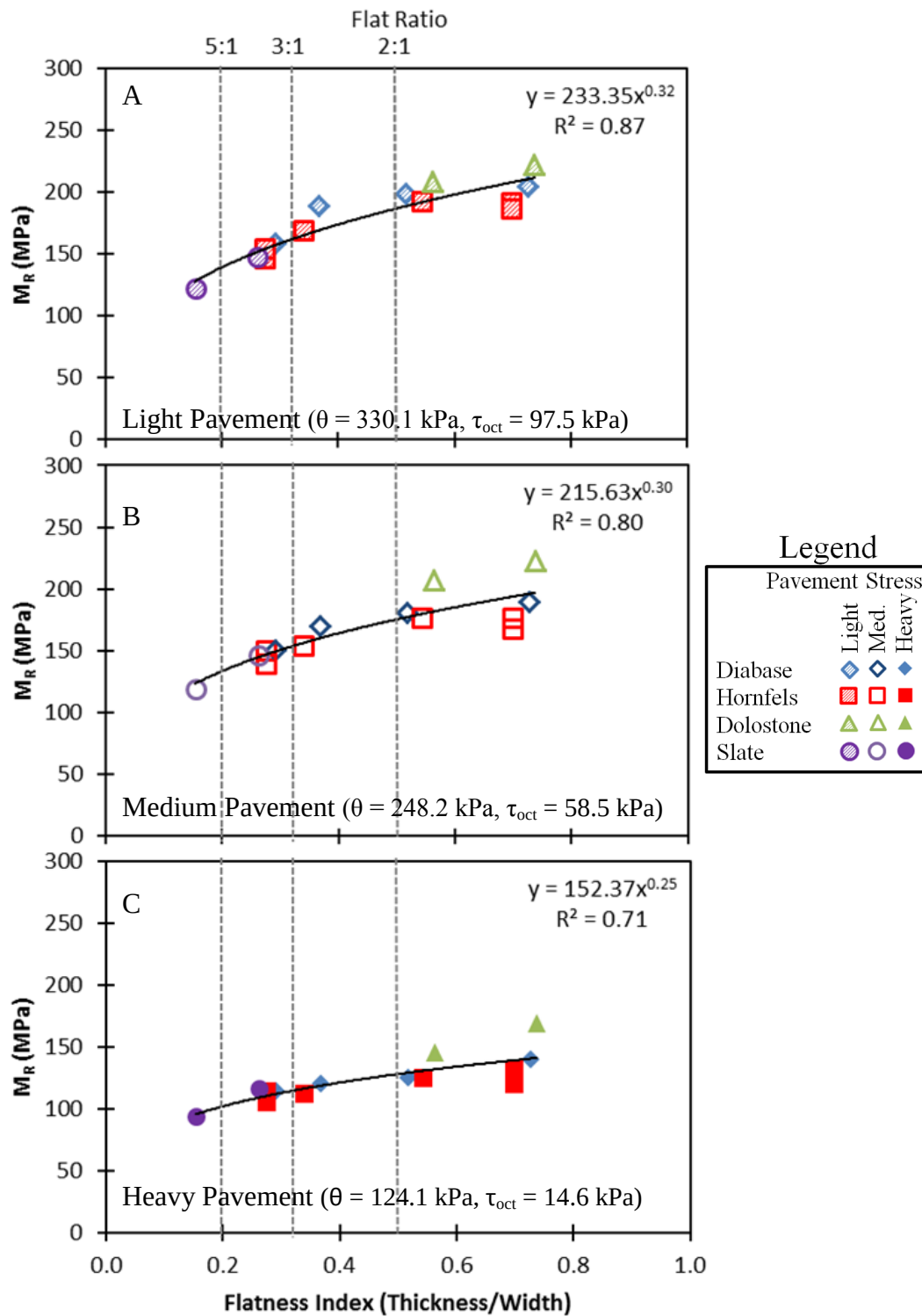


Figure 44. M_R vs. Flatness Index for Open-Graded Samples for A) Light Pavement Structure, B) Medium Pavement Structure, and C) Heavy Pavement Structure

When looking at all three graphs in **Figure 44**, both open-graded dolostone samples showed the highest M_R values across all three pavement stresses when compared to the other open-graded materials. The slate sample with 5:1 ratio coarse particles (OS100%F $\uparrow$ 5:1) showed the lowest M_R values across all three pavement stresses. The slate sample with coarse particles in the 3:1 to 5:1 flat ratio range (OS100%F $\uparrow$ 3:1 $\downarrow$ 5:1) was more comparable in M_R values to the flat 3:1 ratio diabase and hornfels samples (OD100%F $\uparrow$ 3:1, OH100%F $\uparrow$ 3:1, and OH100%F $\uparrow$ 3:1#2).

The effect of particle shape for each of the four individual aggregate types is consistent with an increase in M_R value as the particles become more equidimensional. Based on this observation, and that the OS100%F $\uparrow$ 3:1 $\downarrow$ 5:1 sample M_R value was very similar to the OD100%F $\uparrow$ 3:1, OH100%F $\uparrow$ 3:1, and OH100%F $\uparrow$ 3:1#2 M_R values, all four aggregates could be grouped together and evaluated based on the changes of shape. Regression analysis was used to determine the relationship between the flatness index and the estimated M_R for all of the open-graded materials.

As can be observed in **Figure 44**, there is a clear correlation that as the flatness index value increases (the particle are becoming more equidimensional) the M_R value also increases. The change of M_R value across the range of flatness index value is most pronounced when the highest pavement stresses associated with the light pavement structure is used, as shown in **Figure 44A**. The slope of the fitted curve is similar between **Figure 44A** and **Figure 44B**, but shows a marked decreases in **Figure 44C**. This decrease would indicate that the effect of particle shape on the M_R decreases as the amount of stress decreases.

The decrease in M_R value because of the particle shape could be caused by variation in matrix structure that is observed with the change in densities between the open-graded materials of the same mineralogy (**Table 12**). The matrix structure of the open-graded materials was in a Low SS state and the PS particles are the load-carrying skeleton of the material. The density of the PS matrix decreases as the particles became more flat, so therefore the matrix structure could become less elastically stable with the decrease in the number of contact points between the PS particles. This finding falls in line with research conducted by Kolisoja (1997) in relation to compaction, which stated that as the number of particle contact increased, the resilient modulus increased due to decrease in average contact stress.

No major effect of mineralogy is observed on the estimated M_R values for the open-graded materials. This is based on the fact that all mineralogies have similar M_R values when similar shapes are compared. This observation is supported by the fact that the matrix structure of the UAB materials is in a Low SS state, which would negate the effect of the SS particles, and that the PS particles are all similar due to the fact that all are 100% crushed material. The SS structure is not carrying load, but is only filling the voids between the PS particles.

Comparison Between Dense- and Open-Graded M_R Interpreted Results

Comparison between dense and open-graded M_R samples is presented in **Figure 45**. **Figure 45A** and **Figure 45B** were created by combining all three predicted M_R values for the four UAB mineralogies for the dense- and open-gradations (as shown in **Figure 43** and **Figure 44**). **Figure 45C** was created by creating trend lines fitted to the dense-

graded M_R points for the light, medium, and heavy pavement stress and plotting the trends lines presented in **Figure 44** for the open-graded M_R points for the light, medium, and heavy pavement stresses. **Figure 45C** allowed for general comparison of the predicted M_R at the three stresses between the dense- and open-graded materials

It is observed in **Figure 45C** that for the heavy and medium pavement stress conditions, the open-graded samples showed higher M_R values than dense-graded samples. A variation between dense- and open-graded M_R samples is not observed at the light pavement stresses since the estimated trend lines are almost plotted on top of each other.

Generally, the open-graded materials show higher M_R values than the dense-graded materials. This trend can be attributed to variation in matrix structure between the two gradations. The open-graded materials have a Low SS matrix that resulted in higher M_R values because the SS is not disrupting the interlocking of the PS particles. The dense-graded materials have a High SS matrix, which has resulted in the PS skeleton being disrupted by the SS. The disruption of the interlocking of the PS particles reduces the M_R value of the material at the heavy and medium pavement stresses. The reduction of the M_R value when comparing Low and High SS matrix structures has been observed by Yideti et al. (2014) and Richardson and Lusher (2009). The lack of variation between the dense- and open-graded M_R values at the light pavement stress could be attributed to a threshold of stress where the M_R responses of the matrix structures are similar.

The SS particles that carry the load for the dense-graded materials may have lower M_R values at lower stresses, but have better M_R response at higher stresses.

Therefore, the ability of the SS to carry the load for the dense-graded materials approaches the ability of the PS of the open-graded materials as the pavement stresses increases across the three pavement structures.

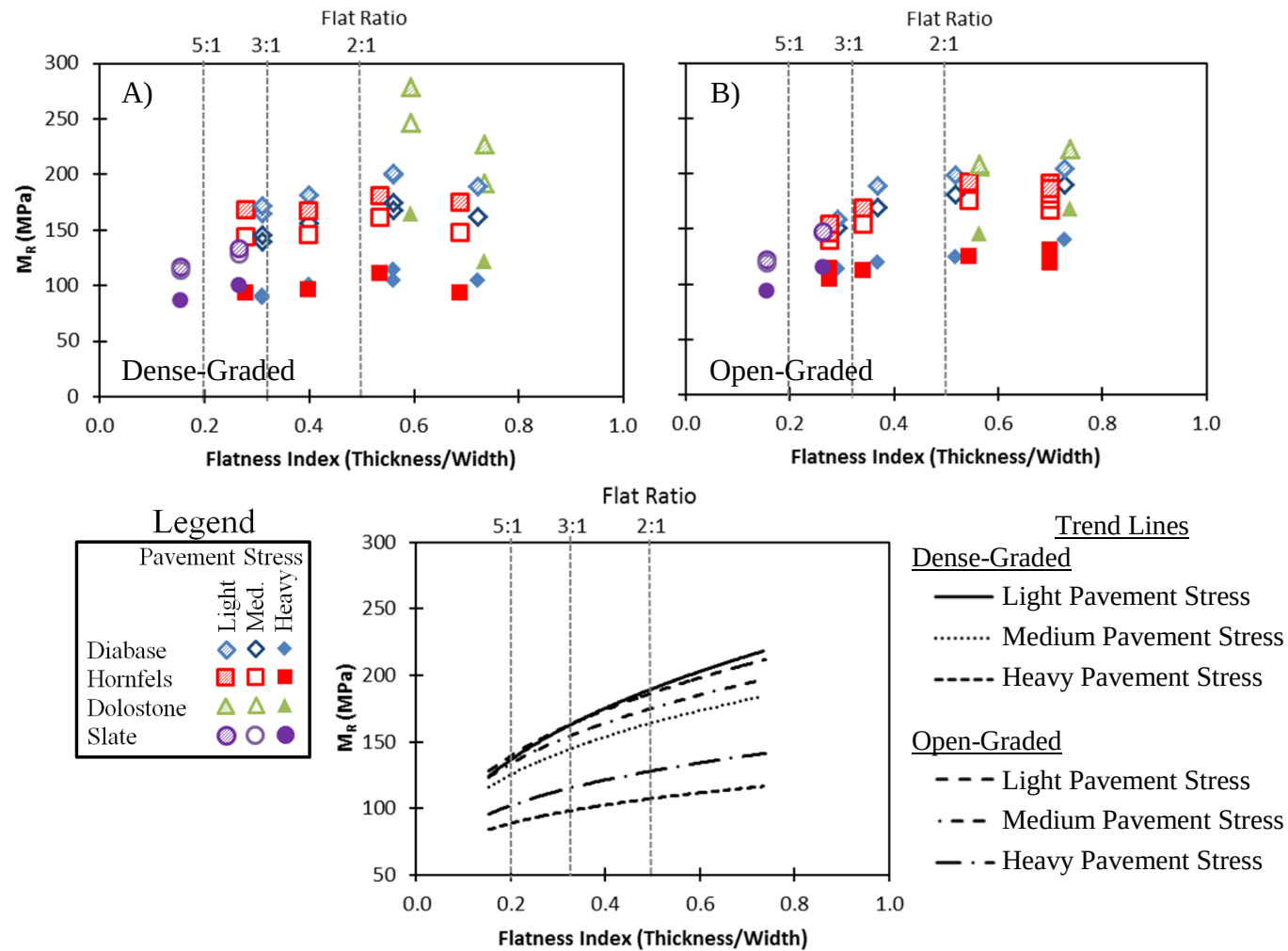


Figure 45. Comparison Between Dense- and Open Graded M_R Results;
A) Dense Graded M_R Results, B) Open-Graded M_R Results, and C) Comparison Between Dense- and Open-Graded

5.3.2 Interpretation of Permanent Deformation Results

The PD tests for both dense- and open-graded materials were evaluated based on the final PD accumulation at the end of the 10,000 cycle loading sequence, as well as the shakedown range (whether the material classifies as in range A, B, or C) (both values for every permanent deformation tests was shown in **Table 20** and **Table 21**). Additionally, the PD tests were evaluated base on the overall trends regarding specifically particle shape and mineralogy.

Dense-Graded Medium Pavement Stress

The interpreted results for the dense-graded UAB materials tested at the medium pavement stress (confining pressure of 41.4 kPa and deviator pressure of 124.1 kPa) is presented in **Figure 46**. **Figure 46A** presents the final ϵ_p when specifically looking at only the variation in flatness index across the tested samples. An interpreted line was fit to the data by regression analysis (**Figure 46A**) and shows a low correlation between increases in accumulated PD and an increase in flatness index value.

When the mineralogy of each sample is examined (**Figure 46B**), the diabase and hornfels samples show peak PD at a flatness index value of ~ 0.6 (non-equidimensional particle shape), while the dolostone and slate sample showed increases in PD as the flatness value increases. Most of the material was classified in shakedown range B, with the exception of the diabase samples with a flatness index value at or above ~ 0.4 which are only slightly past the shakedown range B classification (values shown in **Table 20**).

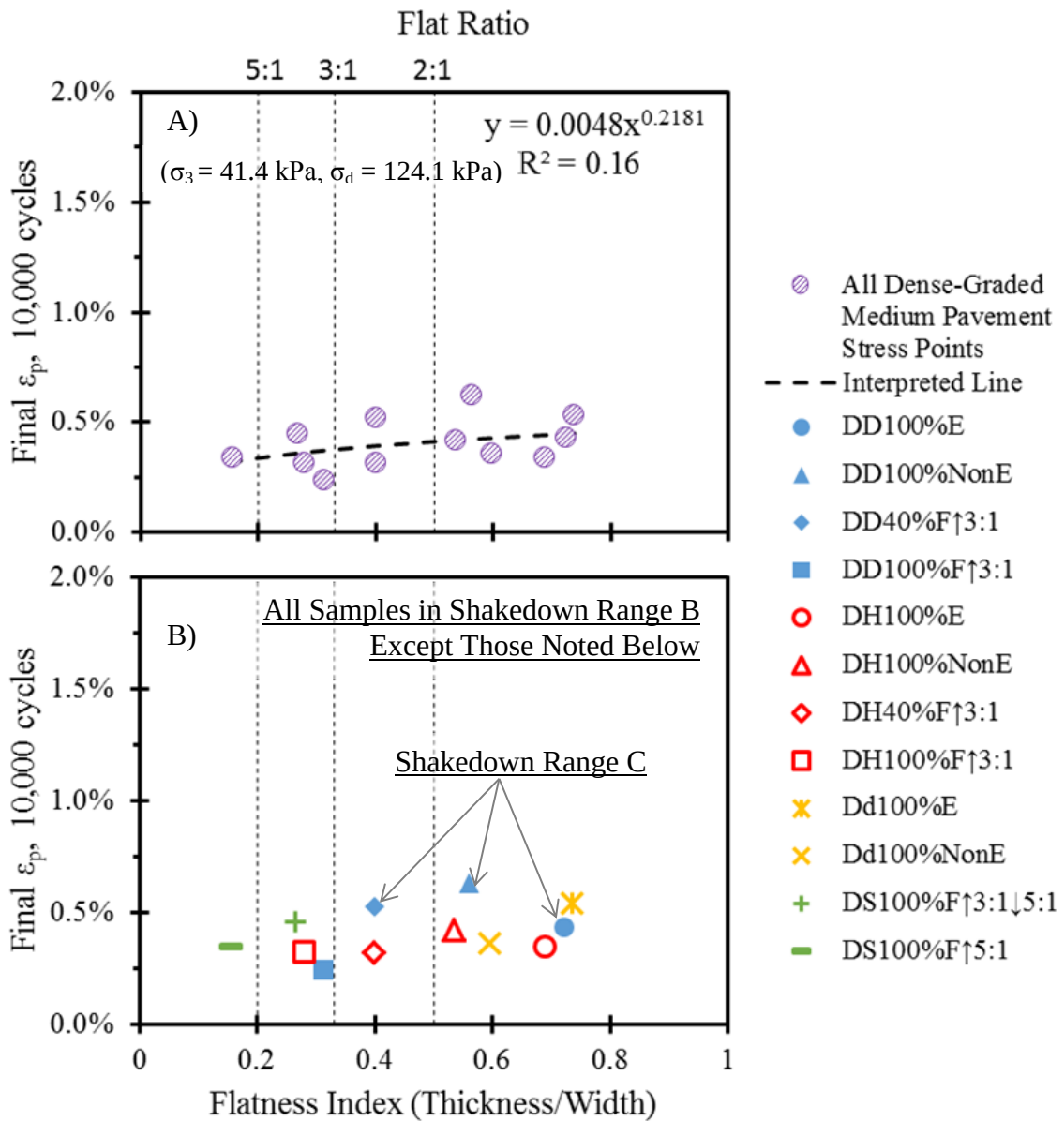


Figure 46. Dense-Graded Medium Pavement Stress Permanent Deformation Interpreted Results; A) View Based on Particle Shape and B) View Based on Minerology with Shakedown Range Classification Noted

Overall, at the medium pavement stress there is a low correlation between an increase in flatness index and increases in accumulated PD. Additionally, all shakedown

range values are similar indicating a similar PD. Neither particle shape nor mineralogy showed a strong observed trend with the PD accumulation.

Dense-Graded Light Pavement Stress

Figure 47 shows the interpreted results for the dense-graded UAB material at the light pavement stresses (confining pressure of 103.4 kPa and deviator pressure of 103.4 kPa). There is a marked increase in PD accumulation for the light pavement stress when compared to the medium pavement PD accumulation. The samples most effect by the increase in stresses was the slate samples, showing a two to three fold increase in PD, and the UAB dense-graded sample with a flatness index value greater than 0.5, with these six samples showing an average 3 fold increase in PD. These two groups of samples show the lowest resistance to PD with the increase in stress.

Two hypothesized mechanism are thought to account for these large increases. For the diabase, hornfels and dolostone samples, the large increase in PD is because the equidimensional and non-equidimensional particles are more likely to reorient and move when compared to 3:1 flat particles. The large increase in PD accumulation in the slate samples is thought to be cause by the lower resistance of the load carrying SS because of the mineralogy. The mechanism based on particle shape will first be discussed (in the diabase, hornfels, and dolostone samples) and then the mechanism based on the mineralogical differences will be discussed.

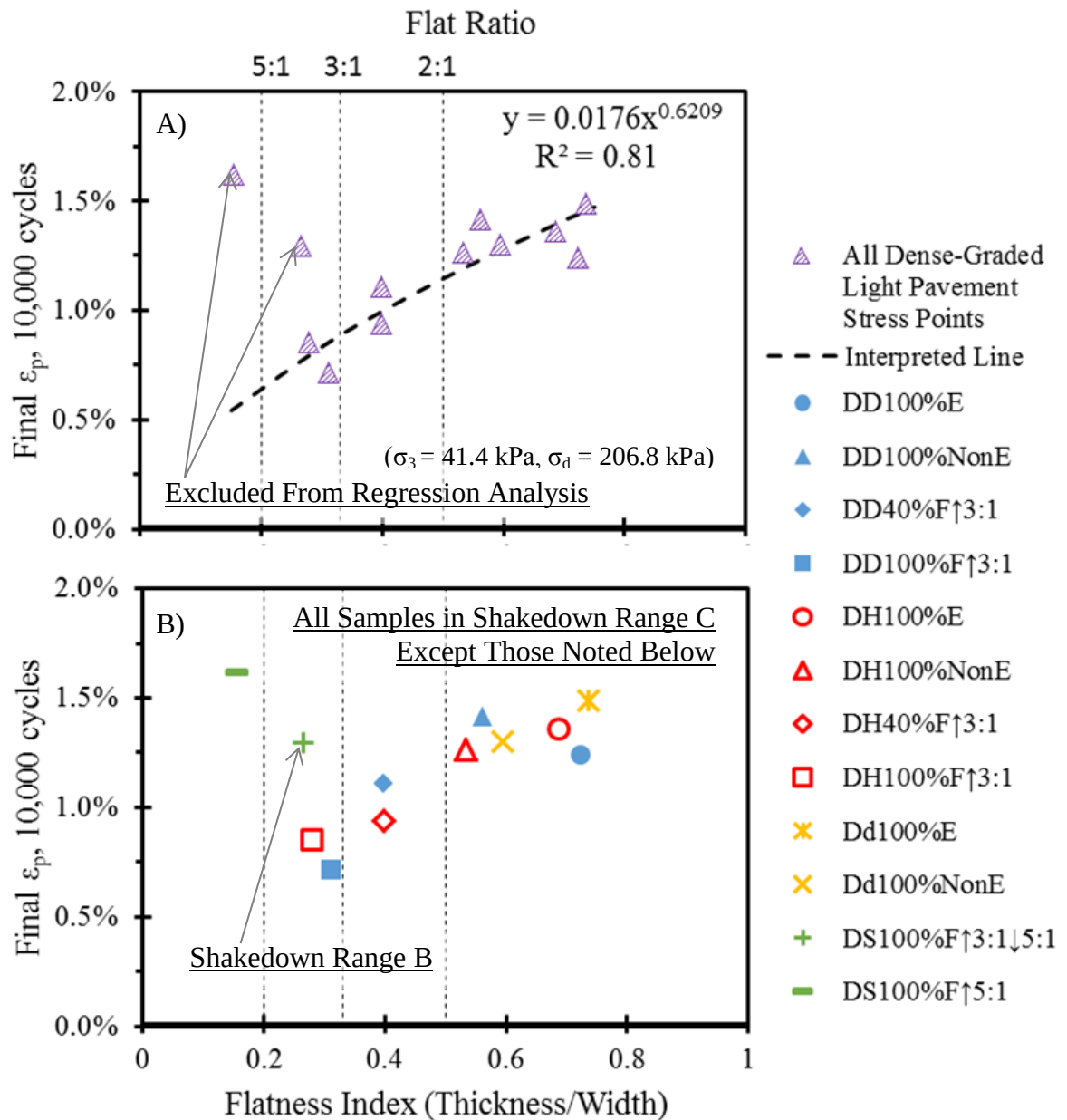


Figure 47. Dense-Graded Light Pavement Stress Permanent Deformation Interpreted Results; A) View Based on Particle Shape and B) View Based on Minerology with Shakedown Range Classification Noted

In **Figure 47A**, when the relationship between the PD accumulation and the flatness index is investigated for the durable aggregate (excluding slate as discussed above), there is a strong correlation ($R^2 = 0.81$) between increased flatness index and increasing PD. This observed trend could be caused by reduced movement of floating PS particles in the SS because of the variation in particle shape.

The PS particles are floating in the SS particles because the dense-graded UAB materials have a High SS matrix. As the particles become more flat (as there is an increase in flat particles at the 3:1 ratio) there is a greater resistance to reorientation. A flat particle at the 3:1 ratio has more surface area than a non-equidimensional or equidimensional particle retained at the same sieve; therefore, there would be more contact points per particle between the PS and SS particles. This increase in contact points would reduce movement/reorientation of PS particles floating in the SS and reduce the total PD of the sample.

The increase in PD accumulation because of mineralogical variation is discussed. The slate mineralogy has a much lower durability than the other three mineralogies (as shown in the micro-deval tests, **Figure 40**). This lower durability reduces the ability of the slate SS particles to resist PD. In order to show this variation in SS, **Figure 48** shows the DD100%F $\uparrow$ 3:1, DH100%F $\uparrow$ 3:1, and DS100%F $\uparrow$ 3:1 $\downarrow$ 5:1 sample light pavement stress PD tests. The slate sample shows twice the amount of PD accumulation during the first 1,000 cycles of the test when compared to the diabase and hornfels samples. This increased initial PD is thought to be attributed to the low durability of the SS particles in the slate UAB sample.

After the initial PD during the first 1,000 cycles, the DD100%F $\uparrow$ 3:1, DH100%F $\uparrow$ 3:1, and DS100%F $\uparrow$ 3:1 $\downarrow$ 5:1 samples show similar calculated shakedown range values; 4.6×10^{-4} , 4.4×10^{-4} , and 3.5×10^{-4} respectively. Since all three samples had a similar shakedown range values, the cause of the increase in PD for the DS100%F $\uparrow$ 3:1 $\downarrow$ 5:1 is because of the reduced strength of the slate SS particles during the early loading cycles. This observation was not observed in the medium pavement stress PD, but this could be due to the fact that the stresses were not high enough in the medium pavement PD to distinguish between the stronger SS.

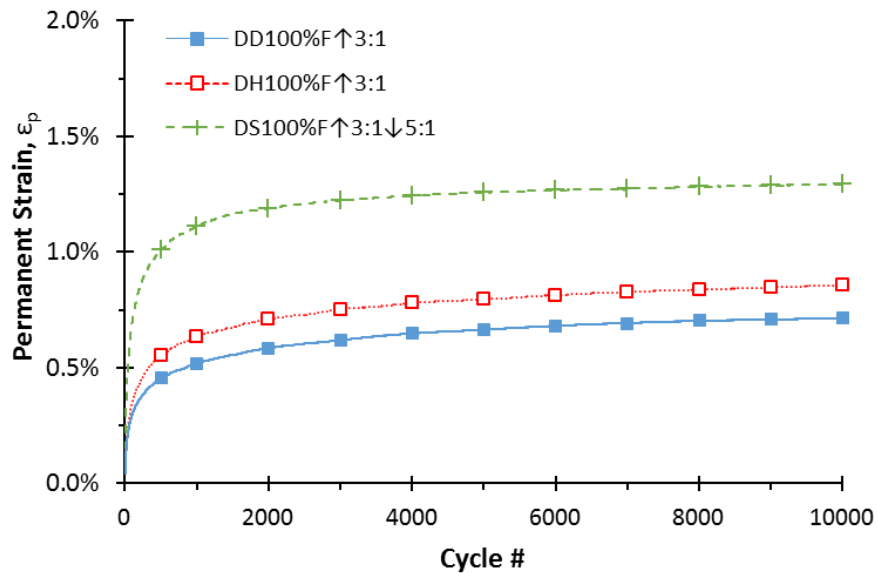


Figure 48. Permanent Deformation Light Pavement Stress Comparison Between DD100%F $\uparrow$ 3:1, DH100%F $\uparrow$ 3:1, and DS100%F $\uparrow$ 3:1 $\downarrow$ 5:1

A similar logic can be applied to explain the observed variation in final PD accumulation between the two slate samples (DS100%F $\uparrow$ 3:1 $\downarrow$ 5:1 and DS100%F $\uparrow$ 5:1). There is a reduced SS particle strength between the DS100%F $\uparrow$ 3:1 $\downarrow$ 5:1 and DS100%F $\uparrow$ 5:1 because of varying compaction levels of the SS. The DS100%F $\uparrow$ 5:1 sample has a lower compacted %RUW_{SS} number when compared to the DS100%F $\uparrow$ 3:1 $\downarrow$ 5:1 (91 as opposite to 97) which means reduced compaction and particle interlocking for the SS structure of the DS100%F $\uparrow$ 5:1 sample. This reduced compaction and particle interlocking in the SS leads to a higher PD accumulation during the first 1,000 cycles, 1.1% as opposed to the 1.3% shown in **Figure 49**. After that initial PD, the DS100%F $\uparrow$ 3:1 $\downarrow$ 5:1 and DS100%F $\uparrow$ 5:1 samples have similar shakedown range values; 3.5×10^{-4} and 5.3×10^{-4} .

The reduced compacted state of the SS structure in the DS100%F $\uparrow$ 5:1 could be attributed to the very large flat ratios of the PS particles which hinder the compaction of SS particles. So, the flat particles at the 5:1 increased the PD accumulation because of their effect on the compacted density of the SS particles. The reduced compaction density in the slate sample was shown in **Figure 36**.

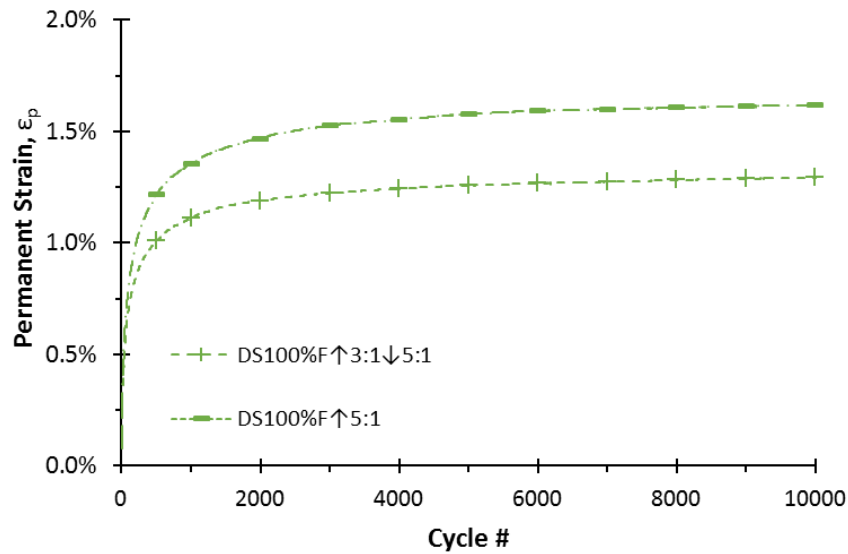


Figure 49. Permanent Deformation Light Pavement Stress Comparison Between DS100%F↑3:1↓5:1 and DS100%F↑5:1

Summary of Dense-Grade Interpreted Results

The results of the dense-graded PD tests show that particle shape and the mineralogy of the SS has an effect on the accumulated PD. There was a low correlation between increases in PD as the flatness index value increased in the medium pavement stress PD tests. The PD accumulation at the light pavement stresses showed a strong correlation between increases in PD for the diabase, hornfels, and dolostone samples with an increase in flatness index value. The light pavement stresses PD tests also showed the importance of SS mineralogy since the slate samples had a marked increase in PD accumulation during the first 1,000 cycles of the test.

Open-Graded Medium Pavement Stress

Figure 50 shows the interpreted results for the open-graded UAB PD results tested at the medium pavement stress. When just the particle shape is considered (**Figure**

50A), there is a small increase in PD accumulation with an increase in flatness index value. When the individual mineralogies are investigated (**Figure 50B**), the diabase and hornfels samples have consistent PD accumulation across the complete range of particle shapes investigated. The slate samples show the lowest PD accumulation out of all samples and the dolostone samples frame the final ϵ_p ranges measure for the diabase and hornfels samples. Overall, the PD accumulation at the medium pavement stress shows little variation because of either particle shape or mineralogy.

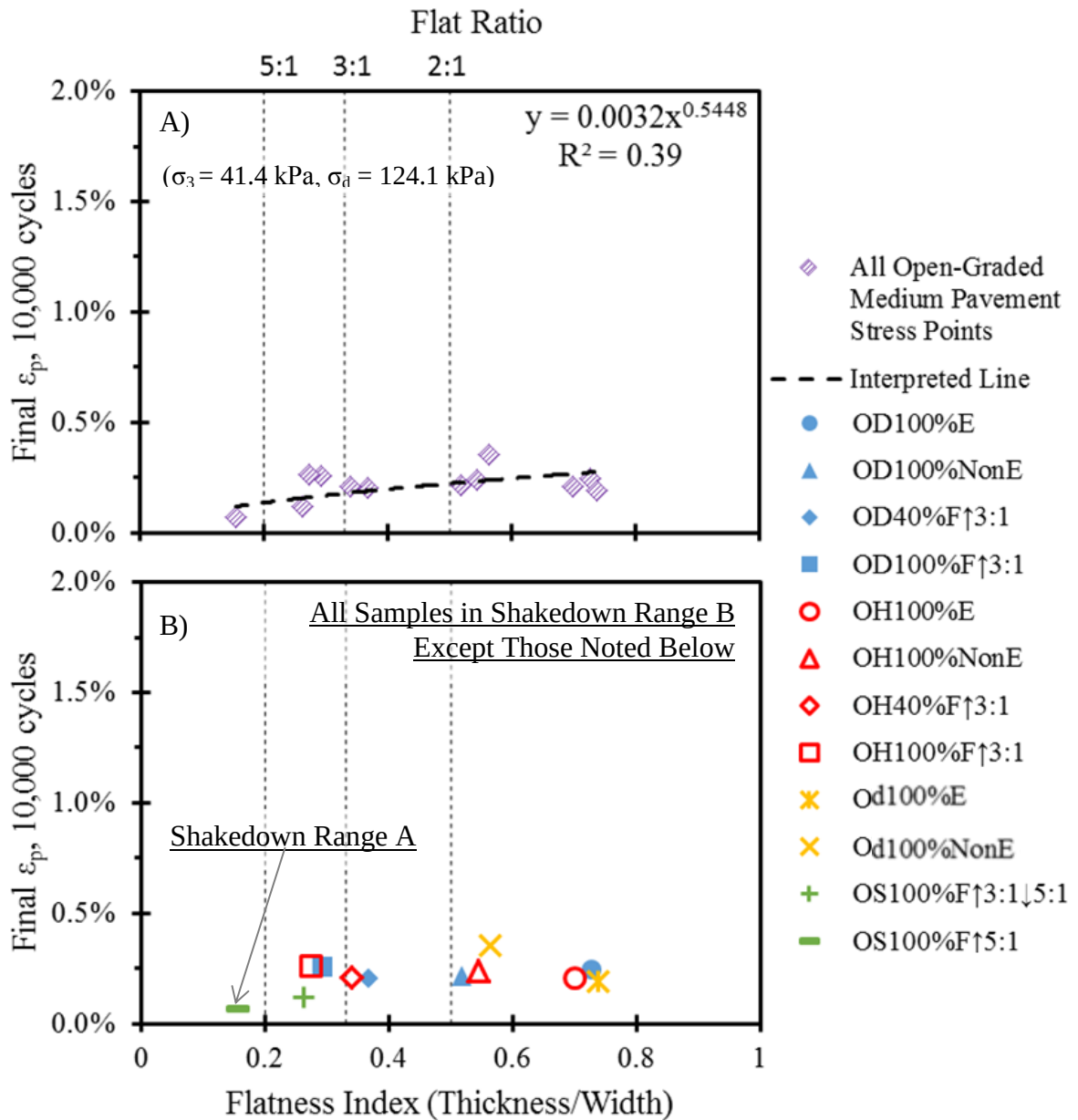


Figure 50. Open-Graded Medium Pavement Stress Permanent Deformation Interpreted Results; A) View Based on Particle Shape and B) View Based on Mineralogy with Shakedown Range Classification Noted

Open-Graded Light Pavement Stress

Interpreted results for the open-graded light pavement stresses are presented in **Figure 51**. When looking at the effect of particle shape (regardless of mineralogies) in **Figure 51A**, there is a general trend of increasing PD with increasing flatness index. The large variations observed within some samples of similar shape could be caused by varying interlocking characteristics of PS as specified by the Bailey method. As discussed in section 5.1.3, the matrix structures of some of the open-graded materials are in a transition zone between the Low SS or High SS, while some are in the Low SS.

In order to show this variation in matrix interlocking characteristics of samples of similar particle shapes, **Figure 51A** was marked with the $\%LUW_{PS}$ as conducted by the Bailey method in **Figure 52**. When looking in Zone 1 (with samples included being the equidimensional diabase, hornfels, and dolostone samples), the sample are all classified by the Baily method as being in a transition zone between High SS and Low SS since the $\%LUW_{PS}$ values are between 90 and 95. So, the PS particles could or could not be in contact. Based on observations by Yideti et al. (2013), when the PS particles are in contact there is increased interlocking and resistance to PD. The variation in final ϵ_p between samples classified in the matrix transition zone could suggest the degree of interlocking of the PS.

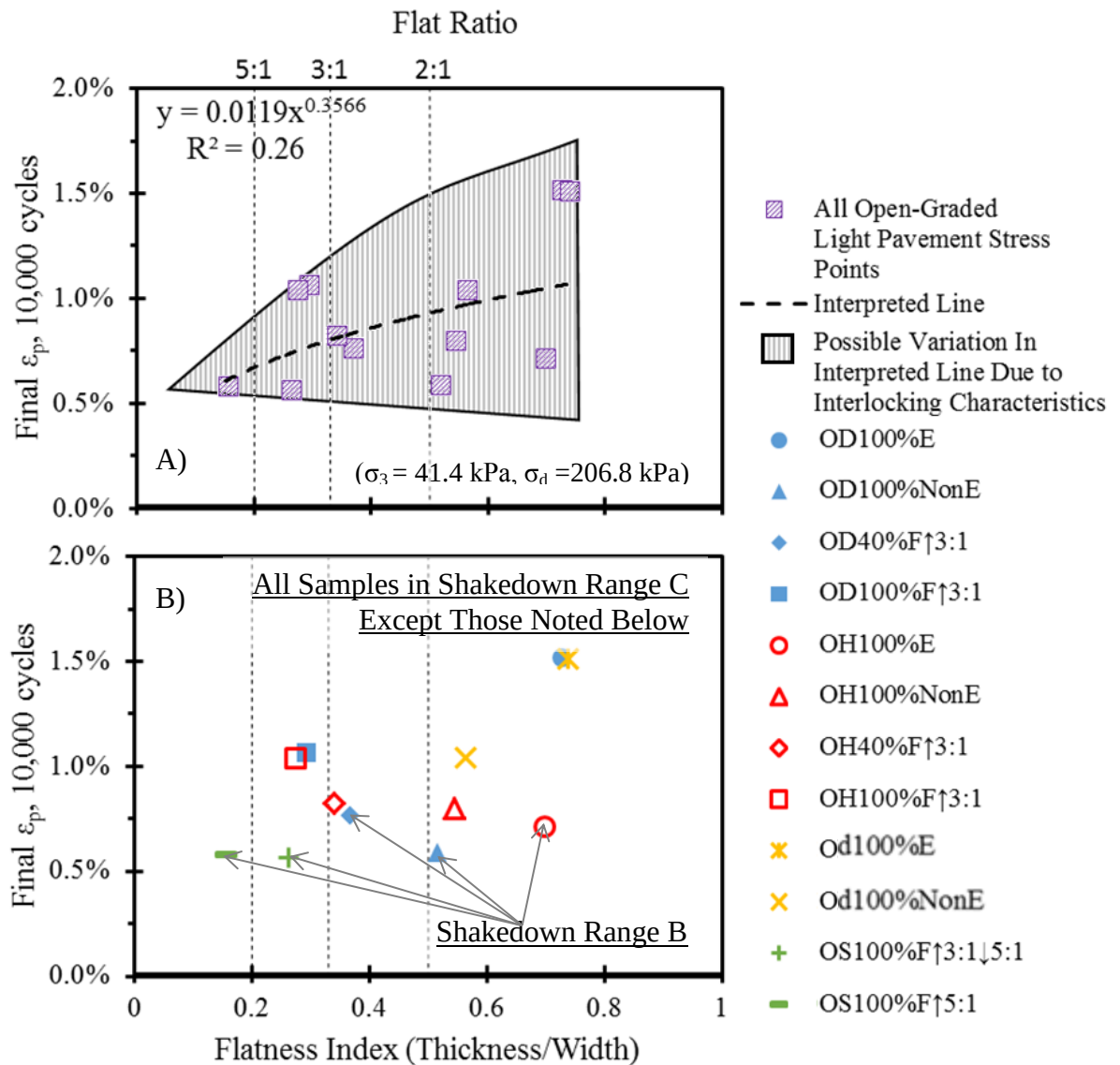


Figure 51. Open-Graded Light Pavement Stress Permanent Deformation Interpreted Results; A) View Based on Particle Shape and B) View Based on Minerology with Shakedown Range Classification Noted

With these assumptions, the large variation in final ϵ_p in Zone 1 (**Figure 52**) could be attributed to the samples at the 1.5% final ϵ_p being more in the High SS matrix while the 0.7% final ϵ_p sample had more PS contact points. In Zone 2 (non-equidimensional diabase, hornfels, and dolostone), all three samples are classified in the transition zone. Based on the observed final ϵ_p , the 1.0% final ϵ_p sample had less particle contacts between the PS than the other two samples in the same zone. In Zone 3 (samples with 100% flat particles at the 3:1 or 5:1 ratio), all sample are classified as in the Low SS state since the %LUW_{PS} values are above 95. The added benefit of the increased %LUW_{PS} is shown since the 104 and 107 samples (slate mineralogy) had half the PD accumulation than the 99 and 97 samples (diabase and hornfels).

Overall for the open-graded tests at the light pavement stress, little definitive observations with regard to the effect of particle shape can be made since half of the samples are in the transition zone between the Low SS and High SS matrix structures. In order to narrow down the effect of particle shape in an open-graded gradation, a gradation would have to be selected and tested where all samples are firmly in the Low SS matrix structure and have similar interlocking characteristics (%LUW_{PS} $\approx$ 103).

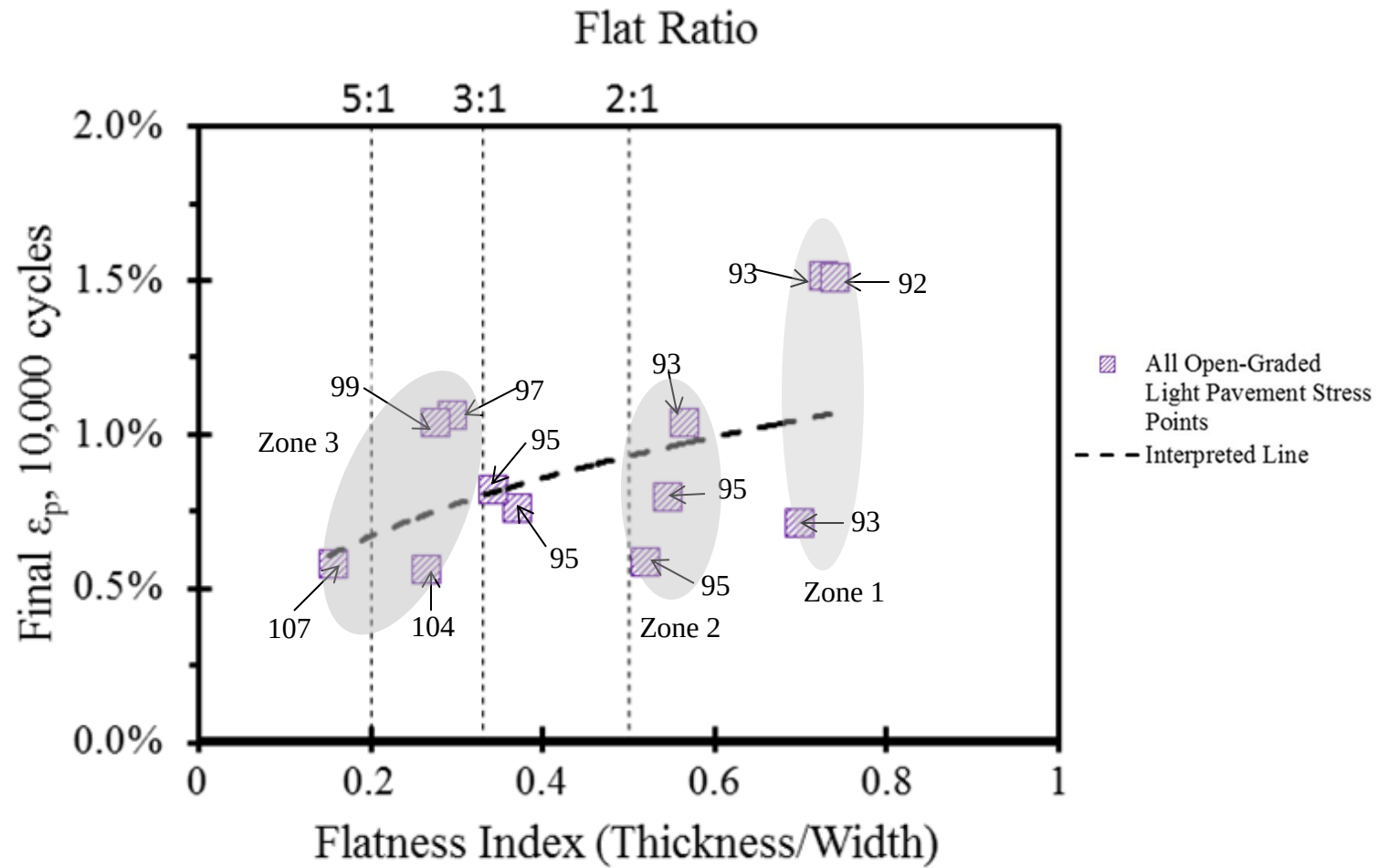


Figure 52. Open-Graded Medium Pavement Stress Permanent Deformation Interpreted Results with %LUW_{PS} Values Noted

Summary of Open-Grade Interpreted Results

There is a minor increase in final ϵ_p with increasing flatness index in the open-graded UAB samples subjected to the medium pavement stress (ranging between 0.07% and 0.35%). The effect of particle shape is more pronounced in the light pavement stress permanent deformation tests, showing an increase in permanent deformation with increasing flatness index. This observed relationship in the light pavement stress permanent deformation tests is not definitive since the type of interlocking of the PS were in a transition zone for half of the samples tested causing large variation in final ϵ_p .

Comparison Between Dense- and Open-Graded Permanent Deformation Interpreted Results

The comparison between the open and dense-graded PD results are plotted in **Figure 53**. **Figure 53B** shows that the open-graded materials consistently performed better in the final PD accumulation for the medium pavement stresses. The comparison between the dense- and open-graded materials PD in the light pavement stresses show that the open-graded materials generally perform better. The observation that there is more resistance to PD accumulation with increasing PS interlocking (the dense-graded with the no PS interlocking) was also noted by Yideti et al. (2013).

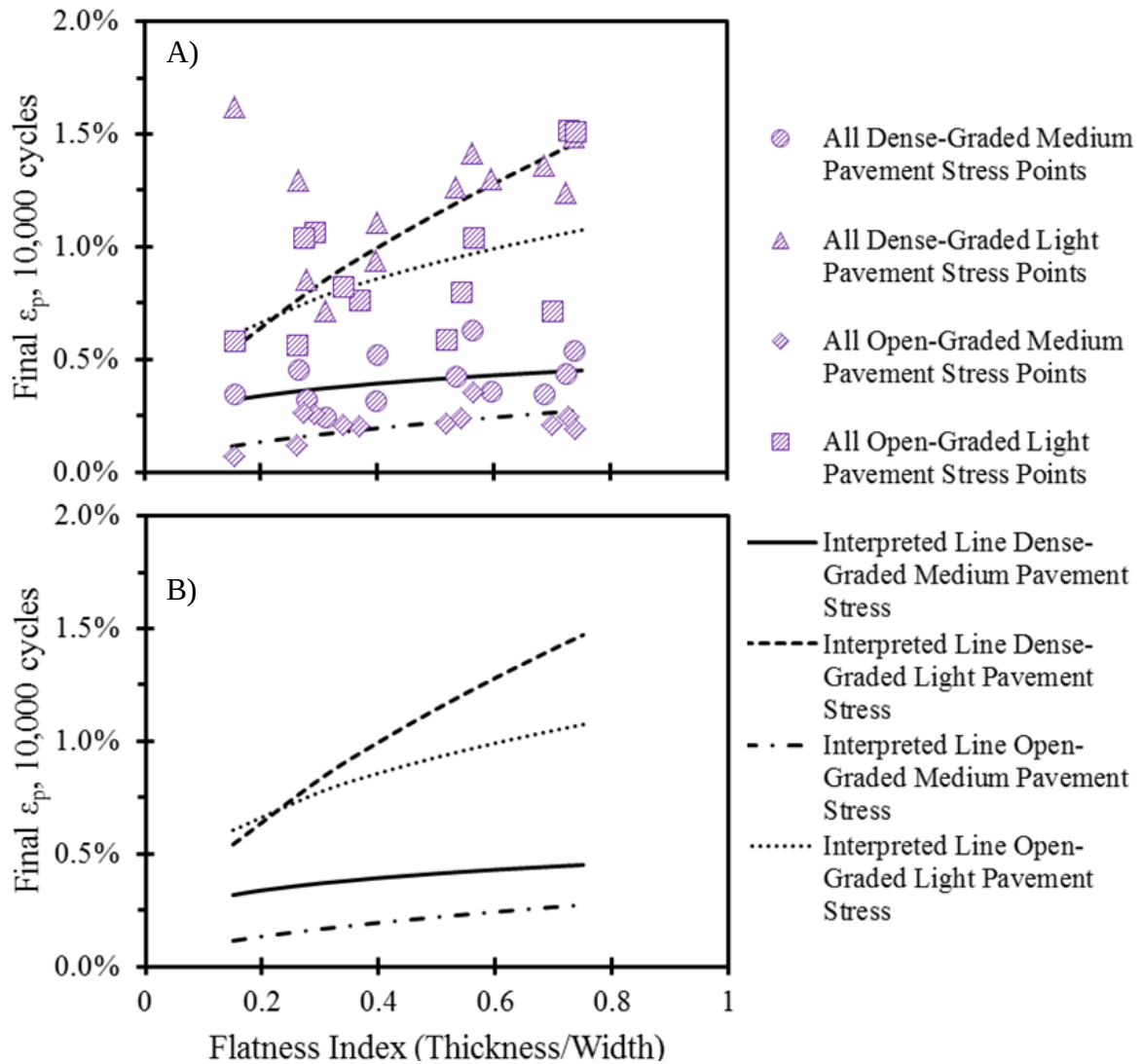


Figure 53. All Permanent Deformation Tests; A) Interpreted Lines with Data Points and B) Interpreted Lines without Data Points

5.3.3 Interpretation of Elastic Response Obtained from Permanent Deformation Tests

As discussed in section 5.2.3, the M_R value (Equation 1) at the last five cycles for all dense- and open-graded PD tests were averaged and recorded. This value was termed the elastic response of the material in this thesis since the M_R term is specifically used for

the elastic response of a material that has been determined from a specific loading sequence, such as the AASHTO T-307 fifteen stage loading sequence. These values will be different in magnitude than the estimated M_R values in section 5.3.1, but could be useful in conforming or rejecting trends observed in section 5.3.1 for both the dense- and open-graded M_R tests.

The observed elastic responses of the dense-graded PD tests are presented in **Figure 54** and will be compared to trends observed in **Figure 43**. The observation that the dolostone samples had the highest M_R value and that the slate samples had the lowest M_R value are supported. The observed trend that as the particle shapes became flatter that there was a decrease in M_R is not observed, nor that there is a possible peak at a flatness index value of 0.6 regardless of mineralogy. The M_R values are fairly constant for a particular mineralogy with changes in flatness index values, which is why the interpreted lines for each mineralogy showed consistent M_R values with changes in flatness index values. The observation that the SS load carrying capacity affects the M_R values is supported since there is separation between samples of different mineralogies, indicating that the SS characteristics are important for the High SS matrix structure.

The observed elastic responses of the open-graded PD tests are presented in **Figure 55** and will be compared to trends observed in **Figure 44**. The strong correlation between and increase in flatness index value and elastic response is supported in **Figure 55** since regression analysis yielded R^2 values in the eighties. Additionally, the observed lack of variation in elastic response due to mineralogical effects is also supported. The

elastic response of PD tests show good agreement with trends observed in the M_R for both the dense- and open-graded materials.

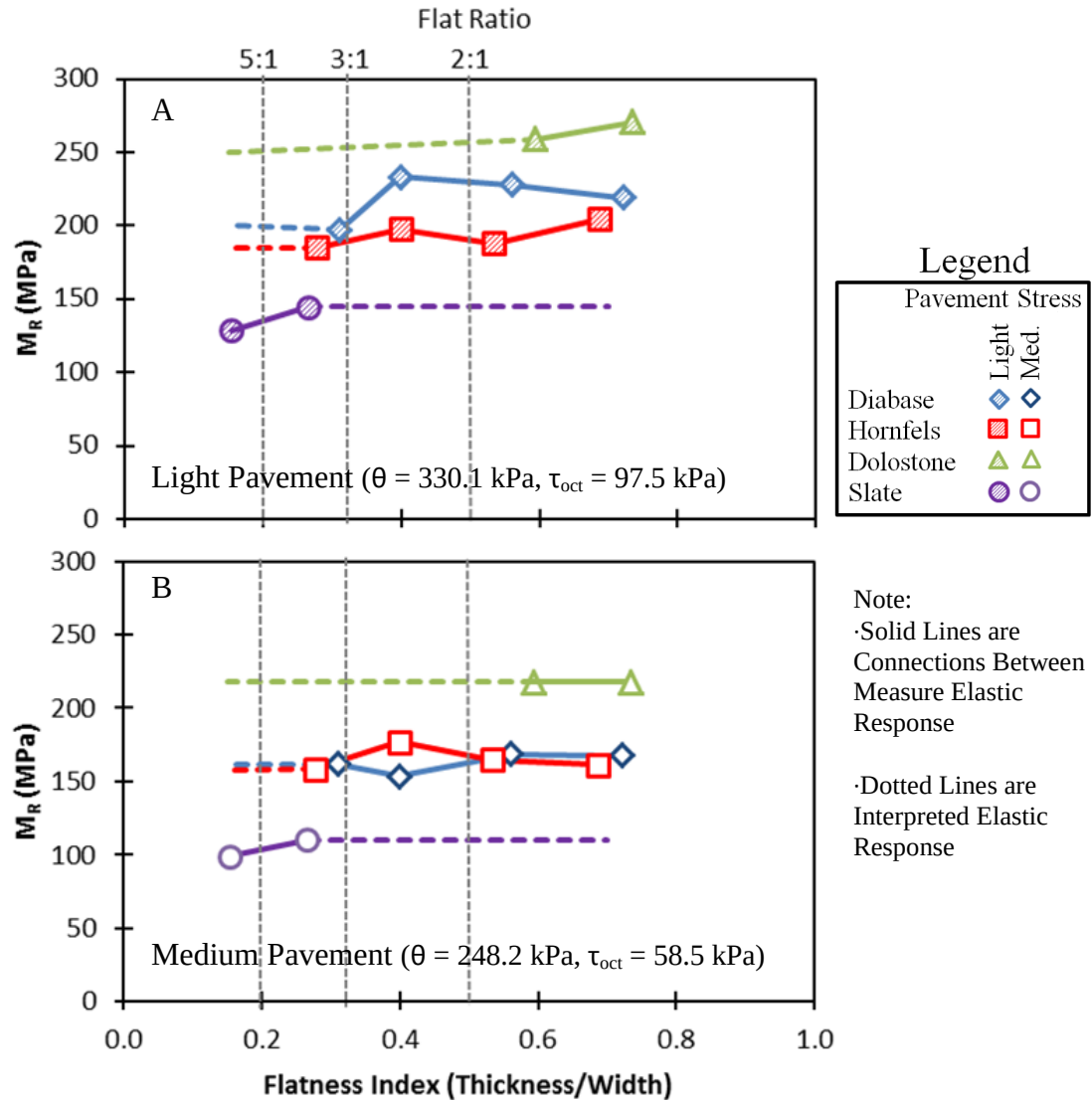


Figure 54. Elastic Response of Averaged Last Five Cycle for Dense-Graded Permanent Deformation Tests; A) Light Pavement Stress, and B) Medium Pavement Stress

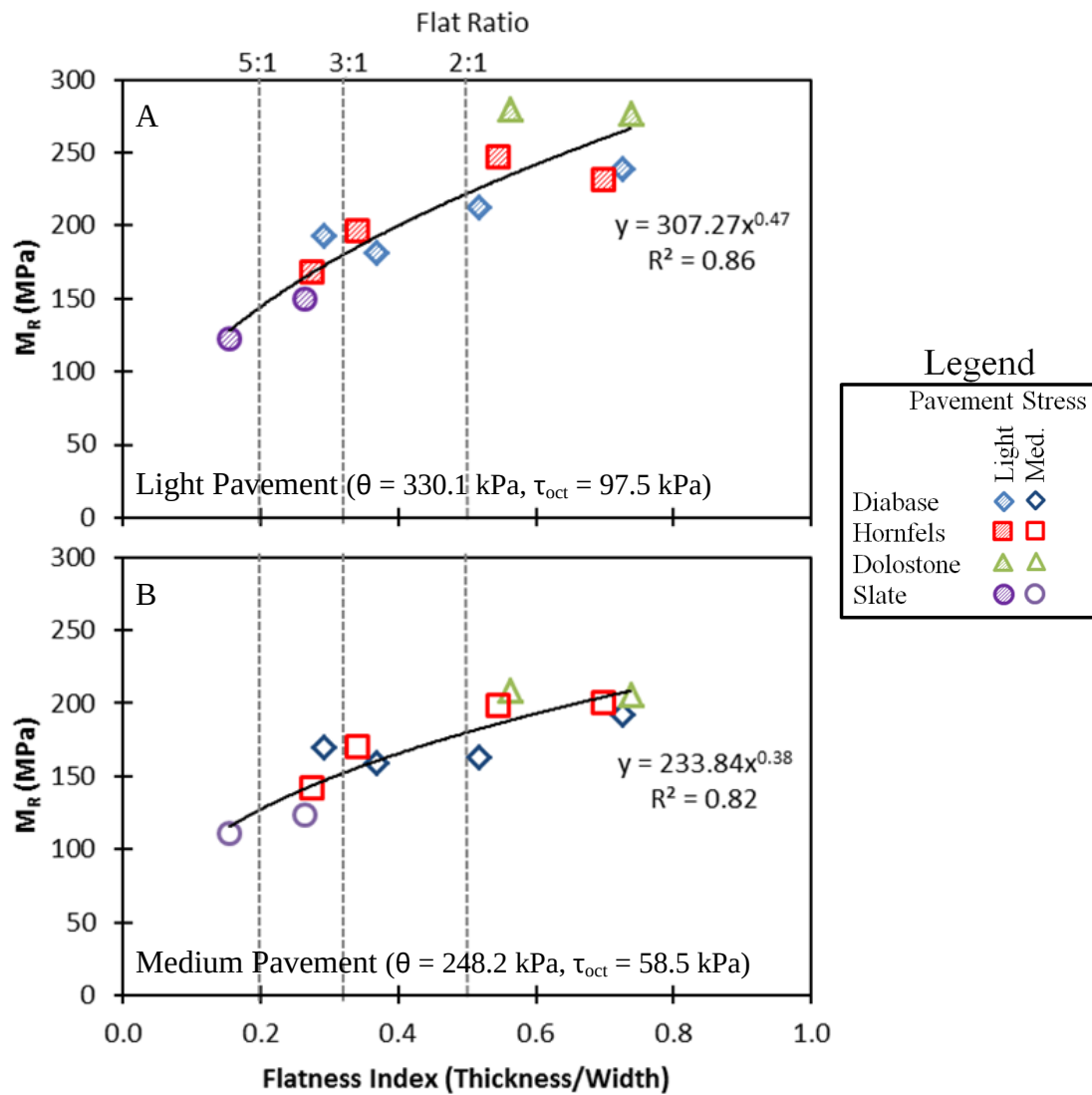


Figure 55. Elastic Response of Averaged Last Five Cycle for Open-Graded Permanent Deformation Tests; A) Light Pavement Stress, and B) Medium Pavement Stress

5.4 Summary of Notable Trends

All particle shape analysis, compaction, matrix characterization, MD, M_R , and PD tests were interpreted and discussed in the results section. Noteworthy trends from those tests are presented below.

- The Flatness Index and Elongated Index values confirmed that the particle shape separation process resulted in consistent creation of particle shapes within and between varying mineralogies
- The effect of particle shape on compaction density was much more pronounced in the open-graded than the dense-graded UAB material, with an observed decrease in compacted density of the open-graded materials due to a decrease in flatness index value across all four mineralogies
- The matrix structure of the dense-graded and open-graded materials was substantially different and the Bailey method allowed for successful quantification of the varying matrix structures and allowed for defining of particle interlocking in the PS and SS
- MD results indicated that the mineralogy is substantially more important in the durability of the aggregate when compared to the effect of particle shape, but a marked increase in particle loss was observed with the increase in flatness index values studied for the diabase and hornfels samples

- The overall variation in M_R between both dense- and open-graded materials decreased with the decrease in overall estimated stress (heavy pavement stress showed least amount of variation)
- M_R results of open-graded aggregate showed a strong trend between increasing M_R value and increased flatness index, dense-graded showed much smaller increase in M_R (peak at ~ 0.6 flatness index value) and was effect by the mineralogy of the SS
- PD tests conducted at a medium pavement structure stress ($\sigma_3 = 41.4$ kPa, $\sigma_d = 124.1$) showed weak correlation between an increase in flatness index value and an increase in PD accumulation for both dense- and open-graded materials
- PD tests conducted at a light pavement structure stress ($\sigma_3 = 41.4$ kPa, $\sigma_d = 206.8$) showed a moderate to strong correlation between an increase in flatness index and an increase in PD in the dense-graded material
- PD tests conducted at a light pavement structure stress ($\sigma_3 = 41.4$ kPa, $\sigma_d = 206.8$) showed a weak correlation between an increase in flatness index and an increase in PD in the open-graded material, since the degree of interlocking of PS also played a major role in the PD accumulation
- Similar trends were observed between the elastic response of the PD tests and M_R tests for both the dense- and open-graded materials

6: PRACTICAL IMPLICATIONS

In order to better understand if the observed variation in performance due to particle shape would affect the design of road structures, the MEPDG software AASHTOWare Pavement ME Design Version 2.0 was used to design a three layer flexible pavement structure. The AASHTOWare Pavement ME Design software estimates performance of the pavement structure over a given design life; the specifics of the MEPDG design process were discussed in section 2.2. The inputs needed for the design by this process involves traffic characteristics, materials/foundation characteristics, climate data, and the performance/reliability criteria used to evaluate the performance of the pavement structure.

The effect of particle shape on the performance of a road structure was investigated by the process outlined in **Figure 56**. The pavement model inputs were first defined (which will be defined in the next section), then the amount of traffic that the pavement structure could withstand and still pass the performance/reliability criteria was noted. This allowed for the pavement structure to be compared based on the amount of “load” the structures could withstand and still perform satisfactorily. The initial design period was 20 years based on recommendations by Virginia Department of Transportation’s (VOT) Guidelines for the 1993 AASHTO Pavement Design (2003). Only the estimated M_R results for the base layer could be directly input into

AASHTOWare Pavement ME Design software from the laboratory test conducted in this study, so therefore the permanent deformation testing was not used for evaluation of pavement performance.

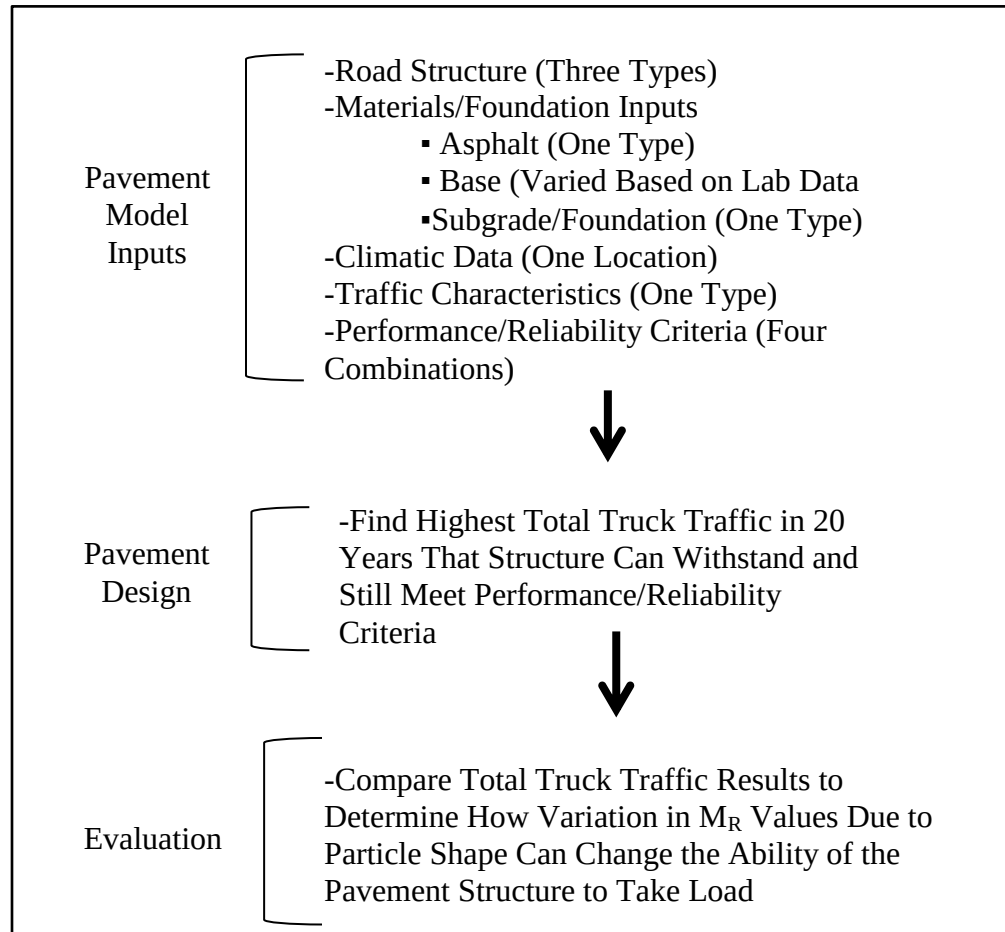


Figure 56. Overview of AASHTOWare Pavement ME Design Process Used to Investigate Effect of UAB Particle Shape on Performance of a Pavement Structure

6.1 Design Inputs for MEPDG Design Examples

The model inputs used in the AASHTOWare Pavement ME Design software are defined below. English units are displayed in this section since the inputs in the AASHTOWare Pavement ME Design software inputs were in English units.

Road Structure

Three road structure profiles were investigated based on typical pavement profiles given by Barksdale and Itani (1989). The three types of road structures represented were light, medium, and heavy pavement structures as seen in **Table 22**. All roads investigated had one lane in each direction.

Table 22, Light, Medium, and Heavy Road Structures
(Barksdale & Itani, 1989)

	Road Structure		
	Light	Medium	Heavy
Asphalt (in.)	2	4	6
Base (in.)	10	10	18

Traffic Inputs

The AASHTOWare Pavement ME Design software has varying default truck distributions. The TTC 12 truck distribution was selected since the distribution was applicable to a wide range of road types (principal arterials to local routes). The TTC 12 truck distribution has a large percentage of buses (40%) and five axel trucks (25%). A 2%

annual growth rate for all vehicle types and a vehicle operation speed of 45 mph was selected based on values used by Schwartz and Carvalho (2007).

Material Inputs

The asphalt concrete materials properties that were input into the MEPDG design process is presented in **Table 23**. The binder grade and material properties were those used by Schwartz and Carvalho (2007) to represent a Maryland asphalt which in this study was assumed to be similar to those used in Virginia. The subgrade properties used for modeling are shown in **Table 24**. The soil properties are based on default value given in the MEPDG software for AASHTO A-7-6 soil and the M_R value was selected based on recommendations given in Mechanistic-Empirical Pavement Design Guide (AASHTO, 2008) for that soil type. Additionally, the use of a competent subgrade (fairly strong M_R value of 11,500 psi) was selected since the pavement structure used in the particle implications assumed a competent subgrade. Larger pavement structure would need to be used if a weak subgrade was encountered in the field to reduce stress on the subgrade to acceptable levels.

Table 23. Asphalt Concrete Properties (Schwartz & Carvalho, 2007)

General Properties	
Reference Temperature (°F)	70
Poisson's Ratio	0.35
Volumetrics	
Effective Binder Content (%)	9
Air Voids (%)	6.2
Total Unit (pcf)	148
Gradation	
% Passing ¾ inch sieve	96
% Passing 3/8 inch sieve	69
% Passing No. 4 sieve	13
% Passing No. 200 sieve	6
Thermal Properties	
Thermal Conductivity asphalt (BTU/hr-ft-°F)	0.67
Heat Capacity asphalt (BTU/lb-°F)	0.23
Binder Grade	PG 70-22

Table 24. Subgrade Properties

AASHTO Classification of soil	A-7-6
Strength Properties	
M _R (psi)	11,500
Poisson's Ratio	0.35
Coeff. of lateral pressure (K _o)	0.5
Gradation and Plasticity Index	
Plasticity Index	40
% Passing No. 4	99
% Passing No. 200	90
Calculated/Derived Parameters (level 3)	
Max dry unit weight (pcf)	91.6
Specific gravity of soils, G _s	2.7
Saturated hydraulic conductivity (ft/hr)	4.498e-06
Optimum gravimetric water content (%)	25.3

The base material inputs used were selected based on two observations in the M_R testing. The first observation was that there was a peak M_R value at a flatness index value of 0.6 for the dense-graded materials specific to their mineralogy. The second observation was that the open-graded M_R results showed a continuous increase in M_R value with increasing flatness index. These two observations lead to eight dense-graded materials being selected. The highest and lowest M_R value for each aggregate type were used in the dense-graded material to evaluate if the variation in M_R observed, due to particle shape in the dense-graded samples, could cause variation in pavement performance. Additionally, two open-graded materials were selected at the high and low M_R values since there is a marked increase in M_R with increasing flatness index regardless of mineralogy.

The interpreted M_R (those estimated from the MEPDG model in section 5.3.1) could be directly used since the stresses used to estimate the light, medium, and heavy pavement structures are the same structure as stated in **Table 9**. The base samples and associated M_R values used in the design runs are present in **Table 25**. The rest of the base properties (such as density and GSD) were input based on the measured value of that sample during laboratory testing. If the value was not measured (such as the saturated hydraulic conductivity) the software default values were used.

Table 25. Base M_R Values Used in Each Pavement Structure

Sample Name	Pavement Structure [M_R (psi)]		
	Light	Med.	Heavy
DD100%NonE#2	29179	25315	16641
DD100%F↑3:1	23945	20293	12873
DH100%NonE	26349	23488	16122
DH100%F↑3:1	24531	20932	13646
Dd100%E	32942	27889	17761
Dd100%NonE	40376	35732	23896
DS100%F↑3:1↓5:1	19405	18708	14558
DS100%F↑5:1	17024	16595	12638
Od100%E	32288	32262	24476
OS100%F↑5:1	17811	17304	13598

Climate Model

A D.C. climate model was used, specifically the Ronald Reagan Washington National Airport climate model.

Performance/Reliability Criteria

Two performance criterion given by the Mechanistic-Empirical Pavement Design Guide (2008) for primary and secondary roads were used as the limiting damage criteria, **Table 26**. The reason for selection of IRI, bottom up AC cracking, and total permanent deformation for the performance criteria is discussed in section 2.2. Additionally, two levels of reliability recommended by the Mechanistic-Empirical Pavement Design Guide (2008) were used for each performance criteria, **Table 27**.

Table 26. Performance Criteria for All Pavement Structures Investigated

Road Type	IRI (in./mi)	Bottom Up AC Cracking (% lane area)	Total Permanent Deformation (in.)
Primary	200	20	0.5
Secondary	200	35	0.65

Table 27. Levels of Reliability Used in Trial Designs

Functional Classification	Level of Reliability (%)
Principal Arterials	90
Local	75

6.2 Results of the Variation in Resilient Modulus on Pavement Structure Performance

The results of the analysis are present in **Figure 57** and **Figure 58**. The total truck traffic is the amount of traffic (both lanes) that the AASHTOWare Pavement ME Design software estimated the structure could withstand and still fulfill the performance/reliability requirements. The performance criteria that reached the limit first in all pavement analysis performed was the total PD.

When looking at the variation in traffic caused by the performance/reliability criterion in **Figure 57** and **Figure 58**, it is shown that the primary road performance criterion at 90% reliability has the lowest estimated total truck traffic for all pavement types and the secondary road performance criterion at 75% has the highest estimated total truck traffic. This is logical given that the limiting performance criteria gets higher and the reliability gets low between the primary performance at 90% reliability and the secondary performance at 75% reliability.

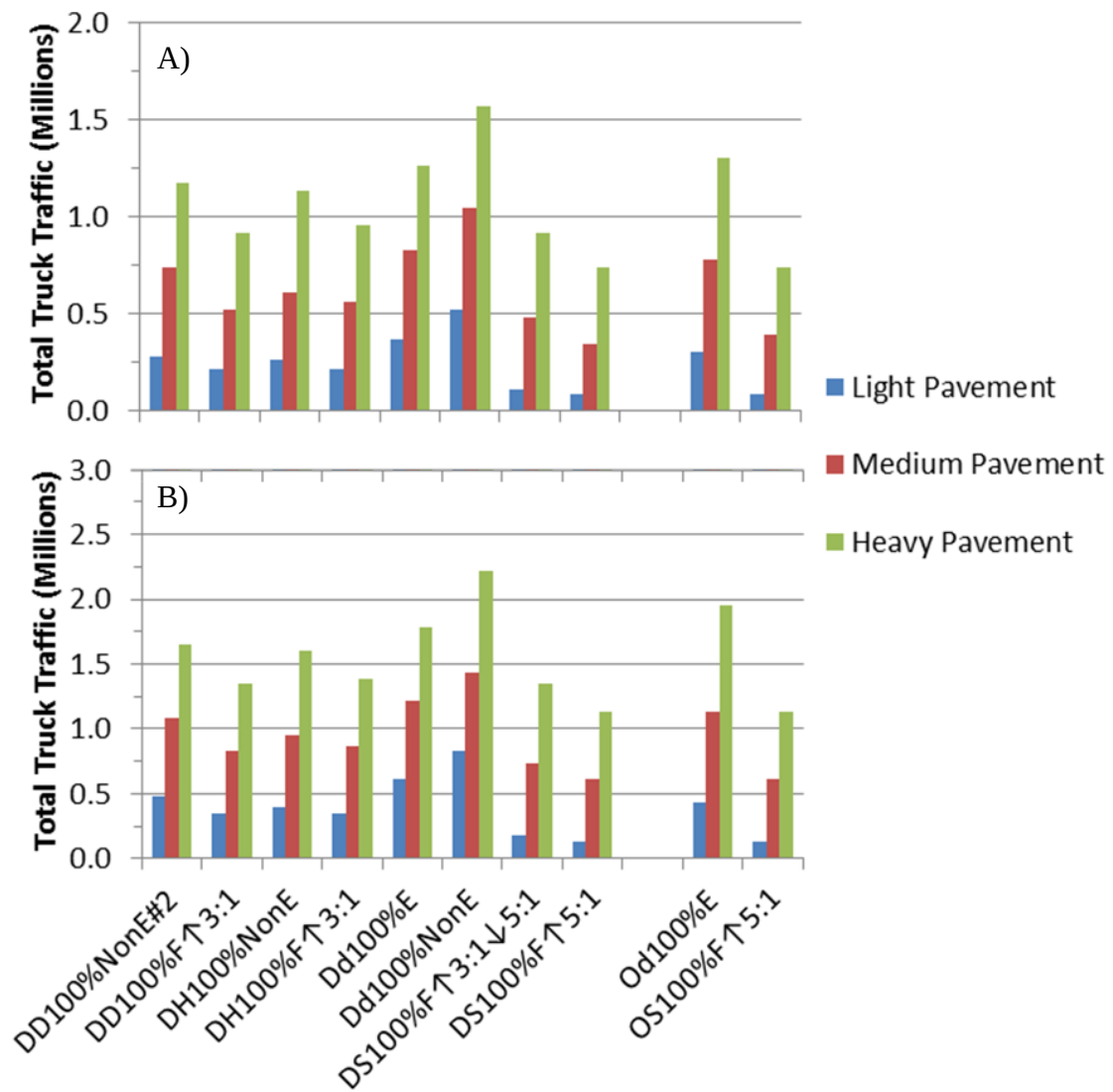


Figure 57. Primary Performance Criteria; A) 90% Reliability and B) 75% Reliability

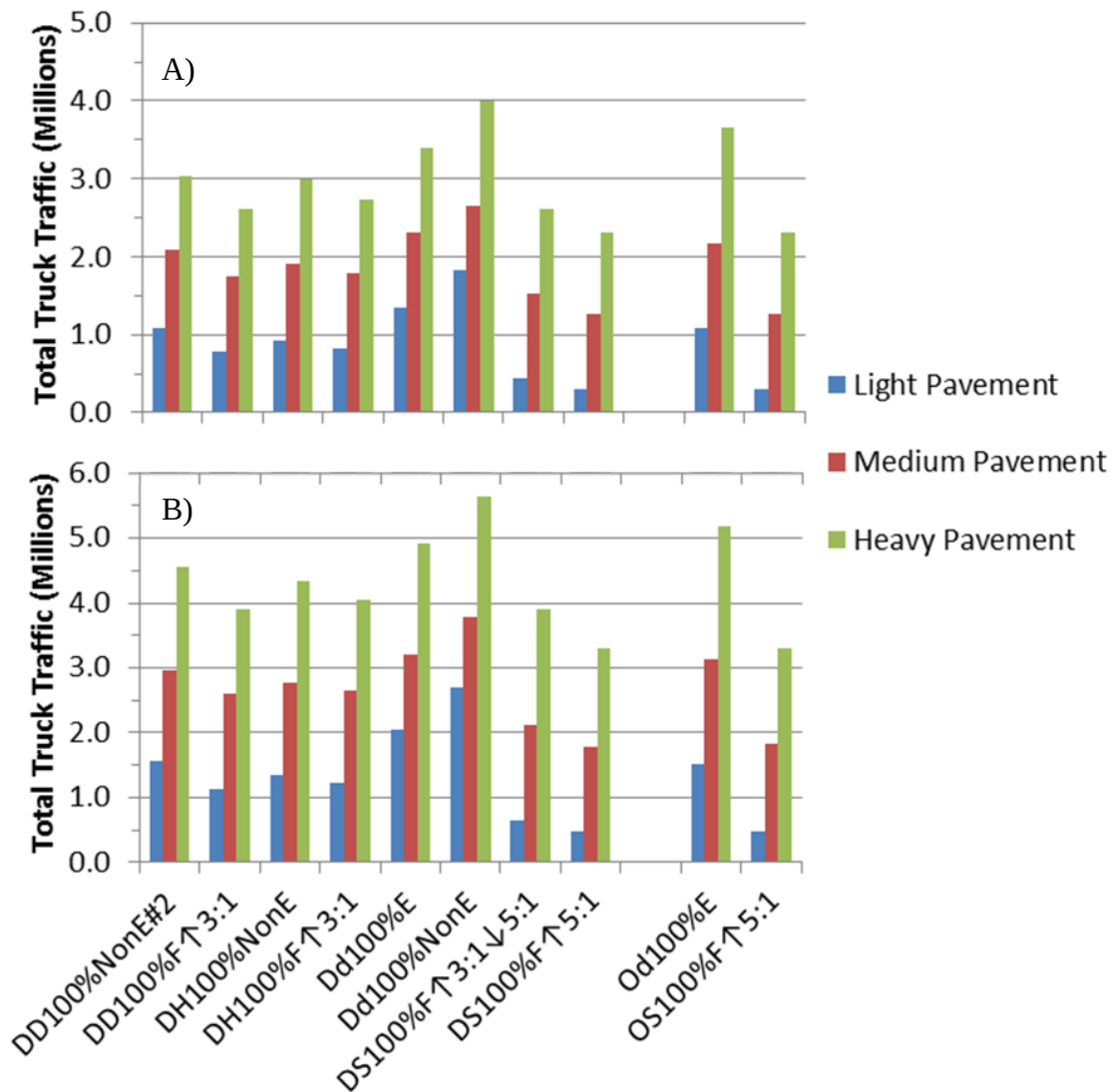


Figure 58. Secondary Performance Criteria; A) 90% Reliability and B) 75% Reliability

Focusing on the two open-graded materials, the COV (one standard deviation) was used to compare the change in performance as the particles become flatter (Od100%E to OS100%F↑5:1) in all three structures and all four performance criteria, **Table 28**. The COV of decreased as the pavement thickness increased showing that the importance of particle shape decreased with increasing pavement structure. The COV

across performance criteria/reliability combinations stayed roughly the same, which shows that the reduction in pavement performance stayed roughly the same at all performance criteria combinations. The open-graded materials showed that if particle shape varied across the whole flatness index that was studied in this thesis there could be a significant decrease in load carry capacity, most pronounced with the light pavement structure.

Table 28. COV between OD100%E and OS100%F $\uparrow$ 5:1 Between All Pavement Types and Performance Criteria/Reliability

Pavement Type	Performance Criteria			
	Primary		Secondary	
	90%	75%	90%	75%
Light	0.56	0.54	0.56	0.52
Medium	0.33	0.30	0.27	0.26
Heavy	0.28	0.27	0.23	0.22

When looking at the performance of the dense-graded gradations in **Figure 57** and **Figure 58**, it is shown through all pavement types and performance criteria that the dolostone samples showed the highest total truck traffic, the slate materials show the lowest total truck traffic, and the diabase/hornfels samples fall in between. These observations are consistent with the fact that this was how the M_R distributions were observed in **Figure 43**. However, the research question yet to be answered was whether the observed variation in particle shape within the same mineralogy would cause a decrease in load carry capability of the overall pavement structure.

In order to understand the variability of pavement performance due to specifically particle shape, the COV was first compared between all dense-graded pavement performance and the durable dense-graded pavement performance, **Table 29**. This shows that when the slate material (that had weak SS structure resulting in lower M_R values) is excluded from the COV calculation for all pavement types the variations between pavement performance decreases, showing that the slate materials was an outlier in the dense-graded materials. The next comparison was made in individual aggregate types and the COV due to particle shape within the same mineralogy; **Figure 57**, **Figure 58**, and **Table 30**. There was a peak strength (highest total truck traffic) in the pavement structures at the non-equidimensional particle shape for the dolostone, diabase, and hornfels dense-graded base pavement structures, but **Table 30** shows that the COV within the same mineralogy because of the variation in particle shape is very low at the secondary performance criteria for both medium and heavy pavement structures. This indicates that most of the performance variation in the dense-graded base pavement structures was due to the variation in M_R value cause by the SS.

Table 29. COV Between All Dense-Graded Gradation and Durable Dense-Graded Aggregates (Excluding Slate)

Performance Criteria Pavement Type	All Dense-Graded				Durable Dense-Graded			
	Primary		Secondary		Primary		Secondary	
	90%	75%	90%	75%	90%	75%	90%	75%
Light	0.51	0.51	0.49	0.48	0.34	0.34	0.32	0.32
Medium	0.32	0.26	0.22	0.21	0.25	0.2	0.15	0.13
Heavy	0.22	0.20	0.17	0.16	0.18	0.17	0.15	0.13

Table 30. COV Between All Dense-Graded Gradation Individual Minerologies

Performance Criteria Pavement Type	Dolostone				Diabase				Hornfels				Slate			
	Primary		Secondary		Primary		Secondary		Primary		Secondary		Primary		Secondary	
	90%	75%	90%	75%	90%	75%	90%	75%	90%	75%	90%	75%	90%	75%	90%	75%
Light	0.18	0.22	0.23	0.23	0.24	0.21	0.21	0.19	0.13	0.08	0.07	0.07	0.16	0.20	0.25	0.22
Medium	0.24	0.19	0.13	0.09	0.16	0.12	0.10	0.11	0.05	0.07	0.05	0.03	0.22	0.14	0.13	0.13
Heavy	0.18	0.14	0.11	0.11	0.15	0.15	0.12	0.10	0.12	0.10	0.06	0.05	0.15	0.12	0.09	0.12

6.4 Summary of Practical Implications

The predicted performances of the three pavement structures by the AASHTOWare Pavement ME Design software have shown that the strength of the base material (the M_R value) directly affects the pavement structure's ability to support traffic. The dense-graded materials showed major variation in total truck traffic values with varying mineralogies (dolostone when compared to the slate sample), but only showed minor variation, COV averaging a low 0.14 across all pavement structures and performance/reliability criteria, because of the variation in particle shape in the same mineralogy. The open-graded materials showed a major variation in pavement performance in the light pavement structures (average COV equal to 0.55), but close to half the effect on pavement performance for the medium and heavy pavement structures (average COV equal to 0.29 and 0.25 respectively). These results show that the variation of M_R value because of the increase in flat particles for the open-graded materials would be greatest at a pavement structure with a thin asphalt layer (2 to 4 inches). The use of thin asphalt layers with open-graded materials is uncommon in practice, but has been investigated in recent years by the University of Oklahoma (Khoury, Zaman, Ghabshi, & Kazmee, 2010).

7: CONCLUSIONS

This thesis was performed to evaluate the gaps in the literature as previously outlined and to develop guidelines as it relates to the effect of particle shape to the performance of UAB. In order to accomplish this goal, an extensive laboratory study was undertaken to evaluate the effect of particle shape within the same mineralogy and across varying mineralogies. The four aggregates evaluated in the study consisted of three durable aggregates and one non-durable aggregate that were mixed in dense- and open-graded gradations in order to allow for an investigation over a range of aggregate types and gradations. This type of investigation has not been performed before with this level of emphasis on particle shape. Therefore the results of this study would provide experiential data to inform and support state guidelines.

The mineralogy of an aggregate material had a sustainably more pronounced effect on the performance of material with regard to durability (MD test) than particle shape. The Bailey method allowed for effective investigation into the structure of the UAB matrix and allowed for defining of the load carrying structure and level of particle-to-particle contact in the PS. In this study, the effect of particle shape with regard to M_R was found to be the most pronounced in the open-graded gradations and resulted in a marked decrease in the flexible pavement performance for the light pavement structure, but a less pronounced effect on medium and light pavement structures. The dense-graded

materials on the other hand, showed that the mineralogy of the SS has a notably larger effect on M_R value than the particle shape of the PS. Furthermore, the effect of particle shape on the PD behavior in both the dense- and open-graded materials was minor at the medium pavement stresses. This result indicates that particle shape is not important as long as the stress on the base layer is not high enough to cause the granular material to behave in an unstable manner with regard to PD.

Overall, the only observed scenario where limiting of particle shape in a base layer would sustainably benefit the performance of a pavement structure would be if an open-graded material was used in conjunction with a thin flexible pavement surface. This pavement structure was investigated in the practical implications section and the pavement structures with the flatter coarse particles showed reduced ability to take load. However, it should be noted that the use of a thin flexible pavement surface with open-graded materials is very uncommon in practice.

Therefore, based on the data collected in this extensive study, it is recommended that state guidelines be amended to no restriction in terms of particle shape of UAB. However, this is based on the current practice of not combining unbound, open-graded base material with thin asphalt pavement surfaces. If this practice changes then these guidelines should be reevaluated. Removing these state guidelines would not change many transportation agency practices because these state guidelines do not exclude many possible aggregate types. Nevertheless, removing particle shape restrictions could allow for previously excluded materials to be used in base applications, which could have potential cost savings.

7.1 Limitations

As in all study, there were several limitations on this study, which are outlined below:

- Only four aggregates were used in this study, therefore the effect of mineralogy was not fully investigated.
- All samples investigated were 100% crushed aggregate. Consequently, the effect of particle shape when angularity is reduced was not explored.
- The PD testing protocol only involved two stress conditions, which limit the universality of observed PD behavior.
- Open-graded aggregate materials experienced the same confining pressure during laboratory testing as dense-graded material, which may not be true in the field.
- The effect of elongated particle shape was not investigated and therefore not captured by the particle ranges created in this study.
- Practical implications were evaluated based on a single subgrade condition, which simulated a moderately stiff ground condition. The findings may differ if the analyses were also performed with very soft ground conditions.
- Practical implications were only investigated using a flexible pavement structure.

7.2 Suggested Future Studies

The effect of particle shape on permeability was not investigated, which could have a marked effect on the overall performance of a pavement structure, therefore permeability could be investigated with varying particle shapes of similar and varying mineralogies. This may be more important for dense-graded aggregates than the open-graded aggregates (which are considered freely draining material). There were marked differences in both dense- and open-graded permanent deformations tests under high stresses; for that reason there could be observed variations in shear strength caused by particle shape, which was not investigated in this study. The Bailey method for characterization of aggregate matrix structures allowed for proper characterization of the two aggregate gradations in this study, hence this method could be explored and validated in order that it would be more generally applied to classifying unbound aggregate behavior with regard to specifically permanent deformation behavior. Open-graded materials are commonly used for rigid pavement base layers; investigation into the effect of changing particle shape on the performance of a rigid pavement structure could be investigated.

APPENDIX A: PARTICLE SHAPE ANALYSES

			ASTM D4791 (% by weight)										
			Particle Shape Analysis					Method A			Method B		
Mineralogy	Test	Sample Name	Elongated Index	Flatness Index	Flatness			Elongated			Flat and Elongated		
					2:1	3:1	5:1	2:1	3:1	5:1	2:1	3:1	5:1
Diabase	Compaction	DD100%E	0.763	0.756	2	0	0	0	0	0	31	1	0
		OD100%E	0.755	0.664	16	0	0	0	0	0	48	1	0
	MR and PD	DD100%E	0.737	0.722	2	0	0	0	0	0	39	0	0
		OD100%E	0.725	0.727	7	0	0	2	0	0	37	0	0
	Micro-Deval	D100%E	0.729	0.693	6	0	0	3	0	0	58	1	0
	Compaction	DD100%NonE	0.650	0.540	39	0	0	24	0	0	88	51	0
		OD100%NonE	0.621	0.590	25	0	0	20	0	0	93	40	0
	MR and PD	DD100%NonE	0.655	0.561	30	0	0	20	0	0	86	34	1
		OD100%NonE	0.638	0.517	50	0	0	17	0	0	91	48	1
	Micro-Deval	D100%NonE	0.638	0.502	54	1	0	20	0	0	96	55	5

Mineralogy	Test	Sample Name	ASTM D4791 (% by weight)										
			Particle Shape Analysis		Method A						Method B		
			Elongated Index	Flatness Index	Flatness			Elongated			Flat and Elongated		
					2:1	3:1	5:1	2:1	3:1	5:1	2:1	3:1	5:1
Diabase	Compaction	DD40%F↑3:1	0.722	0.363	66	38	2	8	0	0	97	60	18
		OD40%F↑3:1	0.662	0.411	63	35	2	17	0	0	98	72	14
	MR and PD	DD40%F↑3:1	0.681	0.399	64	34	0	9	2	0	89	65	10
		OD40%F↑3:1	0.692	0.368	63	37	0	8	1	0	91	63	9
	Micro-Deval	D40%F↑3:1	0.687	0.363	62	38	3	16	0	0	92	69	19
	Compaction	DD100%F↑3:1	0.708	0.304	100	92	11	8	0	0	100	100	53
		OD100%F↑3:1	0.73	0.292	100	84	4	1	0	0	100	100	29
	MR and PD	DD100%F↑3:1	0.725	0.311	98	75	0	6	0	0	100	99	25
		OD100%F↑3:1	0.702	0.292	100	87	3	4	0	0	100	100	38
	Micro-Deval	D100%F↑3:1	0.739	0.291	99	85	1	3	0	0	100	99	33

Mineralogy	Test	Sample Name	ASTM D4791 (% by weight)										
			Particle Shape Analysis					Method A			Method B		
			Elongated Index	Flatness Index	Flatness			Elongated			Flat and Elongated		
					2:1	3:1	5:1	2:1	3:1	5:1	2:1	3:1	5:1
Hornfels	Compaction	DH100%E	0.750	0.721	3	1	0	1	0	0	37	1	0
		OH100%E	0.704	0.704	5	0	0	6	0	0	53	1	0
	MR and PD	DH100%E	0.762	0.686	4	0	0	1	0	0	42	1	0
		OH100%E	0.745	0.698	4	0	0	1	0	0	56	0	0
	Micro-Deval	H100%E	0.742	0.698	7	0	0	0	0	0	44	1	0
	Compaction	DH100%NonE	0.606	0.528	38	3	0	36	0	0	96	59	3
		OH100%NonE	0.576	0.509	42	3	0	27	1	0	98	61	1
	MR and PD	DH100%NonE	0.642	0.534	46	0	0	19	0	0	93	52	0
		OH100%NonE	0.632	0.543	35	2	0	21	0	0	88	38	2
	Micro-Deval	H100%NonE	0.651	0.547	30	0	0	22	0	0	88	28	0

Mineralogy	Test	Sample Name	ASTM D4791 (% by weight)										
			Particle Shape Analysis		Method A						Method B		
			Elongated Index	Flatness Index	Flatness			Elongated			Flat and Elongated		
					2:1	3:1	5:1	2:1	3:1	5:1	2:1	3:1	5:1
Hornfels	Compaction	DH40%F↑3:1	0.668	0.371	69	36	1	20	0	0	98	77	16
		OH40%F↑3:1	0.660	0.385	64	42	9	17	0	0	94	72	25
	MR and PD	DH40%F↑3:1	0.650	0.398	50	33	1	19	0	0	96	65	17
		OH40%F↑3:1	0.645	0.340	67	37	3	18	0	0	95	67	23
	Micro-Deval	H40%F↑3:1	0.685	0.369	57	35	4	22	0	0	90	64	22
	Compaction	DH100%F↑3:1	0.677	0.277	100	82	5	5	0	0	100	100	49
		OH100%F↑3:1	0.676	0.270	100	93	14	8	0	0	100	100	57
	MR and PD	DH100%F↑3:1	0.699	0.277	100	89	3	12	0	0	100	100	48
		OH100%F↑3:1	0.685	0.274	100	92	8	8	0	0	100	100	44
	Micro-Deval	H100%F↑3:1	0.675	0.278	99	92	6	8	0	0	100	100	49

Mineralogy	Test	Sample Name	ASTM D4791 (% by weight)										
			Particle Shape Analysis					Method A			Method B		
			Elongated Index	Flatness Index	Flatness			Elongated			Flat and Elongated		
					2:1	3:1	5:1	2:1	3:1	5:1	2:1	3:1	5:1
Dolostone	Compaction	Dd100%E	0.775	0.771	1	0	0	1	0	0	25	0	0
		Od100%E	0.783	0.721	3	0	0	0	0	0	30	0	0
	MR and PD	Dd100%E	0.802	0.735	2	0	0	0	0	0	21	2	0
		Od100%E	0.768	0.738	8	0	0	0	0	0	27	0	0
	Micro-Deval	d100%E	0.774	0.726	6	0	0	0	0	0	27	1	1
	Compaction	Dd100%NonE	0.664	0.560	19	4	1	14	0	0	77	21	2
		Od100%NonE	0.677	0.536	32	4	0	8	0	0	92	25	1
	MR and PD	Dd100%NonE	0.685	0.594	22	0	0	11	0	0	81	14	0
		Od100%NonE	0.687	0.563	25	2	0	4	0	0	86	16	1
	Micro-Deval	d100%NonE	0.690	0.513	42	5	0	11	0	0	89	32	2

					ASTM D4791 (% by weight)										
					Method A						Method B				
					Particle Shape Analysis		Flatness			Elongated			Flat and Elongated		
Mineralogy	Test	Sample Name	Elongated Index	Flatness Index	2:1	3:1	5:1	2:1	3:1	5:1	2:1	3:1	5:1		
Slate	Compaction	DS100%F↑3:1↓5:1	0.645	0.271	100	94	0	22	1	0	100	100	75		
		OS100%F↑3:1↓5:1	0.595	0.252	100	98	6	36	0	0	100	100	84		
	MR and PD	DS100%F↑3:1↓5:1	0.640	0.265	100	96	0	18	1	0	100	100	83		
		OS100%F↑3:1↓5:1	0.648	0.262	100	99	3	20	0	0	100	100	66		
	Micro-Deval	S100%Flat↑3:1↓5:1	0.628	0.270	100	98	1	27	2	0	100	100	74		
	Compaction	DS100%F↑5:1	0.639	0.150	100	100	94	18	0	0	100	100	100		
		OS100%F↑5:1	0.667	0.154	100	100	86	14	0	0	100	100	100		
	MR and PD	DS100%F↑5:1	0.658	0.154	100	100	95	22	0	0	100	100	100		
		OS100%F↑5:1	0.678	0.154	100	100	94	9	0	0	100	100	99		
	Micro-Deval	S100%Flat ↑5:1	0.677	0.161	100	100	81	9	1	0	100	100	100		

APPENDIX B: RESILIENT MODULUS TEST RESULTS

Evaluation of M_R Regression Analysis to Measured M_R Results

The MEPDG model was chosen as the M_R model since it best represented the measure M_R values during laboratory testing. To illustrate this five selected M_R tests were selected and regression analysis was performed for the three models discussed in section 2.1.1. The laboratory results and the associated model R^2 values are presented below.

DD100E (11/6/2014)

Summary Resilient Modulus Regression Values							
K-θ Model		Uzan Model			MEPDG Model		
$r^2 =$	0.9753	$r^2 =$		0.9881	$r^2 =$		0.9806
K (psi)	n	k1	k2	k3	k1	k2	k3
2048.1155	0.6875	757.7054	0.8256	-0.1497	918.9235	0.7600	-0.2792

DH100%F↑3:1(12/08/2014)

Summary Resilient Modulus Regression Values							
K-θ Model		Uzan Model			MEPDG Model		
$r^2 =$	0.9821	$r^2 =$		0.9890	$r^2 =$		0.9857
K (psi)	n	k1	k2	k3	k1	k2	k3
1984.2358	0.6631	719.9147	0.7577	-0.1033	826.8037	0.7199	-0.2223

DS100F↑5:1(1/15/2015)

Summary Resilient Modulus Regression Values							
K-θ Model		Uzan Model			MEPDG Model		
$r^2 =$	0.8693	$r^2 =$		0.9704	$r^2 =$		0.9739
K (psi)	n	k1	k2	k3	k1	k2	k3
2058.9904	0.6087	507.6669	0.9257	-0.4378	828.1622	0.8690	-1.0282

OH100%E (2/13/2015)

Summary Resilient Modulus Regression Values							
K-θ Model		Uzan Model			MEPDG Model		
$r^2 =$	0.7211	$r^2 =$		0.8334	$r^2 =$		0.7435
K (psi)	n	k1	k2	k3	k1	k2	k3
3931.1689	0.5376	749.1805	0.8947	-0.3873	1217.6820	0.6688	-0.5209

OS100%F↑3:1↓5:1 (2/19/2015)

Summary Resilient Modulus Regression Values							
K-θ Model		Uzan Model			MEPDG Model		
$r^2 =$	0.7021	$r^2 =$		0.7121	$r^2 =$		0.8226
K (psi)	n	k1	k2	k3	k1	k2	k3
5022.4950	0.4331	688.9001	0.8346	-0.4295	1116.8331	0.8205	-1.0412

The results shown above represent general results for the dense-graded materials showing that all three model regression analysis had high R^2 values. For the open-graded materials, the model regression analysis showed that the highest R^2 for the three model alternated between the Uzan and MEPDG model. Based on these general trends, the MEPDG model was selected to best generally represent the laboratory measured data.

Example of Calculation of Estimated M_R values from Table 9. Suggested Stress States for Laboratory Permanent Deformation (Barksdale & Itani, 1989)

Table 9. Suggested Stress States for Laboratory Permanent Deformation (Barksdale & Itani, 1989)

Pavement Structure	Stress State		
	σ_3 (kPa)	σ_3 (psi)	σ_1 / σ_3
Light	41.4	6	6
Medium	41.4	6	4
Heavy	31.0	4.5	2

The confining pressure and three ratios between sigma 1 and sigma 3 are given for all pavement structures by Barksdale and Itani (1989). With this information the bulk stress and octahedral shear stress can be calculated, which is show in the table below for all three pavement structures.

Pavement Structure	σ_3 (psi)	σ_1 (psi)	θ (psi)	τ_{oct} (psi)
Light	6	36	48	14.14
Medium	6	24	36	8.49
Heavy	4.5	9	18	2.12

Then, this information can be input into the MEPDG models with the associated k parameters for that model. For example, the DD100%E sample ($k_1 = 918.9$, $k_2 = 0.760$, and $k_3 = -0.279$) M_R value is calculated for the light pavement stress using the MEPDG model below.

$$M_R = 918.9 * 14.696 * (48/14.696)^{0.760} * ((14.14/14.696) + 1)^{-0.279} = 27504.87 = \underline{27505 \text{ psi}}$$

This process was followed for each interpreted M_R calculated from the MEPDG.

Dense-Graded M_R

Sample Name: DD100%E

Date Tested:11/06/2014

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
2.980	3	3.128	0.0154	12.07	0.2	0	13420.0	280.930
3.058	6	6.325	0.1033	15.5	0.05	0	12317.0	297.260
3.046	9	9.527	0.0354	18.67	0.06	0	14179.5	59.025
5.003	5	5.129	0.0172	20.14	0.03	0	15911.0	93.297
5.039	10	10.530	0.0384	25.64	0.05	0	18880.5	157.640
4.969	15	15.670	0.0404	30.58	0.08	0	19646.5	66.080
9.982	10	9.956	0.0288	39.9	0.03	0	28595.0	117.450
9.972	20	20.290	0.0443	50.21	0.06	0	29160.0	171.640
9.958	30	30.550	0.1869	60.42	0.09	0	32157.5	105.150
14.990	10	10.030	0.0080	55.01	0.03	0	34045.0	187.950
14.990	15	14.960	0.0228	59.94	0.04	0	34898.5	206.640
14.970	30	29.950	0.0288	74.87	0.07	0	39267.0	210.400
18.870	15	14.690	0.0406	71.29	0.04	0	39864.0	398.500
18.690	20	19.700	0.0141	75.78	0.05	0	40095.0	131.080
18.520	40	39.230	0.0389	94.78	0.07	0	47709.0	271.350

Sample Name: DD100%NonE

Date Tested:11/06/2014

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.085	3	3.151	0.0218	12.41	0.03	0	11487.5	107.150
3.101	6	6.304	0.0137	15.61	0.04	0	13084.0	91.500
3.090	9	9.402	0.0395	18.67	0.06	0	15353.0	113.570
5.103	5	5.099	0.0288	20.41	0.03	0	17256.0	79.079
5.098	10	10.400	0.0347	25.69	0.05	0	19230.5	68.589
5.028	15	15.660	0.0288	30.74	0.06	0	22105.0	76.714
10.030	10	10.110	0.0326	40.21	0.03	0	27398.0	191.160
10.040	20	20.300	0.0731	50.42	0.06	0	30521.5	249.280
10.020	30	30.430	0.0426	60.49	0.08	0	33770.5	69.086
15.010	10	10.060	0.0136	55.09	0.03	0	35760.5	260.130
15.010	15	15.140	0.0110	60.17	0.04	0	34219.0	161.090
15.000	30	30.020	0.0328	75.03	0.06	0	42095.5	131.670
19.970	15	14.210	0.0154	74.11	0.03	0	47181.0	642.420
20.000	20	19.200	0.0312	79.21	0.03	0	49476.0	166.600
19.990	40	37.820	0.0528	97.78	0.07	0	51967.5	234.580

Sample Name: DD100%NonE#2

Date Tested:11/07/2014

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.088	3	3.095	0.0070	12.36	0.02	0	15540.5	141.810
2.995	6	6.322	0.0446	15.31	0.04	0	14022.0	141.650
3.081	9	9.516	0.0034	18.76	0.06	0	15320.0	112.630
4.971	5	5.258	0.0259	20.17	0.04	0	15497.5	102.660
4.998	10	10.470	0.0208	25.47	0.05	0	19594.0	64.008
5.057	15	15.630	0.0413	30.81	0.07	0	22256.0	194.280
10.050	10	10.050	0.0319	40.2	0.03	0	29928.0	256.830
9.983	20	20.300	0.0503	50.25	0.06	0	31613.5	33.603
10.020	30	30.360	0.0492	60.43	0.08	0	35389.5	155.530
14.950	10	9.584	0.0132	54.44	0.02	0	42645.0	351.590
14.980	15	14.870	0.0324	59.79	0.04	0	37774.5	149.400
14.970	30	30.000	0.0376	74.91	0.07	0	42396.0	266.830
20.020	15	14.720	0.0188	74.78	0.03	0	44337.0	291.060
19.990	20	19.920	0.0370	79.87	0.05	0	44257.0	223.440
19.970	40	39.460	0.0484	99.38	0.07	0	51174.0	238.910

Sample Name: DD40%F↑3:1

Date Tested:11/07/2014

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.008	3	3.179	0.0045	12.2	0.03	0	11269.5	116.600
3.102	6	6.314	0.0377	15.62	0.05	0	12514.0	61.000
2.984	9	9.489	0.0257	18.44	0.07	0	13501.5	242.120
5.097	5	5.059	0.0157	20.35	0.02	0	18455.0	191.250
5.033	10	10.460	0.0241	25.56	0.06	0	17360.5	80.647
4.978	15	15.800	0.0457	30.74	0.08	0	18371.5	79.359
10.070	10	10.060	0.0494	40.27	0.04	0	26929.0	190.340
10.010	20	20.460	0.0618	50.49	0.07	0	30151.0	28.708
10.070	30	30.760	0.0463	60.98	0.1	0	30892.0	104.420
15.080	10	9.812	0.0127	55.04	0.03	0	34533.5	379.030
15.020	15	15.120	0.0251	60.16	0.05	0	32828.5	167.350
15.070	30	30.380	0.0493	75.58	0.08	0	38242.0	137.260
20.030	15	14.670	0.0058	74.75	0.04	0	38747.5	212.120
19.970	20	19.790	0.0489	79.71	0.04	0	43236.5	352.990
20.050	40	39.140	0.0474	99.29	0.07	0	47972.0	212.600

Sample Name: DD100%F↑3:1

Date Tested:11/05/2014

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.117	3	3.013	0.0280	12.36	0.03	0	10687.3	208.440
3.078	6	6.334	0.0098	15.57	0.05	0	10367.5	39.811
3.035	9	9.612	0.0322	18.72	0.08	0	12240.0	39.269
5.081	5	5.007	0.0151	20.25	0.03	0	16479.5	55.454
5.047	10	10.610	0.0424	25.75	0.07	0	15785.0	74.703
5.079	15	15.830	0.0205	31.07	0.08	0	16764.0	59.269
10.020	10	10.260	0.0501	40.32	0.04	0	22556.0	63.189
10.030	20	20.710	0.0543	50.81	0.08	0	25750.0	113.270
10.040	30	31.060	0.0497	61.19	0.1	0	28471.0	68.677
15.020	10	10.270	0.0373	55.32	0.04	0	28394.0	206.640
15.030	15	15.230	0.0094	60.32	0.05	0	29466.5	104.860
15.050	30	30.360	0.0485	75.5	0.08	0	36225.5	48.105
19.990	15	14.990	0.0094	74.95	0.04	0	37593.5	130.320
20.020	20	19.660	0.0119	79.73	0.05	0	38276.5	110.480
20.010	40	38.960	0.0404	99	0.08	0	42689.0	91.378

Sample Name: DD100%F↑3:1#2

Date Tested:11/07/2014

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.065	3	3.141	0.0128	12.34	0.03	0	10617.5	153.000
3.106	6	6.280	0.0225	15.6	0.05	0	12183.0	77.361
3.105	9	9.547	0.0166	18.86	0.07	0	12302.0	80.909
5.042	5	5.087	0.0142	20.21	0.03	0	14591.0	161.370
5.046	10	10.660	0.0396	25.8	0.07	0	15474.0	51.001
5.051	15	15.920	0.0420	31.07	0.08	0	17675.5	108.600
10.070	10	10.070	0.0146	40.28	0.03	0	26846.5	169.460
10.030	20	20.400	0.0610	50.48	0.07	0	26752.5	201.660
10.060	30	30.620	0.0308	60.8	0.1	0	30492.0	345.860
15.070	10	9.992	0.0238	55.21	..03	0	30163.5	293.890
15.080	15	14.910	0.0411	60.16	0.04	0	33207.5	210.440
15.070	30	30.280	0.0400	75.48	0.08	0	37380.5	97.282
19.970	15	14.700	0.0156	74.63	0.04	0	38818.0	159.630
20.010	20	19.670	0.0334	79.71	0.04	0	42599.5	175.370
20.000	40	38.920	0.0267	98.91	0.08	0	44576.0	116.480

Sample Name: DH100%E

Date Tested:12/08/2014

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
2.993	3	2.994	0.0223	11.97	0.02	0	12158.0	286.960
3.019	6	6.273	0.0307	15.33	0.05	0	11185.5	73.969
3.077	9	9.418	0.0262	18.65	0.06	0	12702.5	57.333
4.988	5	5.081	0.0268	20.05	0.03	0	15431.5	120.560
5.095	10	10.430	0.0248	25.71	0.06	0	16141.0	168.450
4.984	15	15.570	0.0277	30.52	0.08	0	17495.5	108.270
10.030	10	9.906	0.0150	39.99	0.03	0	25221.0	192.680
10.030	20	20.000	0.0360	50.09	0.06	0	27435.0	88.515
10.020	30	30.280	0.0412	60.34	0.09	0	28325.5	80.163
15.010	10	9.806	0.0236	54.84	0.03	0	29313.0	128.090
14.940	15	14.550	0.0276	59.38	0.04	0	33370.0	40.046
14.950	30	30.100	0.0518	74.95	0.08	0	34092.0	86.959
20.000	15	14.320	0.0347	74.32	0.03	0	36566.5	270.000
19.970	20	18.760	0.0170	78.66	0.04	0	39244.5	187.400
19.990	40	37.560	0.8685	97.52	0.07	0	50304.0	218.430

Sample Name: DH100%NonE

Date Tested:12/08/2014

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
2.992	3	2.944	0.0218	11.92	0.02	0	17297.5	361.810
3.035	6	6.202	0.0052	15.31	0.04	0	12780.5	163.710
3.059	9	9.502	0.0620	18.68	0.07	0	12750.5	555.260
4.972	5	5.115	0.0105	20.03	0.03	0	17329.0	163.490
5.075	10	10.540	0.0228	25.76	0.06	0	16844.5	97.045
5.066	15	15.690	0.0379	30.89	0.07	0	19701.0	102.830
10.010	10	10.170	0.0365	40.19	0.04	0	29098.0	128.710
10.000	20	20.060	0.0452	50.07	0.06	0	28846.0	166.160
10.010	30	30.280	0.0408	60.3	0.09	0	30842.5	146.020
15.040	10	9.792	0.0194	54.9	0.02	0	34985.0	448.030
15.020	15	14.550	0.2332	59.6	0.03	0	35531.0	490.350
15.030	30	29.520	0.0400	74.6	0.07	0	40247.5	151.320
19.980	15	14.540	0.0142	74.49	0.03	0	41553.5	203.650
20.010	20	18.980	0.2698	79.01	0.04	0	43520.5	603.910
19.970	40	38.650	0.0262	98.55	0.08	0	47036.5	166.360

Sample Name: DH40%F↑3:1

Date Tested:12/08/2014

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
2.994	3	2.982	0.0124	11.97	0.02	0	12582.5	166.800
3.005	6	6.344	0.0253	15.36	0.05	0	11250.5	44.535
3.054	9	9.578	0.0290	18.74	0.07	0	12513.5	106.380
4.983	5	5.140	0.0283	20.09	0.03	0	15245.0	186.380
5.049	10	10.460	0.0284	25.6	0.06	0	16825.5	103.440
4.990	15	15.760	0.0493	30.73	0.08	0	17627.5	141.890
10.010	10	10.130	0.0364	40.15	0.04	0	23995.0	143.450
10.010	20	20.250	0.0194	50.28	0.07	0	26536.0	73.731
10.030	30	30.310	0.0296	60.4	0.09	0	29243.0	117.960
15.040	10	9.388	0.0090	54.51	0.02	0	34543.5	212.020
15.020	15	14.200	0.9451	59.24	0.04	0	33264.0	243.190
14.990	30	29.950	0.0267	74.93	0.08	0	35614.0	98.785
20.000	15	14.470	0.1286	74.48	0.03	0	39113.5	535.510
20.000	20	19.450	0.0121	79.45	0.05	0	39801.0	121.780
19.960	40	38.450	0.0313	98.34	0.08	0	44765.5	258.750

Sample Name: DH100%F↑3:1

Date Tested:12/08/2014

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.089	3	3.020	0.0066	12.29	0.03	0	11502.5	102.650
3.101	6	6.232	0.1022	15.54	0.06	0	10956.0	92.249
3.025	9	9.388	0.0562	18.46	0.07	0	13473.5	55.334
5.097	5	4.907	0.0264	20.2	0.03	0	15560.5	186.320
5.061	10	10.370	0.0176	25.55	0.06	0	17022.5	44.799
5.052	15	15.670	0.0141	30.83	0.08	0	18152.5	54.923
10.050	10	10.110	0.0226	40.27	0.04	0	24168.5	211.100
9.997	20	20.280	0.0488	50.27	0.07	0	25792.0	61.766
10.050	30	30.150	0.0359	60.31	0.1	0	28154.0	81.453
15.010	10	9.759	0.0198	54.79	0.03	0	27588.5	79.256
15.050	15	14.910	0.0104	60.06	0.05	0	29760.5	133.820
15.030	30	29.720	0.2463	74.82	0.07	0	34634.0	154.850
50.010	15	14.450	0.0251	74.48	0.03	0	35487.0	152.430
19.990	20	19.380	0.0219	79.33	0.04	0	39984.0	51.668
19.980	40	39.320	0.0599	99.26	0.09	0	40992.5	170.770

Sample Name: Dd100%E

Date Tested:1/13/2015

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.004	3	3.023	0.0183	12.03	0.02	0	14250.0	43.607
2.995	6	5.997	0.0321	14.98	0.03	0	16449.0	125.170
2.999	9	8.820	0.0742	17.82	0.05	0	17384.0	140.220
4.999	5	5.167	0.0581	20.16	0.03	0	19163.5	68.460
5.029	10	9.790	0.0394	24.88	0.05	0	20059.0	58.875
4.975	15	15.010	0.0472	29.93	0.06	0	22428.5	111.340
9.960	10	10.020	0.0222	39.9	0.03	0	29918.5	88.820
9.948	20	20.090	0.1027	49.93	0.05	0	36368.0	548.710
9.949	30	30.090	0.0599	59.94	0.08	0	35959.5	381.700
15.000	10	9.841	0.0441	54.85	0.03	0	38196.5	148.130
14.980	15	15.030	0.0596	59.97	0.04	0	37960.5	211.380
14.970	30	29.500	0.0471	74.41	0.05	0	49473.0	305.500
20.020	15	14.650	0.0190	74.41	0.03	0	57654.0	441.820
19.980	20	19.510	0.0501	79.45	0.04	0	50931.5	349.870
19.990	40	38.770	0.0416	98.74	0.06	0	57788.5	287.950

Sample Name: Dd100%NonE

Date Tested:1/13/2015

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.042	3	2.898	0.0298	12.02	0.01	0	36786.0	1505.200
3.028	6	6.401	0.0243	15.48	0.03	0	21420.5	197.140
2.975	9	9.730	0.0437	18.65	0.05	0	20154.5	99.793
4.992	5	5.071	0.0121	20.05	0.02	0	23858.5	209.620
4.961	10	10.130	0.0270	25.01	0.03	0	30418.0	125.410
4.978	15	15.020	0.0409	29.95	0.05	0	28029.5	76.963
9.988	10	9.607	0.0426	39.57	0.02	0	41676.5	168.920
9.963	20	19.530	0.0481	49.42	0.04	0	43110.0	268.190
9.941	30	29.600	0.0509	59.42	0.06	0	45657.5	232.800
14.960	10	9.400	0.0546	54.29	0.02	0	59334.5	577.610
14.970	15	14.190	0.0175	59.12	0.02	0	49437.5	291.240
14.970	30	28.970	0.0400	73.87	0.05	0	55379.0	405.530
19.960	15	14.410	0.0173	74.29	0.2	0	60733.0	525.750
19.970	20	19.790	0.0340	79.69	0.04	0	53957.0	127.050
19.900	40	37.600	0.0228	97.3	0.05	0	75516.0	448.020

Sample Name: DS100%F↑3:1↓5:1

Date Tested:1/15/2015

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.036	3	2.913	0.0121	12.02	0.02	0	13786.5	452.140
3.084	6	6.106	0.0388	15.36	0.04	0	12256.5	74.729
2.989	9	9.287	0.0237	18.26	0.08	0	11657.5	19.607
4.985	5	4.784	0.0642	19.74	0.02	0	18175.0	251.600
5.028	10	10.160	0.0055	25.25	0.06	0	15714.5	30.118
4.977	15	15.270	0.0215	30.21	0.09	0	15760.0	82.384
10.010	10	9.742	0.0194	39.77	0.04	0	24819.5	93.654
9.992	20	19.830	0.0321	49.81	0.08	0	22542.0	96.390
9.959	30	29.840	0.0223	59.72	0.012	0	22718.5	88.653
15.000	10	9.776	0.0698	54.77	0.03	0	27816.0	45.521
14.990	15	14.550	0.0111	59.53	0.05	0	26686.5	66.072
14.940	30	29.550	0.0373	74.38	0.01	0	27982.5	54.967
19.990	15	14.430	0.0221	74.4	0.04	0	31574.5	64.378
20.000	20	19.340	0.0247	79.33	0.06	0	32000.5	102.430
19.990	40	38.920	0.0180	98.9	0.12	0	31325.5	199.640

Sample Name: DS100%F↑5:1

Date Tested:1/15/2015

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.035	3	2.791	0.0179	11.89	0.01	0	14634.0	1192.900
2.982	6	6.143	0.0081	15.09	0.05	0	10099.2	34.668
2.992	9	9.408	0.0119	18.38	0.09	0	10354.3	26.463
4.984	5	4.899	0.0160	19.85	0.03	0	15367.0	138.120
4.986	10	10.170	0.0408	25.13	0.07	0	14176.0	87.794
4.985	15	15.290	0.0788	30.24	0.09	0	14669.5	51.367
9.940	10	9.680	0.0251	39.5	0.03	0	24658.5	549.340
9.937	20	19.810	0.0225	49.62	0.08	0	21198.0	106.150
9.926	30	29.960	0.1358	59.73	0.13	0	20846.0	81.461
14.950	10	9.608	0.0197	54.46	0.03	0	28793.0	211.650
14.980	15	14.860	0.0196	59.81	0.05	0	25549.0	65.809
14.970	30	29.890	0.1645	74.8	0.12	0	24839.5	85.301
19.970	15	14.380	0.0311	74.29	0.04	0	31967.5	242.750
19.980	20	19.290	0.0171	79.22	0.06	0	31315.0	144.410
19.980	40	38.760	0.0185	98.7	0.12	0	29601.0	126.030

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Sample Name: OD100%E

Date Tested:2/12/2015

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.084	3	3.090	0.0160	12.34	0.02	0	19646.0	136.490
3.067	6	6.356	0.0394	15.56	0.04	0	15954.0	199.870
3.000	9	9.390	0.0483	18.39	0.06	0	16158.0	132.710
4.961	5	4.978	0.0211	19.86	0.02	0	23744.5	543.740
4.977	10	10.290	0.0255	25.22	0.04	0	22265.0	178.800
5.045	15	15.260	0.0550	30.4	0.06	0	24302.0	35.358
9.977	10	9.989	0.0192	39.92	0.03	0	31691.5	181.140
9.968	20	19.950	0.0397	49.86	0.06	0	34370.5	61.304
9.940	30	29.740	0.1813	59.56	0.08	0	35979.0	389.550
14.960	10	9.373	0.0429	54.26	0.02	0	42495.0	591.670
15.000	15	14.820	0.0235	59.83	0.04	0	37988.5	194.920
14.950	30	29.240	0.0363	74.1	0.06	0	44409.5	182.940
20.000	15	14.200	0.0054	74.21	0.02	0	49657.0	116.750
20.000	20	19.450	0.0087	79.43	0.04	0	48524.5	265.510
19.990	40	39.210	0.0661	99.19	0.08	0	46441.5	122.250

Sample Name: OD100%NonE

Date Tested:2/20/2015

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.011	3	2.996	0.0263	12.03	0.02	0	18018.0	428.840
2.989	6	6.059	0.0451	15.03	0.04	0	12521.5	87.618
2.974	9	9.137	0.0847	18.06	0.06	0	13985.0	113.760
4.963	5	5.039	0.0329	19.93	0.02	0	20581.0	134.440
4.991	10	10.020	0.0243	24.99	0.04	0	21410.5	73.966
4.991	15	15.330	0.0355	30.3	0.06	0	21334.5	114.700
9.970	10	9.785	0.0979	39.7	0.03	0	31109.5	255.660
9.936	20	19.900	0.0247	49.71	0.05	0	36417.0	268.410
9.959	30	29.850	0.0380	59.73	0.08	0	35139.0	253.370
14.970	10	9.718	0.0215	54.61	0.02	0	45180.0	314.830
14.980	15	14.880	0.0211	59.81	0.03	0	40526.0	272.830
14.960	30	29.450	0.0290	74.33	0.06	0	44846.5	88.674
19.980	15	14.670	0.0723	74.61	0.03	0	52531.0	269.330
19.930	20	19.980	0.0332	79.78	0.04	0	48802.5	135.080
19.930	40	39.410	0.0292	99.19	0.07	0	52273.5	241.340

Sample Name: OD40%F↑3:1

Date Tested:2/25/2015

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.030	3	2.959	0.1713	12.05	0.02	0	14234.0	898.520
3.072	6	6.124	0.0275	15.34	0.04	0	14454.5	47.958
3.072	9	9.464	0.0133	18.68	0.05	0	15059.5	54.050
5.044	5	5.041	0.0127	20.17	0.02	0	19215.0	395.510
5.040	10	10.030	0.0201	25.15	0.04	0	22916.5	77.959
5.048	15	15.280	0.0869	30.43	0.07	0	20645.0	128.990
10.030	10	9.561	0.0406	39.64	0.03	0	31254.5	227.190
10.030	20	19.570	0.0124	49.65	0.05	0	32217.5	79.255
10.000	30	29.680	0.0135	59.7	0.08	0	31296.0	51.151
15.040	10	9.757	0.0178	54.87	0.03	0	34458.0	199.750
15.030	15	14.670	0.0202	59.78	0.03	0	37021.0	120.760
15.040	30	29.760	0.0307	74.88	0.07	0	36825.5	258.420
19.990	15	14.860	0.0184	74.82	0.03	0	41167.5	129.600
20.000	20	20.170	0.0438	80.19	0.04	0	41185.0	154.087
20.000	40	39.840	0.0616	99.84	0.08	0	46619.0	118.470

Sample Name: OD100%F↑3:1

Date Tested:2/12/2015

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
2.977	3	2.879	0.0104	11.81	0.02	0	15907.0	378.170
2.983	6	6.332	0.0371	15.28	0.05	0	11460.5	76.710
3.061	9	9.561	0.0145	18.74	0.06	0	13350.5	81.923
4.993	5	4.631	0.0034	19.61	0.02	0	23068.5	388.400
4.981	10	10.580	0.0490	25.52	0.06	0	16837.5	109.070
4.981	15	15.730	0.0625	30.67	0.08	0	17805.0	89.752
10.010	10	10.030	0.0101	40.05	0.03	0	28536.0	210.520
9.957	20	20.220	0.0406	50.09	0.07	0	26971.0	113.590
9.954	30	30.460	0.0199	60.32	0.1	0	27496.5	73.083
14.980	10	9.824	0.0186	54.75	0.03	0	31927.0	481.940
15.000	15	14.930	0.0336	59.93	0.04	0	32046.5	97.315
14.970	30	19.180	0.2701	74.1	0.07	0	35692.5	139.800
19.990	15	14.440	0.0167	74.42	0.03	0	39600.5	358.890
19.970	20	19.370	0.0376	79.28	0.04	0	39984.5	137.900
19.950	40	39.040	0.0607	98.88	0.09	0	41770.5	218.390

Sample Name: OH100%E

Date Tested:2/13/2015

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.010	3	2.697	0.0204	11.73	0.02	0	27862.0	1027.900
2.972	6	6.227	0.0327	15.14	0.05	0	13815.5	29.867
3.059	9	9.150	0.0419	18.33	0.06	0	14167.0	76.004
5.020	5	4.967	0.0279	20.03	0.03	0	19625.0	112.400
50.560	10	10.340	0.0712	25.51	0.06	0	19100.5	127.340
5.016	15	15.350	0.0225	30.4	0.09	0	20354.0	57.484
10.010	10	8.829	2.5977	38.85	0.04	0	26010.5	455.200
10.030	20	19.940	0.0175	50.03	0.09	0	30309.5	67.053
10.020	30	29.770	0.1519	59.82	0.12	0	31077.0	74.202
15.040	10	9.483	0.0090	54.61	0.04	0	37037.0	319.390
15.000	15	14.720	0.0092	59.71	0.06	0	32806.0	143.810
15.020	30	29.310	0.0425	74.38	0.11	0	42676.0	124.810
19.980	15	14.140	0.0192	74.09	0.05	0	49598.0	169.870
19.970	20	18.920	0.0128	78.83	0.06	0	48359.0	180.930
19.990	40	39.180	0.0442	99.15	0.13	0	47163.5	163.900

Sample Name: OH100%E#2

Date Tested:2/27/2015

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.071	3	2.732	0.0133	11.95	0.03	0	21073.0	201.840
3.073	6	6.054	0.0174	15.27	0.05	0	14964.5	62.681
3.085	9	9.459	0.0267	18.72	0.07	0	15206.5	52.682
5.068	5	4.879	0.0109	20.08	0.03	0	20944.5	188.010
5.069	10	10.200	0.0164	25.41	0.06	0	19417.0	27.822
5.038	15	15.450	0.0353	30.57	0.09	0	20175.5	61.320
10.010	10	10.030	0.0872	40.06	0.05	0	24182.0	169.480
10.030	20	20.220	0.1510	50.3	0.09	0	27814.0	187.940
10.010	30	29.730	0.0286	59.76	0.13	0	30454.5	80.610
15.050	10	9.953	0.0127	55.11	0.04	0	35165.0	197.530
15.400	15	15.180	0.0505	60.29	0.06	0	34141.5	199.400
15.050	30	30.460	0.0560	75.6	0.12	0	38575.0	82.197
20.010	15	15.230	0.0226	75.24	0.06	0	38223.0	113.510
20.000	20	20.140	0.0417	80.15	0.07	0	38631.0	216.260
20.030	40	39.870	0.0307	99.96	0.14	0	43140.5	386.140

Sample Name: OH100%NonE

Date Tested:2/20/2015

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
2.999	3	3.014	0.0332	12.01	0.02	0	17316.0	291.620
2.996	6	6.173	0.0532	15.16	0.04	0	13854.5	99.770
3.000	9	8.972	0.0847	17.97	0.05	0	15320.5	154.360
4.989	5	4.938	0.0087	19.91	0.03	0	17848.0	83.553
5.018	10	10.030	0.0370	25.08	0.04	0	21610.0	339.920
4.970	15	14.980	0.0258	29.89	0.06	0	22581.5	103.670
9.947	10	9.658	0.0152	39.5	0.03	0	30206.0	71.150
9.950	20	19.620	0.2618	49.47	0.05	0	32644.0	242.040
9.944	30	29.660	0.0135	59.5	0.08	0	33972.0	277.700
14.950	10	9.740	0.1319	54.58	0.02	0	42042.0	564.940
14.960	15	14.800	0.0147	59.67	0.03	0	40503.5	175.440
14.970	30	29.820	0.0343	74.74	0.06	0	39951.5	135.360
19.940	15	14.640	0.1758	74.45	0.02	0	46615.5	381.490
19.920	20	19.340	0.0847	79.09	0.04	0	45165.0	321.500
19.940	40	39.720	0.0391	99.54	0.07	0	46346.0	228.060

Sample Name: OH40%F↑3:1

Date Tested:2/25/2015

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.065	3	2.956	0.0075	12.15	0.02	0	16835.0	196.780
3.049	6	6.285	0.0087	15.43	0.04	0	12134.5	56.206
3.061	9	9.366	0.0609	18.55	0.06	0	14138.5	63.911
5.057	5	4.757	0.0270	19.93	0.02	0	19912.0	112.260
5.060	10	10.200	0.0158	25.38	0.05	0	18191.5	82.072
5.060	15	15.390	0.0266	30.57	0.07	0	18328.5	79.586
10.010	10	9.905	0.0773	39.94	0.04	0	23795.0	103.280
10.030	20	19.700	0.1942	49.79	0.06	0	27957.0	142.520
10.020	30	29.440	0.0325	59.5	0.09	0	28451.0	95.680
15.030	10	9.911	0.0126	55	0.03	0	32772.5	252.820
15.030	15	14.870	0.0084	59.97	0.05	0	30842.0	121.260
15.030	30	30.130	0.0595	75.2	0.08	0	35115.5	97.517
19.990	15	14.620	0.0145	74.58	0.04	0	38082.5	94.237
19.980	20	19.800	0.0300	79.74	0.04	0	39630.0	125.090
20.000	40	39.630	0.0148	99.63	0.09	0	40212.5	112.070

Sample Name: OH100%F↑3:1

Date Tested:2/13/2015

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.018	3	2.844	0.0129	11.9	0.02	0	15125.0	209.590
3.094	6	6.005	0.0106	15.29	0.05	0	11476.5	50.648
3.034	9	9.252	0.0117	18.35	0.08	0	11064.0	29.100
5.069	5	4.832	0.0156	20.04	0.03	0	18412.0	69.032
5.038	10	10.180	0.0659	25.3	0.07	0	15638.5	43.552
5.017	15	14.950	0.0219	30	0.09	0	17468.0	21.268
10.030	10	9.387	0.0108	39.48	0.04	0	26792.0	36.378
10.040	20	19.810	0.0102	49.92	0.09	0	26501.5	28.661
9.998	30	30.110	0.2201	60.1	0.14	0	25833.0	24.168
15.030	10	9.612	0.0154	54.71	0.04	0	35861.0	109.730
15.020	15	14.440	0.0263	59.49	0.06	0	31671.5	56.323
15.020	30	29.580	0.0173	74.68	0.12	0	32070.0	22.412
19.970	15	14.250	0.0292	74.17	0.05	0	33934.5	40.566
19.990	20	19.090	0.0358	79.06	0.07	0	35625.5	36.636
19.990	40	38.750	0.0415	98.72	0.14	0	37143.0	11.585

Sample Name: OH100%F↑3:1#2

Date Tested:2/27/2015

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.078	3	2.853	0.0073	12.09	0.01	0	13115.0	375.030
3.084	6	6.026	0.0149	15.28	0.04	0	14226.5	139.000
3.041	9	8.958	0.0739	18.08	0.06	0	13936.5	140.640
5.049	5	5.046	0.0216	20.19	0.03	0	26622.0	1062.900
5.044	10	9.949	0.0170	25.08	0.06	0	16807.0	89.646
5.032	15	15.090	0.0340	30.19	0.09	0	19089.0	41.217
10.010	10	10.050	0.0444	40.07	0.04	0	29090.0	224.260
10.000	20	20.130	0.0209	50.14	0.09	0	28158.0	184.570
10.020	30	30.590	0.0326	60.66	0.14	0	27151.5	57.565
15.040	10	9.957	0.0136	55.08	0.04	0	38174.5	456.000
15.040	15	14.790	0.0167	59.9	0.06	0	32623.0	136.240
15.020	30	30.060	0.0225	75.13	0.12	0	33499.5	128.260
20.000	15	15.020	0.0105	75.01	0.06	0	37127.5	177.660
19.990	20	20.040	0.0258	80.01	0.8	0	35653.5	93.853
19.980	40	39.780	0.0685	99.73	0.15	0	38362.0	285.100

Sample Name: Od100%E

Date Tested:2/19/2015

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.049	3	2.918	0.0409	12.07	0.01	0	25770.0	496.980
3.080	6	5.953	0.0206	15.19	0.03	0	17744.0	250.020
3.095	9	9.114	0.0335	18.4	0.05	0	18401.0	114.640
5.044	5	4.807	0.0534	19.94	0.02	0	26810.5	372.540
5.039	10	10.210	0.0262	25.33	0.04	0	23948.0	157.550
5.027	15	15.440	0.0804	30.52	0.06	0	24801.5	149.850
10.020	10	9.542	0.0130	39.6	0.02	0	44763.5	236.000
10.020	20	19.490	0.0456	49.54	0.05	0	40106.0	390.170
10.020	30	28.340	0.0514	58.41	0.03	0	43851.5	337.500
15.030	10	9.676	0.0146	54.75	0.001	0	80326.0	376.940
15.020	15	14.570	0.0085	59.64	0.03	0	49416.0	236.530
15.020	30	29.710	0.0568	74.78	0.05	0	53737.0	374.270
19.970	15	14.630	0.2020	74.53	0.02	0	72434.0	169.530
19.990	20	20.140	0.0183	80.1	0.03	0	67069.0	377.900
19.960	40	40.190	0.0530	100.1	0.06	0	64230.0	379.700

Sample Name: Od100%NonE

Date Tested:2/19/2015

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.028	3	2.866	0.0152	11.95	0.01	0	43472.0	496.980
3.039	6	5.910	0.0267	15.03	0.03	0	16332.5	250.020
3.038	9	8.979	0.0074	18.09	0.06	0	14884.0	114.640
5.028	5	4.826	0.0235	19.91	0.02	0	24694.0	372.540
5.049	10	9.923	0.0251	25.07	0.04	0	24613.5	157.550
5.078	15	15.130	0.0553	30.37	0.06	0	23908.5	149.850
10.010	10	10.100	0.0244	40.13	0.02	0	39547.0	236.150
10.030	20	19.890	0.0195	49.99	0.05	0	40418.0	151.700
10.000	30	29.310	0.0317	59.31	0.06	0	42862.0	390.700
15.020	10	9.714	0.0322	54.76	0.01	0	55795.5	371.500
15.010	15	15.130	0.0325	60.17	0.03	0	53185.5	376.940
15.040	30	29.970	0.0389	75.1	0.05	0	49832.5	236.530
20.010	15	14.550	0.0422	74.58	0.02	0	69787.5	374.270
20.010	20	20.010	0.0296	80.03	0.03	0	59629.0	169.530
20.000	40	39.490	0.0462	99.47	0.06	0	58182.0	377.900

Sample Name: OS100%F↑3:1↓5:1

Date Tested:2/19/2015

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.059	3	2.852	0.0156	12.03	0.02	0	22551.0	343.310
3.052	6	6.059	0.0408	15.22	0.06	0	13753.5	106.110
3.062	9	9.187	0.0168	18.37	0.08	0	13392.0	81.548
5.050	5	4.601	0.0031	19.75	0.04	0	21512.5	215.550
5.054	10	10.340	0.0121	25.5	0.07	0	16659.0	63.039
5.058	15	15.520	0.0318	30.69	0.09	0	17659.0	66.961
10.030	10	9.936	0.0155	40.04	0.05	0	28819.5	144.510
10.020	20	20.150	0.1308	50.21	0.09	0	25505.5	191.010
10.030	30	30.410	0.0391	60.51	0.14	0	25620.0	83.833
15.040	10	9.137	1.4197	54.25	0.04	0	32220.5	4494.300
15.050	15	14.720	0.0164	59.87	0.06	0	32640.5	185.780
15.000	30	30.020	0.0373	75.03	0.12	0	31197.5	66.390
19.990	15	14.620	0.0397	74.58	0.06	0	36870.0	149.500
19.980	20	19.930	0.0177	79.89	0.08	0	35107.5	161.660
19.970	40	39.200	0.0365	99.12	0.15	0	36909.0	145.840

Sample Name: OS100%F↑5:1

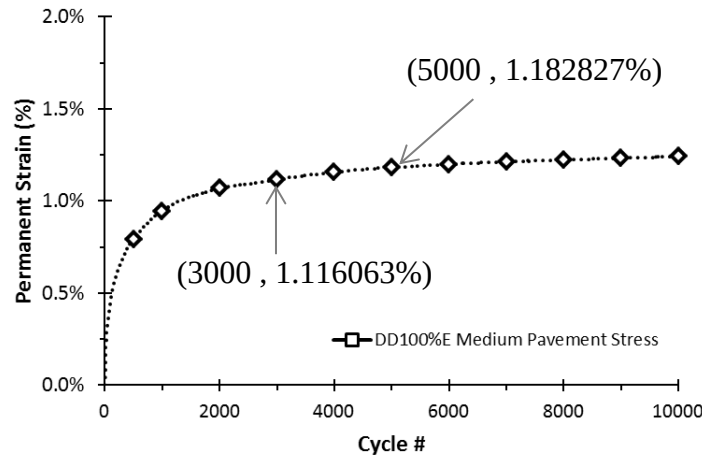
Date Tested:2/19/2015

Confining Stress S3 (psi)	Nom. Max Deviator Stress (psi)	Mean Deviator Stress (psi)	Std. Dev. Deviator Stress (psi)	Mean Bulk Stress (psi)	Mean Resilient Strain (%)	Std. Dev. Resilient Strain (psi)	Mean Resilient Modulus (psi)	Std. Dev. Resilient Modulus (psi)
3.052	3	2.933	0.0179	12.09	0.02	0	14157.0	152.570
3.082	6	5.942	0.0314	15.19	0.05	0	10139.2	47.009
3.090	9	9.085	0.0128	18.35	0.07	0	10238.3	61.650
5.077	5	4.842	0.0117	20.07	0.03	0	17153.0	234.250
5.034	10	10.140	0.0103	25.25	0.07	0	13281.0	42.914
5.068	15	15.100	0.0332	30.3	0.09	0	14680.5	68.336
10.020	10	9.723	0.0932	39.79	0.04	0	22186.5	98.902
10.010	20	19.910	0.0542	49.94	0.08	0	20804.5	67.732
9.990	30	29.580	0.0243	59.55	0.12	0	21710.0	93.432
15.030	10	9.491	0.1081	54.59	0.03	0	28033.5	262.530
15.020	15	14.380	0.1704	59.43	0.05	0	25934.5	298.280
15.020	30	28.830	0.0324	73.88	0.1	0	26541.5	29.506
19.990	15	14.010	0.0158	73.98	0.04	0	30576.5	90.323
20.010	20	19.750	0.0086	79.78	0.06	0	29138.0	60.177
19.970	40	39.370	0.0395	99.28	0.11	0	29562.0	155.500

APPENDIX C: PERMANENT DEFORMATION TEST RESULTS

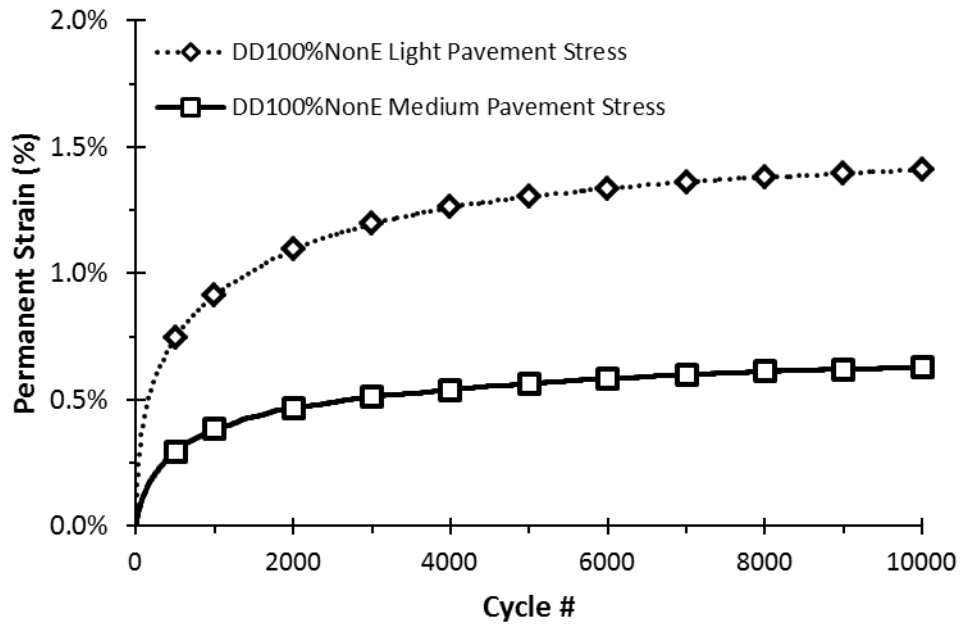
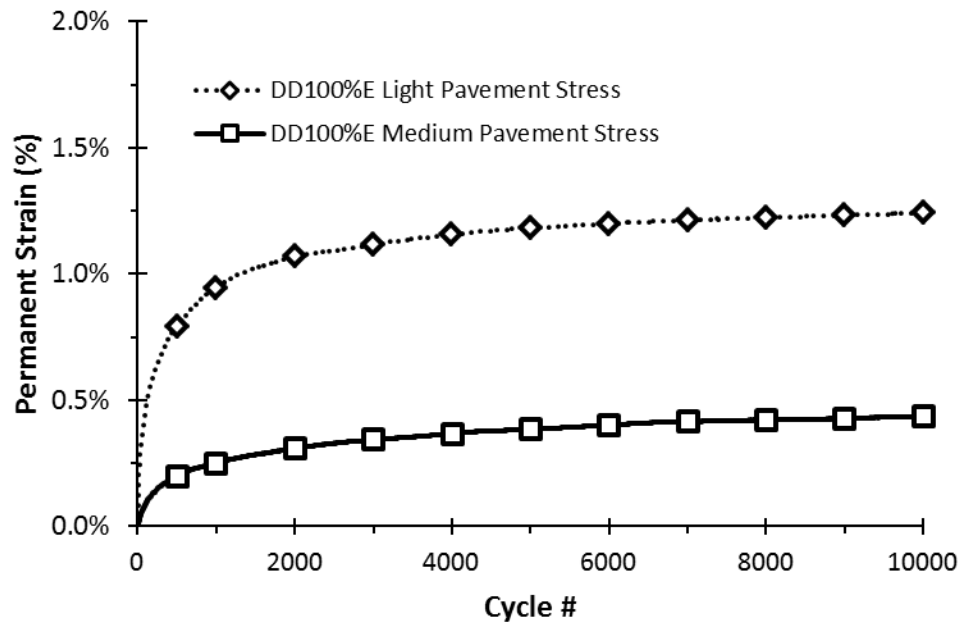
Example of European Standard (BS EN 13286-7 Annex C (2004)) Classification of Shakedown Range

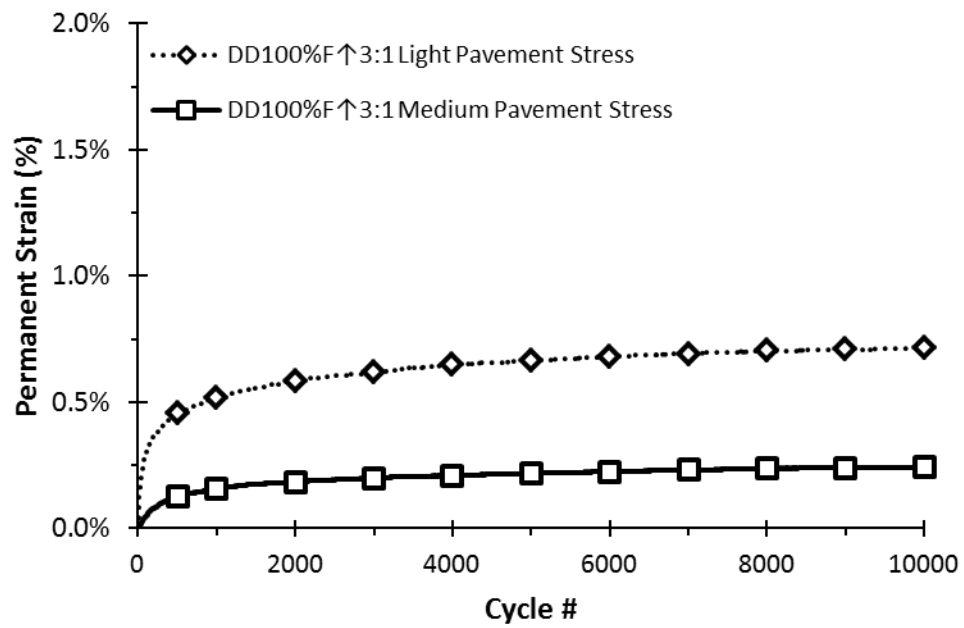
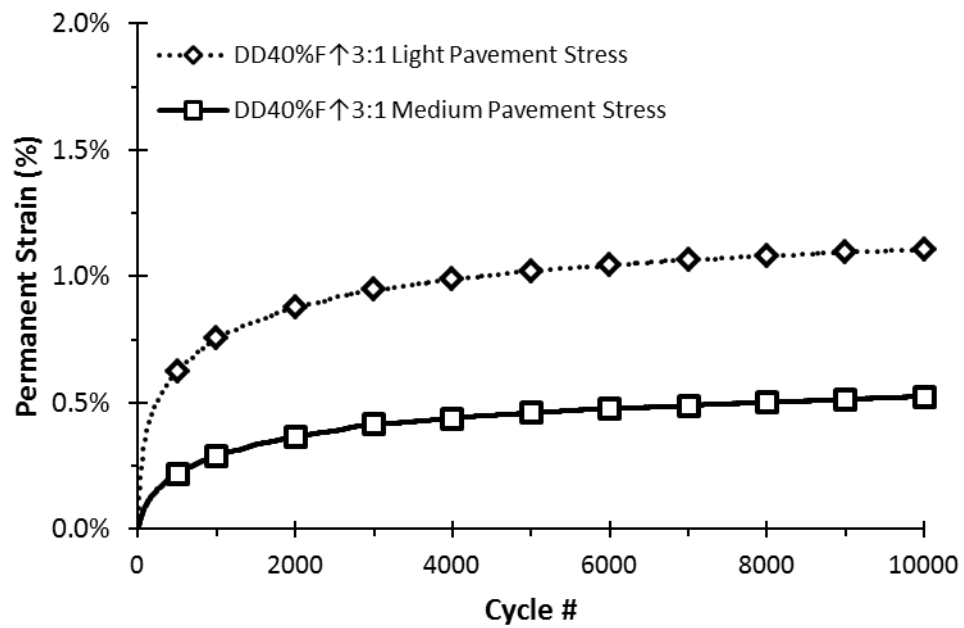
The DD100%E sample tested at light pavement stress in the permanent deformation test is presented below with the associated permanent strain values at 3,000 and 5,000 cycles. The strain value at 5,000 cycles is subtracted from the strain value at 3,000 cycles and then compared to the given shakedown ranges in the European standard.

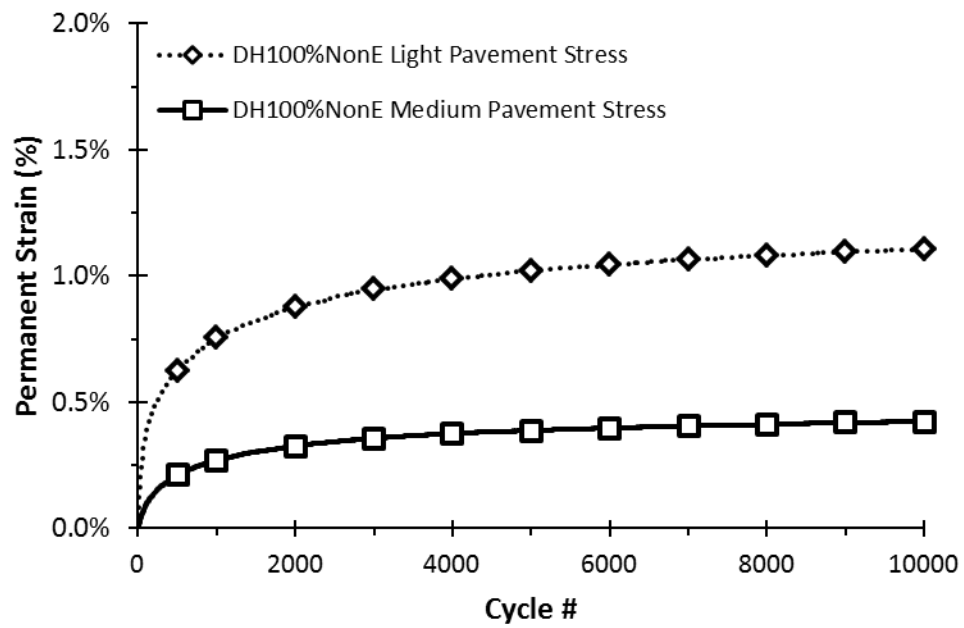
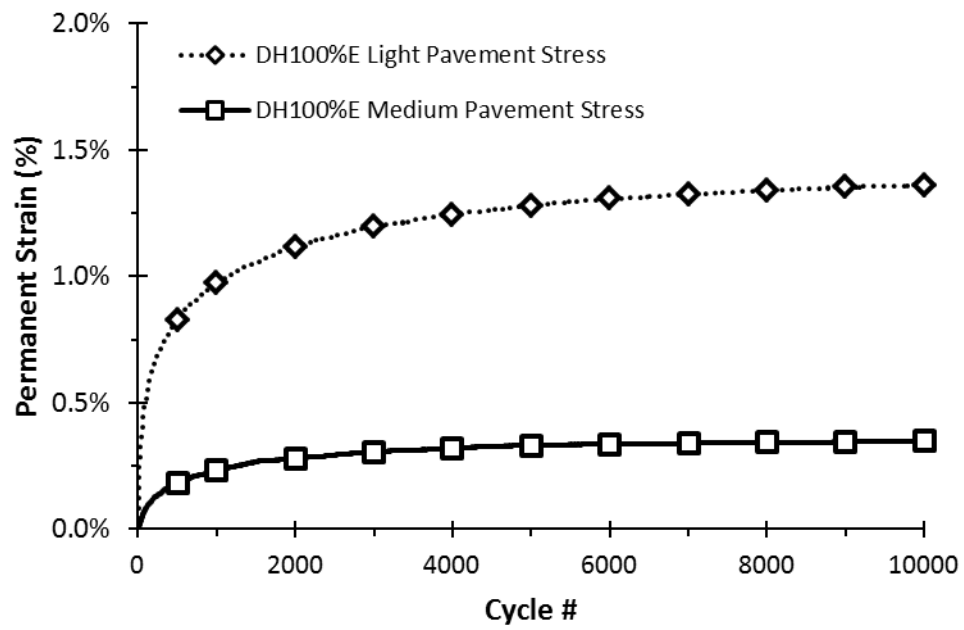


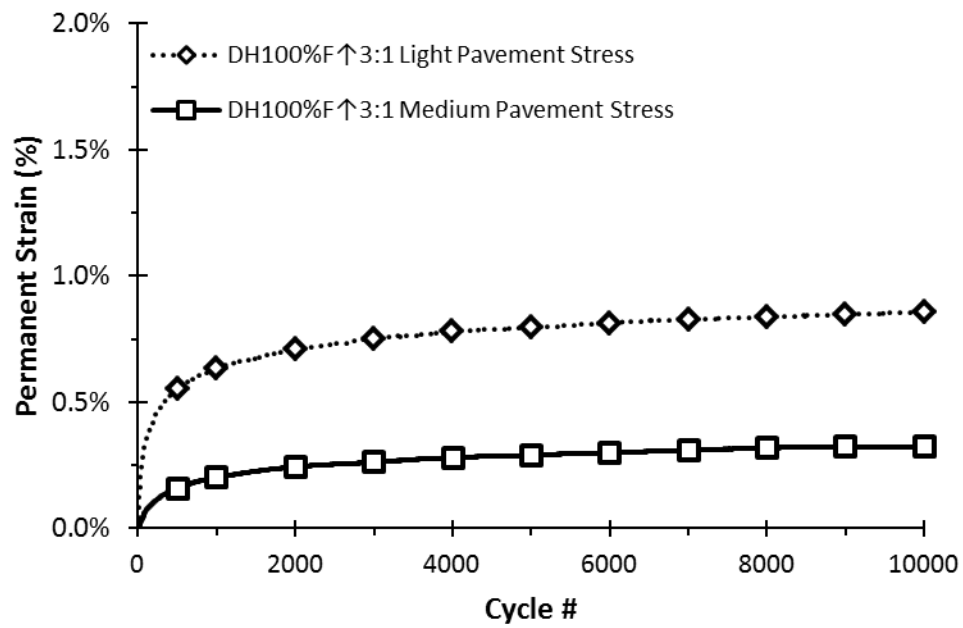
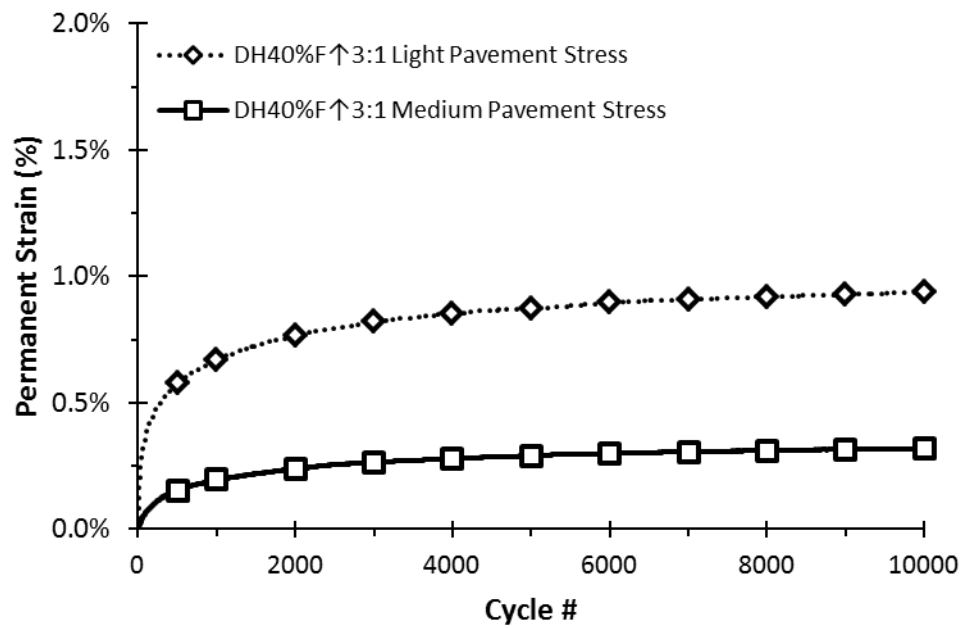
The strain at 5,000 cycles (1.182827%) minus the strain at 3,000 cycles (1.116063%) is 0.000066764 $\sim 6.7 \times 10^{-4}$. This value falls above the 0.4×10^{-3} value that separates the B and C ranges. So, DD100%E is classified in shakedown range C. All permanent deformation tests were classified by this procedure.

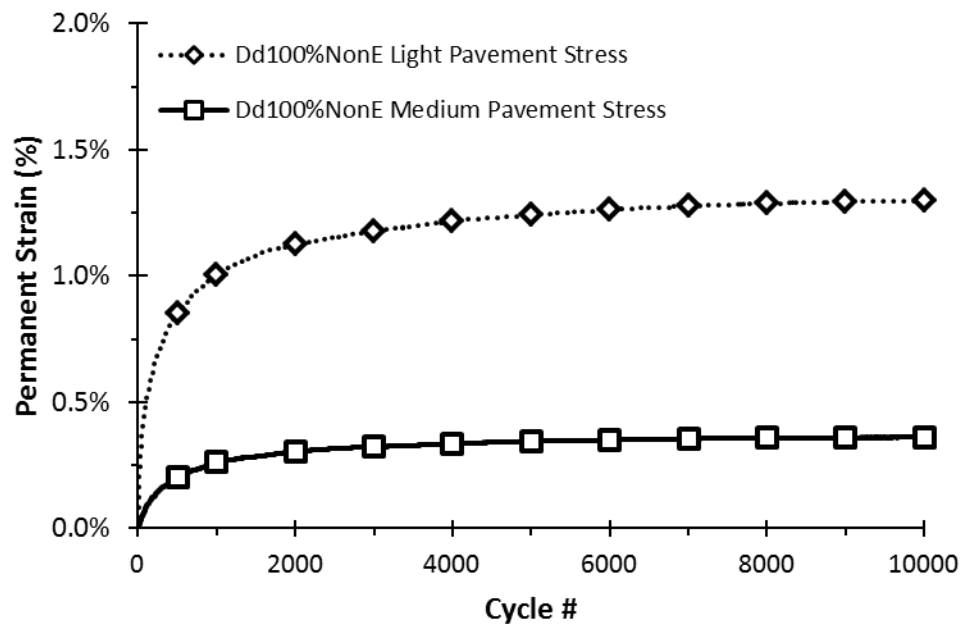
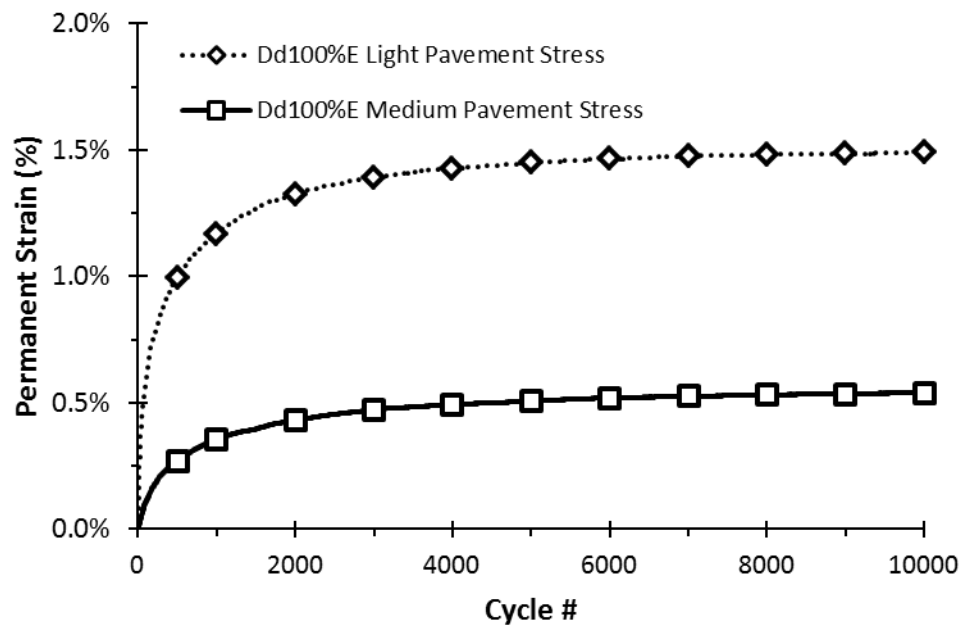
Dense-Graded Permanent Deformation

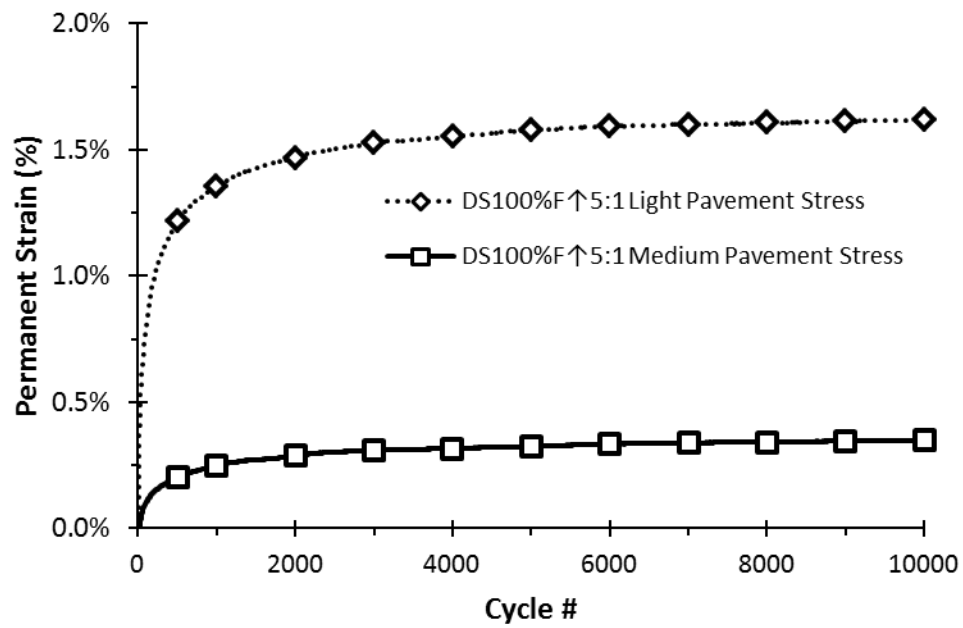
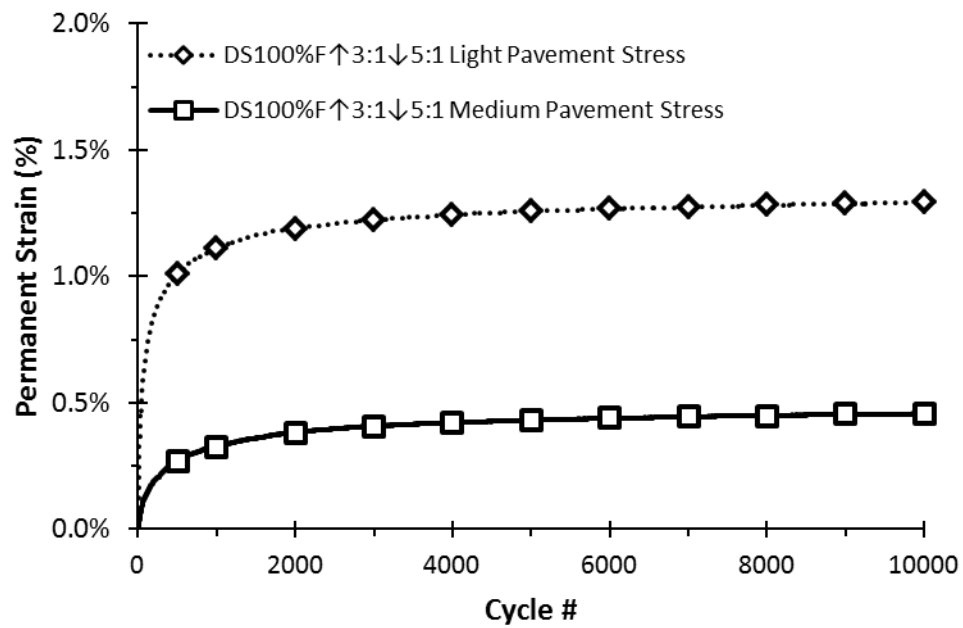




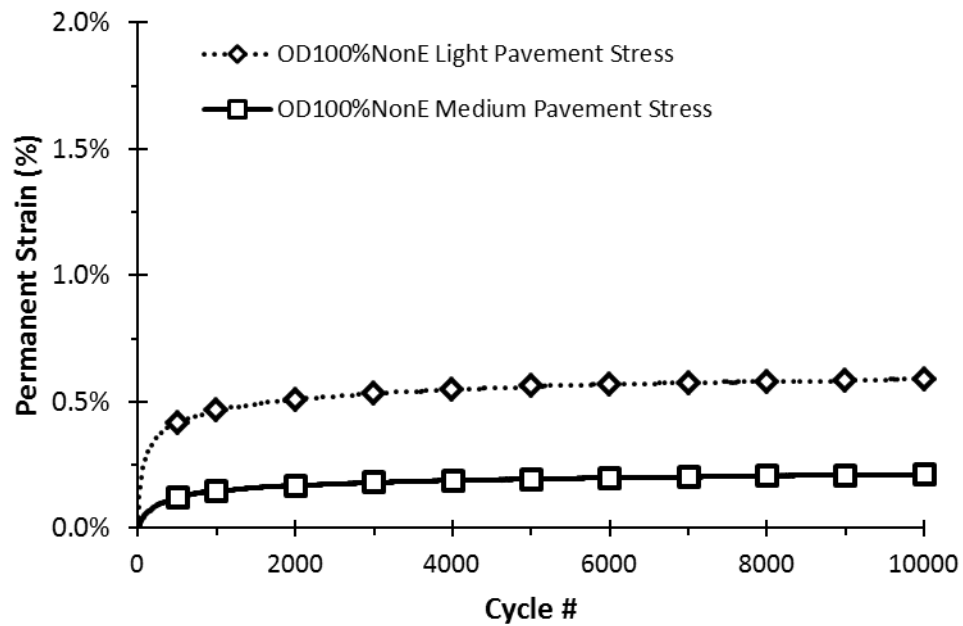
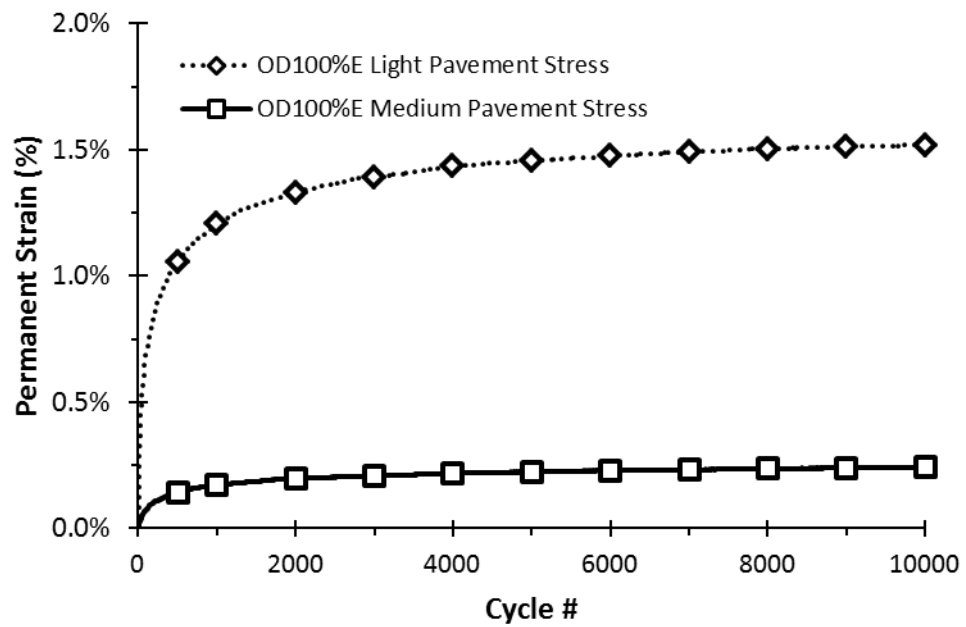


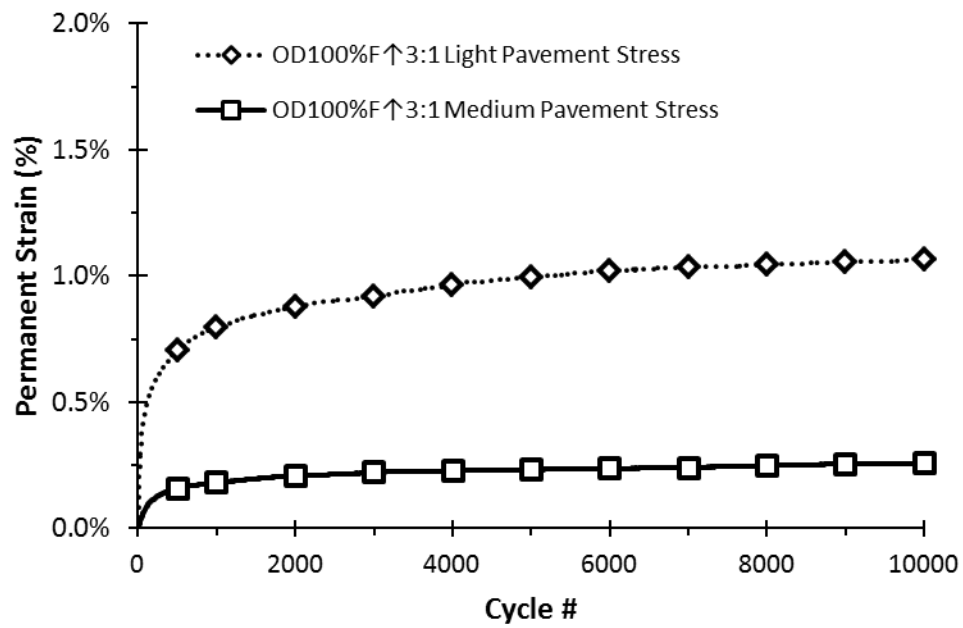
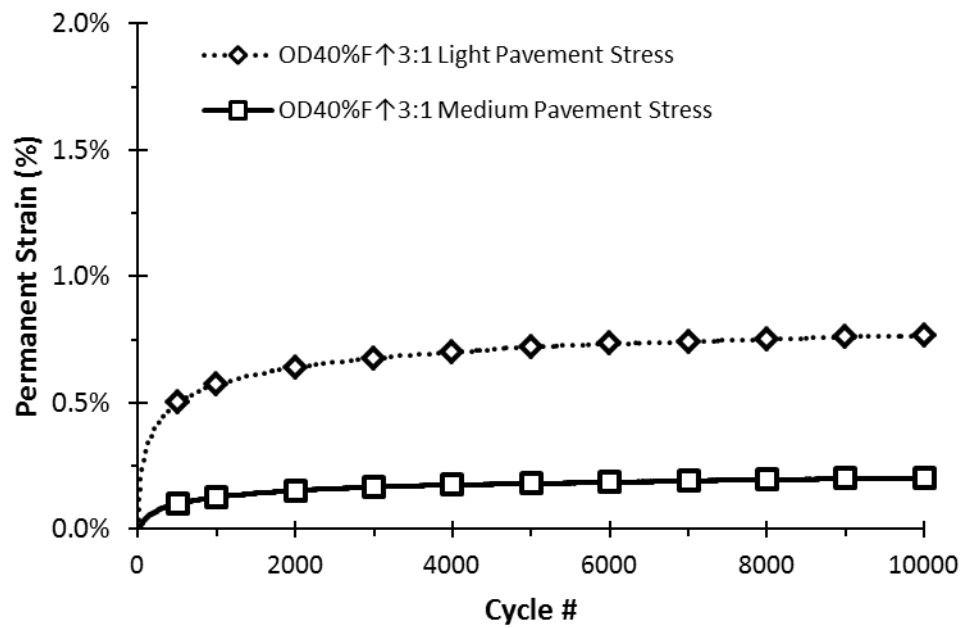


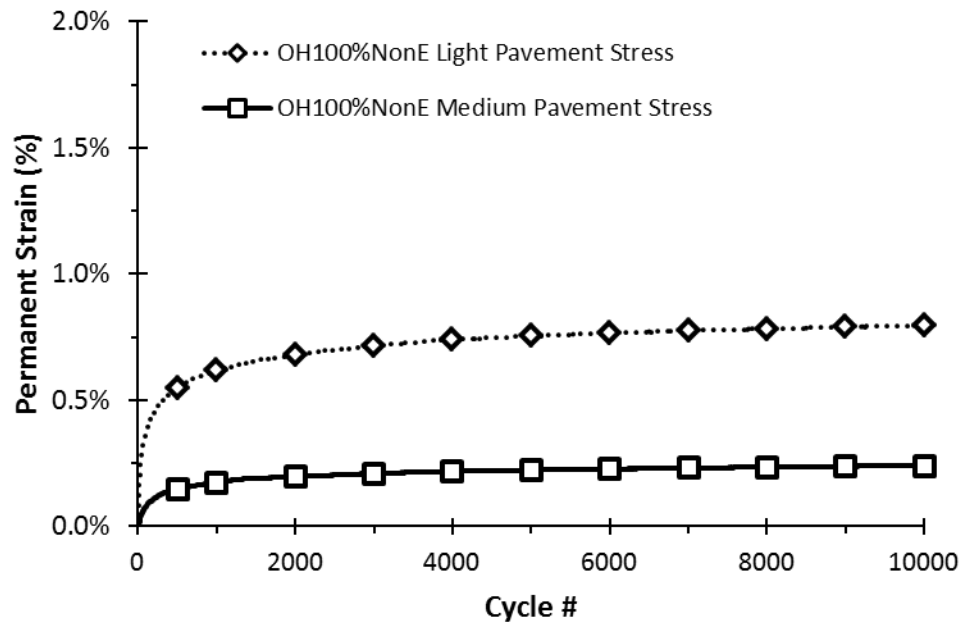
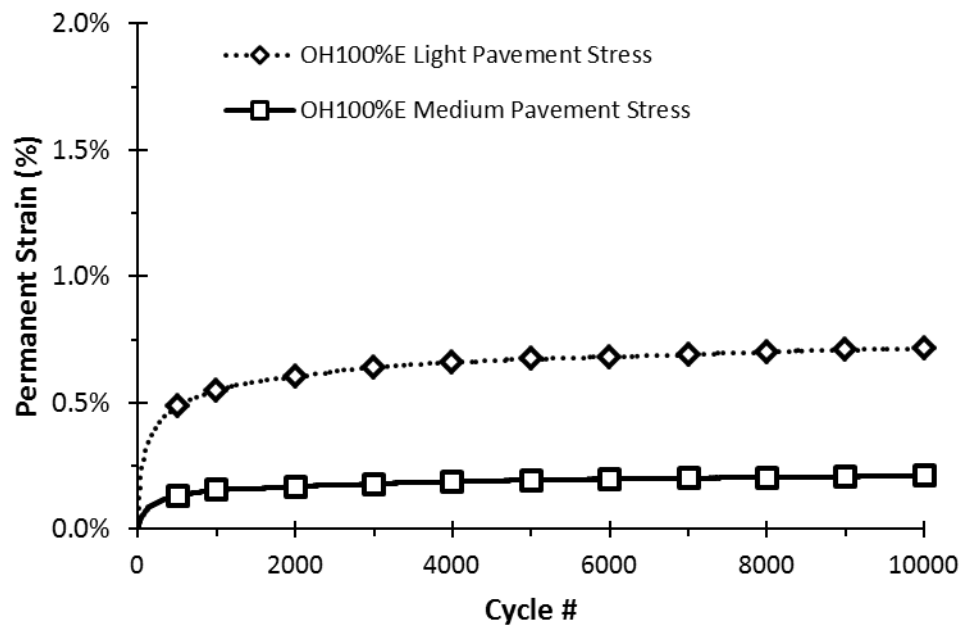


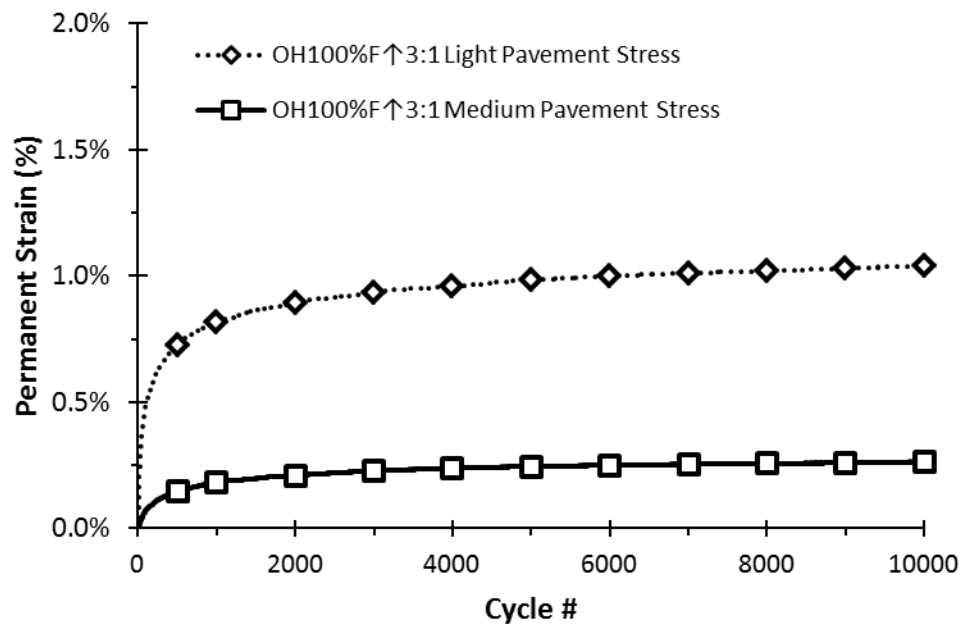
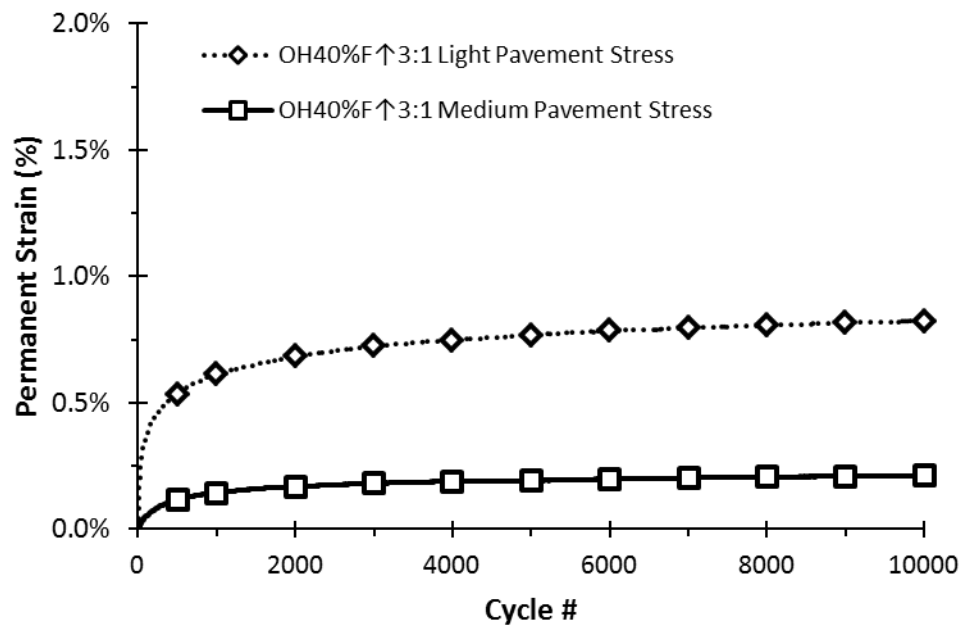


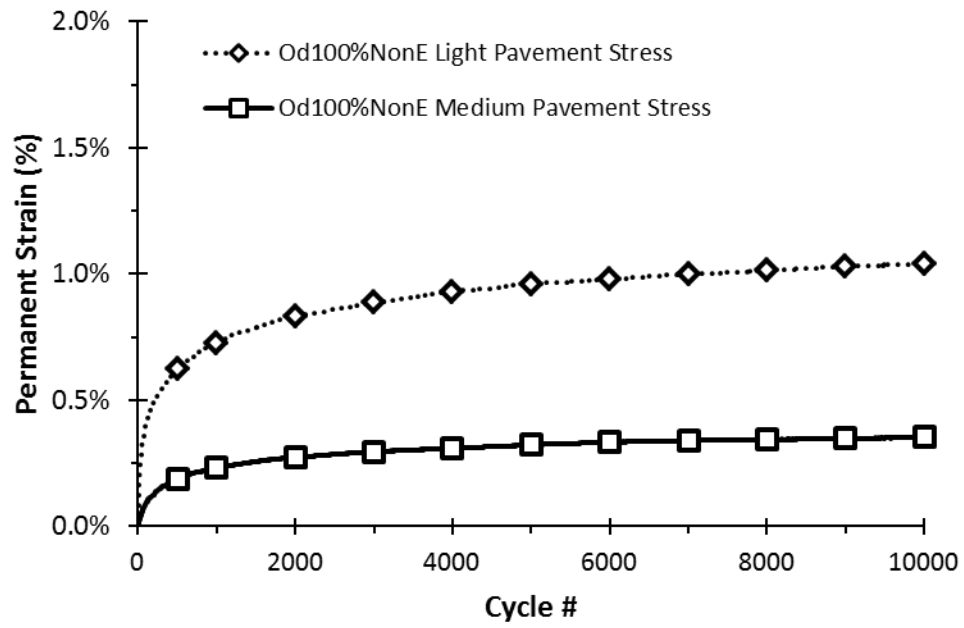
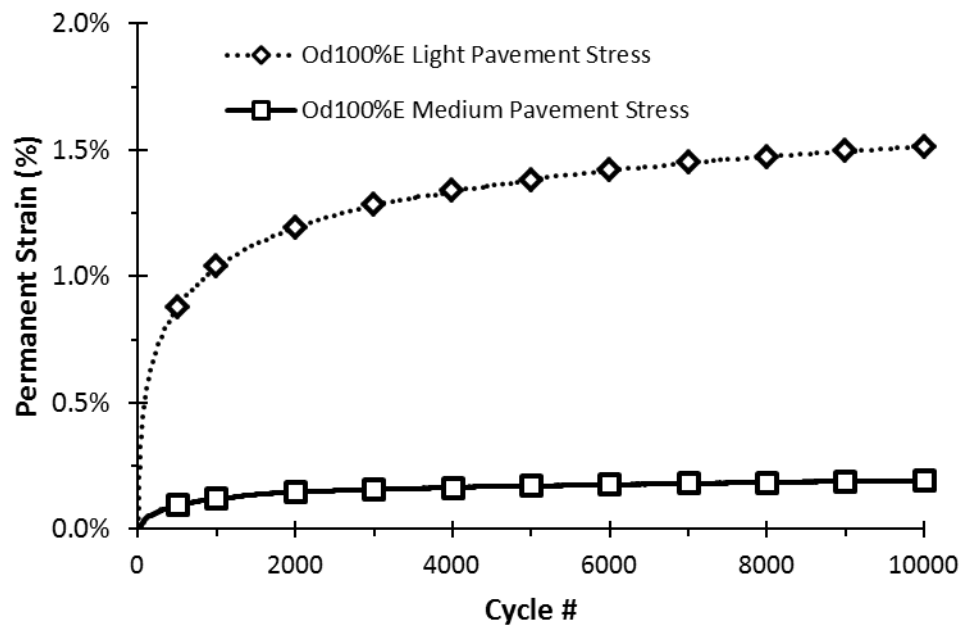
Open-Graded Permanent Deformation

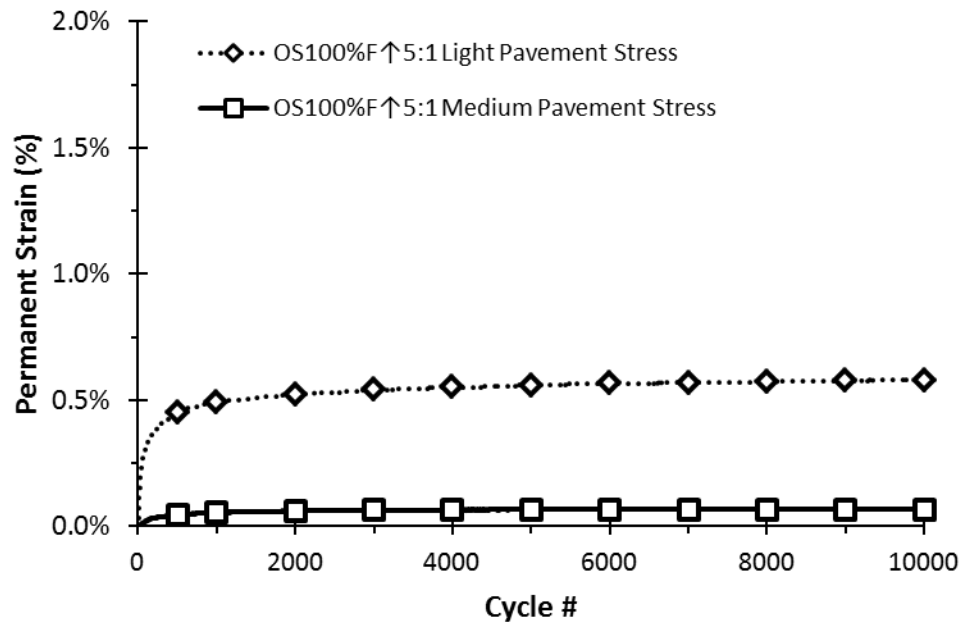
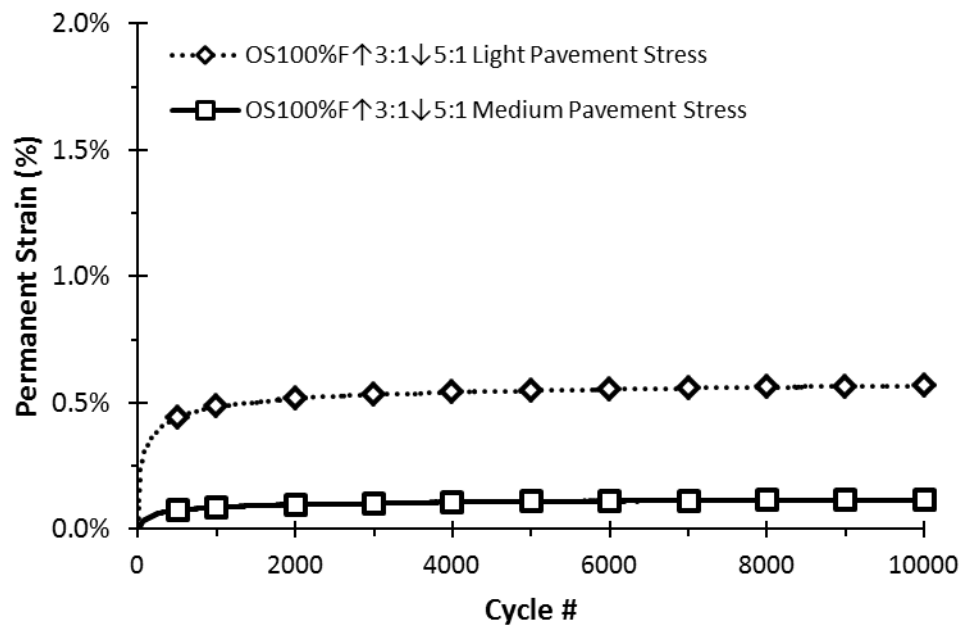












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BIOGRAPHY

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