# STABILITY AND PERMEABILITY OF PROPOSED AGGREGATE BASES IN OKLAHOMA

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Prepared by: Naji N. Khoury Musharraf Zaman Rouzbeh Ghabchi Hassan Kazmee

School of Civil Engineering and Environmental Science The University of Oklahoma Norman, Oklahoma 73019

> Submitted to: Ginger McGovern Planning and Research Division Engineer 200 N.E. 21<sup>st</sup> Street Oklahoma City, OK 73105



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Aggregate base is an important component of a pavement structure. It supports the asphalt concrete (AC) layer and reduces the wheel load-induced stresses on the underlying layers. It also functions as a drainage layer. Consequently, it is important to understand the drainage and strength characteristics of aggregate bases. Permeability (k) and resilient modulus  $(M_R)$  of aggregate bases are used frequently to identify drainage and strength characteristics of aggregate bases. Historically, gradation specifications used by Oklahoma Department of Transportation (ODOT) have led to aggregate bases with very low permeability. Variations in permeability within the same gradation envelope are observed frequently. Also, ODOT currently lacks laboratory and field data for resilient modulus  $(M_R)$  and permeability (k) for commonly used aggregates and gradations. To this end, a combined study is undertaken to generate pertinent laboratory and field data on aggregates for different gradations, including new gradations. Specifically, the present study focuses on the effect of gradation and compaction energy on  $M_R$  and k of aggregates from three commonly used sources in Oklahoma, namely Anchor Stone, Dolese and Martin Marietta. The current study was originally planned as a laboratory study, called Phase 1 in this report. A field component was added subsequently, called Phase 2 in this report, in cooperation with the Oklahoma Aggregate Association (OKAA). In Phase 1, five different gradations, namely, ODOT Type A, Modified AASHTO #57 (M-AASHTO #57), Modified AASHTO #67 (M-AASHTO #67), OKAA Type N, and OKAA Type K, were used in laboratory testing of limestone aggregates from the Anchor Stone quarry. For each gradation, both lower (LL) and upper (UL) limits were used and specimens were prepared using two different levels of compaction, namely standard Proctor and modified Proctor. Permeability (k) was measured using the falling head approach, while resilient modulus  $(M_R)$  was evaluated using the AASHTO T-307 test method. In Phase 2, the number of gradations was narrowed down to three, namely M-AASHTO #57, OKAA Type M and ODOT Type A. These three gradations were tested with all three aggregate sources, as mentioned earlier. To simulate an open-graded base course, M-AASHTO #57 and OKAA Type M specimens were compacted with standard Proctor effort, whereas, modified Proctor effort was used for ODOT Type A specimens to replicate dense-graded base condition. In addition to laboratory testing, in-situ drainage and strength characteristics of representative gradations were evaluated and compared in Phase 2. Accordingly, a 500-ft (152.4 m) long test section was constructed on Timberdell Road in Norman with three selected gradations, namely M-AASHTO #57, OKAA Type M and ODOT Type A. Field tests (falling weight deflectometer (FWD), dynamic cone penetrometer (DCP) and permeability) were conducted during construction and after the test section being opened to traffic. Laboratory test results show that permeability decreases with the increase in compaction level, percent fines and dry density, as expected. ODOT Type A UL shows the highest Maximum Dry Unit Weight with modified Proctor effort for Anchor Stone and Dolese aggregates. Lower limits show higher permeability values in comparison to those of upper limit for the selected gradations. Lower limit of M-AASHTO #67 and #57 satisfies the minimum drainage requirement suggested by the FHWA guideline. The resilient modulus values increased with the increase in dry density and compaction level. Also, coarser LL provided higher M<sub>R</sub> values compared to finer UL because of increased aggregate interlocks. For open-graded base layers permeability can increase due to the increase in angularity of aggregates even in the presence of fines. Field data reveal that traffic-induced compaction led to an increase in  $M_R$  values and decrease in permeability, which conforms to the findings from the laboratory testing. Regression models were developed correlating moisture content, dry unit weight, gradation characteristics and compaction methods to permeability and resilient modulus using laboratory test results. Based on the statistical parameters, these models were found to be significant in predicting the k and M<sub>R</sub> values of aggregates and gradations used in this study. The laboratory and field data from this study could be used for local calibration of the mechanistic-empirical pavement design guide (MEPDG) for pavements with similar attributes.

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	SI (	METR	RIC) C	ON	/ER	SION	FACT	ORS	
Ap	proximate	Conversi	ons to SI l	Jnits	Аррі	roximate (	Conversio	ns from SI	Units
Symbol	When you know	Multiply by	To Find	Symbol	Symbol	When you know	Multiply by	To Find	Symbol
		LENGTH					LENGTH		
in	inches	25.40	millimeters	mm	mm	millimeters	0.0394	inches	in
ft	feet	0.3048	meters	m	m	meters	3.281	feet	ft
yd	yards	0.9144	meters	m	m	meters	1.094	yards	yd
mi	miles	1.609	kilometers	km	km	kilometers	0.6214	miles	mi
		AREA					AREA		
in²	square inches	645.2	square millimeters	mm	mm²	square millimeters	0.00155	square inches	in²
ft²	square feet	0.0929	square meters	m²	m²	square meters	10.764	square feet	ft²
yd²	square yards	0.8361	square meters	m²	m²	square meters	1.196	square yards	yd²
ac	acres	0.4047	hectares	ha	ha	hectares	2.471	acres	ac
mi²	square miles	2.590	square kilometers	km²	km²	square kilometers	0.3861	square miles	mi²
		VOLUME	I		VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL	mL	milliliters	0.0338	fluid ounces	fl oz
gal	gallons	3.785	liters	L	L	liters	0.2642	gallons	gal
ft³	cubic feet	0.0283	cubic meters	m³	m³	cubic meters	35.315	cubic feet	ft³
yd³	cubic yards	0.7645	cubic meters	m³	m³	cubic meters	1.308	cubic yards	уd³
		MASS					MASS		
oz	ounces	28.35	grams	g	g	grams	0.0353	ounces	oz
lb	pounds	0.4536	kilograms	kg	kg	kilograms	2.205	pounds	lb
т	short tons	0.907	megagrams	Mg	Mg	megagrams	1.1023	short tons	т
	(2000 lb)							(2000 lb)	
TEMPERATURE (exact)				TEMPE	RATURE	(exact)			
°F	degrees	(°F-32)/1.8	degrees	°C	°C	degrees	9/5+32	degrees	°F
	Fahrenheit	<b>、</b>	Celsius			Celsius		Fahrenheit	
FORCE and PRESSURE or STRESS			F	ORCE and	PRESSUR	E or STRES	SS		
lbf	poundforce	4.448	Newtons	Ν	N	Newtons	0.2248	poundforce	lbf
lbf/in²	poundforce per square incl	<b>6.895</b> h	kilopascals	kPa	kPa	kilopascals	0.1450	poundforce per square inch	lbf/in²

The contents of this report reflect the views of the author(s) who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the views of the Oklahoma Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation. While trade names may be used in this report, it is not intended as an endorsement of any machine, contractor, process, or products.

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# **Final Report**

# Submitted to:

Ginger McGovern Planning and Research Division Engineer Oklahoma Department of Transportation 200 N.E. 21<sup>st</sup> Street Oklahoma City, Oklahoma 73105

# **Prepared by:**

Naji Khoury<sup>1</sup>, Musharraf Zaman<sup>2</sup>, Rouzbeh Ghabchi<sup>3</sup> and Hassan Kazmee<sup>3</sup> <sup>1</sup>Assistant Professor of Civil and Environmental Engineering, Temple University, Philadelphia, Pennsylvania 19122

<sup>2</sup>David Ross Boyd Professor and Associate Dean for Research, College of Engineering <sup>3</sup>Graduate Research Assistant, School of Civil Engineering and Environmental Science The University of Oklahoma, Norman, Oklahoma 73019

# Submitted by:

Office of Research Services The University of Oklahoma 201 David L. Boren Blvd., Suite 150 Norman, Oklahoma 73019

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# **1.1 Introduction**

Presence of water can significantly influence the stiffness and stability of a pavement structure and is considered an important factor in the mechanistic design (Huang, 2004; Baumgardner, 1992; MacMaster et al., 1982). Water can enter a pavement structure through the top (infiltration through cracks and/or joints), bottom (rise of water table due to capillary action), and sides (lateral seepage of water from saturated ditches). If the penetrated water is not removed quickly, the base layer becomes saturated (TRB, 2010; Apul et al., 2002; Barksdale, 1996). Under wheel loading, the trapped water leads to excess pore-water pressure causing a loss in strength and increase in pavement distress. Trapped water contributes to pavement distresses such as pumping, D-cracking, faulting, blowup, frost action, shrinkage cracking, and potholes (Randolph et al., 2000; Cedergren, 1994; Forsyth, et al., 1987; Raad, 1982). Thus, lack of adequate drainage can lead to greatly reduced service life and premature failure in pavements.

Several methods have been introduced in the past to remove water from a pavement structure, including using an open-graded base as a drainage layer (Apul et al., 2002; Mallela et al., 2000). The Federal Highway Administration (FHWA) recommends the use of permeable base layers for all Interstate pavements (TRB, 2010; FHWA, 1990). The World Road Association requires permeable bases for all concrete pavements, while the U.S. Army Corps of Engineers requires permeable bases only for pavements over 200 mm (7.9 in) thick, making it optional for thinner pavements (Apul et al., 2002; USACE, 1992). Drainable base materials should not only possess high permeability but also enough strength and stiffness so as to sustain traffic loads (Liang et al., 2006). Some open graded base materials with high permeability (*k*)

and void ratio can have stability and stiffness issues. Consequently, study of stability and stiffness of drainable aggregate bases is an important topic for researchers, state agencies and industry (Blanco et al., 2004; Apul et al., 2002; Cedergren, 1988; Forsyth et al., 1987; Cedergren, 1974).

The Oklahoma Department of Transportation (ODOT) currently uses three different gradations (*ODOT Type A*, *ODOT Type B* and *ODOT Type C*) for its base layers (ODOT, 2009). A recent laboratory study by Khoury and Zaman (2007) revealed that each of these gradations has coefficients of permeability that are more than 1000 times lower than the values recommended by FHWA. Although the benefits of a permeable open-graded base layer are well recognized by ODOT and the industry, successful use of such designs is hindered by the lack of adequate laboratory and field data in Oklahoma. The present study was undertaken to address this gap. The study was supported jointly by ODOT and the Oklahoma Aggregate Association (OKAA).

#### **1.2 Need and Scope**

Aggregate base layer is an integral and important part of a pavement structure (Huang, 2004; Siswosoebrotho et al., 2005; Cheung and Dawson, 2002). Ideally, it should be designed to satisfy three requirements: (i) to provide adequate stability to the surface layer, (ii) to provide adequate drainage within the structure, and (iii) to reduce the time the drainage layer remains fully saturated to a relatively short duration (Liang et al., 2006).

From practical and economic considerations, it is difficult, if not impossible, to design a base layer that will never become fully saturated. Thus, it is important to design a base layer such that it will be able to provide adequate drainage whether fully or partially saturated, and thereby

limit the time the layer remains fully saturated (Khoury and Zaman, 2007; Crovetti and Dempsey, 1993).

With increased awareness of drainability, many states either have adopted or are in the process of adopting permeable bases as an integral part of a pavement structure. The required gradation and coefficient of permeability, however, remain uncertain (Kozeliski, 1992; Mathis, 1990; Baldwin, 1987). According to Cedergren (1994), an open-graded base/subbase layer should have a coefficient of permeability between 3.5 cm/sec (10,000 ft/day) and 35 cm/sec (100,000 ft/day). McEnroe (1994) noted that a base layer with a coefficient of permeability (k) of less than 0.017 cm/sec (48 ft/day) will be practically impermeable, whereas a layer with k values lower than 0.038 cm/sec (107 ft/day) will remain 85% saturated, and a base with k values of about 0.074 cm/sec (209 ft/day) will attain 50% drainability. According to FHWA (1990), a base layer with a minimum k value of 0.35 cm/sec (1,000 ft/day) should provide excellent drainage.

The coefficient of permeability (*k*) of aggregate bases, which dictates the drainability of a pavement structure, depends on various factors including gradation and particle packing of the aggregate, temperature and viscosity of the fluid, and degree of saturation (Huang, 2004; Das, 2002; Barksdale, 1996). Gradation characteristics are generally represented by percent passing No.200 sieve, effective diameter ( $D_{eff}$ ), particle size finer than 60% passing ( $D_{60}$ ), particle size finer than 10% passing ( $D_{10}$ ), particle size finer than 30% passing ( $D_{30}$ ), coefficient of uniformity ( $C_U$ ), and coefficient of gradation ( $C_C$ ) (Huang, 2004). The particle packing is generally taken into account using dry density, void ratio, and porosity of compacted aggregates in the base. Many of these factors are included in the present study.

Mineralogical composition is dependent on specific aggregate type and its source. Therefore, the coefficients of permeability of two similar aggregates obtained from two different quarries could be quite different (Khoury and Zaman, 2007; Randolph et al., 1996). Consequently, the applicability and accuracy of existing empirical models to estimate permeability of aggregate bases need to be investigated (Khoury and Zaman, 2007; Hatanaka et al., 2001; Fwa et al., 2001). In the present study, empirical models are developed for aggregates from three different sources, namely Anchor Stone, Dolese and Martin Marietta.

In 2002 AASHTO mechanistic design guide (AASHTO, 2002), structural stability of an aggregate base layer is measured in terms resilient modulus ( $M_R$ ) and layer coefficient (AASHTO, 2002). The gradation and compaction of representative specimens used to determine  $M_R$  in the laboratory must reflect in-situ gradation and compaction level. ODOT currently lacks laboratory data ( $M_R$  and k) for commonly used aggregates and gradations. Also, ODOT currently lacks field data on  $M_R$  and k of aggregate bases. To this end, this study examines the effect of gradation and compaction energy on  $M_R$  and k of aggregates from three commonly used sources in Oklahoma, namely Anchor Stone, Dolese and Martin Marietta.

To examine the effect of gradations on stability and drainage, three different gradations (*ODOT Type A*, *M-AASHTO #57* and *OKAA Type M*) are considered for each aggregate type. Two of these gradations (*M-AASHTO #57* and *OKAA Type M*) are being considered by ODOT and OKAA for possible field applications.

Originally this project was funded as a two-year laboratory study. In the first year (October 1, 2006-September 30, 2007), extensive laboratory tests (specific gravity, absorption, abrasion, gradation, moisture-density, resilient modulus, and permeability) were conducted on a limestone aggregate from Anchor Stone quarry in Tulsa, Oklahoma. In July 2007, the scope of the original study was changed in consultation with ODOT and Oklahoma Aggregate Association (OKAA) by adding a field component. A test section was constructed on Timberdell

Road in Norman with three different gradations (*M-AASHTO #57*, *OKAA Type M* and *ODOT Type A*). A different gradation was used in each sub-section. In addition to monitoring constructability, field tests (falling weight deflectometer (FWD), dynamic cone penetrometer (DCP) and permeability) were conducted during construction and after the test section was opened to traffic. Results from these field tests are reported here and compared with the laboratory results, when feasible.

# **1.3 Contents of this Report**

This report contains seven chapters and six appendices. Following introduction, need and scope in Chapter 1, a detail literature review is presented in Chapter 2. Materials used in this study, including their sources, geological features and gradations, are discussed in Chapter 3. Construction of the test section is summarized in Chapter 4. Pertinent laboratory and field test methods are summarized in Chapter 5, with an emphasis on the methods used in this study. Laboratory and field tests conducted and their discussions are presented in Chapter 6, while a summary of this study and pertinent conclusions and recommendations are presented in Chapter 7. Details of laboratory and field data are presented in appendices, as necessary.

## **2.1 Introduction**

The literature survey conducted in this study focused on the effects of gradation and compaction energy on the hydraulic conductivity and resilient modulus ( $M_R$ ) of aggregate bases. In addition this chapter contains a review of pertinent studies on measurements of permeability and determination of resilient modulus and the field test methods, falling weight deflectometer (FWD) and dynamic cone penetration (DCP), used in this study.

#### **2.2 Hydraulic Conductivity of Unbound Aggregate Bases**

## 2.2.1 Backgrounds

Moisture can enter a base layer through various sources including infiltration through surface cracks and joints, backflow of saturated side drainage, and rise of the water table (Apul et al., 2002; Barksdale, 1996). Infiltration of rain water through cracks and joints in the surface course is believed to account for most of the water in pavement (Crovetti and Dempsey, 1993; Cedergren, 1974). Three approaches have been identified that reduce the risks of excessive moisture in pavements: waterproofing the surface layer, use of surface drainage, and use of subsurface drainage within the pavement. Of these, use of subsurface drainage is considered most effective (Flynn, 2000; Mallela, 2000). Consequently, a large number of previous studies have focused on the use of subsurface drainage systems and the use of open-graded base layers to address pavement drainage (AASHTO, 2002).

The drainage efficiency of an unbound base layer is characterized by its hydraulic conductivity, which depends on such factors as aggregate gradation, shape, abrasion and

compaction. Viscosity of the fluid and degree of saturation are also important to pavement drainage (Das, 2002; Barksdale, 1996).

According to the Colorado Department of Transportation (CDOT) Drainage Design Manual (2004), drainage layers should be designed to ensure removal of at least 50% of the water within the system (under fully saturated conditions) within the first hour after a rainfall event. ODOT recommends a minimum permeability of 1,000 ft/day to achieve this level of drainage. ODOT also recommends that a base layer be composed of crushed angular aggregates with 100% passing the 1½ in. sieve and less than 2% passing the No. 16 sieve.

## 2.2.2 Laboratory Measurements of Hydraulic Conductivity

Hydraulic conductivity of granular materials is generally determined in the laboratory using either a constant head test or a falling head test. A constant head method (AASHTO T 215-70) requires a steady flow of water under a given hydraulic head. This method, however, may not be suitable for unbound aggregates having large permeability because the setup would require large reservoirs at both ends of a specimen. Also, maintaining constant water levels at both ends would require precise flow control (Fwa et al., 1998). On the other hand, in a falling head test using ASTM D 5084 – 00 test standard (ASTM, 2003), both the hydraulic gradient and the specific discharge vary with time, making it easier to conduct this test on a compacted aggregate base specimen. The falling head test method was used in this study to measure the hydraulic conductivity of compacted aggregate specimens.

Determination of hydraulic conductivity is generally based on Darcy's law that assumes laminar flow through a specimen (Das, 2002):

$$q = \frac{Q}{A} = ki \tag{2.1}$$

Where, Q = volumetric discharge in cm<sup>3</sup>/sec (ft<sup>3</sup>/day),

2-2

q = specific discharge in cm/sec (ft/day),

 $A = \text{cross-sectional area in cm}^2$  (ft<sup>2</sup>),

k = coefficient of permeability in cm/sec (ft/day), and

i = hydraulic gradient in cm/cm (ft/ft).

Although laminar flow is used widely, several researchers have noted that flow through unbound aggregates can become turbulent; therefore, Equation (2.1) may not apply (Barksdale, 1996; Jones and Jones, 1989; Scheidegger, 1963; Muskat, 1937). Consequently, modifications to Darcy's equation have been suggested to account for turbulent flow (see e.g., Fwa et al., 1998; Bear, 1972; Scheidegger, 1963; Muskat, 1937). For example, Fwa et al. (1998) used the following modified equation to analyze the results of their falling head permeability tests:

$$v = k_1 \cdot i^n \tag{2.2}$$

where, v = specific discharge velocity in cm/sec (ft/day),

 $k_1$  = experimental coefficient in cm/sec (ft/day), and

n = a second coefficient (unitless).

For laminar flow, n becomes equal to 1 and Equation (2.2) becomes identical to Equation (2.1). For turbulent flow, n is generally assumed to be 0.5 (Fwa et al., 1998).

A commonly used formula for determining the coefficient of permeability when using the falling head approach in the laboratory is:

$$k = 2.303 \ \frac{aL}{At} \log \frac{h_1}{h_2}$$
(2.3)

where, a = cross-sectional area of top cylinder in cm<sup>2</sup> (ft<sup>2</sup>),

L =length of the specimen in cm (ft),

A =cross-sectional area of soil specimen in cm<sup>2</sup> (ft<sup>2</sup>),

t = time required for water to drop from  $h_1$  to  $h_2$  in sec,

 $h_1$  = initial head difference at t = 0 in cm (ft), and

 $h_2$  = final head difference at  $t = t_2$  in cm (ft) (Das, 2002).

Equation (2.3) does not take into account turbulent conditions encountered in flow through unbound aggregates and thus may yield inaccurate results for open-graded aggregate bases. Use of Equation (2.2) may be more appropriate for such cases (Fwa et al., 1998).

## 2.2.3 In-Situ Measurements of Hydraulic Conductivity

In many cases, determination of the coefficient of permeability k from laboratory tests on small specimens may not be representative of the overall field conditions. In-situ permeability tests, on the other hand, allow researchers to test a much larger volume of materials and include flow through secondary features such as macropores, fissures, and slickenside in a manner that cannot be simulated properly in small laboratory specimens (Daniel, 1989). The equations discussed in the preceding section were developed for one-dimensional flow, in which water (or another fluid) travels in a straight line perpendicular to the cross-sectional area of a specimen. In field permeability tests, however, the flow is primarily radial, and the linear flow assumptions are usually not valid (Daniel, 1989; Thiruvengadam et al., 1997). A common method used to determine permeability in the field involves measurements of the change in water levels in open standpipes. This method was used in this study and will be discussed subsequently in this report.

### 2.2.3.1 Boutwell Permeameter Theory

Daniel (1989) reviewed a number of in-situ permeability test methods. According to that review, one of the common field methods is the Boutwell borehole test (Boutwell and Derick, 1986). This test is based on the concept that by varying the geometry of the wetted zone, the relative effect of vertical and horizontal conductivities can be varied in a specified manner. The borehole is drilled and a casing is placed and sealed, as depicted in Figure 2.1. After conducting the falling head permeability test, hydraulic conductivities for each stage are calculated using Equations (2.4) and (2.5), suggested by Hvorslev (1949).

$$k_1 = \frac{\pi d^2}{11D(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$$
(2.4)

$$k_2 = \frac{A}{B} \ln\left(\frac{H_1}{H_2}\right) \tag{2.5}$$

where A and B are calculated from the following equations:

$$A = d^2 \left\{ \ln \left[ \frac{L}{D} + \sqrt{1 + \left(\frac{L}{D}\right)^2} \right] \right\}$$
(2.6)

$$B = 8D \frac{L}{D} (t_2 - t_1) \left\{ 1 - 0.562 \exp\left[-1.57 \left(\frac{L}{D}\right)\right] \right\}$$
(2.7)

The degree of anisotropy of the media, *m*, is expressed as:

$$m = \sqrt{\frac{k_h}{k_v}} \tag{2.8}$$

Using Equations (2.6) and (2.7), Equations (2.4) and (2.5) can be written as a ratio as:

$$\frac{k_1}{k_2} = \frac{\ln\left[\frac{L}{D} + \sqrt{1 + \left(\frac{L}{D}\right)^2}\right]}{\ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}$$
(2.9)

where  $k_h$  and  $k_v$  represent the hydraulic conductivity in the horizontal and vertical directions, respectively. The value of *m* can be found using Figure 2.2. Knowing *m*, the horizontal and vertical permeabilities can be calculated from the following equations:

$$k_h = m k_1 \tag{2.10}$$

$$k_v = \frac{1}{m} k_1 \tag{2.11}$$

Where,  $k_1$  and  $k_2$  are the permeability values calculated from Stage 1 and Stage 2 of the Boutwell test (Boutwell and Derick, 1986). The formulations above are developed for clayey soils where consolidation of structured layers can influence the permeability. In other words, in Figure 2.1, Case (a) represents a situation where vertical flow is dominant and Case (b) a situation where horizontal flow (radial) is dominant. An aggregate base, however, does not involve any consolidation. Consequently, the effect of permeability is considered equal in both horizontal and vertical directions. Due to the nature of the flow in an aggregate base, radial flow will be dominant (Thiruvengadam, 1997), making Case (b) of the Boutwell test appropriate for evaluation of permeability. Case (b) also allows for the fact that some water would permeate through the bottom of the casing. Considering the flow of water in the aggregate base during field testing, the amount of flow through the bottom of the borehole would be negligible because the permeability of the subgrade soil will be considerably lower than that of the base layer. For these reasons, the use of the Boutwell method may not be a good choice for determination of insitu permeability of an aggregate base.

## 2.2.3.2 Porous Probe Theory

Porous probes are pushed or driven into the soil. Both falling and constant head tests can be performed using a porous probe. Figure 2.3 shows the porous probe discussed by Olson and Daniel (1981). As described by Daniel (1989), the following equation could be used to determine permeability from a falling head test:

$$k = \frac{\pi d^2 / 4}{F(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$$
(2.12)

In Equation 2.12, shape factor F is a function of the casing length (L), which is equal to the thickness of the aggregate base layer in the present study, d represents the diameter of the

standpipe and D represents the diameter of the borehole. The parameter F is calculated for Case A and Case B as follows (Daniel, 1989):

Case A: 
$$F = \frac{2\pi L}{\ln\left[\frac{L}{D} + \sqrt{1 + \left(\frac{L}{D}\right)^2}\right]}$$
(2.13)

Case B: 
$$F = \frac{2\pi L}{\ln\left[\frac{L}{D} + \sqrt{1 + \left(\frac{L}{D}\right)^2}\right]} - 2.8 D$$
(2.14)

Case B is close to the geometry and mechanics of the falling head field permeability test performed for the aggregate base in the present study. The thickness of the aggregate base layer will be used as length L, shown in Figure 2.3(b). Also, the permeability of the stabilized subgrade layer beneath the aggregate base is considerably lower than that of the base layer. Therefore, Case B is closer to the in-situ permeability measurements conducted in this study than Case A.

As discussed by Das (1976), to derive Equation (2.12), by equating the discharge rate through the stand pipe and discharge rate from the permeable media as in Equation (2.15).

$$q = kiA = -a\frac{dh}{dt} \tag{2.15}$$

where  $a = \text{stand-pipe cross-sectional area in ft}^2$ ,

q = specific discharge in ft/day,

- A = area of aggregate base perpendicular to flow ft<sup>2</sup>,
- L = thickness of aggregate base in ft,

 $h_1, h_2$  = primary and secondary readings of water head in stand pipe in ft,

t = time of change in head in stand-pipe in sec,

k = coefficient of permeability in ft/day, and

i = hydraulic gradient in ft/ft.

Considering the shape of the flow boundaries beneath the stand pipe, depicted in Figure 2.4, the continuity equation can be rewritten as:

$$k\frac{h}{R_e}\pi DL = -\frac{\pi d^2}{4} \cdot \frac{dh}{dt}$$
(2.16)

Let

$$F = \frac{\pi DL}{R_e} \tag{2.17}$$

Then

$$k dt = -\frac{\pi d^2/4}{F} \cdot \frac{dh}{h}$$
(2.18)

Integrating Equation (2.18) for the time interval  $t_1$  and  $t_2$  and the corresponding height intervals  $h_1$  and  $h_2$ , one can obtain the following equation:

$$k \int_{t_1}^{t_2} dt = -\frac{\pi d^2/4}{F} \int_{h_1}^{h_2} \frac{dh}{h}$$
(2.19)

The corresponding permeability is given by:

$$k = \frac{\pi d^2 / 4}{F(t_2 - t_1)} \ln \frac{h_1}{h_2}$$
(2.20)

Equation (2.20) is the same as Equation (2.12) in form, as suggested by Daniel (1989), and very similar to that of Hvorslev (1949).

# 2.2.3.3 Permeameter used by Bouchedid and Humphrey (2005)

A similar method was used by Bouchedid and Humphrey (2005) to measure the field hydraulic conductivity of a granular base. The device used by these researchers to measure permeability is shown in Figure 2.5. Equation (2.21) will be used in Chapter 5 of this report to calculate field permeability using the falling head method.

$$k = \frac{q}{CrH}$$
(2.21)

where q = flow rate,

C =conductivity coefficient,

r = radius of the cavity,

H = d + P + L = pressure head,

d = head measured from surface of pavement,

P = pavement thickness, and

L = depth of hole measured from bottom of pavement.

In Equation (2.21), the conductivity coefficient *C* is assumed to be a dimensionless constant that depends on the shape of the cavity. For a circular-shaped well, *C* is considered to be approximately 22. The validity of this formula is limited to  $H/r \ge 5$  and groundwater deeper than 7H, if 15 cm <r<50 cm (6 in. < r < 20 in.). This method determines the order of magnitude of the permeability (Lacroix, 1960).

A comparison of Equations (2.21) and (2.20) reveals that factor F in Equation (2.20) is similar to C in Equation (2.21). F, however, is calculated using Equations (2.13) and (2.14), and varies with L (thickness of aggregate base) and D (diameter of the standpipe), while C is assumed to be 22 for a range of variation in H (pressure head) and r (radius of the cavity). Thus, it is evident that Equation (2.20) is likely to provide a better estimation of hydraulic conductivity than Equation (2.21).

## 2.2.4 Permeability of Aggregate Bases

Shah (2007) studied the effect of five different gradations including M-AASHTO #57, OKAA Type N, OKAA Type K, M-AASHTO #67 and ODOT Type A and compaction energy on the hydraulic conductivity and resilient modulus of Anchor Stone aggregates. In that study, it was observed that the lower limit of Modified M-AASHTO #67 gradation compacted using the standard Proctor method had the highest coefficient of permeability (approximately 0.62 cm/sec or 1,777 ft/day). The  $M_R$  value obtained for this gradation was approximately 235 MPa (34 ksi) at a confining pressure of 69 kPa (10 psi) and a bulk stress of 276 kPa (40 psi). ODOT Type A gradation compacted using the standard Proctor method had a range of permeability from  $1.8 \times 10^{-6}$  cm/sec to 0.0046 cm/sec (0.005 to 13 ft/day), whereas the modified Proctor method changed the coefficient of permeability range to  $1.376 \times 10^{-4}$  cm/sec and  $1.4 \times 10^{-4}$  cm/sec (0.39 and  $3.23 \times 10^{-7}$  ft/day). The study determined that lower limit *M-AASHTO* #57 and lower limit M-AASHTO #67 gradations were recommended as drainable bases. (1990). It was also found that the lower limit of a gradation, which is coarser, has a higher coefficient of permeability than the upper limit for all the gradations evaluated. No marked difference was observed in  $M_R$  values amongst the gradations tested although the modified Proctor method produced higher  $M_R$  values compared to the standard effort for all the gradations evaluated.

Khoury and Zaman (2007) evaluated the effect of three gradations on the modulus and permeability of two limestone aggregate bases widely used in Oklahoma. The gradations used in that study consisted of upper limit *ODOT Type A*, lower limit *ODOT Type B*, and lower limit *ODOT Type C*. The moisture-density relationships were obtained in accordance with the AASHTO T 180-01 test method. The degree of compaction achieved varied from 92% to 100% of maximum dry density. Khoury and Zaman (2007) also found that the permeability values

obtained for these three gradations were lower than those obtained from existing models. This raises questions concerning the accuracy and applicability of the existing empirical models for estimating permeability. Also, it suggests the dependency of permeability on the mineralogical and physical properties of specific aggregate bases, as noted previously by Hatanaka et al. (2001), Fwa et al. (2001), and Randolph et al. (1996). Type B gradation, which is the coarsest gradation of the three, had the highest coefficient of permeability values, whereas Type A, the finest gradation, had the lowest. It was also noted that the most permeable of the three gradations, Type B, had a coefficient of permeability that was at least 1000 times lower than that recommended by FHWA (Khoury and Zaman, 2007). It was determined that there is a need for further studies on the effect of compaction on aggregate degradation.

Bouchedid and Humphrey (2005) performed laboratory and in-situ hydraulic conductivity tests on field cores as well as laboratory molded specimens. It was noted that the hydraulic conductivity of the base layer decreased with an increase in both fines content (% passing No. 200 sieve) and the coefficient of uniformity. Gradation analysis showed that for the Maine Department of Transportation Type D gradation, fines increased by a factor of 1.7 after 12 years in service. Bouchedid and Humphrey (2005) correlated percent of fines (*F*) and coefficient of uniformity (*C*<sub>U</sub>) to the coefficient of permeability (*k*) determined via triaxial permeability tests showing a coefficient of determination ( $\mathbb{R}^2$ ) of 0.63. The results are shown in Table 2.1. This correlation is only valid for compacted and semi-rounded aggregate particles with the percent passing No. 200 sieve between 3 and 14 and a coefficient of uniformity between 10 and 80. According to the study, the use of a permeable base was expected to increase pavement life in Maine by 271%, thereby providing a savings of approximately \$400,000 per mile of pavement (Bouchedid and Humphrey, 2005). Siswosoebrotho et al. (2005) examined the influence of the amount of fines and the plasticity index of these fines on the strength and permeability of base courses. It was noted that aggregates, with or without fines, gained strength from grain-to-grain contact; however, aggregate gradations with enough fines to fill all the voids were found to increase the shear resistance. Permeability measurements were conducted using the falling head approach. It was concluded that permeability decreased with the increase in fines and plasticity index. Use of 4% passing No. 200 sieve in a gradation reduced the permeability values considerably; however, specimens with more than 4% fines exhibited less reduction in permeability. These researchers also noted that aggregate gradation and other properties play a vital role on the performance of granular bases.

In a related study, Blanco et al. (2004) assessed the drainage and strength characteristics of an aggregate base, gradation Type 5, commonly used by the Missouri Department of Transportation. Laboratory hydraulic conductivity values were in the range of  $9 \times 10^{-2}$  to  $7 \times 10^{-7}$  cm/sec (255 to  $2 \times 10^{-3}$  ft/day). Although the need for effective drainage is well known, the study found that an aggregate base with Type 5 gradation did not satisfy the permeability requirements typically recommended for good drainage.

Tan et al. (2003) studied the effect of clogging of permeable bases by introducing sand and residual soil over the specimens and flushing them with water to allow these soils to settle within the specimen. As expected, they observed that the hydraulic conductivity of specimens decreased with an increase in the amount of clogging soils. Limiting the number of particle sizes within a specimen increased the amount of clogging soil needed to completely fill all the voids, and reduced the permeability of the specimen considerably. Parra and Blanco (2002) assessed the drainability of a well-graded aggregate by performing a series of constant head permeability tests. The hydraulic conductivity values obtained from the laboratory tests were compared with the values obtained via commonly used expressions, namely the Hazen equation (Hazen, 1930), Sherard equation (Sherard et al., 1984), and Moulton equation (Moulton, 1980). It was concluded that the Hazen and Sherard equations often over-predict field hydraulic conductivity values, whereas the Moulton equation better estimates the field hydraulic conductivity values of aggregate bases.

Hatanaka et al. (2001) studied the permeability characteristics of gravelly soils by conducting permeability tests on high quality undisturbed gravel specimens using a large-scale triaxial cell. The constant head test was used to measure the vertical and horizontal permeability of gravelly soil specimens. Hatanaka et al. (2001) observed that the effect of confining pressure on permeability is directly related to the effect of void ratio. Their study revealed that the presence of the large size particles improve the permeability. It was concluded that the in-situ permeability of gravelly soils could be estimated from reconstituted samples in the laboratory. In addition, no relationship was evident between the physical properties and the hydraulic conductivity of the specimens.

Mallela et al. (2000) developed a framework for the consideration and design of subsurface drainage systems in jointed concrete pavements. The decision to include drainage in a pavement structure should be based on cost effectiveness. The California Department of Transportation recommends the use of permeable bases under all new concrete pavements, while the Wisconsin Department of Transportation recommends basing the design of pavement structures on the traffic loads. The Minnesota Department of Transportation uses a detailed approach, using subgrade class, traffic load and volume, pavement type, and functional

classification in its decision making. Mallela et al. (2000) also provided a means for determining the stability of an aggregate base using gradation characteristics. These researchers noted that stability of a base increased with an increase in the coefficient of uniformity ( $C_U$ ). Based on their research on untreated permeable bases, they recommended the use of a gradation with a  $C_U$  value greater than 4 and a coefficient of gradation ( $C_C$ ) value between 0.6 and 1.6. Mallela et al. (2000) also showed that the presence of a permeable layer would not guarantee sustained pavement life if the design did not account for the expected inflow of water to the system.

Fwa et al. (1998) designed a laboratory experiment for permeability measurements based on the falling head approach. Results obtained via the falling head approach were confirmed using the constant head approach. The results obtained from these two approaches were nearly identical, thus signifying the success of the falling head approach. Velocity (v) was plotted against hydraulic gradient (i), and the coefficient of permeability ( $k_I$  in cm/sec) was obtained using regression models. Fwa et al. (1998) modified Darcy's equation, which is generally used for laminar flow, to better simulate the turbulent flow commonly observed in aggregate bases. However, in their study, the flow in the fine aggregates (retained on No. 50 sieve) was close to laminar, whereas the flow in glass spheres (11 mm and 16 mm diameter) was turbulent.

Tandon and Picornell (1997) emphasized the need for using drainage requirements in evaluating aggregate bases for the design of pavement structures. Tandon and Picornell (1997) recommended the use of *M*-AASHTO #57 and #67 gradations as base layers. While open-graded drainage layers would certainly increase the permeability of aggregate bases, these gradations might not have adequate stiffness and strength due to the lack of mechanical interlock of the coarse aggregates.

Randolph et al. (1996) developed a large-scale permeameter and a testing procedure to evaluate the hydraulic conductivity of six base and subbase gradations made up of three different base types. Randolph et al. (1996) noted that although the aggregate base should be compacted vertically, the hydraulic conductivity test should be run horizontally because the flow through pavement bases is mainly horizontal. Their research provided a range of permeability values for the gradations tested. For No. 57 medium gradation of limestone, a coefficient of permeability value of 12 cm/sec (34,600 ft/day) was obtained, whereas for the limestone No. 67 medium gradation, a value of 17 cm/sec (48,000 ft/day) was obtained. Both gradations, thus, satisfied the requirements set forth by FHWA (1990). However, their research did not compare the effect of compaction energy on the density and the hydraulic conductivity of the aggregate bases. The compaction energy applied to compact the specimens would most certainly provide different optimum moisture contents and maximum dry densities. These properties of base gradations would, in turn, influence the hydraulic conductivity of the compacted specimen.

Elsayed (1995) studied the effects of large-sized aggregates and asphalt stabilization on the permeability of aggregate bases. Three different aggregates were tested for the determination of the coefficient of permeability. Elsayed (1995) strongly recommended the use of Reynolds number for determining the transition from laminar flow to turbulent flow. Particle migration issues were encountered when using two different permeameters, and thus, Elsayed (1995) chose to use the Barber and Sawyer (1952) permeameter, which was designed to reduce particle migration problems. Using the coefficients of permeability for all three aggregates, Elsayed (1995) developed regression models for unbound aggregates and stabilized aggregates. The regression model for the unbound aggregates had a  $R^2$  of 0.78 and was assumed to be able to predict the coefficient of permeability of unbound aggregates within the range of 0.18 cm/sec to 0.71 cm/sec (500 to 2,000 feet/day). This model is shown in Table 2.1. The equation correlates the coefficient of permeability with the void ratio, percent passing No. 30 sieve, and percent passing No. 200 sieve. No considerable differences in the coefficients of permeability were found between base with a top size of 38 mm (1.5 in) and that with a top size of 63 mm (2.5 in).

A prominent advocate for permeable layers, Cedergren (1994) described a pavement as the most unusual structure designed by Civil Engineers. Since pavements are designed and constructed relatively flat, water enters the structure through the top, bottom, and sides but drains out very slowly. Furthermore, Cedergren (1994) emphasized that a good internal drainage system within a pavement structure would at least triple the life of the whole structure. It was also noted that an open-graded drainage layer having a coefficient of permeability between 3.5 cm/sec (10,000 ft/day) and 35 cm/sec (100,000 ft/day) should be used as base/subbase layers in pavement structures.

Crovetti and Dempsey (1993) focused their efforts on examining the effects of cross slope, longitudinal gradient, and drainage layer dimensions on the hydraulic conductivity of a base layer. The effective grain size, porosity, and percent fines were found to be the most significant properties affecting hydraulic conductivity. Crovetti and Dempsey (1993) also indicated a need to study density requirements during the construction of a base layer, construction stability, and degradation of particles during the service life of a pavement. Their study also emphasized the need to assess the effect of aggregate degradation on permeability.

Hajek et al. (1992) evaluated the field performance of open-graded drainage layers on the performance of pavement structures. It was suggested that incorporating a permeable layer within a pavement structure did not automatically guarantee increased pavement life. The entire internal drainage system incorporating the permeable layer should be designed with sufficient

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permeability for better pavement performance. Increased fines content in gradations increased the likeliness of clogging within the aggregate base layer, which in turn reduced the hydraulic conductivity of this layer.

Liang and Lytton (1989) developed a comprehensive model for predicting the effects of air temperature, wind speed, rainfall, frost, and thawing actions on the performance of pavements. They divided the United States into nine climatic regions for simulation patterns that could be used in pavement design and analysis. From their data, Oklahoma is placed in region II-B, which means that there are moderate chances of moisture being present in pavement structures during a typical year. Category B indicates the presence of freeze-thaw cycles in pavement surface and base and occasional moderate freezing of the subgrade (Liang and Lytton, 1989). The model proved to be realistic in simulation purposes and thus shows the need to remove moisture from within pavement structures in Oklahoma, thereby reducing the effects of the freeze-thaw cycles.

Marek (1977) demonstrated via experiments that the aggregate gradations of the base/subbase layers significantly influence the density obtained with any compaction effort. According to this study, strength characteristics of the granular base improved with an increase in density, however, increased density obtained through the addition of excess fines proved detrimental to the strength characteristics of the base.

## 2.2.5 Existing Coefficient of Permeability Models

Carrier (2003) discussed two regression models that are widely used to calculate the hydraulic conductivity of sands and porous media. He also showed that the century-old Hazen formula provided less accurate results than the Kozeny-Carman formula. Both equations are shown in Table 2.1. The Kozeny-Carman formula takes into account many important factors

affecting permeability including the void ratio of the specimen, diameter of the particles in the specimen, and the shape factor of the particles. Carrier (2003) suggests that although the Kozeny-Carman formula is a better approximation of the hydraulic conductivity of porous media, this formula is not free of inaccuracies. Therefore, additional tests need to be conducted to modify the Kozeny-Carman formula to better approximate hydraulic conductivities of granular bases.

Kamal et al. (1993) conducted permeability tests on unbound granular aggregates and developed an equation (Table 2.1) correlating the coefficient of permeability with the effective particle size  $(D_{10})$ , particle size at which 20% of the base by weight is smaller  $(D_{20})$ , and the void ratio. Tests were performed on eight gradations ranging from well graded to open graded mixes.

Sherard et al. (1984) developed a simple equation correlating permeability with particle size at which 15% of the base by weight is smaller ( $D_{15}$ ). This equation, as shown in Table 2.1, was determined by conducting 6 tests on each of 15 different sands and gravels. Using a similar method Hazen (1930) developed an equation correlating the coefficient of permeability with the effective grain size ( $D_{10}$ ) of saturated sands. The disadvantage of both these equations is their dependency on gradation without considering the degree of packing or porosity.

Moulton (1980) developed a regression equation (shown in Table 2.1) from statistical analysis and a nomograph correlating the coefficient of permeability to the effective particle size  $(D_{10})$ , porosity, and percent passing No. 200 sieve. The equation was derived using unbounded aggregate bases using a specific gravity value of 2.70.
### 2.3 Resilient Modulus of Unbound Aggregate Bases

#### 2.3.1 Introduction

Structural stability of a base layer can be evaluated in the laboratory by determining the resilient modulus ( $M_R$ ) of compacted specimens. Deformation of a pavement structure under vehicular traffic loading is generally divided into two components, namely resilient (elastic or recoverable) and plastic (permanent or irrecoverable). After numerous applications of traffic loading, the increment of irrecoverable deformation becomes much smaller than the increment of recoverable deformation. Therefore, the recoverable deformation or resilient deformation of each layer becomes important in pavement design. These resilient characteristics of unbound aggregates are used in the design of the base layer. Mathematically, resilient modulus ( $M_R$ ) is defined as:

$$M_R = \frac{\sigma_d}{\varepsilon_r} \tag{2.22}$$

where,  $\sigma_d$  = applied deviator stress in MPa (psi), and

### $\varepsilon_r$ = recoverable or the resilient strain in m/m (in/in).

Over the past two decades, the flexible pavement design philosophy has shifted from an empirical approach to a more mechanistic approach, including using resilient modulus to relate stress-strain response under cyclic loading (Timm and Priest, 2006; Ashteyat, 2004; AASHTO, 2002). In 1982, AASHTO introduced the T 274-82 test method for determination of  $M_R$ . This method was modified in 1992 (Nazarian and Feliberti, 1993; Claros et al., 1990; Ho, 1989). Among the improvements, the modified test method (AASHTO T294-92) better simulated the loading characteristics of pavement structures and introduced separate procedures for subgrades and bases. It used a haversine waveform to simulate traffic loading and required external LVDTs to measure deflections accurately during testing. This test method was further modified in 2000

(ASHTO T 307-99) to use different loading and confining stresses and measurement devices with increased accuracy (Ping and Ling, 2007).

The repeated load triaxial test is one of the most commonly used test methods for determination of the resilient modulus (Titi et al., 2006). In a repeated load triaxial test, a specimen is subjected to cyclic stress (haversine-shaped load pulse) and static-confining pressure in a triaxial chamber. Several factors affect the resilient modulus of unbound aggregate bases, including stress conditions, moisture conditions, density, and geological features (Khoury and Zaman, 2007; Richter, 2006; Tian et al., 1998; Zaman et al., 1994; Karasahin et al., 1993).

For design purposes, resilient modulus is generally expressed as a function of state of stress. The AASHTO Pavement Design Guide (2002) recommends the use of the following general model for  $M_R$ :

$$M_R = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3}$$
(2.23)

where,  $M_R$  = resilient modulus in MPa (psi),

 $\theta$  = bulk stress =  $\sigma_1 + \sigma_2 + \sigma_3$  in MPa (psi),

 $\sigma_1$  = major principal stress in MPa (psi),

 $\sigma_2$  = intermediate principal stress in MPa (psi),

( $\sigma_2 = \sigma_3$  for cylindrical specimens),

 $\sigma_3$  = minor principal stress = confining pressure in MPa (psi),

 $P_a$  = atmospheric pressure in MPa (psi),

 $\tau_{oct}$  = octahedral shear stress in MPa (psi), and

 $k_1, k_2, k_3 =$  regression constants.

The octahedral shear stress is related to the principal normal stresses as follows:

$$\tau_{oct} = \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2}$$
(2.24)

The three regression coefficients are determined outside the design guide software with  $k_1$  and  $k_2$  being generally positive and  $k_3$  negative. A coefficient of determination (R<sup>2</sup>) of 0.90 or more should be obtained, if not the test results and equipment should be checked (AASHTO, 2002).

The behavior of granular soils has often been correlated with bulk stress, as shown in Equation (2.25).

$$M_R = k_1 \theta^{k_2} \tag{2.25}$$

where,  $M_R$  = resilient modulus in MPa (psi),

 $k_1$  and  $k_2$  = regression constants where  $k_1$  is in MPa (psi), and

 $\theta$  = bulk stress in MPa (psi) (Ping et al., 2001).

Unbound aggregate bases have  $k_1$  values in the range of 10 MPa to 82 MPa (1,500 psi to 12,000 psi) and  $k_2$  values in the range of 0.3 to 0.7 (Richter, 2006; Ping et al., 2001).

# 2.3.2 Resilient Modulus of Aggregate Bases

Khoury and Zaman (2007) evaluated the effect of three gradations on the modulus of two limestone aggregate bases used in Oklahoma. It was observed that the resilient modulus increased with an increase in bulk stress and confining pressure. Higher resilient modulus values were observed for denser gradations, as expected. A statistical correlation was reported between the resilient modulus value and  $C_U$ ,  $C_C$ , and percent passing No. 200 sieve. The statistical model had an R<sup>2</sup> value of 0.95 and is shown in Table 2.2. It was evident that resilient modulus was sensitive to fines (percent passing No. 200 sieve).

In evaluating different laboratory compaction methods for preparation of cyclic triaxial samples, Hoff (2004) compared four different compaction techniques. Samples compacted using

the modified Proctor method showed less resistance to permanent deformation as compared to those compacted by the other methods (vibratory hammer, vibratory table, and gyratory compactor). This is due to the fact that modified Proctor would produce a dense specimen compared to other compaction techniques.

Ping et al. (2001) used AASHTO T 292-97 (AASHTO, 2002) test procedure to compare laboratory-determined resilient modulus values under both field and laboratory conditions. The specimens were compacted using the modified Proctor values (*OMC* and  $\gamma_{d max}$  values) in order to simulate field conditions immediately after construction. These researchers also compared the laboratory-determined *OMC* values with those determined in-situ and found that while the laboratory-determined MDD values were slightly higher than the in-situ values, the values were comparable under normal environmental conditions. The comparison also showed that the laboratory compacted specimens had an average resilient moduli of 1.1 times higher than the average resilient moduli of the excavated specimens. This could, however, be attributed to the higher densities achieved in the laboratory compaction. It was concluded that the laboratorydetermined resilient modulus of the compacted specimens could represent the actual resilient behavior of a granular base in flexible pavements.

Zaman et al. (1994) evaluated the resilient modulus of six commonly used base/subbase aggregate gradations in Oklahoma. The effects of gradation, compaction method, specimen size, and testing method on the resilient modulus were incorporated. It was concluded that the gradation influenced the density of aggregate bases; however, stress state would be much more significant in determining the resilient modulus values than gradation. Resilient modulus values varied in the range of 20% to 50% between different aggregate types when gradation and bulk stress was constant.

Kamal et al. (1993) conducted repeated load triaxial tests to evaluate the resilient modulus of aggregate bases. Resilient modulus values increased with increased particle size, confining pressure, and deviatoric stress. This suggested the dependency of the resilient modulus on gradation and applied stresses. The resistance to permanent deformation was also less for the open-graded specimens, as compared to the well-graded specimens.

Raad et al. (1992) observed that the open-graded bases are more resistant to pore water pressure build-up than dense-graded specimens. Their comparison of the resilient modulus values of dense-graded and open-graded specimens revealed that dense-graded specimens had a higher resilient modulus value, but dense-graded specimens were also susceptible to excess pore water pressure development under undrained conditions.

Barksdale and Itani (1989) found that rounded gravel had lower resilient modulus values than angular particles and that increase in the fines content resulted in lower resilient modulus values. Increasing the density from 95% to 100% increased the resilient modulus values by 50% to 160% at a low bulk stress of 0.103 MPa (15 psi) and by 15% to 25% at a high bulk stress of 0.690 MPa (100 psi).

Thom and Brown (1987) carried out resilient modulus tests on crushed rock aggregates with changes in grading, degree of compaction, and moisture content. A decrease in stiffness with increase in the moisture content was observed due to the development of pore pressures. However, pore pressures only developed at saturation levels greater than 85%. It was also observed that stiffness increased when the specimen was dried. Thom and Brown (1987) also reported that the effect of moisture would be more apparent with an increase in the fines content. They noted that density had a relatively small influence on the resilient modulus values of crushed aggregates.

Rada and Witczak (1981) analyzed the results of more than 270 separate resilient modulus tests on granular bases. They observed that the regression parameters (i.e.  $k_1$  and  $k_2$ ) in the bulk stress model (Equation 2.7) had an inverse relationship for granular bases and that different aggregate types had different  $k_1$  and  $k_2$  values.  $k_1$  values increased with an increase in the compaction effort. It was also observed that  $k_1$  values increased with an increase in fines content up to an optimum amount of fines, beyond which it decreased. Rada and Witczak (1981) also found a critical degree of saturation at 80-85% above which the base bases exhibited lower resilient modulus values. They determined that the effect of moisture content could change the  $k_1$  values from 20.68 MPa to 6.89 MPa (3,000 psi to 1,000 psi) as the specimen moved from a dry state to a saturated state. This would further change the resilient modulus values from 275 MPa to 69 MPa (40,000 psi to 10,000 psi). It was also stated that increasing compaction density results in a corresponding increase in the resilient modulus.

Hicks and Monismith (1971) evaluated the effect of the degree of saturation on the regression constants in the bulk stress model (Equation 2.7). Well-graded, sub-angular, partially crushed gravel and rock were used in their study. As specimens changed states from relatively dry to partially saturated,  $k_1$  values decreased whereas  $k_2$  values barely changed. These researchers further evaluated the effect of gradation, particle shape and density on  $k_1$  and  $k_2$  values. It was observed that  $k_1$  values decreased as the fines content increased for the partially crushed aggregate. However, for the crushed aggregate, an opposite trend of an increase in  $k_1$  values with increasing fines content was observed.  $k_1$  values were always larger for the crushed aggregates than for the partially crushed base. It was also observed that  $k_1$  values increased with increasing density, whereas  $k_2$  values remained relatively constant.

#### 2.4 Dynamic Cone Penetration (DCP) Test on Unbound Aggregate Bases

# 2.4.1 Introduction

The dynamic cone penetration (DCP) test is a rapid and fairly versatile test for in-situ evaluation of soils and some aggregates. Its correlations to the California bearing ratio (CBR), unconfined compressive strength, resilient modulus, and shear strengths, and its use in performance evaluation of pavement layers make it an attractive tool. DCP tests are widely used to evaluate the in-situ strength of fine grained and granular subgrades, granular base and subbase materials, and weakly cemented materials. Many useful correlations between the DCP penetration index and other material properties are reported in the literature (Amini, 2003).

Many agencies use DCP to check subgrade stability before and during construction. The subgrade must be sufficiently stable to prevent excessive rutting and shoving during and after construction, and must also provide adequate support for the placement and compaction of the layers to be constructed. For these reasons, the DCP was used in this study to measure field stability of aggregate bases.

## 2.4.2 DCP Test on Aggregate Bases

Roy (2007) conducted DCP tests on granular materials. According to Roy's study, with adequate care taken to control data errors and characterize inherent data variability, the DCP test data can be considered representative of in-situ materials characteristics. Also, correlations are available for converting the DCP penetration rate to the California bearing ratio (CBR) and  $M_R$ . It was concluded that the DCP blow rate is an indicator of the ultimate bearing capacity of the material being penetrated. Also, it was found that the blow rate (DCP index) for a granular pavement layer is analogous to the AASHTO structural layer coefficient for that layer.

According to Amini (2003), there are several reasons, including time and economy, that DCP can be used as a tool for characterizing the pavement layers. In addition, many correlations between DCP index and various parameters of the aggregate bases (e.g. resilient modulus) exist that permit the estimations of various parameters/properties with given DCP index (e.g., Allersma, 1988; Bester and Hallat, 1977; Bukoski and Selig, 1981; Chen et al., 1999; Chen et al., 2001; and Chan and Armitage, 1997). A new standard test method, ASTM D6951, for use of the DCP in five shallow pavement applications has been recently developed (ASTM, 2003).

Siekmeier et. al. (1998) developed base compaction specifications using the DCP test. It was shown that the DCP method is an appropriate substitute for the specified density method when assessing aggregate base materials. It was recommended that the DCP method be used as an option for determining acceptable aggregate base conditions. They concluded that accurate and repeatable DCP tests depended on seating the cone tip properly and beginning the test consistently. Also, it was recommended that the cone tip be seated by one full drop of the hammer.

	2.1 Coefficient of		
Equation	Author	Property	Comments
$k = C_H (D_{10})^2$	Hazen (1892)	$C_H$ = Hazen coefficient (unitless) $D_{10}$ = particle size for 10% finer soil (cm) k = permeability (cm/sec)	Based on saturated sands
$k = \frac{6.214 \times 10^5 \ D_{10}^{1.478} \ n^{6.654}}{F^{0.597}}$	Moulton (1980)	$D_{10}$ = particle size for 10% finer soil (cm) n = porosity (unitless) F = % passing No. 200 sieve k = permeability (cm/sec)	Based on unbound aggregate bases
$k = 0.35 \ (D_{15})^2$	Sherard et al. (1984)	$D_{15}$ = particle size for 15% finer soil (mm) k = permeability (cm/sec)	Based on dense sands and gravels
$k = -69.2 - 22.10D_{10} + 24.7D_{20} + 228e + 6.96(D_{10})^2 - 1.56(D_{20})^2$	Kamal et al. (1993)	$D_{10}$ = particle size for 10% finer soil (mm) $D_{20}$ = particle size for 20% finer soil (mm) e = void ratio (unitless) k = permeability (10 <sup>-3</sup> m/s)	Based on unbound aggregate bases
$k = -710.27 + 2606.96e + \frac{7597.23}{pass30} - 13.44F$	Elsayed (1995)	e = void ratio (unitless) pass30 = % by wt. passing No. 30 sieve F = % passing No. 200 sieve k = permeability (ft/day)	Based on unbound aggregate bases
$k = 1.99 \times 10^4 \times D_{eff}^2 \times \left(\frac{1}{SF^2}\right) \times \left[\frac{e^3}{(1+e)}\right]$	Carrier III (2003)	$f_i$ = fraction of particles between 2 sieves (%) $D_{eff}$ = effective diameter (cm) SF = shape factor (unitless) e = void ratio (unitless) k = permeability (cm/sec)	Based on porous media
$\log k = -2.74487 - (0.0939125F) - (0.00743402C_U)$	Bouchedid and Humphrey (2005)	F = % passing No. 200 sieve $C_U = \text{coefficient of uniformity (unitless)}$ k = permeability (cm/sec)	Based on unbound aggregate bases

# Table 2.1 Coefficient of Permeability Models

Equation	Author	Property	Comments	
$M_R = 163.45 - 0.956 C_U - 6.055C_C + 8.508F$	Khoury and Zaman (2007)	$C_U$ = coefficient of uniformity (unitless) $C_C$ = coefficient of gradation (unitless) F = % passing No. 200 sieve $M_R$ = resilient modulus (MPa)	Based on unbound aggregate bases $M_R$ determined at a bulk stress of 172 kPa (25 psi)	
$M_R = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3}$	AASHTO Pavement Design Guide (2002)	$\begin{split} \Theta &= \text{bulk stress} = \sigma_1 + \sigma_2 + \sigma_3 (\text{MPa}) \\ \sigma_1 &= \text{major principal stress} (\text{MPa}) \\ \sigma_2 &= \text{intermediate principal stress} = \sigma_3 (\text{MPa}) \\ \sigma_3 &= \text{minor principal stress} = \text{confining} \\ \text{pressure (MPa)} \\ P_a &= \text{atmospheric pressure (MPa)} \\ \tau_{oct} &= \text{octahedral shear stress} (\text{MPa}) \\ \kappa_1 &= \text{regression constant} (\text{MPa}) \\ k_2, k_3 &= \text{regression constants} (\text{unitless}) \\ M_R &= \text{resilient modulus} (\text{MPa}) \end{split}$		
$M_R = k_1  \Theta^{k_2}$	Hicks and Monismith (1971)	$k_1$ = regression constant (psi) $k_2$ = regression constant (unitless) $\Theta$ = bulk stress (psi) $M_R$ = resilient modulus (psi)	Based on unbound aggregate bases	

# Table 2.2 Resilient Modulus Models



Figure 2.1 Boutwell Two-Stage Permeameter





Figure 2.3 Hydraulic Conductivity from Porous Probe Tests: (a) Case A-Probe with Permeable Base; (b) Case B-Probe with Impermeable Base (After Olson and Daniel, 1989)





Figure 2.5 Schematic of Field Permeability Test used by Bouchedid and Humphrey (2005)

## 3.1 General

Aggregate properties including gradations, specific gravity, shape, size, and texture used in the present study are discussed in this chapter. An overview of aggregate origins is also presented. The results presented in this chapter are used to rationalize the stiffness and permeability results presented subsequently in this report.

#### **3.2 Aggregate Origins**

Three types of aggregates from Oklahoma were used in this study. In Phase 1, aggregates were collected from Anchor Stone quarry located at Owasso, Tulsa County. In Phase 2, two additional aggregates, Dolese from the Hartshorne quarry in Pittsburg County and Martin Marietta from Sawyer in Choctaw County, were collected.

## **3.3 Collection of Aggregates**

In each case, bulk aggregates were collected from a well mixed stockpile consisting of particles ranging from 50 mm (2 in) to fines (% passing No.200 sieve). The bulk samples were shoveled into polythene bags, sealed to avoid any contamination and hauled to Broce Lab for testing. At least 100 bags, each weighing approximately 9 kg (20 lb), were collected and stored under a covered shed.

Before the start of any testing, moisture was removed from the bulk aggregates by ovendrying for 24 hours in a pan. Using a mechanical sieve shaker, the dry aggregates were sieved in accordance to the sieve sizes recommended for each gradation to be tested (ODOT standard specifications for highway construction, 1999; ASTM C136). Particle sizes larger than 37.5 mm (1.5 in) were removed from the bulk aggregates. All particle sizes larger than 0.075 mm (No.200 sieve) were washed to remove any fines. The washed aggregates were once again oven-dried for 24 hours and then stored in sealed buckets in the laboratory for further testing.

# **3.4 Geological History of Aggregate Origins**

According to the ODOT Material Division Aggregate Information Report (2009), the Anchor Stone and Dolese aggregates used in this study are limestone, while the Martin Marietta aggregates are primarily sandstone. To gain an understanding of the mineralogical aspects, a geologic history of each aggregate source was determined.

### 3.4.1 Anchor Stone Quarry

The Anchor stone quarry is located at 14901 East 66<sup>th</sup> Street, North Owasso in Tulsa County. This county falls in the geomorphic province of Claremore Cuesta Plains in Oklahoma (Figure 3.1). This particular area is comprised of resistant Pennsylvanian sandstones and limestones which form cuestas between broad shale plains. During the crustal unrest in the Pennsylvanian period both orogeny and basin subsidence occurred in the south while gentle raising and lowering of broad areas occurred in the north (Oklahoma Geological Survey, 2008).

Uplifts in Colorado and New Mexico gave rise to the mountain chain referred to as the Ancestral Rockies. Sediments deposited earlier in the Wichita, Arbuckle, and Ouachita Uplifts were lithified, deformed, and uplifted to form major mountains, while nearby basins subsided rapidly and received sediments eroded from the highlands. Pennsylvanian rocks are dominantly marine shale, but beds of sandstone, limestone, conglomerate, and coal are also encountered. Pennsylvanian strata, commonly 2,000 ft (609.6 m) to 5,000 ft (1524 m) thick in shelf areas in the north, are up to 16,000 ft (4876.8 m) in the Anadarko Basin, 15,000 ft (4572 m)in the Ardmore Basin, 13,000 ft (3962.4 m) in the Marietta Basin, and 18,000 ft (5486.4 m) in the Arkoma Basin (Oklahoma Geological Survey, 2008).

## 3.4.2 Dolese Quarry

The geological history of this quarry can be traced back to the Hogback Frontal Belt in Oklahoma (Figure 3.1). The Hogback Frontal Belt was formulated from the thrust blocks of steeply dipping Pennsylvanian sandstones and limestones. Hogback ridges raise 500 ft (152.4 m) to1,500 ft (457.2 m) above adjacent shale valleys (Oklahoma Geological Survey, 2008). As noted in the preceding section, aggregates from this quarry have the same geological features (limestone originated in the Pennsylvanian period) as those from the Anchor Stone quarry.

#### 3.4.3 Martin Marietta Quarry

Most of the land area of Choctaw County falls under a particular geomorphic province named the "Dissected Coastal Plain" (Figure 3.1). This geomorphic province is composed of mostly unlithified, south-dipping Cretaceous sands, gravels, clays and some limestones from the Gulf Coastal Plain which are dissected by streams.

All of the rocks in this geologic province are sedimentary rocks that were formed during Mesozoic-Cretaceous Age (Oklahoma Geological Survey, 2008). Shale, sandstone and limestone occur up to about 2,000 (609.6 m) to 3,000 ft (914.4 m) in the gulf coastal plain. A major unconformity is exposed throughout the southeast, where Cretaceous strata rest on rocks from the Precambrian to Permian. Uplift of the Rocky Mountains in the late Cretaceous and early Tertiary caused a broad uplift of Oklahoma, imparting an eastward tilt that resulted in final withdrawal of the sea, thus promoting sedimentary deposits turning into rocks (Oklahoma Geological Survey, 2008).

# **3.5 Physical Properties of Aggregates**

Physical properties of aggregates are important to overall pavement performance, and are closely related to their mineral and chemical compositions. As such, knowledge of the geological

history of aggregate sources is important. The following physical properties were considered in this study:

- Gradations.
- Grain Size Distribution of Fines (passing No.200 sieve).
- Specific Gravity.
- Abrasion Resistance.
- Aggregate Texture and Shape.

## 3.5.1 Aggregate Gradations

Particle size distribution or gradation is one of the most influential characteristics of aggregates. As evident from the studies discussed in Chapter 2, both stiffness (indicator of stability) and permeability of aggregates are heavily influenced by their gradations (Shah, 2007; Khury and Zaman, 2007; Christopher and McGuffey, 1997). Nominal maximum aggregate size and its percentage in the gradation are also important to the mix designs and workability of aggregate bases.

As noted previously, in Phase 1 of this study five different gradations, namely *M*-*AASHTO #57*, *M*-*AASHTO #67*, *ODOT Type A*, *OKAA Type K* and *OKAA Type N*, were selected. These gradations were only used for Anchor Stone aggregates. In Phase 2 after consultation with ODOT, the number of gradations was reduced to three: *M*-*AASHTO #57*, *ODOT Type A* and *OKAA Type M* (Khoury and Zaman, 2007). These three gradations were used for the Dolese and Martin Marietta aggregates. Gradations used in Phase 1 and Phase 2 are shown in Table 3.1 and graphically illustrated in Figure 3.2 and Figure 3.3, respectively. Both lower and upper limits for each specified gradation were used to prepare specimens for permeability and resilient modulus testing.

*M-AASHTO #57* and *M-AASHTO #67* have been recommended for permeable bases by pavement engineers (Tandon and Picornell, 1997). In this study, however, both of these gradations were modified in order to introduce additional fines. The *M-AASHTO #57* and *#67* gradations are open-graded having similar fines contents, in the range of 0 to 5%. *OKAA Type M*, *OKAA Type N* and *OKAA Type K* gradations have been proposed by Oklahoma Aggregate Association (OKAA) for possible use as base layers. *OKAA Type M*, *OKAA Type N* and *OKAA Type K* gradations for the range of 0 to 5%, 0 to 7% and 0 to 10%, respectively. Including *Type A* gradation provided baseline data and a basis for comparison of performance with other gradations. It is important to note that Type A is the densest gradation used in this study, having fines content in the range of 4 to 12%.

When comparing the results of this study, one needs to keep in view that most of upper limits of gradations considered have appreciable amounts of fines (particles passing a No.200 sieve). These fines can have significant influence on the flow behavior. A relatively small increase in the amount of fines can substantially reduce the permeability of an aggregate base. To determine the percentages of different particle sizes within the fines, hydrometer tests were carried out in accordance with the D 422 test method (ASTM, 1999). Results from hydrometer testing are shown in Table 3.2 through Table 3.4 and are used in the calculation of effective diameters for the selected gradations. Results from the ASTM D 422 test for effective diameter are shown in Chapter 6 and enumerated in Table 6.4 through Table 6.7. Those tables show that dense gradations tend to have low effective diameter. For example, in Phase 1 for lower limits of *M-AASHTO #57* (Open graded base course), effective diameters were reported as 1.31 in. (33.3 mm) and 1.51 in. (38.4 mm), respectively with standard compaction effort. Whereas, lower

limits of *ODOT Type A* (Dense graded base course) had effective diameter of 0.15 in. (3.8 mm) and 0.13 in. (3.3 mm) respectively, for the same standard compaction.

# 3.5.2 Specific Gravity

AASHTO M 132 (AASHTO, 2002) test method defines specific gravity as the ratio of the mass of a unit volume of a material at a stated temperature to the mass of the same volume of gas-free distilled water at the same temperature (Das, 1983). This property is very important in case of aggregate bases. In the current study, specific gravities of different aggregates were used to compute void ratios of specimens for selected gradations. Specific gravities of the different aggregate types were determined using the CoreLok method and are shown in Table 3.5 through Table 3.7. The tables show that the bulk specific gravity (Saturated Surface Dry Condition) of Anchor Stone aggregate was 2.606. Meanwhile, specific gravities for Dolese and Martin Marietta aggregates were reported as 2.683 and 2.670, respectively. Again, percent absorption was highest for Anchor Stone aggregate with an absorption of 1.71%. On the other hand, Dolese and Martin Marietta aggregates had approximately the same percent absorption (0.45% and 0.42%, respectively).

## 3.5.3 Abrasion Resistance

The abrasion resistance of aggregates is important during construction as well as during the service life of a pavement. In general, aggregates should be hard and tough enough to resist crushing, degradation and disintegration from such activities as manufacturing, stockpiling, placing and compaction. Also, the aggregates should be strong enough to sustain dynamic loading and creeping introduced by heavy traffic running over the pavement.

A common test used to characterize toughness and abrasion resistance is the Los Angeles (L.A.) abrasion test (ASTM C535). For the L.A. abrasion test, a portion of an aggregate sample

retained on the 1.70 mm (No. 12) sieve is placed in a large rotating drum that contains a shelf plate attached to the outer wall. A specified number of steel balls are then placed in the machine and the drum is rotated for 500 revolutions at a speed of 30 - 33 revolutions per minute (RPM). The material is then extracted and separated into material passing the 1.70 mm (No. 12) sieve and material retained on the 1.70 mm (No. 12) sieve. The retained material (larger particles) is then weighed and compared to the original sample weight. The difference in weight is reported as a percent of the original weight and called the "percent loss." Aggregates from the three different aggregate sources were tested for Los Angeles Abrasion loss. Results from these tests are summarized in Table 3.5, Table 3.6 and Table 3.7. From these results, it was clear that Dolese aggregates suffered the highest abrasion loss of 33.2%. The other two aggregates show a similar range of abrasion loss. Anchor Stone aggregates showed 23.6% abrasion loss; whereas, Martin Marietta aggregates showed 25.3% abrasion loss.

# 3.5.4 Aggregate Texture and Shape

According to Loudon (1953), particle angularity and texture can influence mechanical properties of aggregates. More angular aggregates are likely to have a higher porosity, and hence a higher permeability. An increase in particle angularity and temperature and a decrease in relative density and plasticity index can significantly increase the flow of water through a granular medium. According Randolph et al. (2000), flow through a medium depends on tortuosity of the medium. Elongated or irregular particles are likely to create flow paths which are more tortuous than those around nearly spherical particles.

Particles with a rough surface texture provide more frictional resistance to flow than smooth textured particles (Randolph et al., 2000). Both of these conditions tend to reduce the permeability of aggregates for a selected gradation. Surface texture is an indicator of the pattern

and the relative roughness or smoothness of aggregate particles. A rough surface texture gives the fines something to grip, producing a stronger bond, and thus promoting stability. However, this may also decrease the void spaces in the aggregate matrix resulting in reduced permeability.

To evaluate the texture and angularity of the selected aggregates, an Aggregate Imaging System (AIMS) developed by Masad et al. (2005), was used in this study. The test sample consisting of 56 particles for coarse aggregates are placed on specified grid points on the measurement tray for scanning, as shown in Figure 3.4. For fine aggregates, a handful of the aggregate is spread uniformly on the measurement grid for scanning. A built-in camera unit captures images of the aggregates in black, white and gray format. The software system evaluates the images and determines aggregate texture, angularity, sphericity and 2D form (Masad et al., 2005). An Aggregate Shape Classification Chart (Figure 3.5) suggested by Masad et al. (2005), is used to categorize the different types of aggregates.

For open-graded bases, larger size particles are expected to play a governing role on the evaluation of permeability, therefore, texture, sphericity, 2D form and angularity properties were determined for coarse aggregates. AIMS results obtained for ½ in. (12.7 mm) and 3/8 in. (9.5 mm) aggregates are summarized in Table 3.8 through Table 3.10 and illustrated in Appendix F. From Table 3.8, it can be shown that even after being categorized in the same group for angularity, the angularity varies over a wide range. Different aggregates show different permeability values even for the same gradation. That may be due to variation in the angularity and texture properties of aggregates. Figure 3.5 shows that texture values of less than 165 are classified as polished aggregates. All three types of aggregates showed average texture values below 165 for ½ in. (12.7 mm) and 3/8 in. (9.5 mm) size aggregates, meaning they all fall in the category of polished texture. However, the average amounts of the representative "polished"

texture aggregates vary from one type to another. For example, Dolese #4 (4.75 mm) sieve sizes aggregates had 99.10% polished aggregates; whereas, Anchor Stone and Martin Marietta aggregates had 91.07% and 97.32% polished aggregates.

Figure 3.5 defines the range for sub-rounded aggregates with values varying between 2,100 and 4,000. Table 3.8, Table 3.9 and Table 3.10 show that for the mentioned sieve sizes (½ in., 3/8 in. and #4 sieves, respectively), Anchor Stone aggregates had the highest average gradient angularity (3,329.53 for ½ in sieve size). Average gradient angularity values for Dolese and Martin Marietta aggregates were reported as 2546.97, 2715.35, 3182.46 and 2920.58, 2920.46, 3062.61 for ½ in. (12.7 mm) and 3/8 in. (9.5 mm) and #4 (4.75 mm) sieve sizes, respectively. For all of the aggregates and both of the above mentioned sieve sizes, average radius angularity was between 10 and 16, which defines the aggregates as angular aggregates as per Figure 3.5. However, the percentage of aggregates for the representative angular type is different over a very wide range. For instance, ½ in. (12.7 mm) Martin Marietta aggregates had 40.74 and 43.64% radially angular aggregates. #4 (4.75 mm) Anchor Stone aggregates had the highest percentage (56.86%) of radially angular aggregates. Dolese Stone and Martin Marietta aggregates had 41.35 and 52.00% of radially angular aggregates, respectively for #4 sieve.

On the basis of sphericity of <sup>1</sup>/<sub>2</sub> in. (12.7 mm) sieve size, average value of sphericity for Anchor Stone, Dolese and Martin Marietta aggregates were 0.69, 0.72 and 0.64, respectively. Figure 3.5 shows that Anchor Stone and Martin Marietta aggregates fall in the category of low sphericity type, whereas Dolese aggregates were of moderate sphericity type. Table 3.9 shows that 3/8 in. (9.5 mm) Dolese and Martin Marietta aggregates belong to low sphericity type, and Anchor Stone aggregates fall in the moderate sphericity type.

Sieve	e Size		G	radation Sp	ecification (	% Passing)		
(in)	(mm)	ODOT Type A	Modified AASHTO #57 (1)*	Modified AASHTO #57 (2)**	Modified AASHTO #67	ОКАА Туре М	OKAA Type N	OKAA Type K
1.5 in	37.5	100 - 100	100 - 100	100-100	100 - 100	100-100	100 - 100	100 - 100
1 in	25.4	-	95 - 100	95-100	-	70-100	-	95 - 100
3/4 in	19	40 - 100	-	-	90 - 100	55-100	40 - 80	-
1/2 in	12.5	-	25 - 60	25-60	-	-	-	-
3/8 in	9.5	30 - 75	-	-	20 - 55	30-60	25 - 60	-
No. 4	4.75	25 - 60	0 - 10	0-10	0 - 12	-	10-50	5 - 75
No. 8	2.36	-	0 - 5	0-5	0 - 8	-	-	-
No. 10	2	20 - 43	-	-	-	5-25	-	-
No. 20	0.85	-	-	-	-	-	-	0 - 30
No. 40	0.425	8 - 26	-	0-0	0 - 6	0-10	-	-
No. 200	0.075	4 - 12	0 - 5	-	0 - 5	0-5	0 - 7	0 - 10

Table 3.1 Gradation Specifications Used in the Study

\*Gradation used in Phase 1;

\*\*Gradation used in Phase 2.

Time (min)	Tem p (°C)	Actual Hydr. Reading	Control Fluid Reading	Corr. Hydr. Reading	% Finer	Eff. Depth L (cm)	K	D (mm)
1.0	22	47	1	47	94.0	8.59	0.01332	0.03903
2.0	22	41	1	41	82.0	9.57	0.01332	0.02913
4.0	22	27	0.5	27.5	55.0	11.78	0.01332	0.02286
6	22	14	0.5	14.5	29.0	13.92	0.01332	0.02028
7	22	7	0.5	7.5	15.0	15.06	0.01332	0.01954
8	22	3	0.5	3.5	7.0	15.72	0.01332	0.01867
10	22	1	0.5	1.5	3.0	16.05	0.01332	0.01687
13	22	1	0.5	1.5	3.0	16.05	0.01332	0.01480
15	22	1	0.5	1.5	3.0	16.05	0.01332	0.01377
16	22	1	0.5	1.5	3.0	16.05	0.01332	0.01334
19	22	1	0.5	1.5	3.0	16.05	0.01332	0.01224

Table 3.2 Hydrometer Test on Anchor Stone Fines

Table 3.3 Hydrometer Test on Dolese Fines

Time (min)	Temp (°C)	Actual Hydr. Reading	Corr. Hydr. Reading	% Finer	Eff. Depth L (cm)	K	D (µm)
0.5	23	60.0	61.2	87.0	6.6	0.01332	49.41
1.0	23	58.0	59.2	82.5	7.1	0.01332	36.21
2.0	23	55.0	56.1	78.0	7.6	0.01332	26.48
4	23	52.0	53.0	72.8	8.2	0.01332	19.42
8	23	48.5	49.5	67.5	8.8	0.01332	14.20
11	23	45.0	45.9	63.8	9.2	0.01332	12.39
13	23	42.5	43.4	62.3	9.3	0.01332	11.50
15	23	41.5	42.3	60.8	9.5	0.01332	10.80
18	23	40.5	41.3	59.3	9.7	0.01332	9.95
21	23	39.5	40.3	56.3	10.0	0.01332	9.36
24	23	37.5	38.3	53.3	10.3	0.01332	8.90
27	23	35.5	36.2	51.8	10.5	0.01332	8.46
30	23	34.5	35.2	51.0	10.6	0.01332	8.06
60	23	34.0	34.7	45.0	11.2	0.01332	5.87
120	23	30.0	30.6	37.5	12.0	0.01332	4.30

	1 40	510 5.4 Hyu			i till ivitally	etta i mes	
Time (min)	Temp (°C)	Actual Hydr. Reading	Corr. Hydr. Reading	% Finer	Eff. Depth L (cm)	K	D (mm)
0.5	23	46.0	46.9	69.0	8.6	0.01358	56.28
1.0	23	43.0	43.9	64.5	9.1	0.01358	40.92
2.0	23	40.0	40.8	60.0	9.6	0.01358	29.71
4	23	38.5	39.3	57.8	9.8	0.01358	21.28
8	23	35.5	36.2	53.3	10.3	0.01358	15.42
11	23	33.0	33.7	49.5	10.7	0.01358	13.41
13	23	32.5	33.2	48.8	10.8	0.01358	12.38
15	23	31.5	32.1	47.3	11.0	0.01358	11.61
18	23	31.0	31.6	46.5	11.0	0.01358	10.64
21	23	30.0	30.6	45.0	11.2	0.01358	9.92
24	23	29.5	30.1	44.3	11.3	0.01358	9.32
27	23	28.0	28.6	42.0	11.5	0.01358	8.88
30	23	27.5	28.1	41.3	11.6	0.01358	8.45
60	23	25.5	26.0	38.3	12.0	0.01358	6.06
120	22	23.0	23.5	34.5	12.4	0.01358	4.36

Table 3.4 Hydrometer Test on Martin Marietta Fines

Table 3.5 Anchor Stone Aggregate Specific Gravity, Absorption and L.A. Loss

			Average
	Trial 1	Trial 2	Value
Bulk Specific Gravity :	2.564	2.56	2.562
Bulk Specific Gravity (SSD):	2.604	2.608	2.606
Apparent Sp. Gravity:	2.67	2.688	2.679
Absorption, %:	1.55	1.86	1.71
% Abrasion Loss (500 revs.):		23.6	

			Average
	Trial 1	Trial 2	Value
Bulk Specific Gravity :	2.664	2.678	2.671
Bulk Specific Gravity (SSD):	2.68	2.686	2.683
Apparent Sp. Gravity:	2.707	2.7	2.704
Absorption, %:	0.6	0.31	0.45
% Abrasion Loss (500 revs.):		33.2	

Table 3.6 Dolese Aggregate Specific Gravity, Absorption and L.A. Abrasion Loss

Table 3.7 Martin Marietta Aggregate Specific Gravity, Absorption and L.A. Abrasion Loss

			Average
	Trial 1	Trial 2	Value
Bulk Specific Gravity :	2.658	2.662	2.659
Bulk Specific Gravity (SSD):	2.67	2.673	2.67
Apparent Sp. Gravity:	2.69	2.692	2.689
Absorption, %:	0.44	0.42	0.42
% Abrasion Loss (500 revs.):		25.3	

Table 3.8 Summary of AIMS Results on aggregates retaining on 1/2" sieve

Aggragata		Classification based on Average Value (Retaining on 1/2 in. sieve)								
Sources	Average Texture	%	Avergae Gradient Angularity	%	Average Radius Angularity	%	Form (Sphericity)	%		
Anchor Stone	Polished-Smooth	50.00	Sub-Rounded	89.29	Angular	40.74	Low Sphericity	43.14		
Delege	(100.58~105) Polished	57.14	(2100< <b>3529.53</b> <4000) Sub-Rounded	77 60	(10<11.24<10) Angular	12 61	Moderate Sphericity	62.06		
Dolese	( <b>151.29</b> <165)	37.14	(2100< <b>2546.97</b> <4000)	//.08	(10< <b>10.35</b> <16)	45.04	(0.7< <b>0.72</b> <0.8)	03.90		
Mantin Maniatta	Polished	83 33	Sub-Rounded	76 10	Angular	54.00	Low Sphericity	37.38		
	( <b>121.62</b> <165)	05.55	(2100< <b>2920.58</b> <4000)	70.19	(10< <b>11.36</b> <16)	54.00	(0.6< <b>0.64</b> <0.7)			

		5		$\overline{\mathcal{U}}$	0	0				
A gano goto	Classification based on Average Value (Retaining on 3/8 in. sieve)									
Sources	Avergae Texture	%	Avergae Gradient Angularity	%	Average Radius Angularity	%	Form (Sphericity)	%		
Anchor Stone	Polished (135.09<165)	73.21	Sub-Rounded (2100< <b>3069.40</b> <4000)	76.79	Angular (10< <b>10.84</b> <16)	39.29	Moderate Sphericity (0.6< <b>0.69</b> <0.7)	35.71		
Dolese	Polished (126.73<165)	80.18	Sub-Rounded (2100<2715.35<4000)	77.68	Angular (10< <b>11.39</b> <16)	51.43	Low Sphericity (0.6< <b>0.67</b> <0.7)	37.84		
Martin Marietta	Polished (112.75<165)	91.96	Sub-Rounded (2100< <b>2920.46</b> <4000)	82.14	Angular (10< <b>12.25</b> <16)	47.52	Low Sphericity (0.6< <b>0.69</b> <0.7)	35.71		

Table 3.9 Summary of AIMS Results on aggregates retaining on 3/8" sieve

Table 3.10 Summary of AMIS Results on Aggregates Retaining on #4 Sieve

	Classification Based on Average Value (Retaining on #4 Sieve)									
Aggregate Sources	Average Texture	%	Average Gradient Angularity	%	Average Radius Angularity	%	Form (Sphericity)	%		
Anchor Stone	Polished (0< <b>93.07</b> <165)	91.07	Sub-Rounded (2100< <b>3064.2</b> <4000	71.43	Angular (10< <b>11.67</b> <16)	56.86	Low Sphericity (0.6< <b>0.62</b> <0.7)	28.57		
Dolese	Polished (0< <b>82.53</b> <165)	99.10	Sub-Rounded (2100< <b>3182.46</b> <400	80.73	Angular (10< <b>11.43</b> <16)	41.35	Low Sphericity (0.6< <b>0.62</b> <0.7)	29.59		
Martin Marietta	Polished (0< <b>92.17</b> <165)	97.32	Sub-Rounded (2100< <b>3062.61</b> <400	80.56	Angular (10< <b>11.67</b> <16)	52.00	Low Sphericity (0.6< <b>0.61</b> <0.7)	32.71		



Figure 3.1 Geomorphic Divisions in Oklahoma



Figure 3.2 Gradation Specifications of Phase 1



Figure 3.3 Gradation Specifications of Phase 2



Figure 3.4 Aggregates Positioned on AIMS Tray (after Masad, 2004)



# 4.1 General

A 500 ft (152.4 m) test road was constructed on Timberdell Road between Asp Avenue and Jenkins Avenue in the University of Oklahoma Norman Campus (Figure 4.1). The test road involved three different test sections involving two new aggregate base designs and one standard design, for comparison. This chapter describes the construction of the test road and the associated testing.

# 4.2 General Information on the Test Road

A photographic view of the original road (before construction) is shown in Figure 4.2. It was a paved road with no well defined drainage system on the north side and no drainage ditches between the road and the neighboring field (Figure 4.2). Also, the existing pavement had numerous potholes, alligator cracking and other distresses.

## 4.3 Test Road Sections

To evaluate the effect of different drainable aggregate base designs, the test road was divided into three sections, as shown in Figure 4.3. The first section, TS-1, started at the intersection of Asp Ave and Timberdell Road and had a length of approximately 200 ft (60.96 m). A typical profile of this section is shown in Figure 4.4. Evidently, TS-1 consisted of four layers. The top layer was 4-in. thick hot mix asphalt (HMA) layer, underlain by an 8.0 in. (203.2 mm) thick aggregate base layer having *M*-*ASHTO* #57 gradation. The third layer was 6.0 in. (152.4 mm) thick subgrade soil stabilized with 15% Cement Kiln Dust. The bottom layer was A-4 (3) type existing subgrade soil, according to the AASHTO classification. The second section, TS-2, started at the west end of TS-1 and was approximately 200 ft (60.96 m) long. This section also included a 4.0 in. (101.6 mm) thick HMA layer constructed on the top of an 8.0 in. (203.2

mm) thick aggregate base of *OKAA Type M* gradation (Figure 4.5). The third and fourth layers in this section were identical to those in TS-1, as shown in Figure 4.5. The third section, TS-3, started at the west end of TS-2 and extended about 100 ft (30.5 m). eastward, as shown in Figure 4.6. Construction wise, this section was identical to TS-2 except the aggregate base layer was constructed with *ODOT Type A* aggregates.

## 4.4 Pre-Construction Laboratory Testing

Bulk subgrade soils were collected from the site before construction and pertinent laboratory tests (namely, sieve analysis including hydrometer and Atterberg limits) were conducted to classify them according to the AASHTO classification system. The subgrade soil was found to be primarily silt, having a classification of A-4(3). Based on the OHD-L 50 guidelines, it was determined that 8% CKD would be required to stabilize the existing subgrade soil. A Standard Proctor test was conducted on CKD-soil mix according to the AASHTO T 90 (AASHTO, 2002) test method. From these test results, the maximum dry unit weight ( $\gamma_{dmax}$ ) and the optimum moisture content were found to be 111 pcf (17.4 kN/m<sup>3</sup>) and 14.5%, respectively. These results were used in the construction to ensure that the field densities throughout the test sections met the ODOT requirements.

#### **4.5 Overview of Construction**

The construction of the test road was divided into four phases. The first phase consisted of removing the old, damaged pavement (Figure 4.7), the second phase consisted of constructing the stabilized subgrade layer (Figure 4.9). In the third phase the different aggregate bases were laid (Figure 4.14). The last phase involved paving the road with HMA, as shown in Figure 4.25.

### 4.6 Treatment of the Existing Subgrade

The existing subgrade was graded and leveled to assure conformity with the typical sections and grades, as specified by ODOT (1999). A motor grader was used for this purpose, as shown in Figure 4.8. The CKD was spread and mixed with the existing soil, as shown in Figure 4.10. Water was added to the CKD-soil mixture which was then compacted using a sheep-foot roller as shown in Figure 4.11. A nuclear density gauge (Figure 4.12), was used to measure the *in-situ* density of the compacted stabilized-subgrade layer. The nuclear density values were compared with the laboratory moisture-density results to ensure that the compaction level achieved in the field was acceptable in accordance with the ODOT specifications (ODOT, 1999).

# 4.7 Construction of Aggregate Base Layer

Aggregates used in the construction were hauled from the Dolese Bros Co. Quarry, located in Davis, Oklahoma, on Friday, December 7, 2007. Prior to laying the aggregate base layer, a separator fabric was placed on the finished subgrade to prohibit fine subgrade soils from contaminating the aggregate base as well as prevent water from penetrating and collecting in the subgrade (Figure 4.13). The aggregates were spread using a bottom-dump truck, as shown in Figure 4.15. The un-compacted thickness of the aggregate base, called loose lift thickness, was kept about an inch (25.4 mm) more than the desired thickness of 8.0 in. (203 mm). A vibratory roller manufactured by Ingersoll Rand (IR) was used to compact the aggregate layers (Figure 4.16). Several passes were made in heavy vibratory mode followed by several passes in static mode (no vibration) to reach the desired density. A nuclear density gauge was used to check the quality of compaction. The results of the density tests are shown in Table 4.1.

#### 4.8 Problems Encountered During the Construction of the Aggregate Bases

After spreading the aggregate bases, an ice storm occurred forcing the work to be shut down for one week. On Monday, December 17, 2007, the work was resumed and the compaction of the aggregate base was completed on Wednesday, December 19, 2007.

# 4.9 Paving

The paving work started on December 18, 2007. The HMA layer was laid first on the west bound lane and then on the eastbound lane. Paving was performed with a paver, as shown in Figure 4.21. Two passes with heavy vibratory mode and one pass with static mode (no vibration) of an Ingersoll Rang (Figure 4.22) roller were used to achieve the desired density. A light duty roller, manufactured by Ferguson (Figure 4.23), was used to smooth the HMA surface and remove the marks of the vibratory roller.

#### 4.10 Constructability of M-AASHTO #57 and OKAA Type M Aggregate Bases

Field observations revealed that the spreading technique, using a bottom dump truck, caused some aggregate segregation, as shown in Figure 4.17, which in turn caused variations in permeability and modulus of elasticity results throughout the aggregate base types and sections. It was observed that the level of segregation was the highest in TS-1 (with *M-AASHTO #57* gradation), followed by TS-2 (with *OKAA Type M* gradation). The lowest segregation was found in TS-3 (with *ODOT Type A* gradation). No quantitative analyses were conducted to assess the level of segregation. In future projects, it is recommended that an aggregate spreader be used to uniformly spread the aggregate over the entire width and length of a section.

A smooth roller was used for the compaction of the aggregate bases. Initially, multiple passes with vibration mode were applied on the aggregate bases in TS-1 and TS-2, having *M*-AASHTO #57 and OKAA Type M gradations, respectively. The vibratory mode caused the
aggregate to move/flow during compaction, as shown in Figure 4.18. As a result, it was decided to use additional passes without vibration (static mode) to achieve a more compacted and stable base layer, as shown in Figure 4.19. A photographic view of TS-1 (*M-AASHTO #57* gradation) and TS-2 (*OKAA Type M* gradation) after compaction and prior to placing the HMA layer is shown in Figure 4.20. The research team also visually inspected the aggregate base layer after the paving-related work started. Figure 4.21 and Figure 4.24 show a photograph of the HMA layer being placed along the west bound lane. There was no major aggregate flow or movement under the pavers and compactors in any sections. Therefore, it was concluded that both *M-AASHTO #57* (in TS-1) and *OKAA Type M* (in TS-2) provided a relatively stable base during the compaction of the aggregate base and the construction of the HMA layer.

Type of Aggregate	Stat # E		w(%)	Density	Stat # W		Sta # W	Density
	0+35	0.35	2.3	116.5	0+45	0.45	2.5	116.9
	0+55	0.55	2.5	118	0+65	0.65	2.4	116.6
	0+75	0.75	2.6	118.6	0+85	0.85	3.0	116.7
W-AA3HTO NO. 37	1+15	1.15	2.5	113.1	1+25	1.25	3.5	116.6
	1+35	1.35	2.4	113.4	1+45	1.45	2.8	115.4
	1+75	1.75	3.1	116.5	1+85	1.85	2.4	121.6
	2+15	2.15	2.1	117.7	2+25	2.25	2.1	122.6
	2+35	2.35	2.5	118	2+45	2.45	2.4	123
	2+56	2.56	2.6	124.9	2+65	2.65	2.7	123.5
	2+75	2.75	2.9	115.2	2+85	2.85	2.8	126.8
OKAA Type w	3+15	3.15	2.8	117.7	3+25	3.25	2.7	129
	3+37	3.37	2.5	120.5	3+45	3.45	3.0	126.5
	3+55	3.55	2.2	125.1	3+65	3.65	2.8	125.5
	3+75	3.75	2.1	121.4	3+85	3.85	2.3	126.6
ODOT Type A	4+15	4.15	4.8	140.2	4+25	4.25	4	139.2
	4+35	4.35	4.4	133.1	4+45	4.45	4.1	132.9
	4+55	4.55	4.1	133.8	4+65	4.65	4.3	139.9

Table 4.1 A Summary of Field Density of Aggregate Base



Figure 4.1 Location of Timberdell Rd. in the University of Oklahoma Norman Campus



Figure 4.2 A Photographic View of Timberdell Road



Figure 4.3 Longitudinal Section of the Test Road



Figure 4.4 A Sketch of the Cross-Section of TS-1 with M-AASHTO #57 Aggregate Base



Figure 4.5 A Sketch of the Cross-Section of TS-1 with OKAA Type M Aggregate Base



**CKD Stabilized Layer** 

Figure 4.6 Sketch of the Cross-Section of TS-1 with ODOT Type A Aggregate Base



Figure 4.7 Milling the Existing Pavement



Figure 4.8 Motor Grader Used in Subgrade Layer Preparation



Figure 4.9 Mixing the CKD and the SubgradeSoil



Figure 4.10 Mixing the CKD and the Subgrade Soil



Figure 4.11 Adding Water to CKD-Soil Mix and Compacting the CKD- Soil Mix



Figure 4.12 Nuclear Density Measurement of the Compacted Subgrade



Figure 4.13 Placing Separator Fabrics on the CKD-Stabilized Subgrade



Figure 4.14 Spreading the Aggregate Using Bottom Dump Truck



Figure 4.15 Spreading Aggregate Bases



Figure 4.16 Vibratory Compaction of Aggregate Base Layer



Figure 4.17 Aggregate Segregation throughout M-AASHTO #57 and OKAA Type M



Figure 4.18 Movement of Aggregate Base when Compacting with Vibration Mode



Figure 4.19 Compacting Aggregate Base without Vibration



Figure 4.20 Aggregate Base Prior to Laying Hot Mix Asphalt



Figure 4.21 Placing of Hot Mix Asphalt on the Top of Compacted Aggregate Base



Figure 4.22 Compacting the HMA Layer using an Ingersoll Rand Compactor



Figure 4.23 A Pneumatic Compactor



Figure 4.24 Aggregate Base layer During Paving-Related Work



Figure 4.25 Compacted Finish of Hot Mix Asphalt Layer

# **5.1 Introduction**

This chapter presents the laboratory and field testing methods used in this study. The laboratory tests included moisture-density relationship, falling head permeability, and resilient modulus. The field tests included falling head permeability, dynamic cone penetrometer (DCP), and falling weight deflectometer (FWD). Traffic counts performed at the test section on the Timberdell Road are also presented in this chapter. The analysis of laboratory data is outlined and the test matrix for each test is included.

### **5.2.** Laboratory Testing

### 5.2.1 Moisture-Density Relationship

The density of each layer within a pavement structure has a significant effect on the stability of that layer (Hoff, 2004). Density of a layer correlates to the degree of compaction of the layer as well as the particle shape and gradation (Barksdale, 1996). Since permeability and resilient modulus ( $M_R$ ) are also dependent on the degree of compaction, both of these parameters can be correlated to dry density or dry unit weight. Specimens for permeability and resilient modulus testing are generally compacted at near optimum moisture content (*OMC*) and maximum dry unit weight ( $\gamma_{dmax}$ ). Therefore, it was necessary to determine the moisture-density relationships for each aggregate type and the selected gradations used in this study.

During Phase 1 of this study, upper and lower limits of five different gradations (*M*-AASHTO #57, *M*-AASHTO #67, OKAA Type K, OKAA Type M and ODOT Type A) were selected for the Anchor Stone aggregate. In Phase 2, however only upper and lower limits of three selected gradations (*M*-AASHTO #57, OKAA Type M and ODOT Type A) were used for the Dolese and Martin Marietta aggregates. Two different compaction methods, namely standard

Proctor and modified Proctor, demonstrated the effect of compaction on *OMC* and  $\gamma_{dmax}$  values. The standard Proctor test was performed according to the AASHTO T 99-01 test method (AASHTO, 2002), while the modified Proctor test was conducted according to the AASHTO T 180-01 test method (AASHTO, 2002).

Specimens were compacted using an automated mechanical compactor which could be adjusted for compaction according to either standard Proctor or modified Proctor (see Figure 5.1). The mechanical compactor can be set to count the number of blows applied. This compactor also allows the mold to rotate at a set number of revolutions per minute to assure uniform compaction. Figure 5.1 shows a photograph of the mechanical compactor used.

At least four specimens were compacted for each gradation to obtain an acceptable moisture-density relationship curve. The *OMC* and  $\gamma_{dmax}$  values were determined from these curves. The test matrix of the standard and modified Proctor tests are shown in Table 5.1,Table 5.2 and Table 5.3 for Anchor Stone, Dolese and Martin Marietta aggregates, respectively. A total of 44 standard Proctor and 50 modified Proctor tests were conducted.

### 5.2.2 Permeability Testing

### 5.2.2.1 Permeability Device

The coefficient of permeability values for various gradations were measured using a new permeability device that was fabricated at the University of Oklahoma Broce Laboratory. This device is similar to the one used by Fwa (1998). The device consists of a steel cylinder, with a diameter of 6.25 in. (158.75 mm) and a height of 6.5 in. (165 mm, as shown in Figure 5.2), a bottom mold (Figure 5.2), two cylindrical porous stones each having a diameter of 6 in. (150 mm) and thickness of 0.5 in. (12.7 mm, Figure 5.3), a long vertical inlet cylinder with an attached scale and transparent tube (Figure 5.4), rubber gaskets (Figure 5.5), a reservoir tank, a

stopwatch, hose clamps, pressure gauge, air pressure connection, and water connections to the inlet cylinder. The assembled device is shown in Figure 5.10 (a) for highly permeable materials and in Figure 5.10 (b) for materials with low permeability.

# 5.2.2.2 Specimen Preparation and Test Setup

Cylindrical specimens were compacted in a standard Proctor mold, having a diameter of 6.0 in. (150 mm) and a height of 4.584 in. (120 mm). During the Proctor testing, visual observations were made on the stability of the specimen being compacted. Open-graded aggregate specimens can have low stability, and thus they can collapse as soon as removed from the mold (Hatanaka et al., 2001; Hatanaka et al., 1997). Therefore, if a particular gradation was deemed unstable, specimens for such gradations were molded a day in advance and kept in a freezer for 24 hours without removing from the mold. This enabled extraction of the specimen from the mold with minimal disturbance and base loss. This procedure of freezing the base before extraction has been previously used by Hatanaka et al. (2001) and Hatanaka et al. (1997). To further reduce specimen disturbance, a hydraulic jack was used to extract the specimen from the mold. The test matrix followed during the course of permeability testing is shown in Table 5.4 through Table 5.6 for Anchor Stone, Dolese and Martin Marietta aggregates respectively. A total of 38 tests were conducted on Anchor Stone specimens, while 12 tests were conducted on Dolese and Martin Marietta specimens each.

After measuring the weight of the frozen specimen, the extracted specimen was placed on a porous stone and a steel permeability mold was slid onto the specimen. As shown in Figure 5.6, this mold contained a 0.025 in. (0.635 mm) thick rubber membrane, which was folded over the top and bottom of the mold. O-rings were placed on the membrane at both ends to provide an air tight seal between the membrane and the mold, as shown in Figure 5.6. The membrane was pulled onto the mold wall using vacuum, making sliding of the mold on the specimen easier. Once the specimen was in place, the vacuum could be released from the membrane, allowing the membrane to hold the specimen. These steps were performed quickly and carefully, so as to avoid any melting of the specimen and to prevent any loss of aggregate.

A hose clamp was tightened on the bottom O-ring as shown in Figure 5.6 and a rubber gasket was seated on the bottom mold (Figure 5.7). This gasket helped eliminate any water leakages that could occur in the space between the steel mold and the bottom mold. A second porous stone was placed on the top of the specimen. The specimen, together with the mold and porous stones, was then positioned on the bottom permeability mold, as shown in Figure 5.8 and Figure 5.9. As stated earlier, short circuiting of the water flow adjacent to the permeameter wall, in permeability testing using a rigid wall permeameter, can result in erroneous measurements. In order to eliminate this problem, a flexible wall permeameter with confining air pressure (see Figure 5.10) was used here. Confining air pressure constricts the membrane onto granular particles of the specimen, thus ensuring full contact with the specimen and reducing the flow of water between the membrane and the granular particles. An air pressure of 10 psi (69 kPa ) was used in this study for all tests. Additional details on the new permeameter are given by Shah (2007).

#### 5.2.2.3 Permeability Test

After sealing the entire apparatus and making sure that there were no leakages, the apparatus was placed inside a water reservoir and the specimen was allowed to saturate for approximately 15 minutes before running the test. After saturation, the cylinder (Figure 5.10 (a)) was filled with water up to at least the 19.7 in. (50 cm ) mark. Water level readings were taken every 2 seconds. For dense specimens, the water level changes were extremely slow and thus

readings were taken as deemed necessary. The height difference between the water level in the reservoir and a reference point on the scale was also taken each time a test was conducted. Permeability is a function of the unit weight and the viscosity of water which in turn depends on the temperature of the water (Das, 2002). Therefore, the temperature of the water in the reservoir was also recorded each time a test was conducted. A photographic view of the setup is shown in Figure 5.10 (a).

Due to difficulties in obtaining enough gradient for the dense graded samples having very low permeability, a modified version of the aforementioned permeameter was used, which is shown in Figure 5.10 (b). In this permeameter, the diameter of the inlet cylinder was reduced from 6 in. (15.24 cm) to 1 in. (2.54 cm) and the length of the cylinder was increased from 33 in. (83.82 cm) to 96 in. (243.84 cm) to achieve a higher hydraulic gradient. This setup increased the falling head rate and hence the discharge rate than in the other device. This setup, especially for gradations having very low permeability (which may take more than 24 hours to produce a recordable drop in water head), made the permeability measurement practical.

After the completion of each permeability test, the specimen was removed from the mold and oven dried for 24 hours. The dried specimen was weighed and then washed on a No. 200 sieve so as to remove the fines. These washed aggregates were dried again for 24 hours, reweighed, and sieved. This provided the actual post-compaction gradation of the specimen for which the coefficient of permeability value was obtained. Gradation-related parameters, namely  $D_{60}$ ,  $D_{30}$ ,  $D_{10}$ ,  $D_{eff}$ ,  $C_U$  and  $C_C$  were obtained from the post-compaction sieve analysis.

# 5.2.2.4 Calculation of Permeability

The measured data showing the change in the water level height with time was entered in an MS Excel spread sheet. The velocity (v) and hydraulic gradient (i) values were calculated in the spread sheet and plotted on a graph. The falling head approach employed by Fwa et al. (1998) (see Equation 5.1) was used in the analysis of the permeability results:

$$v = k_1 . i^n \tag{5.1}$$

where v = specific discharge velocity in ft/day (cm/sec),

 $k_1$  = coefficient of permeability in ft/day (cm/sec),

and n =experimental coefficient (unit less).

A power trend-line was fitted through the points in the graph of v versus *i*. The coefficient of permeability values were then corrected for temperature using Equation (5.2) and Table 5.7 and finally reported at a temperature of 20°C.

$$k_{20\,^{\circ}C} = \left(\frac{\eta_{T\,^{\circ}C}}{\eta_{20\,^{\circ}C}}\right) k_{T\,^{\circ}C} \tag{5.2}$$

where  $k_{T^{\circ}C}$  and  $k_{20^{\circ}C}$  = coefficient of permeability at T°C and 20°C, respectively,

and  $\eta_{T^{\circ}C}$  and  $\eta_{20^{\circ}C}$  = viscosity (N.m<sup>-2</sup>.sec) of water at temperatures T<sup>°</sup>C and 20<sup>°</sup>C, respectively.

As an example, the permeability results obtained for the *OKAA Type N* specimens prepared using the lower limit (LL) gradation and compacted using the standard Proctor effort is shown in Figure 5.11. Figure 5.12 shows the corresponding relationship between specific discharge and hydraulic gradient. The permeability results are discussed in Section 6.3.

# 5.2.3 Resilient Modulus Testing

A servo-controlled hydraulic actuator (MTS) capable of applying load pulse duration of 0.1 sec and a rest period of 0.9 sec was used for resilient modulus testing. The load was

controlled and measured using a 5,000 lbs (22.25 kN) load cell mounted inside the triaxial chamber, in compliance with AASTHO T-307 (AASHTO, 2002). The load cell had an output of 2 mV/V and a resistance of 350 ohms. The recoverable vertical deformation was measured by two externally-mounted linear variable differential transducers (LVDT). The testing equipment was calibrated before running any tests. The resilient modulus test on the aggregate base specimens was performed according to the AASHTO T 307-99 test method (AASHTO, 2002).

A cylindrical split steel mold, shown in Figure 5.13, with a diameter of 6.0 in. (152.4 mm) and a height of 12.0 in. (304.8 mm), was used for specimen preparation. The mold was modified by making two holes, each 3.0 in (76.2 mm) from the edge of the mold. These holes were used for creating a vacuum within the mold. As mentioned earlier, some open-graded specimens were found to be unstable (under no confining pressure) and chances of these specimens collapsing under their own weight when removed from the mold were extremely high. To avoid specimen collapse, a 0.025 in. (0.635 mm) thick rubber membrane was used inside the mold and the mold was subjected to vacuum during the compaction process, as shown in Figure 5.14. Further details of this method are given by Shah (2007).

After compaction, the base of the mold was unscrewed from the rammer platform and the setup transferred to a pan. The extension collar was carefully removed from the mold and the specimen was trimmed from the top until a flat surface was obtained. To level the top surface and make it more uniform, a steel plate was placed on the specimen and the plate was tamped gently using a steel rod. A filter paper and a 0.5 in. (12.7 mm) thick porous disk were placed on the top of the specimen. The base of the mold was then removed by carefully overturning the mold and unscrewing the base. Another filter paper and porous disk were placed on the bottom of the specimen. The O-rings were removed from the mold at this time. The mold with the

specimen was then transported to the triaxial chamber and set on the 6.0 in. (152.4 mm) testing platen in the chamber. Using the drainage lines, vacuum was again applied to the specimen via the bottom platen. This vacuum created suction within the specimen thus allowing the membrane to hold on to the specimen. An aluminum platen with a perforated top was placed on the top of the porous stone and the membrane unfolded from both the top and the bottom of the mold.

The split mold was then removed and to avoid the loss of confining air pressure during the resilient modulus test, a new rubber membrane was placed over the old membrane. Membranes were usually punctured during compaction and thus the use of double membranes was necessary. Figure 5.15 shows a specimen ready for testing. O-rings were placed on both sides of the specimen over the platens to seal the membranes to the platens and reduce the possibility of further air loss. Drainage lines were then connected and the system sealed accordingly. The load cell was connected to the specimen via a steel ball bearing. The LVDT's were attached to the load cell piston, as shown in Figure 5.16. To reduce the risk of any deformations of the specimen, the vacuum was generally applied until at least 5 psi (34.5 kPa) confining pressure was achieved within the system.

After the setup of the specimen was complete and the system sealed, the MTS system was started and the  $M_R$  program commenced. Because resilient modulus tests are very sensitive to the testing procedure and equipment, the test procedure was followed carefully. The load frame was lowered to close proximity of the load cell piston. A ball bearing was also used between the frame and the piston to reduce any eccentricity. The AASHTO T 307-99 test method recommends applying at least 500 repetitions of a load equivalent to maximum axial stress of 15 psi (103.4kPa) to condition the specimen.

Conditioning was used to eliminate the effects of the interval between specimen compaction and loading. Conditioning also reduces the effects of imperfect contact between the top and bottom platens with the specimen. During conditioning, both the confining pressure and the maximum axial stress was set at 15 psi (103.4 kPa) and a contact stress of 10% of the maximum axial stress applied to ensure that the load cell piston was in direct contact with the ball bearing. As soon as the confining pressure reached 5 psi (34.5 kPa), the vacuum was stopped and the drainage lines opened to allow for drained conditions to prevail.

After the conditioning phase was complete, the resilient modulus test followed the loading sequence shown in Table 5.8. The test matrices followed during the course of the resilient modulus testing for Anchor Stone, Dolese and Martin Marietta aggregates are shown in Table 5.9, Table 5.10 and Table 5.11, respectively. If the total vertical permanent deformation reached 5% of the total height of the specimen, the test was terminated and the specimen was considered weak. However, in the present study, none of the tested specimens attained a deformation of 5%.

The resulting loads and vertical deformations for each sequence were recorded in an MS Excel file. These data were then analyzed using a separate program created in MS Excel to obtain the actual bulk stress, cyclic load and resilient modulus for each sequence within a test.

After the completion of the  $M_R$  test, the specimen was removed, weighed, and oven dried for moisture content determination. The dried aggregates were weighed and then washed on a No. 200 sieve to eradicate the fines. These washed aggregates were dried again for 24 hours, reweighed, and sieved. This provided the actual post-compaction gradation of the specimen for which the resilient modulus value was obtained. From this gradation analysis,  $D_{60}$ ,  $D_{10}$ ,  $D_{30}$ ,  $D_{eff}$ ,  $C_U$ , and  $C_C$  were determined.

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#### 5.3. Field Testing

Field permeability, falling weight deflectometer (FWD) and dynamic cone penetrometer (DCP) tests were conducted during and after the construction of the test section. A description of these activities is given in this section.

### 5.3.1 Field Permeability Test

As discussed earlier in Section 2.2.3, no standard devices or techniques are currently available for measuring permeability of aggregate bases in the field. An overview of different techniques was presented in Sections 2.2.3.1 through 2.2.3.3. In the present study, a falling head technique was used. At each location selected for measuring permeability, a 4.5 in. (11.4 cm) diameter hole was drilled through the asphalt layer using a portable coring device (Figure 5.17). The asphalt core was retrieved and the hole was cleaned using compressed air. A clear plastic standpipe with a diameter of 4.0 in. (10.2 cm) and a height of 102.4 in. (260 cm), fitted with a valve (Figure 5.18), was inserted into the hole. The space between the standpipe and the hole was gently and carefully filled with plaster of Paris. The standpipe was held vertically and the plaster of Paris was allowed to set. In order to conduct the test, the valve was closed and the standpipe was filled with water. As the valve was opened and the water level in the standpipe began to fall, the time taken by the water level to fall a known distance (marked on the pipe) was recorded. Tests at each location were repeated several times to ascertain reproducibility. Early trials showed large differences, as expected. As the base became saturated with repeated tests, subsequent tests became more reproducible. The amount of time for the water level in the standpipe to fall a known distance varied significantly depending upon the gradation of the aggregate base. For an open graded base (M-AASHTO #57 and OKAA Type M), the water level fell rapidly (within seconds), while for a dense graded base (ODOT Type A) it took much longer (hours). A theoretical framework, as discussed in Section 2.2.3, was needed to determine the coefficient of permeability from the recorded data. The results obtained from the field tests are discussed in Section 6.5.

### 5.3.2 Falling Weight Deflectometer (FWD) Test

The FWD test was performed by ODOT personnel in accordance with the ASTM D 4694 test method (ASTM 2009). It is one of the popular tests for evaluating pavement performance (Abdallah et al., 2001; Navaratnarajah, 2006). Figure 5.19 shows a photograph of the FWD testing in progress. The tests consisted of applying two load pulses by dropping a weight of 132.3 lbs (60 kg) from two predetermined heights (3.9 in.(100 mm) and 15.6 in. (396 mm)). Each load was impounded five times at each location. The resulting load pulse transmitted to the pavement as a half sine wave deformed the pavement into a bowl shape, called a deflection basin. The deflection and load history were recorded and stored for analysis. Deflections were measured with seven velocity transducers mounted on a straight bar. Based on the force imparted to the pavement and the shape of the deflection basin, the stiffness (back-calculated modulus) of the pavement was calculated by using Modulus 6.0 developed by Liu et al. (2000). Outputs of all FWD tests are given in APPENDIX C. The FWD test results are discussed in Section 6.7.

#### 5.3.3 Dynamic Cone Penetrometer (DCP) Test

The Dynamic Cone Penetrometer (DCP) test is designed to provide a measure of the insitu strength of fine grained and granular subgrades, granular base and sub-base materials, and weakly cemented materials. The DCP device (Figure 5.20) used in this study consisted of two 5/8-in. (16 mm) diameter shafts coupled near the midpoint. The lower shaft contains an anvil and a pointed tip, which is driven into unbound materials by dropping a sliding hammer contained on the upper shaft onto the lower anvil. The strength is determined by measuring the penetration of the lower shaft into the unbound materials. This value is recorded in millimeters per blow and is known as the Penetration Index (DCP-I). Penetration depth was recorded after each drop of the DCP hammer. The DCP index (in./blow) was calculated for each drop and DCP-I vs. depth was plotted as a profile for each testing station (see APPENDIX D).

# 5.3.4 Traffic Count

Vehicles were counted manually to approximate the volume and the type of traffic on the test section on an hourly basis. Vehicle counts were conducted on September 29, 2008 from 7:00 A.M. to 7:00 P.M. and on December 5, 2008 from 7:00 A.M. to 4:00 P.M. The vehicle count was recorded for each 15-minute period and the counted vehicles were categorized based on their types. Two research team members participated in these efforts, one counting the west bound traffic (towards Asp Ave.) and the other counting the east bound (towards Jenkins Ave.) traffic. The results are presented in APPENDIX E.

Gradation	-	Standard I AASHTC	Proctor ) T 99	Modified AASHT(	Proctor DT 180	Total
		LL	UL	LL	UL	
ODOT	Type A	4	4	4	4	16
M-AASHTO	#57	4	4	4	4	16
OKAA*	Type M	1	1	-	-	2
OKAA	Type N	4	4	4	4	16
OKAA	Туре К	4	4	4	4	16
M-AASHTO	#67	4	4	4	4	16
Total		21	21	20	20	82

Table 5.1 Moisture-Density Relationship Test Matrix for Anchor Stone Aggregate (Phase 1)

\* OKAA Type M gradation is tested as a part of Phase 2 of this study

Note: LL = Lower Limit; UL = Upper Limit

 Table 5.2 Moisture-Density Relationship Test Matrix for Dolese Aggregate (Phase 2)

Gradation		Standard Proctor AASHTO T 99		Modified AASHTC	Total	
		LL	UL	LL	UL	
ODOT	Type A	-	-	1	1	2
M-AASHTO	#57	1	1	-	-	2
OKAA	Type M	1	1	-	-	2
Total		2	2	1	1	6

Note: LL = Lower Limit; UL = Upper Limit

Table 5.3 Aggregate Moisture-Density Relationship Test Matrix for Martin Marietta (Phase 2)

Gradation		Standard Proctor AASHTO T 99		Modified AASHTO	Total	
		LL	UL	LL	UL	
ODOT	Type A	-	-	1	1	2
M-AASHTO	#57	1	1	-	-	2
OKAA	Type M	1	1	-	-	2
Total		2	2	1	1	6

Note: LL = Lower Limit; UL = Upper Limit

Gradation		Standard Proctor AASHTO T 99		Modified Proctor AASHTO T 180		Total
		LL	UL	LL	UL	
ODOT	Type A	2	1	2	1	6
M-AASHTO	#57	2	2	2	2	8
OKAA*	Type M	2	2	-	-	4
OKAA	Type N	2	1	2	1	6
OKAA	Туре К	2	1	2	1	6
M-AASHTO	#67	2	2	2	2	8
Total		12	9	10	7	38

Table 5.4 Permeability Test Matrix for Anchor Stone Aggregate (Phase 1)

\* OKAA Type M gradation is tested as a part of Phase 2 of this study

Table 5.5 Permeability	Test Matrix for Dolese	e Aggregate (Phase 2)	)
			/

Gradation		Standard Proctor AASHTO T 99		Modified Proctor AASHTO T 180		Total
		LL	UL	LL	UL	
ODOT	Type A	-	-	2	2	4
M-AASHTO	#57	2	2	-	-	4
OKAA	Type M	2	2	-	-	4
Total		4	4	2	2	12

 Table 5.6 Permeability Test Matrix for Martin Marietta Aggregate (Phase 2)

Gradation		Standard Proctor AASHTO T 99		Modified Proctor AASHTO T 180		Total
		LL	UL	LL	UL	
ODOT	Type A	-	_	2	2	4
M-AASHTO	#57	2	2	-	-	4
OKAA	Type M	2	2	-	-	4
Total		4	4	2	2	12

Temperature, T	$\eta_{T^{\circ}C}$	Temperature, T	$\eta_{T^{\circ}C}$
( C)	$\eta_{20^{\circ}C}$	( C)	$\eta_{20^{\circ}C}$
15	1.135	23	0.931
16	1.106	24	0.910
17	1.077	25	0.889
18	1.051	26	0.869
19	1.025	27	0.850
20	1.000	28	0.832
21	0.976	29	0.814
22	0.953	30	0.797

Table 5.7 Temperature Correction Factors (after Das, 2002)

 Table 5.8 Resilient Modulus Test Loading Sequence (after, AASHTO T 307-99)

Sequence No.	Cond. Pressure	Max. Axial Stress	Max. Axial Load	Cyclic Stress	Cyclic Load	Contact Stress	Seating Load	No. of Load Applications
	kPa	kPa	kN	kPa	kN	kPa	kN	
Cond.	103.4	103.4	1.89	93.1	1.70	10.3	0.19	500
1	20.7	20.7	0.38	18.6	0.34	2.1	0.04	100
2	20.7	41.4	0.76	37.3	0.68	4.1	0.08	100
3	20.7	62.1	1.13	55.9	1.02	6.2	0.11	100
4	34.5	34.5	0.63	31.0	0.56	3.5	0.06	100
5	34.5	68.9	1.26	62.0	1.13	6.9	0.13	100
6	34.5	103.4	1.89	93.1	1.70	10.3	0.19	100
7	68.9	68.9	1.26	62.0	1.13	6.9	0.13	100
8	68.9	137.9	2.51	124.1	2.26	13.8	0.25	100
9	68.9	206.8	3.77	186.1	3.39	20.7	0.38	100
10	103.4	68.9	1.26	62.0	1.13	6.9	0.13	100
11	103.4	103.4	1.89	93.1	1.70	10.3	0.19	100
12	103.4	206.8	3.77	186.1	3.39	20.7	0.38	100
13	137.9	103.4	1.89	93.1	1.70	10.3	0.19	100
14	137.9	137.9	2.51	124.1	2.26	13.8	0.25	100
15	137.9	275.8	5.03	248.2	4.52	27.6	0.50	100

Gradation		Standard Proctor AASHTO T 99		Modified Proctor AASHTO T 180		Total
		LL	UL	LL	UL	
ODOT	Type A	1	1	1	1	4
M-AASHTO	#57	1	1	1	1	4
OKAA*	Type M	2	2	0	0	4
OKAA	Type N	1	1	1	1	4
OKAA	Туре К	1	1	1	1	4
M-AASHTO	#67	1	1	1	1	4
Total		7	7	5	5	24

Table 5.9 Resilient Modulus Test Matrix for Anchor Stone Aggregate (Phase 1)

\* OKAA Type M gradation is tested as a part of Phase 2 of this study

Table 5.10 Resilient Modulus	Test Matrix for Dolose A	ggregate (Phase 2)

Gradation		Standard Proctor AASHTO T 99		Modified Proctor AASHTO T 180		Total
		$\mathbf{L}\mathbf{L}$	UL	LL	UL	
ODOT	Type A	0	0	2	2	4
M-AASHTO	#57	2	2	0	0	4
OKAA	Type M	2	2	0	0	4
Total		4	4	2	2	12

Table 5.11 Resilient Modulus Test Matrix for Martin Marietta Aggregate (Phase 2)

Table 5.11 Resilient Modulus Test Matrix for Martin Marietta Aggregate (Phase 2)									
Gradation		Standard Proctor AASHTO T 99		Modified Proctor AASHTO T 180		Total			
		LL	UL	LL	UL				
ODOT	Type A	0	0	2	2	4			
M-AASHTO	#57	1	1	0	0	2			
OKAA	Type M	1	1	0	0	2			
Total		2	2	2	2	8			



Figure 5.1 Automatic Mechanical Compactor



Figure 5.2 Steel Mold with Membrane and Bottom Mold used in Permeability Test



Figure 5.3 Porous Stones used in Permeability Test



Figure 5.4 Inlet Cylinder with Scale and Transparent Tubing used for Permeability Test



Figure 5.5 Rubber Gaskets and Plastic Rings used for Insulation in Permeability Test



Figure 5.6 Permeability Specimen Setup (Step 1)



Figure 5.7 Bottom Mold with Rubber Gasket used for Permeability Test



Figure 5.8 Permeability Specimen Setup (Step 2)



Figure 5.9 Permeability Specimen Setup (Step 3)





(b)


Type N Aggrega	ate Base gr	adation (Lowe	r Limit)		
Date:	-	2/8/2007			
Performed by: Aggregate Type: Compaction Type: Sample # :		AnchorStone Impact Comp (Standard) Type N - Lov	action ver - Std - k - 3		
Pressure:	10	psi	Water Temp:		14.80 °C
L:	4.586	in	Water Level to free sur	face:	12.7 cm
Head, h (cm)	Time, <u>dt</u> (sec)	dh (cm)	Velocity, v cm/sec	į	
40	0	0			
38	7	2	0.29	3.58	
36	14	4	0.29	3.41	
34	22	6	0.27	3.24	
32	32	8	0.25	3.06	
30	40	10	0.25	2.89	
28	51	12	0.24	2.72	
26	61	14	0.23	2.55	
24	72	16	0.22	2.38	
22	86	18	0.21	2.21	
20	96	20	0.21	2.03	
18	113	22	0.19	1.86	
16	128	24	0.19	1.69	
14	147	26	0.18	1.52	
12	166	28	0.17	1.35	
10	188	30	0.16	1.18	
8	216	32	0.15	1.00	

Figure 5.11 Permeability Results (OKAA Type N LL)



Figure 5.12 Permeability Results Plot (OKAA Type N LL)



Figure 5.13 Split Mold for  $M_R$  Specimen Compaction



Figure 5.14 Split Mold with Membrane for  $M_R$  Specimen Compaction



Figure 5.15 Aggregate  $M_R$  Test Specimen



Figure 5.16  $M_R$  Testing Setup



Figure 5.17 Portable Coring Device used in the Study



Figure 5.18 Field Permeability Test Device used in Field Tests



Figure 5.19 Falling Weight Deflectometer Device



Figure 5.20 DCP Test in Progress

### **6.1 Introduction**

This chapter is divided into two parts. The first part includes the laboratory test results for moisture-density relationships, permeability (k) and resilient modulus ( $M_R$ ) and regression models correlating k and  $M_R$  with gradation characteristics and physical properties. The second part presents the field test results (including permeability, falling weight deflectometer and dynamic cone penetrometer).

#### **6.2 Moisture-Unit Weight Relationships**

The optimum moisture content (*OMC*) and maximum dry unit weight ( $\gamma_{dmax}$ ) values obtained for each type of aggregates (i.e., Anchor Stone, Dolese and Martin Marietta) and gradations (*M-AASHTO #57, M AASHTO # 67, ODOT Type A, OKAA Type K, OKAA Type M* and *OKAA Type N*) used in this study are shown in Table 6.1 through Table 6.3 and graphically illustrated in Figure 6.1 through Figure 6.4. From these tables and figures one can conclude that compaction efforts and aggregate gradations have a significant effect on *OMC* and  $\gamma_{dmax}$  values. In general, *OMC* values obtained by the standard Proctor test were found to be greater than those obtained in the modified Proctor test, while the standard  $\gamma_{dmax}$  values were less than those of the modified. As an example, for Anchor Stone aggregate with *OKAA Type N* upper limit (UL) gradation, the average  $\gamma_{dmax}$  of specimens compacted using standard Proctor was 136.9 pcf (21.5 kN/m<sup>3</sup>) while  $\gamma_{dmax}$  for the same aggregate and gradation compacted using modified Proctor was 141.3 pcf (22.2 kN/m<sup>3</sup>), a 3.2% increase in  $\gamma_{dmax}$  value. The corresponding *OMC* values were 7.2% and 5.9% for the same gradation, respectively. The decrease in *OMC* and increase in  $\gamma_{dmax}$  values due to increase in compaction effort are attributed to the fact that high compaction effort:

(1) forces the grain to slide over each other with less water, (2) reduces the volume of water occupying the pores (i.e. lower *OMC*) and (3) produces denser packing and a higher density (Das, 2002; Barksdale, 1996; Marek 1977; Singh and Prakash, 1963).

In addition, the aggregate gradation characteristics – that is, upper limit, lower limit, and % fines, among others– have a significant influence on *OMC* and  $\gamma_{dmax}$  values, as shown in Figure 6.1 through Figure 6.4. The upper limit (UL) of each gradation, in general, exhibits *OMC* and  $\gamma_{dmax}$  values greater than the corresponding values of the lower limit (LL). As an example,  $\gamma_{dmax}$  values for *OKAA Type M* UL gradation of Dolese aggregate were found to be 127 pcf (19.9 kN/m<sup>3</sup>) compared to 116 pcf (18.2 kN/m<sup>3</sup>) for *OKAA Type M* LL, representing a difference of 8% between UL and LL; the corresponding *OMC* values were 4.0% and 2.7% for the same gradations, respectively. This is explained by the fact that the presence of smaller particles in a gradation allows for more particle packing and new particle arrangement. Obviously, this will lead to a reduction in the void ratio and an increase in density (Stein and Dempsey, 2004; Singh and Parkash, 1963). The *OMC* and  $\gamma_{dmax}$  values are also affected by the % fines in a given gradation. *OMC* and  $\gamma_{dmax}$  increased with the percent of fines, as shown in Figure 6.5 and Figure 6.6, respectively.

#### 6.3 Laboratory k Test Results

The variation of k values with gradations, physical properties and compaction efforts is presented in Table 6.4 through Table 6.7 and graphically illustrated in Figure 6.7 through Figure 6.10. According to these tables and figures k of aggregate bases depends on: (1) gradations, (2) compaction efforts and (3) physical properties. The importance of these factors is described in detail in the following sections.

#### 6.3.1 Effect of Gradation on k values

Gradation is one of the factors that have a significant effect on k values of aggregate bases. Figure 6.7 and Figure 6.8 show that the k values of M-AASHTO #67 are greater than the values obtained for M-AASHTO #57, OKAA Type M, OKAA Type K, and OKAA Type N followed by ODOT Type A. For example, the average k value of specimens prepared with LL of M-AASHTO #67 was 1,777 ft/day (0.87 cm/sec.) compared to 13 ft/day (0.006 cm/sec) obtained for specimens prepared with ODOT Type A LL gradation using the standard Proctor. This is expected because the aforementioned gradations have different particle sizes and amount of fines, as shown in Figure 6.11; this figure shows that M-AASHTO #67, M-AASHTO #57 and OKAA Type M has less amounts of fines (0-5%) compared to other gradations and that ODOT Type A is the finest gradation with a content of fines ranging between 4-12%. Given the laboratory results, it is evident that the amount of fines in an aggregate mix significantly influences the permeability values, as shown in Figure 6.12 where k values are plotted versus % fines. In addition, Table 6.4 through Table 6.7 and Figure 6.7 through Figure 6.10 show that the bound limits (i.e., UL and LL) also significantly influence k. Specimens with UL produced kvalues considerably lower than those of specimens prepared with LL, for the same aggregate gradation. For example, Anchor Stone specimens prepared with M-AASHTO #57 LL gradation and compacted using the standard Proctor effort had an average k value of approximately 1,474 ft/day (0.52 cm/sec) compared to 594 ft/day (0.21 cm/sec) obtained for specimens prepared with *M-AASHTO #57* UL gradation. This is explained by the fact that UL, for a given gradation, is much finer than LL. These findings are consistent with the findings of Bennert and Maher (2005) and Zhou et al. (1993). It may be noted that an increase in fines and small-size particles leads to a decrease in permeability for two reasons; firstly the presence of fines and small

particles decreases the total area available for flow and secondly it leads to a decrease in the individual pore size.

#### 6.3.2 Effect of Compaction Efforts and Physical Properties on k Values

According to Figure 6.7 and Figure 6.8 k is inversely proportional to the compaction effort; the higher the compaction effort the lower the k value. For example, specimens of Anchor Stone aggregate prepared with *M*-AASHTO #57 LL and compacted using standard Proctor effort exhibited an average k value of approximately 1,474 ft/day (0.52 cm/sec) compared to 192 ft/day (0.07 cm/sec) for the same gradation using modified Proctor effort, refer to Figure 6.7 and Figure 6.8. These findings are consistent with the findings of Mallela (2000) and Marek (1977). If a high compaction effort (i.e., modified Proctor) is used, the grains are forced to slide over each other and thus produce denser packing (higher density) and lower void ratio. The results revealed that density is also a factor affecting the permeability. k decreases with the increase in density, as shown in Figure 6.13; this is expected due to the reduction in the total area available for flow and reduction in individual pore sizes.

### 6.3.3 Effect of Aggregates Shape and Texture on k

Among the open graded base courses, permeability values of *AASHTO #57* LL gradation, Dolese and Anchor Stone aggregates exhibited same range of permeability values; whereas, Martin Marietta aggregates showed approximately 25% lower permeability (1122 ft/day (0.43 cm/s)). The underlying reason is that even with the same gradation surface texture, the tortuosity can vary within a specimen or from aggregate to aggregate type. Aggregates retaining on ½ in. (12.7 mm) sieve constitute a major portion of lower limit of *AASHTO #57* gradation. According to the Aggregate Imaging System (AIMS) protocol, any aggregate type having the gradient angularity value in between 2100 and 4000, is to categorized as sub-rounded. AIMS analysis in current study showed that, Anchor Stone materials had the highest gradient angularity (from Table 3.8 gradient Angularity = 3329.53, sub-rounded) in comparison to the other two types for aggregates (Dolese and Martin Marietta gradient angularity values are 2546.97 and 2920.58, respectively). Also the permeability values of both lower and upper limit of Anchor Stone aggregates were high. This observation confirms to Park and Smucker's (2005) claim that more angular materials promote larger permeability values. In general, permeability reduces with the increase in percent fines. On the contrary, Anchor Stone aggregates showed the highest permeability value among the three aggregate types even with higher percentage of fines as enumerated in Table 6.4 through Table 6.7. Hence for open-graded base layers permeability can increase due to the increase in angularity of aggregates even in the presence of fines.

On the other hand, permeability of dense graded base condition cannot be related to angularity and texture in a distinct manner. Well interconnected porous path can play a role in case of flow behavior. If the porous spaces in the aggregate fine matrix is not well connected or, if the porous spaces are filled up with fines, then the values of permeability cannot be related to angularity and texture. That's what happens in the dense graded base layers. Similar scenario was found in case of standard Proctor effort on UL of *OKAA Type M* and *ODOT Type A* gradations. Permeability value for *OKAA Type M* UL gradation was the lowest (23 ft/day (0.009 cm/s)) for Dolese aggregate. On the other hand, upper limit of the other two aggregate types showed approximately equal permeability values (from Table 6.5 and Table 6.7, *OKAA Type M* UL permeability of Anchor Stone and Martin Marietta aggregates are 64 ft/day (0.02 cm/sec), respectively.). Aggregates retaining on 3/8 in. (9.5 mm) sieves constitute a major portion (retaining 40% above the 3/8 in. (9.5 mm) sieve) of aggregate size

revealed that all of the three aggregates had polished texture. However, Anchor Stone had the least percentage (from Table 3.9, an average value of 135.09 with 73.21% of representative aggregates) of polished texture among the aggregates and Dolese aggregates had 80.18% of polished structure with an average value of 126.73. According to Randolph et al. (2000), flow through Anchor Stone aggregates was expected to encounter the highest resistance due to larger specific surface area as the texture is rougher for Anchor Stone than Dolese. However, Dolese aggregates showed the least permeability (23 ft/day (0.009 cm/s)) for *OKAA Type M* UL gradation which does not agree with Randolph et al.'s statement. So, permeability of aggregates cannot be attributed to characteristic related to aggregate shape alone for dense graded aggregate base condition. Rather, it could be attributed to aggregate packing, porosity, dry density, different physical properties in a synergistic way.

To assess the effect of aggregate shape on permeability, fractional indices in terms of gradient angularity, radius angularity and texture are introduced. Influence of these indices on the laboratory permeability of *OKAA Type M* aggregate is discussed. The index property terms start with "Fractional" because these properties only encompass specific percentages of post compaction aggregate sizes. These properties are calculated using the weighted average of the 3/8-in (9.75 mm) and #4 (4.75 mm) aggregates. These sizes were selected in calculating the indices because of their dominance in both upper and lower limits of *OKAA Type M* gradation envelope. Corresponding fractional indices along with the total percentage of aggregate coverage are enumerated in Table 6.8 and Table 6.9. Also, these values are presented graphically in Figure 6.14through Figure 6.16.

From Figure 6.14 it is seen that the permeability values increased with an increase in texture, which confirms to the qualitative classification of texture by US National Resources

Conservation Services (US Department of Agriculture, 2010). According to this qualitative classification, coarse particles with rough texture lead to a higher permeability, which confirms to the results in Figure 6.14. Anchor Stone *OKAA Type M* LL had the highest permeability with the highest fractional texture index. The percentage of coarse aggregates (3/8-in and #4) in Anchor Stone was higher than those of the other two types of aggregates (Dolese and Martin Marietta). This observation is consistent with the US National Resources Conservation Services guidelines (US Department of Agriculture, 2010).

Figure 6.15 and Figure 6.16 show the variation of permeability of OKAA Type M LL gradation, with Fractional Radius Angularity index and Fractional Gradient Angularity index, respectively. From Figure 6.15, it was revealed that permeability decreased with increasing Fractional Radius Angularity index. On the other hand, from Figure 6.16 it was observed that permeability increased with an increase in Fractional Gradient Angularity index. Radius angularity is measured by normalizing the difference between the particle radius in a certain direction and that of an equivalent ellipse (Masad, 2005); whereas, gradient angularity is measured by normalizing difference in angles at the edges of aggregates. Higher difference in angles promotes higher angularity. Henceforth, with increasing radius angularity, the particle shape deviates from an elliptical or elongated to a more rounded shape. In the Manual for Seepage Analysis and Control for Dams (U.S, Army Corps of Engineers, 1986), it was reported that the measured permeability is several orders of magnitude lower for angular particles with rough surfaces than for rounded particles with smooth surfaces. Observations from the current study support the aforementioned statement. Table 6.8 shows that Martin Marietta OKAA Type M LL has the lowest permeability with the highest radius angularity index. On the other hand, angles along the aggregate edges tend to promote less obstructed fluid flow path. Consequently,

with the highest Gradient Angularity Index of 3067, Anchor Stone *OKAA Type M* LL was reported to have the highest permeability.

A detailed discussion on the influence of aggregate shape and texture on permeability is beyond the scope of this project. However, these findings have merit for further research on effect of aggregate morphological properties on permeability. Also, further quantitative analysis is required to reach a conclusion as these indices do not cover the entire gradation curve commonly used in Oklahoma.

#### 6.4 Regression Model for the Prediction of k

Since k values are influenced by gradations, compaction efforts and physical properties, regression models were developed correlating k with gradation characteristics and density. The regression models are expected to help pavement engineers and others predict the k values in terms of more common properties. Also, these models may be used in the design of pavement structures (Level 1 or Level 2) using the hierarchal approach, as recommended by the new mechanistic empirical pavement design guide (M-EPDG). The k model is presented in Equation (6.1).

$$k\left(\frac{ft}{Day}\right) = A \times \left(D_{eff}\right)^{B} \times (D_{30})^{C} \times (C_{C})^{D} \times \left(\frac{e}{e+1}\right)^{E}$$
(6.1)

where,

 $D_{eff}$  = effective diameter in 0.01 inch,

 $D_{30}$  = particle size for 30% finer in 0.01 inch,

 $C_C$  = coefficient of gradation (unitless),

e = void ratio (unitless),

A, B, C, D and E = regression model parameters.

Regression models correlating *k* and gradation characteristics and physical properties were developed for each aggregate type. An additional general model was developed for all three aggregates. The regression parameters obtained for these models are shown in Table 6.10. It is evident that the *k* of aggregate bases varied with gradation characteristics, namely,  $D_{eff}$ ,  $D_{30}$ , and  $C_C$ , and physical properties represented in this model by the void ratio. The presence of these parameters is consistent with other studies (Bouchedid and Humphrey, 2005; Amer and Awad, 1974). Amer and Awad (1974) correlated *k* with void ratio,  $C_U$  and  $D_{10}$ Bouchedid and Humphrey (2005) also developed a regression model correlating *k* with percent fines and  $C_U$ . Other researchers (e.g., Carrier, 2003; Elsayed, 1995; Kamal et al., 1993; Sherard et al., 1984; Moulton, 1980) have used other gradation characteristics in developing *k* regression models. Based on R<sup>2</sup> and F values for these models, summarized in Table 6.10, it is evident that these models are statistically significant in predicting the *k* values of aggregates and gradations used in this study.

#### 6.5 Laboratory Resilient Modulus $(M_R)$ Results

The  $M_R$  results of all the gradations and aggregates tested in this study are presented in Table 6.11 through Table 6.15. One way to observe the effect of physical properties and gradation characteristics on  $M_R$  is to evaluate the changes in  $M_R$  values at a specific stress level. Previous studies revealed several models to correlate the  $M_R$  values of pavement bases with stresses. For example, Witczak (2000) and Khoury and Zaman (2007) found 14 different models that are available for predicting the  $M_R$  of unbound materials. The independent variables used in these models were confining pressure ( $\sigma_3$ ) and deviatoric stress ( $\sigma_d$ ), and bulk stress ( $\theta$ ) and octahedral stress ( $\tau_{oct}$ ).

The k- $\theta$  model has been used for unbound granular bases for many years (Witczak, 2000). In 2002, however, the AASHTO pavement design guide recommended the use of a new model, where  $M_R$  is correlated with bulk stress ( $\theta$ ) and octahedral shear stress ( $\tau_{oct}$ ), as shown in Equation (6.2).

$$M_R = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3}$$
(6.2)

where,

 $M_R$ = resilient modulus in psi,

 $\theta$  = bulk stress =  $\sigma_1 + \sigma_2 + \sigma_3$  in psi,

 $\sigma_1$  = major principal stress in psi,

 $\sigma_2$  = intermediate principal stress =  $\sigma_3$  for cylindrical specimens in psi,

 $\sigma_3$  = minor principal stress = confining pressure in psi,

 $P_a$ = atmospheric pressure in psi (14.7 psi),

 $k_1, k_2, k_3 =$  regression constants,

 $\tau_{oct}$  = octahedral shear stress in psi.

$$\tau_{oct} = \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2}$$
(6.3)

In the present study, the new M-E PDG model was used to analyze the laboratory data. The model constants ( $k_1$ ,  $k_2$ ,  $k_3$ ) obtained for each specimen are shown in Table 6.16 and Table 6.17. These values could easily be used for pavement design using both the 1993 AASHTO Design Guide and the M-E PDG (Level 2) provided the state of stress is known from layered elastic analysis or some other means. In this study, the design  $M_R$  values were calculated at a deviatoric stress of 6.0 psi (41.4 kN/m<sup>2</sup>) and a confining pressure of 4.0 psi (27.6 kN/m<sup>2</sup>). A summary of the  $M_R$  values at the aforementioned stresses along with aggregate properties are presented in Table 6.18. It is seen from Table 6.18 that the material factors that influence the  $M_R$  values of aggregate bases are: (1) gradation characteristics, (2) compaction efforts and (3) physical properties. This is consistent with the findings reported by other researchers (Shah, 2007; Khoury and Zaman, 2007; Ping and Ling, 2007; Ping et al., 2001; Tian et al., 1998; Hicks and Monismith, 1971). The influence of these factors on  $M_R$  values is described in the following sections (i.e. Section 6.5.1 through 6.5.2).

### 6.5.1 Effect of Gradations on M<sub>R</sub> Values

Gradation is a critical factor influencing the resilient modulus of aggregate bases, as shown in Table 6.18 and Figure 6.17 through Figure 6.19. In general, specimens prepared with UL have lower  $M_R$  values compared to LL of the same gradation. The difference in  $M_R$  values between coarser LL and finer UL is shown in Table 6.17. It is evident that, in general, coarser LL provided higher  $M_R$  values (2% to 64% higher) compared to finer UL. The reasons for the differences in  $M_R$  values could be that UL aggregates lack larger irregular particles that interlock while large top size particles in the LL gradations interlock and, therefore, provide a strong aggregate structure (Shah, 2007; Ping and Ling, 2007; Kamal et al. 1993; Barksdale and Itani, 1989). Also, the amount of fines in the UL gradation could displace the coarse particles, which leads to an aggregate fines matrix that has reduced  $M_R$  values (Jorenby and Hicks, 1986).

### 6.5.2 Effect of Compaction Efforts and Physical Properties on $M_R$ Values

According to Figure 6.18, Figure 6.19 and Figure 6.20, the variation of resilient modulus with compaction efforts and density could not be clearly determined. For example, specimens prepared with M-AASHTO #57 UL and compacted using modified Proctor had an average  $M_R$  value approximately 12% higher than the corresponding values obtained for specimens compacted using standard Proctor. On the other hand,  $M_R$  values of ODOT Type A UL gradation compacted using modified Proctor had an average  $M_R$  value of 26.1 ksi (179.9 MPa) compared to 26.3 ksi (181.3 MPa) obtained for UL under standard Proctor; no significant changes were observed. This is consistent with some studies in literature that density had negligible effects on  $M_R$  of aggregate bases (Papagiannakis and Masad, 2008).

### 6.5.3 Regression Model for the Prediction of $M_R$ Values

A regression model was developed correlating  $M_R$  values with the original gradation characteristics and physical properties. The stepwise method of multiple linear regression ( $\alpha$  = 0.15) option in SAS 9.1 was used to develop this model. The F-test for multiple regressions was conducted using SAS 9.1 to validate significance of the relationship between  $M_R$  and the independent variables included in the model. The associated probability was designated as  $P_r > F$ or p-value. It was found that the  $M_R$  values were significantly influenced by the compacted specimen characteristics, aggregate properties and stress levels.

# 6.5.3.1 Regression Model

The regression model developed in this study for predicting  $M_R$  values in terms of gradation characteristics and physical properties and stress states is shown in Equation (6.4).

$$M_R(psi) = k_1 p_a \times \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3}$$
(6.4)

In which the regression coefficients  $k_1$ ,  $k_2$  and  $k_3$  are given by:

$$\log k_{1} = 3.82562 - 0.25619 \log \left(\frac{MC}{OMC}\right) - 0.65999 \log(e) - 0.11457 \log(LA) + 0.15481 \log(CE) + 0.22094 \log(D_{30})$$
(6.5)

$$k_{2} = -1.72250 \log\left(\frac{\gamma_{d}}{\gamma_{d \max}}\right) + 0.39185 \log(LA) + 0.18436 \log(D_{30}) - 0.40399 \log(D_{60})$$
(6.6)

$$k_3 = -0.34278 \log(D_{60}) \tag{6.7}$$

$$(R^2 = 0.9330, F-value = 360.39, P_r < 0.0001)$$

where,

*MC* = molded moisture content (%),

*OMC* = optimum moisture content (%),

e = void ratio of the compacted specimen,

LA = Los Angeles abrasion value (%),

 $CE = \text{compaction energy (ft-lbf/ft}^3),$ 

 $D_{30}$  = particle size for 30% finer (in),

 $\gamma_d$  = molded dry density (pcf),

 $\gamma_{d max}$  = maximum dry density (pcf), and

 $D_{60}$  = particle size for 60% finer (in).

In view of relatively high  $R^2$  value obtained for this model, it is evident that the  $M_R$  of aggregate bases tested in this study varied with bulk stress, octahedral shear stress, moisture content, dry density, compaction effort, void ratio,  $D_{30}$  and  $D_{60}$ . Table 6.19 shows the ANOVA table of the model developed. The model yielded an  $R^2$  value of 0.93, an F-value of 360.39 and a  $P_r$  value of less than 0.0001, which indicated that the model was statistically significant in predicting the  $M_R$  values of aggregate bases. A comparison between the predicted  $M_R$  values and the corresponding  $M_R$  values obtained from the laboratory tests is shown in Figure 6.21. From Figure 6.21, it is evident that the data points are close to the 45° line, which indicates that this model could be a good predictor of  $M_R$  values of aggregate bases using gradation characteristics, physical properties and stress states.

### **6.6 Field Permeability Results**

Measured values of field permeability are summarized in Table 6.20 and Table 6.21. It is observed from these tables that the field permeability varied with gradations. According to the field results, the variations of the permeability of the base layers used in the test sections with gradation are, in general, similar to those observed in the laboratory. *M-AASHTO #57* gradation produced the highest average permeability value in the field; while the base layers of *ODOT Type A* gradation produced the lowest *k* value, which is consistent with the findings in the laboratory. The variation in the field permeability values over time was also observed and is presented in Table 6.20 and Table 6.21. Field-measured results show that the average *k* values for *M-AASHTO #57* gradation were 5,790 ft/day (2.04 cm/sec) for the tests performed on April 08, 3,730 ft/day (1.32 cm/sec) in July 08 and 437 ft/day (0.15 cm/sec) in May 09. The average *k* of *OKAA Type M* gradation was 199 ft/day (0.07 cm/sec) in April 08, 165 ft/day (0.06 cm/sec) in

July 08, and 85 ft/day (0.03 cm/sec) in May 09. Comparatively, the average k of *ODOT Type A* aggregate base layers was found to be only 0.1 ft/day ( $3.5 \times 10^{-5}$  cm/sec) during the April and July 08 field testing and 0.02 ft/day ( $7.0 \times 10^{-5}$  cm/sec) during the May 09 testing. The decrease in k over time was likely attributed to additional compaction and reduced air voids caused by traffic. Qualitatively, these findings support those by Tangpithakkul (1997). According to Tangpithakkul (1997), denser aggregate base layers result in much lower k values due to lower void ratios. Cedergren (1974) also reported that the k of gravel specimens decreases by approximately 30% when it becomes about 10% denser. No effort was made to compare changes in density of the aggregate bases with time and traffic. Also, because of inherent difficulties in measuring field permeability accurately and with sufficient degree of reproducibility, it would be appropriate to use these data as a comparative (rather than absolute) indicator of drainage of aggregate bases with different gradations.

Table 6.20 and Table 6.21 show large variations among the field k values for each type of gradation at different locations or stations. For example, station 0+50 EB had an average k of 993 ft/day (0.35 cm/sec), compared to 5,920 ft/day (2.09 cm/sec) for station 1+25 EB and 10,458 ft/day (3.69 cm/sec) for station 0+75 WB. As documented in Chapter 4, segregation was noticed during the spreading of aggregate in the field. These segregations were partly responsible for a non-uniform aggregate base layer resulting in variations in the k values. Figure 6.22 shows the *M*-AASHTO #57 section after construction. As can be seen, the aggregate base layer is not uniform throughout the section.

## **6.7 Falling Weight Deflectometer Results**

The back-calculated modulus ( $M_{FWD}$ ) values for the different time periods, including their standard deviation values, obtained from the FWD tests are summarized in Table 6.20 and Table

6.21. A comparison of the  $M_{FWD}$  values obtained from the tests performed in April 08, and May 09 revealed that for *M-AASHTO* #57 gradation,  $M_{FWD}$  values increased from 11.4 ksi (78,600 kN/m<sup>2</sup>) in April 08 to 20.8 ksi (143,411 kN/m<sup>2</sup>) in May 09, an 82% increase. A similar trend was observed for the *OKAA Type M* gradation where the  $M_{FWD}$  increased from 17.3 ksi (119,279 kN/m<sup>2</sup>) in April 08 to 38.7 ksi (266,827 kN/m<sup>2</sup>) in May 09, a 124% increase. The  $M_{FWD}$  values of the *ODOT Type A* section increased from 23.2 ksi (159,958 kN/m<sup>2</sup>) to 56.9 ksi (392,312 kN/m<sup>2</sup>), about 145% increase over the same period of time. Based on the aforementioned  $M_{FWD}$  results, it is clear that the back-calculated modulus values exhibited an increase for all gradations between April 08 and May 09. The traffic loads have likely caused additional compaction, beyond construction, resulting in an increase in the  $M_{FWD}$  values. These results are consistent with the results reported by Huang (2004) and Zeghal (2003). The FWD results are presented in APPENDIX C.

#### 6.8 Field Dynamic Cone Penetrometer (DCP) Test

As noted earlier, DCP tests were performed in accordance with the ASTM D 6951-03 test method. The test results are summarized in Table 6.20 and Table 6.21. From these tables, it is evident that the section with *M*-AASHTO #57 gradation had the highest DCP-I values followed by the OKAA Type M section and then the ODOT Type A section. These results are consistent with the  $M_R$  and  $M_{FWD}$  values which showed that the *M*-AASHTO #57 gradation was relatively less stable than the OKAA Type M followed by the ODOT Type A. Variations of DCP-I with depth for each section are presented in APPENDIX D.

#### 6.9 A comparison between field and laboratory k values

A comparison between the laboratory and field permeability values is presented in Table 6.22. From this table, the average laboratory *k* values of *OKAA Type M* and *ODOT Type A* were

higher than the field k values obtained in April 08, July 08 and May 09. Also, the average laboratory k values of *M*-AASHTO #57 were found to be lower than the field k values in April 08 and July 08 and higher than the field k values obtained in May 09. There are four possible explanations for the differences between the laboratory and field values: (1) the laboratory specimens were compacted near optimum moisture content and maximum dry density. It is likely that the final conditions of the laboratory specimens are different than the conditions in the field. ; (2) it is also anticipated that the flow conditions are better defined and more controlled in laboratory testing than in field testing. Moreover, the theoretical approach used in the interpretation of field measurements and evaluation of k is an approximate method and may not reflect the boundary and flow conditions in the field. (3) the difference could also be attributed to the nature of the construction in which segregations were observed during spreading of aggregates in the field as noted in Section 4.10.

# 6.10 A Comparison between $M_R$ and $M_{FWD}$ values

A comparison between results from FWD back-calculated modulus ( $M_{FWD}$ ) and laboratory resilient modulus ( $M_R$ ) is presented in Table 6.23. From this table, the average laboratory  $M_R$  values of OKAA Type M and ODOT Type A were lower than the field k values obtained in April 08, July 08 and May 09. Also, the average  $M_R$  values of M-AASHTO #57 were found to be higher than the field  $M_R$  values in April 08 and lower than those obtained May 09. This difference could be explained by the fact that the final conditions (gradations, densities, etc.) of specimens in the laboratory are different that the conditions in the field. This difference could also be attributed to the nature of the construction in which segregations were observed during spreading of aggregates in the field as noted in Section 4.10. This is consistent with findings by Ping et al. (2001) that the moduli obtained from FWD tests were different than the moduli from the laboratory.

Gradation	Compaction Energy	Limit	<b>OMC</b> (%)	$\gamma_{d max}$ (pcf)
Madified	Stondard	Lower	4.2	107.6
	Standard	Upper	4.3	111.4
AASHIO - #57	Modified	Lower	1.5	120.3
#37	Wiodiffed	Upper	3.9	123.5
	Standard	Lower	2.1	120.3
OKAA	Standard	Upper	7.2	136.9
Type N	Modified	Lower	1.9	128.6
	Wiodiffed	Upper	5.9	141.3
	Standard	Lower	1.6	109.5
OKAA	Stanuaru	Upper	7.8	134.3
Туре К	Modified	Lower	1.3	120.3
	Wiodiffed	Upper	5.7	138.8
Modified	Standard	Lower	3.3	103.1
	Standard	Upper	3.5	110.1
#67	Modified	Lower	3.3	116.5
#07	Wioumeu	Upper	2.9	121.6
	Standard	Lower	6.0	132.4
ODOT	Standard	Upper	6.8	138.1
Type A	Modified	Lower	5.0	141.3
	wioumeu	Upper	6.6	152.8
OKAA	Cton dand	Lower	2.2	120.0
Type M	Standard	Upper	4.5	130.0

Table 6.1 Moisture-Density Relationships of Anchor Stone Aggregate

Gradation	Compaction Energy	Limit	<b>OMC</b> (%)	$\gamma_{d max}$ (pcf)
M-AASHTO	Standard	Lower	2.3	111.0
#57	Stanuaru	Upper	3.7	114.0
OKAA Type	Standard	Lower	2.7	116.0
Μ	Stanuaru	Upper	4.0	127.0
ODOT Type	Modified	Lower	5.0	144.0
Α	Moumed	Upper	6.2	146.0

Table 6.2 Moisture-Density Relationships of Dolese Aggregate

Table 6.3 Moisture-Density Relationships of Martin Marietta Aggregate

Gradation	Compaction Energy	Limit	<b>OMC</b> (%)	$\gamma_{d max}$ (pcf)
M-AASHTO	Standard	Lower	3.2	103.0
#57	Stanuaru	Upper	4.5	109.0
ОКАА Туре	Standard	Lower	2.0	112.0
Μ	Stanuaru	Upper	5.2	123.0
ODOT Type	Modified	Lower	6.0	137.0
A	wiouilled	Upper	6.4	136.0

Gradation	Compaction	Limit	MC (%)	<mark>⊮⊿</mark> (pcf)	% Fines	Deff	D <sub>et</sub>	<i>1</i> 0 <sub>10</sub>	D <sub>ad</sub>	c,	C <sub>c</sub>	e	k*
	Energy						(0.0)	1 in)					(ff/Day)
		Louron	4.4	103.9	0.3	15.1	67.1	22.1	43	3.03	1.25	0.54	1757
	Standard	Lower	2.3	104.4	0.4	13.1	61	23.6	43.2	2.88	1.16	0.53	1207
	Statuaru	Linnar	4.8	106.9	3.7	2.2	45.5	14.6	27	3.11	1.09	0.5	340
M-AASHTO		Opper	5.2	110	3.9	2.1	43.9	13.3	26	3.31	1.16	0.45	848
#57		Lower	1.4	116.3	2.8	4.1	34.1	4.5	20.2	7.62	2.68	0.37	281
	Madified	LOWCI	1.3	116.1	1.5	4.3	59.4	13	31.5	4.58	1.28	0.38	102
	IVIOUIIEU	Linnar	3.8	120	5.8	1.4	40.2	4.7	22.3	8.54	2.63	0.33	612
		Opper	4	115.6	5.8	1.4	42.2	5.2	22.8	8.09	2.36	0.38	377
	Standard	Lower	2.1	120	0.9	4.7	85.6	6.5	33.5	13.06	2.01	0.33	522
			2.3	117.7	0.7	5.4	91.5	5.4	30.7	17.05	1.92	0.36	292
OV A A True N		Upper	8.3	135.9	8.4	0.8	32.2	0.4	2.8	86.81	0.63	0.18	0.1**
UKAA Iype N		Louion	1.9	128.9	2.5	2.2	66.7	2.4	23	27.39	3.26	0.24	48
	Modified	Lower	2.4	128.4	2.5	2.3	69	2.6	23.6	27.17	3.18	0.25	42
		Upper	5.4	141.1	10.9	0.6	26.1	0.2	2	106.9	0.64	0.13	0.004**
		Louron	2.3	108.1	0.71	6.8	45.5	19.1	28.1	2.39	0.91	0.48	587
	Standard	Lower	1.7	109.3	0.73	6.8	46.7	19.2	28.6	2.43	0.91	0.46	828
ОКАА Туре К		Upper	8.5	134.1	10.85	0.7	11.1	0.2	3.4	45.43	4.33	0.19	0.06
		Louise	1.8	120	2.55	2.5	39.3	4.4	21.3	8.91	2.62	0.33	108
	Modified	Lower	2.4	118.2	2.62	2.5	39.1	4.4	21.5	8.82	2.67	0.35	54
	MOURIC	Upper	6.1	138.5	13.28	0.6	9.5	0.2	1.9	60.62	2.54	0.16	0.003**

Table 6.4 Anchor Stone Lab. Permeability and Post Compaction Gradation

\* Permeability Values at 20°C

\*\*Permeability coefficients close to zero are calculated to higher decimal points. Otherwise, zero values will introduce more error to the regression model.

(continued)

Gradation	Compaction Energy	Limit	MC (%)	₽ <sub>2</sub> (pcf)	% Fines	D <sub>eff</sub>	D <sub>60</sub>	D <sub>ag</sub>	Dag	C <sub>D</sub>	C <sub>C</sub>	e	k*
							0.0)	1 <b>n</b> )					(ft/Day)
		Lower	3.7	101.9	0.9	7.3	53.8	20.9	37.5	2.57	1.25	0.57	1916
	Standard	Lowei	3.9	102.3	0.9	7.4	54.3	20.8	37.5	2.61	1.25	0.56	1638
M-AASHTO #67	Statkialu	Upper	3.9	109.3	5.2	1.5	38.7	10.7	24.2	3.61	1.42	0.46	1074
			3.7	110.3	5.3	1.5	37.3	9.6	23.4	3.89	1.53	0.45	215
		Lower	3.6	116.3	3.3	2.2	46.6	5.9	24.4	7.84	2.14	0.38	479
	Modified	Lower	4	112.3	2.7	2.6	48.1	8.2	26	5.87	1.71	0.37	649
		Unner	2.7	121.5	7	1.1	34.3	1.2	20	27.59	9.36	0.32	92
		Орры	3.3	119.6	7	1.1	34.1	1.3	20	26.63	9.19	0.34	272
	Standard	Lower	6.4	128.2	5	1.5	120.2	1.7	21.2	70.59	2.2	0.25	21
			5.2	132.6	5.6	1.3	102.3	1.3	17.7	75.76	2.26	0.21	5
ODOT		Upper	6.9	137.6	14.1	0.6	18.5	0.2	2.5	134.1	2.48	0.16	0.005**
Туре А		Lower	5.2	139	7.2	1.1	63.2	0.9	12.8	72.78	3.01	0.15	0.790**
	Modified	Lowei	4.9	141.5	7.4	1	70.5	0.7	12.3	92.73	2.82	0.13	0.020**
		Upper	6.9	152.7	15.5	-	16.7	0.1	1.9	149.8	2.03	0.16	0.001**
		Longe	2.2	112.1	0.7	7.68	13	32.3	72	5.54	1.11	0.47	1546
OK A A True M	Standard	Lowei	2.2	115	0.3	11.74	13.8	31.5	74	5.37	0.97	0.43	1457
UKAA Type M	STATIONALO	Ilmoor	4.6	128.1	6.1	1.14	0.7	9.1	33.5	44.74	3.28	0.29	65
		opper	4.5	129	5.6	1.21	0.7	8.7	33.1	44.21	3.03	0.28	62

Table 6.5 Anchor Stone Lab. Permeability and Post Compaction Gradation

\* Permeability Values at 20°C

\*\* Permeability values are calculated up to one decimal point. Permeability coefficients close to zero are calculated to higher decimal points. Otherwise, zero values will introduce more error to the regression model.

				<b>V</b> a		D <sub>eff</sub>	D <sub>ed</sub>	D <sub>10</sub>	Dag	C <sub>D</sub>	C <sub>c</sub>	е	k*
Gradation	<b>Compaction Energy</b>	Limit	MC (%)	(pcf)	% Fines		.0)	)1 in)					(ff/Day)
		Lower	2.7	111	0.69	9.3	27.6	74.8	82.7	3	2.46	0.5	1941
M-AASHTO	Standard		2.6	112	0.75	8.6	27.6	74.8	82.7	3	2.46	0.49	1120
#57	Staikiatu	Ilmor	3.7	113	0.86	4.8	13	29.5	68.9	5.3	0.97	0.47	27
		Opper	3.9	114	0.74	5.1	13.4	29.5	68.9	5.15	0.94	0.46	31
	Standard	Lower	2.9	116	0.97	5.5	13.8	32.3	77.2	5.6	0.98	0.44	1124
ОКАА Туре			2.8	117	1.09	5.1	12.2	31.1	76.4	6.26	1.04	0.42	1126
Μ	Statituatu	Imper	3.8	128	3.65	2.1	2	10.6	34.2	17.4	1.68	0.3	27
		Оррсі	3.7	128	4.17	1.9	1.7	10.6	34.6	20.95	1.97	0.3	19
		Lower	5.1	143	4.89	1.7	1.6	18.9	77.2	49	2.94	0.17	0.3
ODOT Type	Modified	Lower	4.9	144	5.49	1.6	1.4	15.7	77.6	56.29	2.32	0.16	0.001**
Α	MOUIIKU	Ilmor	6.4	144	13.67	0.7	0.2	2.2	18.9	106.7	1.4	0.16	0.1**
		opper	6.1	144	11.97	0.7	0.2	2.6	18.9	75	1.37	0.16	0.1**

Table 6.6 Dolese Lab. Permeability and Post Compaction Gradation

6-23

\* Permeability Values at 20°C

\*\* Permeability values are calculated up to one decimal point. Permeability coefficients close to zero are calculated to higher decimal points. Otherwise, zero values will introduce more error to the regression model.

	Compaction			r <sub>a</sub>		D <sub>eff</sub>	Deg	D <sub>10</sub>	Dao	Cn	C.	e	k*
Gradation	Energy	Limit	MC (%)	(pcf)	% Fines		(0.0	)1 in)		-324	-16		(ff/Day)
		Lower	3	103	1.1	8.5	21.6	47.2	82.7	3.82	1.25	0.61	1085
M-AASHTO #57 Standard	brebret2		2.9	103	1.1	9.1	23.6	63	82.7	3.5	2.03	0.61	1158
	Unner	4.4	109	1.2	4.9	9.8	26.8	66.9	6.8	1.09	0.52	339	
		Opper	4.4	109	1.1	5	9.8	26.8	66.9	6.8	1	0.52	334
		Lower	2	113	1.1	8	11.8	32.3	78.7	6.67	1.12	0.47	322
OKAA Type	Standard		2.3	113	0.9	8.8	11.4	29.5	80.7	7.07	0.95	0.47	919
Μ	Stationaru	Unner	6.1	123	6.1	1.8	0.9	9.4	32.3	34.17	2.93	0.35	45
		Оррсі	6.7	123	6	1.8	1.1	9.1	32	30	2.42	0.35	70
		Lower	6.8	136	6.6	1.7	0.7	13.4	66.9	89.47	3.58	0.22	0.3**
	Modified	LOWCI	6.3	138	6.5	1.7	0.8	14.9	70.9	85.71	3.82	0.2	0.1**
ODOT Type A	IVIOUIIKU	Upper	5.7	135	13.5	0.8	0.2	1.5	15	74.51	0.74	0.23	0.001**
			6.1	136	14.3	0.8	0.2	1.6	15	76	0.93	0.22	0.001**

Table 6.7 Martin Marietta Lab. Permeability and Post Compaction Gradation

6-24

\* Permeability Values at 20°C

\*\* Permeability values are calculated up to one decimal point. Permeability coefficients close to zero are calculated to higher decimal points. Otherwise, zero values will introduce more error to the regression model.

Origin	Gradation	Aggregate Sizes	Aggregate Percentage	*FTI	*FRAI	*FGAI	Permeability k (ft/day)
Anchor	OKAA Type M LL #1	ing ves	48.32	116.56	11.21	3067.28	1546
Stone	OKAA Type M LL #2	taini 4 sie	49.06	115.53	11.23	3067.12	1457
Dologo	OKAA Type M LL #1	Ret d #∠	48.85	106.16	11.40	2932.77	1124
Doiese	OKAA Type M LL #2	gate ' an	47.88	106.04	11.53	2933.58	1126
Martin	OKAA Type M LL #1	greg 3/8'	44.54	103.03	11.99	2987.94	322
Marietta	OKAA Type M LL #2	Ag on	42.97	102.63	11.99	2991.06	919

Table 6.8 Fractional Morphological Indices for OKAA Type M LL

Table 6.9 Fractional Morphological Indices for OKAA Type M UL

Origin	Gradation	Aggregate Sizes	Aggregate Percentage	*FTI	*FRAI	*FGAI	Permeability k (ft/day)
Anchor	OKAA Type M UL #1	ing ves	56.75	120.39	11.14	3067.4	65
Stone	OKAA Type M UL #2	tain 4 sie	56.03	120.04	11.14	3067.18	62
Dologo	OKAA Type M UL #1	d #	56.64	111.74	11.41	2873.89	27
Doiese	OKAA Type M UL #2	gate '	57.16	111.37	11.41	2877.64	19
Martin	OKAA Type M UL #1	gre 3/8'	56.83	105.05	12.05	2974.08	45
Marietta	OKAA Type M UL #2	Ag on	56.7	105.20	12.06	2972.08	70

Table 6.10 Regression Model Parameters for Permeability Model I for Various Aggregates

A concepto Truno	Re	egression	ANOVA Parameters				
Aggregate Type	Α	В	С	D	Ε	F	$\mathbf{R}^2$
Anchor Stone	$4.6 \times 10^{-8}$	-0.557	3.102	-1.111	20.225	17.24	0.91
Dolese	$10^{6.9}$	0	0	0	8.827	35.21	0.90
Martin Marietta	$10^{13.48}$	0	0	0	22.984	37.33	0.9
All 3 Types	$10^{13.48}$	-0.026	1.550	-2.511	8.504	14.49	0.75

σ3	$\sigma_{d}$	θ					M <sub>R</sub> (j	osi)				
(	(		M-AASH	TO No. 57	OKAA	Type N	OKAA	Туре К	M-AASH	IO No. 67	ODOT	Type A
(psi)	(psi)	(psi)	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper
3	3	12	16316	18058	16760	13441	31516	7019	26100	19879	16103	11917
3	5	14	18621	19879	19258	16390	20881	8864	24758	21947	18085	13976
3	8	17	19514	21169	20761	17863	22900	9955	23004	22904	19778	15459
5	4	19	20202	21776	21032	18546	23235	9875	24236	24452	20361	16219
5	9	24	22732	24746	24284	22449	27425	12065	26068	27170	24194	19111
5	13	28	24097	25945	26079	22271	29778	13288	27014	28074	26393	21002
10	9	39	28539	30373	29933	28723	33964	17023	32942	34505	32227	25679
10	18	48	31767	33757	34125	33386	38580	19901	35748	37595	36167	30846
10	27	57	34416	35321	36316	33617	40405	23497	37400	38937	18466	33187
15	9	54	32293	33327	32354	32250	37368	20723	38617	39896	36549	31870
15	13	58	34447	35602	35147	35869	40124	22823	40158	41456	38386	34557
15	27	72	38913	40334	40792	42623	45677	32636	43230	45948	44233	41055
20	13	73	37554	39246	37410	41248	43807	30565	44148	46606	43406	41180
20	18	78	39698	41427	41396	45012	46603	32599	45701	48483	45783	44254
20	36	96	45391	46487	48409	51508	51883	37415	49253	52683	52097	50181
$\sigma_3$ : confine	ing pressur	e, $\sigma_d$ : devi	atoric stress,	M <sub>R</sub> : resilient	modulus							

Table 6.11  $M_R$  Results of Anchor Stone Aggregate with Standard Proctor

σ <sub>3</sub>	$\sigma_{d}$	θ					$M_R$	(psi)				
(mai)	(mai)		M-AASH	ГО No. 57	OKAA	Type N	OKAA	Туре К	M-AASH	TO No. 67	ODOT	Type A
(psi)	(psi)	(psi)	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper
3	3	12	16370	19714	14096	16963	26116	23698	9052	11366	18930	13587
3	5	14	19089	22199	4644	20070	30426	26953	22523	22139	20894	15278
3	8	17	21175	23804	18416	23124	32751	28941	24327	26234	22873	16812
5	4	19	21469	24361	18820	22130	30679	28410	22678	22283	23614	17762
5	9	24	25884	27872	23113	28038	36370	31516	28697	31652	27574	20003
5	13	28	28075	29715	26449	29962	39266	33784	30860	33580	30152	22022
10	9	39	32902	34058	32739	35338	40669	36867	37156	40387	37204	25672
10	18	48	37807	38565	37870	40578	46461	40463	41980	44427	41879	28906
10	27	57	39557	40970	40814	42066	47624	41086	44895	45413	44464	31842
15	9	54	36002	37324	36094	36631	41958	37510	42046	40031	42692	30249
15	13	58	39614	40276	39753	41402	45769	41252	46296	48332	45826	31667
15	27	72	45555	46182	47052	49798	52164	46913	52284	52999	52048	35570
20	13	73	43168	43809	43815	46529	48384	44812	51417	53085	50969	35615
20	18	78	46252	46578	47102	51237	52082	48178	54485	56064	53792	36827
20	36	96	52522	52746	54169	57396	57740	52728	60803	59849	60222	41480
$\sigma_3$ : confini	ing pressur	e, σ <sub>d</sub> : devi	atoric stress	, M <sub>R</sub> : resilie	nt modulus							

Table 6.12  $M_R$  Results of Anchor Stone Aggregate with Modified Proctor

σ3	$\sigma_{d}$	θ	M <sub>r</sub> (psi)							
			M-AASH	TO No. 57	OKAA	OKAA Type M		Type A		
(psi)	(psi)	(psi)	Lower	Upper	Lower	Upper	Lower	Upper		
3	3	12	16,316	16,991	21,665	17,274	18,930	13,578		
3	5	14	18,621	15,634	20,626	15,993	20,894	15,278		
3	8	17	19,514	16,926	21,505	17,973	22,873	16,812		
5	4	19	20,202	16,993	21,682	17,914	23,614	17,762		
5	9	24	22,732	19,748	24,932	21,558	27,574	20,003		
5	13	28	24,097	21,639	26,742	23,659	30,152	22,022		
10	9	39	28,539	25,530	31,034	27,268	37,204	25,672		
10	18	48	31,767	30,636	35,193	32,222	41,879	28,906		
10	27	57	34,416	32,822	37,604	34,727	44,464	31,842		
15	9	54	32,293	29,752	35,304	30,990	42,692	30,249		
15	13	58	34,447	32,393	37,649	33,860	45,826	31,667		
15	27	72	38,913	38,168	42,722	40,254	52,048	35,570		
20	13	73	37,554	36,574	41,575	38,248	50,969	35,615		
20	18	78	39,698	39,148	44,051	41,047	53,792	36,827		
20	36	96	45,391	45,528	50,394	47,609	60,222	41,480		

Table 6.13  $M_R$  Results of Anchor Stone Aggregate

 $\sigma_3$ : confining pressure;  $\sigma_d$ : deviatoric stress;  $\theta$ : bulk stress;  $M_r$ : resilient modulus

1 able 6.14 $M_R$ Results of Dolese Aggr
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σ3	$\sigma_{d}$	θ	M <sub>r</sub> (psi)							
			M-AASH	TO No. 57	OKAA	Type M	ODOT Type A			
(psi)	(psi)	(psi)	Lower	Upper	Lower	Upper	Lower	Upper		
3	3	12	19,371	16,560	15,960	10,600	19,695	8,567		
3	5	14	16,989	17,144	17,336	12,320	18,325	13,009		
3	8	17	17,971	18,772	17,884	14,252	19,756	15,808		
5	4	19	20,104	19,650	19,798	14,097	20,920	12,023		
5	9	24	22,320	22,912	21,949	18,286	24,019	16,925		
5	13	28	24,262	24,942	23,181	21,328	26,879	18,832		
10	9	39	29,871	30,273	30,143	25,285	32,403	17,833		
10	18	48	33,656	34,846	33,803	31,391	38,036	23,151		
10	27	57	35,084	37,305	35,235	34,751	41,942	26,410		
15	9	54	33,091	34,862	35,490	28,471	37,454	20,154		
15	13	58	36,113	37,744	37,808	32,218	41,078	24,289		
15	27	72	41,094	43,734	43,438	40,161	48,761	31,823		
20	13	73	39,126	42,166	42,881	35,727	45,907	30,088		
20	18	78	42,168	45,108	45,299	39,421	49,594	34,575		
20	36	96	48,270	51,665	51,673	47,925	57,448	40,968		
$\sigma_3$ : c	onfinin	gpress	ure; σ <sub>d</sub> : deviat	oric stress; θ:	bulk stress;	$M_r$ : resilie	nt modulus			

σ <sub>3</sub>	$\sigma_{d}$	θ	M <sub>r</sub> (psi)							
			M-AASHTO No. 57		OKAA	Type M	ODOT Type A			
(psi)	(psi)	(psi)	Lower	Upper	Lower	Upper	Lower	Upper		
3	3	12	11,761	10,541	15,582	12,395	16,811	8,522		
3	5	14	10,659	9,722	13,634	10,614	14,737	9,644		
3	8	17	12,615	11,041	15,315	11,913	16,205	11,509		
5	4	19	13,772	11,893	16,015	12,610	16,980	10,281		
5	9	24	16,269	14,082	18,760	14,690	19,678	13,030		
5	13	28	18,049	15,687	20,816	16,170	21,573	15,211		
10	9	39	22,404	19,726	24,854	19,684	25,426	17,150		
10	18	48	26,033	22,789	28,769	22,716	29,137	20,652		
10	27	57	27,820	24,680	30,890	24,165	31,254	23,637		
15	9	54	25,469	23,087	28,397	22,445	27,972	20,322		
15	13	58	28,002	24,916	30,543	24,078	30,191	22,296		
15	27	72	32,869	29,365	35,594	28,249	35,471	27,471		
20	13	73	31,514	28,444	34,285	27,177	33,455	26,043		
20	18	78	33,748	30,343	36,391	28,878	35,575	28,078		
20	36	96	38,523	35,295	41,262	33,545	41,589	33,837		
$\sigma_3$ : co	onfinin	gpress	ure; $\sigma_d$ : deviat	toric stress; θ: I	bulk stress;	M <sub>r</sub> : resilie	nt modulus			

Table 6.15  $M_R$  Results of Martin Marietta Aggregate

Aggregate Type	Gradation	Limit	Compaction Energy	k <sub>1</sub>	k <sub>2</sub>	k <sub>3</sub>	$R^2$	M <sub>R</sub> (psi)
	M-AASHTO	Lower	Modified	127	0.5	0.12	0.995	31,618
	No. 57	Upper	Modilled	146	0.42	0.13	0.998	33,504
		Lower	Standard	161	0.4	-0.09	0.918	34,084
	M-AASHTO	Upper	Standard	148	0.5	-0.11	0.995	34,519
	No. 67	Lower	Modified	119	0.68	0.03	0.881	34,954
		Upper	Modilled	130	0.62	0.07	0.903	36,114
	ODOT	Lower	Standard	121	0.55	0.03	0.998	31,328
Anchor Stone	Type A	Upper	Standard	91	0.69	-0.03	0.995	26,252
Anchor Stone		Lower	Standard	159	0.35	0.12	0.83	34,084
	OKAA	Upper	Standard	54	0.75	0.13	0.987	17,405
	Туре К	Lower	Modified	194	0.26	0.28	0.984	39,885
		Upper	Modilled	175	0.3	0.17	0.985	36,114
		Lower	Standard	124	0.41	0.22	0.996	29,008
	OKAA	Upper	Standard	106	0.6	0.03	0.989	28,282
	Type N	Lower	Modified	109	0.61	0.11	0.994	30,458
		Upper	widdilled	132	0.5	0.19	0.988	33,359

Table 6.16 Model Constants and Design  $M_R$  Values of Anchor Stone Aggregate

Aggregate Type	Gradation	Limit	Compaction Energy	$\mathbf{k}_1$	$\mathbf{k}_2$	k <sub>3</sub>	$R^2$	M <sub>r</sub> * (psi)	% Increase <sup>#</sup>
Anchor Stone	M-AASHTO	Lower	Standard	1,226	0.463	0.058	0.996	19,860	11
	No. 57	Upper	Standard	1,090	0.505	0.105	0.970	17,862	11
	OKAA Type M	Lower	Standard	1,408	0.431	0.060	0.971	22,675	21
		Upper	Standard	1,141	0.505	0.136	0.978	18,743	21
	ODOT Type A	Lower	Modified	1,429	0.562	-0.017	0.998	23,516	27
		Upper	Modified	1,045	0.520	0.041	1.000	17,111	57
Dalaca	M-AASHTO	Lower	Standard	1,237	0.500	-0.037	0.968	20,075	2
	No. 57	Upper	Standard	1,191	0.555	0.035	0.994	19,633	2
	OKAA Type M	Lower	Standard	1,165	0.587	-0.110	0.993	19,154	33
Dolese		Upper		842	0.668	0.290	0.991	14,428	
	ODOT Type A	Lower	Modified	1,291	0.553	0.085	0.978	21,343	6/
		Upper	Mouned	767	0.543	0.506	0.925	12,984	04
	M-AASHTO	Lower	Standard	813	0.618	0.010	0.986	13,559	13
	No. 57	Upper	Standard	716	0.629	0.002	0.986	11,954	13
Montin Moniette	OKAA Type M	Lower	Standard	1,007	0.525	0.050	0.974	16,508	28
Martin Marietta		Upper	Standard	789	0.535	0.036	0.971	12,948	20
	ODOT Type A	Lower	Madified	1,072	0.471	0.108	0.968	17,459	<i>c</i> 0
		Upper	Modified	647	0.601	0.283	0.991	10,927	00

Table 6.17 Model Constants and Design  $M_R$  Values of Different Gradations

\*Design  $M_r$  value at  $\sigma_3 = 4$  psi;  $\sigma_d = 6$  psi;  $\theta = 18$  psi;  $\tau_{oct} = 0.94$  psi;  $M_r = k_1 P_a x (\theta/P_a)^{k_2} x (\tau_{oct}/P_a + 1)^{k_3}$ ; #Increase in Mr of lower limit as compared to upper limit gradation
Aggregate Type	Gradation	Limit	CE	MC (%)	DD (pcf)	% Fines	D <sub>eff</sub> (in)	$D_{10}(in)$	D <sub>30</sub> (in)	D <sub>60</sub> (in)	C <sub>U</sub>	C <sub>C</sub>	e	Mr* (psi)
	M-AASHTO	Lower	Standard	5.0	106.8	0.530	0.116	0.712	0.264	0.514	2.700	1.410	0.495	19,860
	No. 57	Upper	Stanuaru	3.3	112.8	5.040	0.017	0.469	0.141	0.274	3.320	1.140	0.472	17,862
Anchor Stone	OKAA	Lower	Standard	2.3	119.2	0.840	0.068	0.185	0.323	0.807	4.362	0.698	0.402	22,675
Anchor Stone	Type M	Upper	Stanuaru	4.6	127.8	5.830	0.012	0.008	0.089	0.341	41.238	2.784	0.308	18,743
	ODOT	Lower	Modified	4.6	140.3	6.860	0.011	0.779	0.012	0.207	62.670	4.450	0.191	23,516
	Type A	Upper	WIOUIIIEU	6.9	151.5	15.500	0.007	0.167	0.001	0.019	149.800	2.030	0.103	17,111
M-4 No. OK	M-AASHTO	Lower	Standard	2.0	108.3	0.600	0.104	0.280	0.768	0.866	3.099	2.434	0.505	20,075
	No. 57	Upper	Stanuaru	2.8	113.8	0.940	0.046	0.126	0.299	0.689	5.469	1.031	0.328	19,633
	OKAA	Lower	Standard	2.8	115.8	0.960	0.055	0.128	0.315	0.748	5.846	1.036	0.455	19,154
Dolese	Type M	Upper		4.0	126.1	5.410	0.016	0.017	0.094	0.319	19.286	1.693	0.329	14,428
	ODOT	Lower	Modified	4.2	141.2	5.160	0.017	0.015	0.189	0.772	50.256	3.014	0.195	21,343
	Type A	Upper		6.3	145.2	14.360	0.006	0.002	0.021	0.185	111.905	1.477	0.156	12,984
	M-AASHTO	Lower	Standard	2.6	102.9	0.960	0.092	0.217	0.508	0.807	3.727	1.476	0.607	13,559
	No. 57	Upper	Standaru	4.6	108.9	1.090	0.052	0.098	0.276	0.853	8.668	0.904	0.520	11,954
Martin Marietta	OKAA	Lower	Standard	2.0	111.4	0.900	0.081	0.116	0.325	0.843	7.254	1.078	0.474	16,508
	Type M	Upper	Standaru	6.0	123.4	5.920	0.017	0.008	0.087	0.307	36.792	2.980	0.347	12,948
	ODOT	Lower	Modified	5.5	136.2	7.160	0.016	0.006	0.122	0.705	110.494	3.314	0.219	17,459
	Type A	Upper	withduited	6.2	135.0	13.590	0.008	0.002	0.015	0.150	74.510	0.785	0.243	10,927

Table 6.18 Resilient Modulus Compaction Data

CE: compaction energy; MC: molded moisture content; DD: molded dry density;  $D_{eff}$ : effective diameter;  $D_{10}$ : particle size for 10% finer;  $D_{30}$ : particle size for 30% finer;  $D_{60}$ : particle size for 60% finer;  $C_U$ : coefficient of uniformity;  $C_C$ : coefficient of gradation; e: void ratio; \*Design  $M_r$  value

Independent	Parameter	Standard	Type II	F-value	Pr>F	<sup>a</sup> Significant
Variable	Estimate	Error	Sum of			
			Squares			
Intercept	3.82562	0.10138	3.27702	1424.01	<.0001	Yes
Log(MC/OMC) <sup>b</sup>	-0.25619	0.03568	0.11866	51.56	<.0001	Yes
$Log(e)^{b}$	-0.65999	0.03756	0.71037	308.69	<.0001	Yes
$Log(LA)^{c}$	-0.11457	0.04606	0.01424	6.19	0.0135	Yes
Log(CE)	-0.15481	0.02236	0.11028	47.92	<.0001	Yes
$Log(D_{30})$	0.22094	0.01392	0.57972	251.92	<.0001	Yes
$(\theta/P_a) x$						Yes
Log(DD/MDD) <sup>b</sup>	-1.7225	0.66896	0.01526	6.63	0.0106	
$Log(\theta/P_a) x$						Yes
$Log(LA)^{c}$	0.39185	0.01639	1.31492	571.39	<.0001	
$Log(\theta/P_a) x$						Yes
$Log(D_{30})$	0.18436	0.07909	0.01251	5.43	0.0205	
$Log(\theta/P_a) x$						Yes
$Log(D_{60})$	-0.40399	0.14696	0.01739	7.56	0.0064	
$Log(\tau_{oct}/P_a+1) x$						Yes
$Log(D_{60})$	-0.34278	0.15976	0.01059	4.6	0.0328	

Table 6.19 Analyses of Variance (Measured vs. Predicted  $M_R$ ) Using Original Gradation Data

<sup>a</sup>Significant at probability level (alpha) = 0.15; <sup>b</sup>molded specimen property; <sup>c</sup>aggregate property; MC: molded moisture content; OMC: optimum moisture content; e: void ratio; CE: compaction energy (ft-lbf/ft<sup>3</sup>); D<sub>eff</sub>: effective diameter (in); D<sub>30</sub>: particle size for 30% finer (in); D<sub>60</sub>: particle size for 60% finer (in); LA: Los Angeles abrasion value (%); C<sub>C</sub>: coefficient of gradation;  $\theta$ : bulk stress (psi); P<sub>a</sub>: atmospheric pressure (14.7 psi)

			FWD	Standard	Permeability	DCP Index	Average	Average	Average
Cradation	Station	Data	Modulus	Deviation	k	DCP-I	M <sub>FWD</sub>	k	DCP-I*
Gradation	Station	Date	$\mathbf{M}_{\mathbf{FWD}}$	M <sub>FWD</sub> M <sub>FWD</sub>					
			(ksi)	(ksi)	(ft/day)	(in/blow)	(ksi)	(ft/day)	(in/blow)
MAASUTO	0+50 EB		13.9		993	0.2			
# 57	1+25 EB	Apr-3-08	10.2	2.2	5920	0.2	11.4	5,790	0.2
π 57	0+75 WB		10.1		10458	-			
OKAA	2+65 EB		19.5		2.6	-			
Type M	3+65 EB	Apr-3-08	14.5	2.6	469	-	17.3	199	0.1
Type WI	2+35 WB		17.9		126	0.1			
ODOT	4+65 EB	Apr-3-08	17.4	81	0.05	-	23.2	0.1	0.04
Type A	4+15 WB	71pi 5 00	28.9	0.1	0.17	0.04	23.2		
	0+51 EB				6830	0.2		3,730	0.2
M-AASHTO	1+26 EB	Jul-23-08	N/A**	_	4681	-	-		
# 57	0+85 WB				1871	-			
	1+60 WB				1538	0.1			
	2+25 EB				41	0.1			
	2+66 EB				13	-			
OKAA	3+66 EB	Jul 22 08	N/ / * *		303	0.1		165	0.1
Type M	2+29 WB	Jui-23-08	1N/A	-	198	-	-	105	0.1
	2+89 WB				415	-			
	3+35 WB				20.8	0.1			
ODOT	4+65 EB	Jul 22 09	NI / A **		0.05	_		0.1	0.04
Type A	4+08 WB	Jul-23-08	1N/A	-	0.1	0.04	-	0.1	0.04

Table 6.20 Timberdell Road FWD, k and DCP Tests' Results (April and July 2008)

\* Dynamic cone penetration index (DCP-I); the lower the value, more stable the aggregate base is.

\*\* FWD test results are unreliable due to device over heating and calibration issues; based on the statements made be the ODOT personnel in the field (05/02/09)

			FWD	Standard	Permeability	Standard	DCP Index	Average	Average	Average
Gradation	G4 4*	Date	Modulus	Deviation	k	Deviation	DCP-I	M <sub>FWD</sub>	k	DCP-I*
	Station		M <sub>FWD</sub>	M <sub>FWD</sub>		k				
			(ksi)	(ksi)	(ft/day)	(ft/day)	(in/blow)	(ksi)	(ft/day)	(in/blow)
	0+52 EB		26.4		177	11	0.2			
M-AASHTO # 57	0+86 WB	May 2 00	19.2	5.6	627	69	0.2	20.8	437	0.2
	1+27 EB	viay-2-07	13.7		774	149	0.2			
	1+61 WB		23.8		168	10	-			
	2+27 EB		34.3	10.0	39	3	0.1	38 7	85	0.1
	2+67 EB		41.9		120	8	0.1			
OKAA	3+67 EB	May_2_00	29.2**		1.1	1	-			
Type M	2+30 WB	viay-2-09	46.2	10.7	325	21	0.1	50.7		0.1
	2+90 WB		54.6		14	2	-			
	3+36 WB		25.7		10	2	0.1			
ODOT	4+67 EB	Max_2_00	50.9	8 5	0.03	0	0.07	56.0	0.02	0.07
Type A	4+09 WB	viay-2-09	62.9	0.3	0.01	0	0.06	50.9	0.02	0.07
* Dynamic cone	penetration	index (DCF	P-I); the lov	ver the valu	ie, more stal	ole the agg	gregate base	e is.		
** FWD test performed under the name of this station is done in the location of 3+77 EB (10 ft. away)										

Table 6.21 Timberdell Road FWD, *k* and DCP Tests' Results (May 2009)

		Apr-3-08		Jul-23-08			May-2-09			Laboratory	
	]	Permeability	Average	]	Permeability	Average		Permeability	Average	Average	Lab. Range
		k	k		k	k		k	k	k	k
Gradation	Station	(ft/day)	(ft/day)	Station	(ft/day)	(ft/day)	Station	(ft/day)	(ft/day)	(ft/day)	(ft/day)
	0+50 EB	993		0+51 EB	6830		0+52 EB	177			
M-AASHTO	1+25 EB	5920	5 700	1+26 EB	4681	3 730	0+86 WB	627	127	780	27 ~ 1941
#57	0+75 WB	10458	5,790	0+85 WB	1871	5,750	1+27 EB	774	437		
				1+60 WB	1538		1+61 WB	168			
OKAA	2+65 EB	2.6		2+25 EB	41		2+27 EB	39			
Type M	3+65 EB	469		2+66 EB	13		2+67 EB	120			
	2+35 WB	126	199	3+66 EB	303	165	3+67 EB	1.1	85	574	10 1126
				2+29 WB	198	105	2+30 WB	325	85	574	19~1120
				2+89 WB	415		2+90 WB	14			
				3+35 WB	20.8		3+36 WB	10			
ODOT	4+65 EB	0.05	0.1	4+65 EB	0.05	0.1	4+67 EB	0.03	0.02	0.12	0.001 0.3
Type A	4+15 WB	0.17	0.1	4+08 WB	0.1	0.1	4+09 WB	0.01	0.02	0.12	0.001 ~ 0.5

Table 6.22 Field and	Laboratory	<b>Coefficients</b>	of Permeability	v Values
				/

1. Standard deviation of permeability is calculated for the trials done for each station.

2. Laboratory average *k* shows the average amounts of UL and LL of each gradation for Dolese aggregates.

Gradation	$M_R^*$	Average <i>M<sub>R</sub></i> *	M <sub>FWD</sub> (ksi)			
		(ksı)	(ksi)	Apr-08	May-09	
	LL	20.1	10.0	11 /	20.8	
M-AASHIO #37	UL	19.6	19.9	11.4	20.8	
$OV \Lambda \Lambda$ Type M	LL	19.2	16.8	173	287	
OKAA Type M	UL	14.4	10.8	17.5	38.7	
	LL	21.3	17.2	23.2	56.0	
ODOT Type A	UL	13.0	17.2	23.2	30.9	

Table 6.23  $M_{FWD}$  and Design  $M_R$  values Comparison

\* Design  $M_R$  value from the laboratory





Figure 6.1 Variation of OMC with Gradations of Anchor Stone Aggregate

Figure 6.2 Variation of  $\gamma_{d max}$  with Gradations of Anchor Stone Aggregate







Figure 6.4 Variation of  $\gamma_{d max}$  with Gradations of Different Aggregates







Figure 6.6 Variation of  $\gamma_{d max}$  with % Fines



Figure 6.7 Permeability Variation of Lower Limit of Anchor Stone Aggregate



Figure 6.8 Permeability Variation of Upper Limit of Anchor Stone Aggregate







Figure 6.10 Permeability Variation of Martin Marietta Aggregate







Figure 6.12 Effect of % Fines on Permeability



Figure 6.13 Effect of Density on Permeability



Figure 6.14 Variation of Permeability with Fractional Texture Index



Figure 6.15 Variation of Permeability with Fractional Radius Angularity Index



Figure 6.16 Variation of Permeability with Fractional Gradient Angularity Index



Figure 6.17 Design  $M_R$  of Anchor Stone, Dolese and Martin Marietta Agg. vs. Gradation



Figure 6.18 Effect of Compaction Efforts on M<sub>R</sub> values (Upper Limit)



Figure 6.19 Effect of Compaction Efforts on  $M_R$  values (Lower Limit)







Figure 6.21 Measured and Predicted  $M_R$  Values Using Original Gradation Data



Figure 6.22 Segregation of Aggregate During Construction of *M-AASHTO* #57

### 7.1 Introduction

This chapter presents a summary of this study and conclusions drawn from the data obtained and the analyses performed in the preceding chapters. Finally, recommendations for further research are suggested.

#### 7.2 Summary

A combined laboratory and field study was conducted in two phases to evaluate resilient modulus  $(M_R)$  as a measure of stability and coefficient of permeability (k) as an indicator of drainage of aggregate bases. Aggregates from three different sources in Oklahoma, namely Anchor Stone limestone, Dolese limestone and Martin Marietta sandstone were selected in consultation with the Oklahoma Department of Transportation (ODOT) and the Oklahoma Aggregate Association (OKAA) and tested in the laboratory under simulated traffic loading to evaluate the  $M_R$  in accordance with the AASHTO T 294-94 test method. In Phase I, five different gradations, namely M-AASHTO #57, M-AASHTO #67, ODOT Type A, OKAA Type N and OKAA Type K were selected and used for Anchor Stone aggregates. In Phase 2, the number of gradations was reduced to three, namely M-AASHTO #57, ODOT Type A, OKAA Type M. The laboratory testing program was specifically focused on the effects of compaction (standard versus modified Proctor), gradation and aggregate type and source on  $M_R$  and k values. Conventional tests, namely grain size distribution, moisture-density relationship and Los Angles abrasion, were conducted for each aggregate. Aggregate imaging system was also used to evaluate texture, shape and angularity characteristics. Regression models were developed to estimate  $M_R$  in terms of molded moisture content (%), optimum moisture content (%), void ratio

of the compacted specimen, Los Angeles abrasion value (%), compaction energy (ft-lbf/ft<sup>3</sup>), particle size for 30% finer (in.), molded dry density (pcf), maximum dry unit weight (pcf), and particle size for 60% finer (in.). A falling head permeability apparatus was fabricated and used to determine the drainage characteristics of the selected aggregates under different gradations. Regression models were developed to estimate permeability (*k*) in terms of gradation and compaction characteristics and aggregate type. ANOVA tests performed on these regression models show that they are able to estimate  $M_R$  and *k* with reasonable accuracy and, hence, may be used as tool for design of aggregate bases.

A 500 ft (152.4 m) segment of the Timberdell Road between Asp Avenue and Jenkins Avenue in Norman was constructed with three different gradations, two new gradations (*M*-*AASHTO #57* and *OKAA Type M*) and one standard gradation (*ODOT Type A*), for comparison. In addition to monitoring stability and compaction characteristics during construction using nuclear density gauge, falling weight deflectometer (FWD) and dynamic cone penetrometer (DCP), field tests were conducted to evaluate in-situ modulus ( $M_{FWD}$ ) and permeability (*k*) with time. Also, traffic data were collected selectively. The back calculated modulus ( $M_{FWD}$ ) and DCP index values from the field tests were compared with the laboratory  $M_R$  values. Also, the field permeability values were compared with their laboratory counterpart, for each gradation. Field modului of the aggregate base sections were found to increase and the permeability values were found to decrease, possibly due to additional compaction caused by traffic and other factors. Significant differences in both stability and drainage characteristics within each section were noticed. These variations were partly caused by the segregation of aggregates during spreading. Specific conclusions and recommendations from this study are given below.

# 7.3 Conclusions

From the results and analyses presented in the preceding chapter, the following conclusions are deduced:

- 1. From moisture-density relationship *ODOT Type A* upper limit (UL) showed the highest Maximum Unit Weight ( $\gamma_{d max}$ ) of 152 pcf (23.9 kN/m<sup>3</sup>) for Anchor Stone aggregates with an Optimum Moisture Content (*OMC*) of 6.6% and compacted using modified Proctpr effort. On the other hand, *M-AASHTO #57* lower limit (LL) produced the lowest  $\gamma_{d max}$  (103 pcf (16.2 kN/m<sup>3</sup>)) for Martin Marietta aggregates with an *OMC* of 3.2% and compacted using standard Proctor effort.
- 2. *OKAA Type K* LL gradation showed the lowest *OMC* (1.3%) with an  $\gamma_{d max}$  of 120.3 pcf (18.9 kN/m<sup>3</sup>) for standard Proctor, whereas, the highest *OMC* (6.6%) was recorded for *ODOT Type A* UL compacted using modified Proctor.
- 3. For all aggregate types and gradations, UL gradations showed higher  $\gamma_{d max}$  and *OMC* values in comparison to LL gradations.
- 4. Anchor Stone specimens with *M-AASHTO* #67 LL gradation showed the highest average permeability value (1777 ft/day (0.67 cm/s)) compacted using the standard Proctor. On the contrary, the lowest permeability (0.001 ft/day (4x10<sup>-7</sup> cm/sec)) was reported for *ODOT Type A* UL gradation with all the three aggregate types.
- 5. Variation of permeability was observed even within the same gradation range and same aggregate source. For example, two Dolese specimens with *M-AASHTO #57* LL gradation showed permeability values of 1941 ft/day (0.7 cm/sec) and 1120 ft/day (0.4 cm/sec), respectively. The underlying reason behind this variation is the spatial segregation of aggregates and fines in the specimen during compaction.

- 6. For all aggregate types and gradations, permeability increases when the gradation envelope shifts from upper limit to lower limit. For instance, Anchor Stone *OKAA Type K* UL showed an average permeability of 0.06 ft/day ( $2.28 \times 10^{-5}$  cm/s); where as Anchor Stone *OKAA Type K* LL produced an average permeability of 708 ft/day (0.27 cm/s) which is 11800 times higher than that of the upper limit gradation. Upper limit gradations have smaller aggregates with larger specific surface area compared to those of lower limit gradations. Larger specific area promotes higher resistance against the flow; for these reasons, lower limit gradations showed higher permeability values in comparison to upper limit gradations.
- 7. Permeability is inversely proportional to percent of fines. With 2.5% post compaction fine content and modified Proctor compaction method, Anchor Stone specimens for *OKAA Type N* LL showed an average permeability of 45 ft/day (0.02 cm/sec). On the other hand, with 10.9% post compaction fine and modified compaction effort, *OKAA Type N* UL showed an average permeability of 0.004 ft/day (1.4×10<sup>-6</sup> cm/sec) for the same aggregate type.
- 8. Increase in compaction energy leads to a reduction in permeability, as expected. For all aggregate types and gradations, Specimens compacted, using standard Proctor showed higher permeability values compared to those compacted using modified Proctor.
- 9. The coefficient of permeability (k) also depends on the original gradation characteristics and void ratio. Based on the results, k is found to increase with the increase in void ratio (e). Contrariwise a decrease in coefficient of permeability was observed with the decrease in Coefficient of Curvature  $(C_c)$ .
- 10. According to Federal Highway Administration (FHWA) guidelines, minimum permeability of an aggregate base is to be 1000 ft/day (0.35 cm/s). Throughout the laboratory study none of the upper limit gradations satisfied the FHWA drainage requirement. Moreover, in the

field only *M-AASHTO #57* gradation satisfied the FHWA guideline for minimum drainage requirement.

- 11. For open-graded base layers permeability can increase due to the increase in angularity of aggregates even in the presence of fines. However, as the amount of fines increases and the gradation shifts to a denser aggregate packing, permeability of aggregates can no longer be attributed to characteristic related to aggregate shape alone. Rather, permeability is influenced by a number of factors such as aggregate packing, porosity, dry density, different physical properties in a synergistic way.
- 12. Large variations in field permeability were observed over different stations within the same gradation. This observation confirms the laboratory findings that spatial variation and segregation in the aggregate fine matrix can cause variations in permeability.
- 13. Field gradations involving *M*-AASHTO #57, OKAA Type *M* and ODOT Type A show that these gradations were a mixture (median) of lower and upper limit gradations. From the first field testing, following construction, the *k* values obtained for *M*-AASHTO #57, OKAA Type *M* and ODOT Type A gradations were 5790 ft/day (2.0 cm/sec), 199 ft/day (0.1 cm/sec) and 0.1 ft/day ( $3.5 \times 10^{-5}$  cm/sec), respectively. In the second field testing (after approximately 3.5 months), the in-situ permeability values reduced to 3730 ft/day (1.3 cm/sec), 165 ft/day (0.1 cm/sec) and 0.1 ft/day ( $3.5 \times 10^{-5}$  cm/sec) for *M*-AASHTO #57, OKAA Type *M* and ODOT Type A gradations, respectively. In the third field testing (approximately 10 months from the second field testing), the average *k* values for the corresponding gradations further reduced to 437 ft/day (0.2 cm/sec) , 85 ft/day (0.03 cm/sec) and 0.02 ft/day ( $7.1 \times 10^{-6}$  cm/sec). These reductions reflected 88%, 48% and 80% lower *k* values in the corresponding sections, compared to the *k* values from the second field test. Thus, traffic- induced compaction can

contribute to reduced field permeability, as observed here for all three gradations used in the field for this study.

- 14. Resilient modulus of all aggregate types and gradations increased with the increase in confining pressure. For a confining pressure ( $\sigma_3$ ) of 3 psi (0.02 MPa) and deviatoric stress ( $\sigma_d$ ) of 3 psi (0.02 MPa), Anchor Stone specimens with modified compaction effort for *ODOT Type A* UL showed an  $M_R$  of 13.6 ksi (93.77 MPa), whereas, for a confining pressure ( $\sigma_3$ ) of 20 psi (0.14 Mpa) and deviatoric stress ( $\sigma_d$ ) of 36 psi (0.25 MPa), the same specimens showed an  $M_R$  of 60 ksi (413.69 MPa).
- 15. Resilient Modulus ( $M_R$ ) of specimens for a particular gradation decreased with the increase in fines over the gradation envelope. Lower limit of each gradation produced higher  $M_R$  values compared to those of upper limit. The  $M_R$  values did not correlate well with the octahedral stress but showed better correlations with bulk stress, compaction energy, dry unit weight, percent fines,  $C_U$ ,  $D_{eff}$ , and  $D_{60}$ . The  $M_R$  values decrease with increase in moisture content. No marked difference is observed in  $M_R$  values amongst the gradations tested in this study.
- 16. Based on the laboratory  $M_R$  results, a regression model was developed in this study. According to this model, the highest design resilient modulus (23.5 ksi (162.03 MPa)) (for  $\sigma_3 = 4 psi (0.03 MPa), \sigma_d = 6 psi (0.04 MPa) and \theta = 18 psi (0.12 MPa)$ ) was observed for *ODOT Type A* lower limit gradation of Anchor Stone aggregates.
- 17. Critical analyses of FWD results show that the results were reliable only for April 2008 and May 2009 testing. According to these measurements, average M<sub>FWD</sub> values of *M-AASHTO* #57, OKAA Type M and ODOT Type A gradations measured in April 2008 were 11.4 ksi (78.60 MPa), 17.3 ksi (119.28 MPa) and 23.2 ksi (159.96 MPa), respectively. These values in the next measurements performed in May 2009 increased to 20.8 ksi (143.41 MPa) ,showing

100% increase, 38.7 ksi (266.83 MPa) showing 124% increase and 56.9 ksi (392.31 MPa) exhibiting 145% increase for *M*-AASHTO #57, OKAA Type M and ODOT Type A gradations, respectively. Hence, traffic- induced compaction of aggregate bases can result in an increase in  $M_R$  values. This effect is more pronounced in dense graded aggregates.

#### **7.4 Recommendations**

The following recommendations are made for future studies:

- 1. In the present study, significant variations in in-situ permeability and back-calculated modulus were observed at different locations within the same sub-section or gradation. Segregation of aggregates during construction of the test section is believed to be a major contributing factor for such variations. It is recommended that a different spreading and compaction technique be used in a future study to ensure uniformity within a given section/gradation. A comparison of in-situ flow and stability from such sections will be a more realistic indicator of the influence of gradation on field performance of aggregate bases. Also, field performance should be monitored over a longer period of time than the limited time used in the present study.
- 2. Measurement of in-situ permeability using falling head method for dense gradation (*ODOT Type A*) was found problematic because the change in water level within the standpipe was too slow. Using a system capable of including both elevation head and pressure head will provide more flexibility and accuracy in field permeability measurements of dense graded aggregate bases. It is recommended that such measurements be considered in future studies.
- 3. Due to limited scope, the present study did not consider local validation of the 2002 mechanistic-empirical pavement design guide (MEPDG) for aggregate bases. It is suggested

that a future study be pursued, using data developed in the present study, on local validation of MEPDG for pavements with drainable aggregate bases.

- 4. Although laboratory and field data are tremendously useful, numerical modeling generally provides valuable insights, particularly with respect to parametric studies (i.e. influence of different factors on stability and flow characteristics of aggregate bases). It is recommended that future studies be undertaken on pertinent numerical modeling.
- 5. Because of limited scope, the present study considered only three different gradations (except Anchor Stone) and aggregates from three selected sources. It is suggested that additional gradations (having potential field applications) and aggregates from other sources that are commonly used in Oklahoma be considered in future studies.
- 6. Stability of open graded aggregates, where lack of stability is an issue, can be increased by using geogrids. Currently no uniform specifications are available for use of geogrids with aggregates. It is recommended that future studies be undertaken to obtain necessary data for developing such specifications. Also, future studies should address industry needs.

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## APPENDIX – A

Permeability Testing

Prepared I	oy:								
Date:									
Sample#: 0	ОКАА_Тур	e_M_UL_	Std_k_1						
Compactio	on Type: Ir	npact (Sta	ndard)						
Aggregate	Type: An	chor Stone							
Mold Dian	neter:	6	in			MDD	(pcf)	130	
Mold Heig	ht:	4.584	in			OMO	C (%)	4.52	
Mold Volu	ime:	0.075	ft <sup>3</sup>			Projecte	ed w (%)	4.5	
No. of Lay	ers:	3				Ywet	(pcf)	135	
Blows per	layer:	56				Ydry	(pcf)	129	
Wt. per la	yer:	1850 gm							
Wt. of wat	er:	153	gm						
Point Wei	ght:	5550	gm						
						A	fter Washi	ng	
Sieve #	Size	% Det	Mt (am)	Actual	Molded	%	Cum. %	%	L/O Wt.
Sieve #	(mm)	70 REL.	wt. (gill)	Wt. (gm)	Wt. (gm)	Retained	Retained	Passing	(gm)
1.5 in	37.5	0	0	0	0	0.00	0	100	0
1 in	25.4	0	0	0	0	0.00	0	100	0
3/4 in	19	0	0	0	0	0.00	0	100	0
3/8 in	9.5	40	2220	2220	1575	35.98	35.98	64.02	575.6
#4	4.75	20	1110	1110	877.7	20.05	56.03	43.97	183.6
#10	2	15	832.5	834	712.9	16.28	72.31	27.69	115
#40	0.425	15	832.5	834	623.8	14.25	86.56	13.44	126.2
#200	0.075	5	277.5	270	342.9 10.0	7.85	94.39	5.01	10
Pall	U Total V	J Vt (am)	5550	555/	10.9	0.25	94.04	5.50	1.0
	TOTAL	vt. (gill)	3330	5554	4143.2				1077.2
%Loss =		5 61%	1						
/0 2033 -		5.0170							
Notes:						D	0.0307	cm	
Notes.						D <sub>eff</sub>	0.0307		
						D <sub>10</sub>	0.019	CIII	
Noisture I	Jetermina	ation:				D <sub>30</sub>	0.22	cm	
						D <sub>60</sub>	0.84	cm	
After Pe	rmeability	/ Testing 8	Washing						
Mold Wt.		5780	gm		Gradation Parameters:				
Mold + We	et Agg Wt.	10356	gm						
Com. Agg.	Wt.	4576	gm		After Permeability Testir			Washing	
Pan No.		CK 1							
Pan Wt.		576	gm		Effective :	Size (cm)		0.019	
Pan + Dry /	Agg Wt.	4954	gm		Coefficier	nt of Unifor	mity, C <sub>u</sub>	44.21	
Dry Agg. W	/t.	4378	gm		Coefficient of Curvature, C <sub>c</sub> 3.03				
Actual Wa	ter	198	gm						
MC (%)		4.52	%						

Example Permeability Specimen Preparation Data Sheet - Anchor Stone Aggregate

Type M Ag	ggregate B	ase Gradat	ion (Uppe	<u>r Limit)</u>				
Date:								
Performe	d By:							
Aggregate	<b>Source:</b> A	nchor Ston	е					
Compactio	on Type: In	npact (Stan	idard)					
Sample#:	Type M-Up	per-Std-k-	2					
Pressure:	10 psi	Water Ter	<b>np.:</b> 28 °C					
L: 4.584 in	/ 11.64336	cm	•					
Head, h	Head,h		Time, dt	Velocity,				
(in)	(cm)	dh (cm)	(sec)	v (cm/s)	i			
29	73.66	0	()	(- /-/				
28	71.12	2.54	53.95	0.0471	6.1082			
27	68.58	5.08	111.55	0.0455	5.8901			
26	66.04	7.62	169.45	0.0450	5.6719			
25	63.5	10.16	227.89	0.0446	5.4538			
24	60.96	12.7	288.92	0.0440	5.2356			
23	58.42	15.24	351.64	0.0433	5.0175			
22	55.88	17.78	415.48	0.0428	4.7993			
21	53.34	20.32	482.36	0.0421	4.5812			
20	50.8	22.86	559.89	0.0408	4.3630			
_		ΟΚΑΑ	Type M	UL (And	hor Ston	e)		
	0500			•		•		
0.0	0500							
<b>•</b> •••								
	0430							
	0400	•		- ,	y = 0.0245x <sup>0.</sup>	3532		
it,	0400				$R^2 = 0.977$	8		
elo								
> 0.0								
0.0	0300							
0.	1 0000	Л	5000	5 000	n	5 5000	6.00	00
	4.0000	4	.5000	5.000	0	5.5000	0.000	
			н	ydraulic Gr	adient, i			
Permeabi	lity <i>, k</i> = 0.0	)245cm/s =	69 ft/day					

# Example Permeability Test Data Sheet – Anchor Stone Aggregate

Prepared I	oy: Kazme	е							
Date:									
Sample#:	ОКАА_Тур	e_M_UL_	Std_k_1						
Compactio	on Type: Ir	npact (Sta	ndard)						
Aggregate	Type: Dol	lese							
Mold Dian	neter:	6	in			MDD	(pcf)	127	
Mold Heig	ht:	4.584	in			OMO	C (%)	3.83	
Mold Volu	ime:	0.075	ft <sup>3</sup>			Projecte	ed w (%)	4	
No. of Lay	ers:	3				Ywet	(pcf)	133	
Blows per	layer:	56				Ydry	(pcf)	128	
Wt. per la	yer:	1850	gm						
Wt. of wat	ter:	153	gm						
Point Wei	ght:	5550	gm						
						A	fter Washi	ng	
Sieve #	Size	% Ret	Wt (gm)	Actual	Molded	%	Cum. %	%	L/O Wt.
Sieve n	(mm)	70 Het.	<b>W</b> (1 (Bill)	Wt. (gm)	gm) Wt. (gm) Retained Retain			Passing	(gm)
1.5 in	37.5	0	0	0	0	0.00	0	100	0
1 in	25.4	0	0	0	0	0.00	0	100	0
3/4 in	19	0	0	0	0	0.00	0	100	0
3/8 in	9.5	40	2220	2220	1627.5	37.43	37.43	62.57	564.9
#4	4.75	20	1110	1110	835.1	19.21	56.64	43.36	104
#10	2	15	832.5	834	859.5	19.77	76.41	23.59	123.7
#40	0.425	15	832.5	834	660.7	15.20	91.61	8.39	138.9
#200	0.075	5	277.5	278	206.1	4.74	96.35	3.65	53.1
Pan		5	277.5	2/8	9.1	0.21	96.56	3.44	1
	lotal v	vt. (gm)	5550	5554	4198				985.6
%Loss =		3 65%	1						
Notes:		5.6576				D <sub>eff</sub>	0.0528	cm	
						D <sub>10</sub>	0.05	cm	
Moisture I	Determina	ation:				D <sub>30</sub>	0.27	cm	
						D <sub>60</sub>	0.87	cm	
After Pe	rmeability	/ Testing 8	Washing						
Mold Wt.		5780	gm		Gradation	Paramete	rs:		
Mold + We	et Agg Wt.	10294	gm						
Com. Agg.	Wt.	4514	gm		After Pe	rmeability	Testing &	Washing	
Pan No.		Agg # 12							
Pan Wt.		524	gm		Effective	Size (cm)		0.053	
Pan + Dry	Agg Wt.	4871.6	gm		Coefficier	nt of Unifo	rmity, C <sub>u</sub>	17.40	
Dry Agg. W	/t.	4347.6	gm		Coefficier	nt of Curva	ture, C <sub>c</sub>	1.68	
Actual Wa	ter	166.4	gm						
MC (%)		3.83	%						

# Example Permeability Specimen Preparation Data Sheet - Dolese Aggregate

Туре М А	ggregate B	ase Gradat	ion (Uppe	<u>r Limit)</u>			
Date:							
Performe	d By:						
Aggregate	Source: D	olese					
Compactio	on Type: In	npact (Star	idard)				
Sample#:	Type M-Up	per-Std-k					
Pressure:	10 psi	Water Ter	<b>np.:</b> 20 °C				
<b>.:</b> 4.584 in	/ 11.64336	cm					
Head, h	Head,h	dh (cm)	Time, dt	Velocity,			
(in)	(cm)	un (un)	(sec)	v (cm/s)			
27	68.58	0					
26	66.04	2.54	55.65	0.0456	5.6719		
25	63.5	5.08	112.83	0.0450	5.4538		
24	60.96	7.62	174.36	0.0437	5.2356		
23	58.42	10.16	242.37	0.0419	5.0175		
22	55.88	12.7	312.08	0.0407	4.7993		
21	53.34	15.24	393.24	0.0388	4.5812		
20	50.8	17.78	475.21	0.0374	4.3630		
19	48.26	20.32	566.68	0.0359	4.1449		
18	45.72	22.86	674.83	0.0339	3.9267		
17	43.18	25.4	802.55	0.0316	3.7086		
16	40.64	27.94	946.18	0.0295	3.4904		
15	38.1	30.48	1109.87	0.0275	3.2723		
14	35.56	33.02	1299.4	0.0254	3.0541		
13	33.02	35.56	1504.02	0.0236	2.8360		
12	30.48	38.1	1727.83	0.0221	2.6178		
11	27.94	40.64	1989	0.0204	2.3997		
10	25.4	43.18	2290	0.0189	2.1815		
9	22.86	45.72	2610	0.0175	1.9634		
8	20.32	48.26	3048	0.0158	1 7452		
7	17 78	50.8	3555	0.0143	1 5271		
6 25	15 875	52 705	4111	0.0143	1 3634		
0.25	15.075	52.705	-111	0.0120	1.5054		
0.0	0500	ОКА	А Туре	MUL	(Doles	e)	<b>r•</b>
(s)					معرفاه	- 0.0255	
<u>ق</u> 0.0	300 +				y:	$= 0.0094x^{0.9256}$ $R^2 = 0.0068$	-
^ ^						0.1500	
0.0 <b>Velocit</b>	200	معمو					
0.0	100						
0.0							
0.0	1.0000	2.000	0 3.0	0000	4.0000	5.0000	6.0000
			Hy	draulic Grad	dient, i		-

# Example Permeability Test Data Sheet – Dolese Aggregate

Prepared b	oy:								
Date:									
Sample#: 0	ОКАА_Тур	e_M_UL_	Std_k_2						
Compactio	on Type: Ir	npact (Sta	ndard)						
Aggregate	Type: Ma	rtin Marie	etta						
Mold Dian	neter:	6	in			MDD	(pcf)	123.5	
Mold Heig	ht:	4.584	in			OMO	C (%)	6.66	
Mold Volu	ime:	0.075	ft <sup>3</sup>			Projecte	ed w (%)	6.8	
No. of Lay	ers:	3				Ywet	(pcf)	132	
Blows per	layer:	56				Ydry	(pcf)	123.5	
Wt. per la	yer:	1850	gm						
Wt. of wat	ter:	153	gm						
Point Wei	ght:	5550	gm						
						A	fter Washi	ng	
Sieve #	Size	% Ret	Wt (gm)	Actual	Molded	%	Cum. %	%	L/O Wt.
	(mm)	70 HCt.		Wt. (gm)	Wt. (gm)	Retained	Retained	Passing	(gm)
1.5 in	37.5	0	0	0	0	0.00	0	100	0
1 in	25.4	0	0	0	0	0.00	0	100	0
3/4 in	19	0	0	0	0	0.00	0	100	0
3/8 in	9.5	40	2220	2220	1508.8	35.91	35.91	64.09	598.8
#4	4.75	20	1110	1110	873.7	20.79	56.7	43.3	241
#10	2	15	832.5	834	667.6	15.89	/2.59	27.41	169.1
#40	0.425	15	832.5	834	663.2	15.78	88.37	11.63	1/9.3
#200	0.075	5	277.5	2/8	237.9	5.00	94.03	5.97	92.5
Pan	U tal \\/t (a)	5 m)	277.5		31.0	0.75	94.78	5.22	4.7
10	tai vvt. (gi	E 07%	5550	5554	5962.0				1265.4
Notos:		3.97/0	<u> </u>						
NOLES.						<b>D</b>	0.0459	<b></b>	
						D <sub>eff</sub>	0.0458	CITI	
						D <sub>10</sub>	0.027	cm	
Moisture I	Determina	ation:				D <sub>30</sub>	0.23	cm	
						D <sub>60</sub>	0.81	cm	
After Pe	rmeability	/ Testing 8	k Washing						
Mold Wt.		5780	gm		Gradation	Paramete	rs:		
Mold + We	et Agg Wt.	10262	gm						
Com. Agg.	Wt.	4482	gm		After Pe	<mark>rmeability</mark>	Testing &	Washing	
Pan No.	No. B 12								
Pan Wt. 612		gm		Effective	Size (cm)		0.0458		
Pan + Dry A	Pan + Dry Agg Wt. 4814		gm		Coefficier	nt of Unifor	mity, C <sub>u</sub>	30.00	
Dry Agg. W	ry Agg. Wt. 4202		gm		Coefficier	nt of Curva	ture, C <sub>c</sub>	2.42	
Actual Wat	tual Water 280		gm						
MC (%)		6.66	%						

Example Permeability Specimen Preparation Data Sheet – Martin Marietta Aggregate

OKAA type	e M Aggre	gate Base C	Gradation (L	Jpper Limit	t <u>)</u>		
Date:							
Performe	d By:						
Aggregate	Source: N	lartin Mari	etta				
Compactio	on Type: In	npact (Stan	idard)				
Sample#:	ОКАА Туре	e M-Upper-	-2-Std-k				
Pressure:	10 psi	Water Ter	<b>np.:</b> 24.2 °C				
<b>L:</b> 4.584 in	/ 11.64336	cm					
Head, h	Head,h	dh (cm)	Time, dt	Velocity,	;		
(in)	(cm)	un (chi)	(sec)	v (cm/s)	•		
29	73.66						
28	71.12	2.54	60.05	0.0423	6.1082		
27	68.58	5.08	115.24	0.0441	5.8901		
26	66.04	7.62	175.8	0.0433	5.6719		
25	63.5	10.16	233.99	0.0434	5.4538		
24	60.96	12.7	298.55	0.0425	5.2356		
23	58.42	15.24	362.05	0.0421	5.0175		
22	55.88	17.78	429.34	0.0414	4.7993		
21	53.34	20.32	500.15	0.0406	4.5812		
20	50.8	22.86	568.52	0.0402	4.3630		
19	48.26	25.4	644.43	0.0394	4.1449		
18	45.72	27.94	721.02	0.0388	3.9267		
17	43.18	30.48	799.47	0.0381	3.7086		
16	40.64	33.02	885.15	0.0373	3.4904		
-		ОКА	A Type	MUL	(MM)		

### Example Permeability Test Data Sheet – Martin Marietta Aggregate



### APPENDIX – B

**Resilient Modulus Testing** 



Modified AASHTO #57 LL Standard Effort – Anchor Stone (Phase 1)



Modified AASHTO #57 UL Standard Effort – Anchor Stone (Phase 1)



AASHTO #67 LL Standard Effort – Anchor Stone (Phase 1)



AASHTO #67 UL Standard Effort – Anchor Stone (Phase 1)



OKAA Type K LL Standard Effort – Anchor Stone (Phase 1)



OKAA Type K UL Standard Effort – Anchor Stone (Phase 1)



OKAA Type N LL Standard Effort – Anchor Stone (Phase 1)



OKAA Type N UL Standard Effort – Anchor Stone (Phase 1)



**ODOT Type A LL Standard Effort – Anchor Stone (Phase 1)** 



**ODOT Type A UL Standard Effort – Anchor Stone (Phase 1)** 



Modified AASHTO #57 LL Modified Effort - Anchor Stone (Phase 1)



Modified AASHTO #57 UL Modified Effort – Anchor Stone (Phase 1)



AASHTO #67 LL Modified Effort – Anchor Stone (Phase 1)



AASHTO #67 UL Modified Effort – Anchor Stone (Phase 1)



OKAA Type K LL Modified Effort – Anchor Stone (Phase 1)



OKAA Type K UL Modified Effort – Anchor Stone (Phase 1)



OKAA Type N LL Modified Effort – Anchor Stone (Phase 1)



OKAA Type N UL Modified Effort – Anchor Stone (Phase 1)



**ODOT Type A LL Modified Effort – Anchor Stone (Phase 1)** 



**ODOT Type A UL Modified Effort – Anchor Stone (Phase 1)** 



OKAA Type M LL Standard Effort - Anchor Stone (Phase 2)



OKAA Type M UL Standard Effort - Anchor Stone (Phase 2)



AASHTO #57 LL Standard Effort - Dolese (Phase 2)



AASHTO #57 UL Standard Effort - Dolese (Phase 2)



OKAA Type M LL Standard Effort - Dolese (Phase 2)



OKAA Type M UL Standard Effort - Dolese (Phase 2)



ODOT Type A LL Modified Effort - Dolese (Phase 2)



**ODOT Type A UL Modified Effort - Dolese (Phase 2)** 



AASHTO #57 LL Standard Effort - Martin Marietta (Phase 2)



AASHTO #57 UL Standard Effort - Martin Marietta (Phase 2)







OKAA Type M UL Standard Effort - Martin Marietta (Phase 2)







ODOT Type A UL Modified Effort - Martin Marietta (Phase 2)

## **APPENDIX** – C

Falling Weight Deflectometer Testing

					TTI	MODULUS	ANALYSIS	SYSTE	M (SUMMAS	RY REPORT)			(V	Version 6	.0)
District:							-		Þ	MODULI RANG	E(psi)				
county :							Thicknes	s(1n)	MI	inimum	Maximum	Poisson	Ratio V	alues	
Highway/Ro	oad:				Pavem	ent:	4.5	0		80,000	580,000	H1:	V = 0.3	8	
					Base:		8.0	0		10,000	150,000	H2:	V = 0.3	5	
					Subbas	se :	6.0	0		5.000	150,000	H3 :	V = 0.3	5	
					Subarz	de -	66.0	(hy DB	1	10	000	H4 -	V = 0.4	0	
					o ao gra		00.0.	1 (0) 00	/	20,	000				
Station	Load (1bs)	Measur R1	ed Defle R2	ction ( R3	mils): R4	R5	R6	R7	Calculate SURF(E1)	ed Moduli v BASE(E2)	alues (ksi SUBB(E3)	): A SUBG(E4) E	bsolute RR/Sens	Dpth to Bedrock	
50.000	11,972	22.97	15.83	13.84	7.20	4.15	2.79	1.82	340.9	29.2	84.4	9.2	4.05	84.5	
Mean:		22.97	15.83	13.84	7.20	4.15	2.79	1.82	340.9	29.2	84.4	9.2	4.05	84.5	
Std. Dev:		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.0	0.0	0.0	0.00	0.0	
Var Coeff	(%):	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.0	0.0	0.0	0.00	0.0	

### 0+50 EB (AASHTO #57) Falling Weight Deflectometer Test Result – 07/23/2008

					TTI	MODULUS	ANALYSIS	SYSTE	M (SUMMAR	Y REPORT)			(ν	ersion 6	.0)
District:	17 (Brya	m)							N	ODULI RANG	BE(psi)				
County :	21 (BRAZ	OS)					Thicknes	s(in)	Mi	nimum	Maximum	Poisson	Ratio V	alues	
Highway/R	oad: FM2	818			Paveme	nt:	4.5	0		20,000	460,000	H1:	v = 0.3	8	
							8.0	0		10,000	150,000	H2:	v = 0.3	5	
					Subbas	e :	6.0	0		5,000	150,000	H3:	V = 0.3	5	
					Subgra	de:	58.3	9 (by DB	)	10,	000	H4:	V = 0.4	0	
Station	Load Measured Deflecti Station (lbs) R1 R2 R				mils): R4	R5	R6	R7	Calculate SURF(E1)	ed Moduli v BASE(E2)	values (ksi) SUBB(E3)	: Al SUBG(E4) E	bsolute RR/Sens	Dpth to Bedrock	
85.000	11,950	29.61	21.18	18.37	9.26	5.19	3.42	2.09	360.0	15.7	114.9	6.4	4.01	76.9	
Mean: Std. Dev: Var Coeff	(%):	29.61 0.00 0.00	21.18 0.00 0.00	18.37 0.00 0.00	9.26 0.00 0.00	5.19 0.00 0.00	3.42 0.00 0.00	2.09 0.00 0.00	360.0 0.0 0.0	15.7 0.0 0.0	114.9 0.0 0.0	6.4 0.0 0.0	4.01 0.00 0.00	76.9 0.0 0.0	

0+85 EB (AASHTO #57) Falling Weight Deflectometer Test Result – 07/23/2008

				TTI	MODULUS	ANALYSIS	SYSTE	M (SUMMA)	RY REPORT)			(V	ersion 6.0)	
District:								1	ODULI RANG	HE(psi)				
County :						Thicknes	s(in)	M	inimum	Maximum	Poisson	ı Ratio V	alues	
Highway/Road:				Paveme	ent:	3.8	0		80,000	580,000	H1:	v = 0.3	8	
				Base:		8.0	0		10,000	150,000	H2 :	v = 0.3	5	
				Subbas	se :	6.0	0		5,000	150,000	H3 :	v = 0.3	5	
				Subgra	ide:	117.6	2 (by DB	)	10,	000	H4 :	v = 0.4	0	
Load	Measur	red Defle	ection (	mils):				Calculate	ed Moduli v	alues (ksi	): A	ubsolute :	Dpth to	
Station (1bs)	R1	R2	R3	R4	R5	R6	R7	SURF (E1)	BASE (E2)	SUBB(E3)	SUBG(E4) E	RR/Sens	Bedrock	
126.000 12,038	3 23.76	16.30	13.95	6.91	3.87	2.75	1.77	399.0	40.0	22.3	14.7	4.27	135.4	
Mean:	23.76	16.30	13.95	6.91	3.87	2.75	1.77	399.0	40.0	22.3	14.7	4.27	135.4	
Std. Dev:	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.0	0.0	0.0	0.00	0.0	
Var Coeff(%):	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.0	0.0	0.0	0.00	0.0	

# 1+26 EB (AASHTO #57) Falling Weight Deflectometer Test Result – 07/23/2008

					TTI	MODULUS	ANALYSIS	SYSTE	M (SUMMAS	RY REPORT)			(Vers	ion 6.0)
District: County : Highway/Ro	oad:				Paveme Base: Subbas Subgra	nt: e: de:	Thicknes 3.8 8.0 6.0 107.6	s(in) 0 0 7(by DB	ז א: )	MODULI RANG inimum 80,000 10,000 5,000 10,	3E(psi) Maximum 580,000 150,000 150,000 000	Poissor H1: H2: H3: H4:	Ratio Valu v = 0.38 v = 0.35 v = 0.35 v = 0.35 v = 0.40	es
Station	Load (1bs)	Measur R1	ed Defle R2	ction ( R3	mils): R4	R5	R6	R7	Calculate SURF(E1)	ed Moduli v BASE(E2)	values (ksi) SUBB(E3)	): J SUBG(E4) H	Absolute Dpt SRR/Sens Bed	h to rock
127.000	12,005	23.96	16.32	14.02	7.05	3.89	3.14	1.76	481.0	26.9	61.0	13.0	5.92 12	5.5
Mean: Std. Dev: Var Coeff	(%):	23.96 0.00 0.00	16.32 0.00 0.00	14.02 0.00 0.00	7.05 0.00 0.00	3.89 0.00 0.00	3.14 0.00 0.00	1.76 0.00 0.00	481.0 0.0 0.0	26.9 0.0 0.0	61.0 0.0 0.0	13.0 0.0 0.0	5.92 12 0.00 0.00	5.5 0.0 0.0

1+27 EB (AASHTO #57) Falling Weight Deflectometer Test Result – 07/23/2008

				TTI	MODULUS	ANALYSIS	SYSTE	M (SUMMAS	RY REPORT)			(V	ersion 6	. 0)
District:17 (Brya County :21 (BRA2 Highway/Road: FM2		Paveme Base : Subbas Subgra	nt: e: ide:	Thicknes 4.6 8.0 6.0 88.9	s(in) 0 0 6(by DB	р М: )	00DULI RANG inimum 20,000 10,000 5,000 10,	<pre>IE(psi)     Maximum     460,000     150,000     150,000 000</pre>	Poisson H1: H2: H3: H4:	Ratio V v = 0.3 v = 0.3 v = 0.3 v = 0.4	alues 8 5 5 0			
Load Station (lbs)	Measur R1	red Defle R2	R3 (T	nils): R4	R5	R6	R7	Calculate SURF(E1)	ed Moduli v BASE(E2)	alues (ksi SUBB(E3)	): A SUBG(E4) E	bsolute RR/Sens	Dpth to Bedrock	
160.000 11,841	19.91	14.02	12.32	7.04	4.50	3.11	1.63	312.8	46.9	73.8	10.8	2.85	107.6	
Mean: Std. Dev: Var Coeff(%):	19.91 0.00 0.00	14.02 0.00 0.00	12.32 0.00 0.00	7.04 0.00 0.00	4.50 0.00 0.00	3.11 0.00 0.00	1.63 0.00 0.00	312.8 0.0 0.0	46.9 0.0 0.0	73.8 0.0 0.0	10.8 0.0 0.0	2.85 0.00 0.00	107.6 0.0 0.0	

#### 1+60 WB (AASHTO #57) Falling Weight Deflectometer Test Result – 07/23/2008

				TTI	MODULUS	ANALYSIS	SYSTE	M (SUMMAR	RY REPORT)			(Version 6.0	0)
District: County : Highway/Road:		Pavem Base : Subbas Subgra	int: ie: ide:	Thickness 4.90 8.00 6.00 103.60	s(in) 0 0 0 0 (by DB	м мі	MODULI RANG inimum 60,000 10,000 5,000 10,	<pre>JE (psi) Maximum 550,000 150,000 150,000 000</pre>	Poisson Hl: H2: H3: H4:	Ratio Values v = 0.38 v = 0.35 v = 0.35 v = 0.40			
Load Station (1bs)	Measur R1	ed Deflec R2	R3 (n	nils): R4	R5	R6	R7	Calculate SURF(E1)	ed Moduli v BASE(E2)	values (ksi) SUBB(E3)	: A SUBG(E4) E	bsolute Dpth to RR/Sens Bedrock	
225.000 11,994	15.94	11.16	9.78	5.49	3.45	2.41	1.56	444.8	40.6	138.0	14.9	2.53 122.5	
Mean: Std. Dev: Var Coeff(%):	15.94 0.00 0.00	11.16 0.00 0.00	9.78 0.00 0.00	5.49 0.00 0.00	3.45 0.00 0.00	2.41 0.00 0.00	1.56 0.00 0.00	444.8 0.0 0.0	40.6 0.0 0.0	138.0 0.0 0.0	14.9 0.0 0.0	2.53 122.5 0.00 0.0 0.00 0.0	

2+25 EB (OKAA Type M) Falling Weight Deflectometer Test Result – 07/23/2008

	т	TI MODULUS	ANALYSIS	SYSTEM	(SUMMARY	REPORT)			(Version 6.0)
District:17 (Bryan)					MOL	OULI RANGE (P	osi)		
County :21 (BRAZOS)			Thickness (	in)	Mini	imum	Maximum	Poisson Rati	o Values
Highway/Road: FM2818	Pa	vement:	3.75		20	0,000	480,000	H1: V =	0.38
	Ba	se:	8.00		10	0,000	150,000	H2: V =	0.35
	Su	bbase:	6.00		5	5,000	150,000	H3: V =	0.35
	Su	bgrade:	282.25(	by DB)		10,000	)	H4: V =	0.40
Load Measured	Deflection (mils	) =			Calculated	Moduli valu	ues (ksi):	Absolu	te Dpth to
Station (lbs) R1	R2 R3 R	4 R5	R6 1	R7 8	SURF(E1) E	BASE(E2) SU	JBB (E3) SUE	G(E4) ERR/Se	ns Bedrock
229.000 12,005 14.95 1	0.36 9.00 5	.25 3.63	2.55	1.54	276.7	124.1	34.6	22.8 1.	91 300.0
Mean- 14 95 1	0 36 9 00 5	25 3 63	2 55	1 54	276 7	124 1	34.6	22.8 1	91 300 0
Std Dev. 0.00	0 00 0 00 0	00 0.00	0.00	0 00	0.0	0.0	0.0	0.0 0	00 0.0
Var Coeff(%) · 0.00	0 00 0 00 0	00 0.00	0.00	0 00	0.0	0.0	0.0	0.0 0.	00 0.0

#### 2+29 WB (OKAA Type M) Falling Weight Deflectometer Test Result – 07/23/2008

				TTI	MODULUS	ANALYSIS	SYSTE	M (SUMMAR	Y REPORT)			(Versic	n 6.0)
District:								N	ODULI RANG	E(psi)			
County :						Thickness	s(in)	Mi	nimum	Maximum	Poisson	1 Ratio Values	
Highway/Road:				Pavem	ent:	4.40	)		60,000	550,000	H1	: v = 0.38	
				Base:		8.0	)		10,000	150,000	H2	: V = 0.35	
				Subbas	se :	6.0	)		5,000	150,000	H3	: V = 0.35	
				Subgra	ide:	143.2	(by DB	)	10,	000	H4:	v = 0.40	
Load Station (lbs)	Measur Rl	ed Deflec R2	rtion (r R3	mils): R4	R5	R6	R7	Calculate SURF(E1)	d Moduli v BASE(E2)	values (ksi) SUBB(E3)	: J SUBG(E4) I	Absolute Dpth SRR/Sens Bedro	to ck
266.000 12,038	14.81	10.49	9.30	5.41	3.51	2.54	1.62	403.1	84.6	56.1	18.1	2.89 161.	7
Mean: Std. Dev:	14.81 0.00	10.49	9.30	5.41	3.51	2.54	1.62	403.1 0.0	84.6 0.0	56.1 0.0	18.1	2.89 161. 0.00 0.	7
Var Coeff(%):	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.0	0.0	0.0	0.00 0.	0

2+66 EB (OKAA Type M) Falling Weight Deflectometer Test Result – 07/23/2008

				TTI	MODULUS	ANALYSIS	SYSTE	M (SUMMAR	Y REPORT)			(V	ersion 6.(	D)
District: County : Highway/Road:				Paveme Base: Subbas Subgra	nt: e: de:	Thickness 4.40 8.00 6.00 104.16	(in) ) ) 5 (by DB)	N Mi	CODULI RANG nimum 60,000 10,000 5,000 10,	<pre>HE(psi) Maximum 550,000 150,000 150,000 000</pre>	Poisson H1: H2: H3: H4:	Ratio V v = 0.3 v = 0.3 v = 0.3 v = 0.4	alues 8 5 5 0	
Load Station (lbs)	Measur R1	ed Defleo R2	rtion (r R3	nils): R4	R5	R6	R7	Calculate SURF(E1)	d Moduli v BASE(E2)	alues (ksi SUBB(E3)	): A SUBG(E4) E	bsolute   RR/Sens	Dpth to Bedrock	
267.000 12,005	14.93	10.57	9.39	5.36	3.52	2.47	1.60	442.3	71.3	81.7	15.5	3.06	122.6	
Mean: Std. Dev: Var Coeff(%):	14.93 0.00 0.00	10.57 0.00 0.00	9.39 0.00 0.00	5.36 0.00 0.00	3.52 0.00 0.00	2.47 0.00 0.00	1.60 0.00 0.00	442.3 0.0 0.0	71.3 0.0 0.0	81.7 0.0 0.0	15.5 0.0 0.0	3.06 0.00 0.00	122.6 0.0 0.0	

# 2+67 EB (OKAA Type M) Falling Weight Deflectometer Test Result – 07/23/2008

					TTI	MODULUS	ANALYSIS	SYSTE	M (SUMMAS	RY REPORT)			(Version	6.0)
District	17 (Brva	m)								NODILL RANG	E(nei)			
County :	21 (BRAZ	05)					Thicknes	s(in)	M	inimum	Maximum	Poisson	Ratio Values	
Highway/R	oad: FM2	818			Paveme	nt:	4.1	5		30,000	490,000	H1:	v = 0.38	
					Subbas	e:	6.0	0		5,000	150,000	H2: H3:	v = 0.35 v = 0.35	
					Subgra	de:	102.5	з (Бу DB	)	. 10,	000	H4:	V = 0.40	
	Load	Measur	ed Defle	ction (r	nils):				Calculate	ed Moduli v	values (ksi)	: A	bsolute Dpth to	
Station	(105)	R1	R2	R3	R4	R5	R6	R7	SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4) E	RR/Sens Bedrock	
288.000	12,060	16.91	12.32	10.94	6.61	4.38	3.04	1.78	406.3	79.9	61.8	12.5	2.39 120.7	
Mean: Std. Dev: Var Coeff	(%):	16.91 0.00 0.00	12.32 0.00 0.00	10.94 0.00 0.00	6.61 0.00 0.00	4.38 0.00 0.00	3.04 0.00 0.00	1.78 0.00 0.00	406.3 0.0 0.0	79.9 0.0 0.0	61.8 0.0 0.0	12.5 0.0 0.0	2.39 120.7 0.00 0.0 0.00 0.0	
Var Coeff	(%):	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.0	0.0	0.0	0.00 0.0	

2+88 WB (OKAA Type M) Falling Weight Deflectometer Test Result – 07/23/2008

				TTI	MODULUS	ANALYSIS	SYSTEM	M (SUMMAS	Y REPORT)			(V	ersion 6.0)
District:17 (Bry County :21 (BRA Highway/Road: FM	an) 205) 2818			Paveme Base: Subbas Subgra	nt: e: de:	Thickness 4.1 8.0 6.0	s(in) 5 0 6 (by DB)	Mi Mi	CODULI RANG nimum 30,000 10,000 5,000 10,	E(psi) Maximum 490,000 150,000 150,000 000	Poisson H1: H2: H3: H4:	<pre>Ratio V v = 0.3 v = 0.3 v = 0.3 v = 0.3 v = 0.4 </pre>	alues 5 5 0
Load Station (1bs)	Measur R1	ed Defle R2	ction (r R3	nils): R4	R5	R6	R7	Calculate SURF(E1)	ed Moduli v BASE(E2)	alues (ksi) SUBB(E3)	: A SUBG(E4) E	bsolute : RR/Sens :	Dpth to Bedrock
289.000 11,994	17.09	12.16	10.72	6.53	4.30	2.97	1.76	299.8	88.8	62.4	12.6	2.33	118.5
Mean: Std. Dev: Var Coeff(%):	17.09 0.00 0.00	12.16 0.00 0.00	10.72 0.00 0.00	6.53 0.00 0.00	4.30 0.00 0.00	2.97 0.00 0.00	1.76 0.00 0.00	299.8 0.0 0.0	88.8 0.0 0.0	62.4 0.0 0.0	12.6 0.0 0.0	2.33 0.00 0.00	118.5 0.0 0.0

### 2+89 WB (OKAA Type M) Falling Weight Deflectometer Test Result – 07/23/2008

					TTI	MODULUS	ANALYSIS	SYSTE	M (SUMMAS	RY REPORT)			(Ve	ersion 6.0)
District: County : Highway/Ro	oad:				Paveme Base: Subbas Subgra	nt: e: de:	Thickness 4.00 8.00 6.00 281.92	s(in) 3 0 2(by DB)	м Мі	MODULI RANG inimum 20,000 10,000 5,000 10,	HE (psi) Maximum 490,000 150,000 150,000 000	Poisso H1 H2 H3 H4	n Ratio Va : v = 0.38 : v = 0.35 : v = 0.35 : v = 0.40	lues ; ;
Station	Load (1bs)	Measur R1	ed Defle R2	ction ( R3	mils): R4	R5	R6	R7	Calculate SURF(E1)	ed Moduli v BASE(E2)	values (ksi) SUBB(E3)	): 1 SUBG(E4) 1	Absolute I SRR/Sens B	opth to Medrock
335.000	12,071	19.05	13.78	12.22	7.04	4.46	3.10	1.87	312.7	96.7	11.8	18.9	2.02	300.0
Mean: Std. Dev: Var Coeff	(%):	19.05 0.00 0.00	13.78 0.00 0.00	12.22 0.00 0.00	7.04 0.00 0.00	4.46 0.00 0.00	3.10 0.00 0.00	1.87 0.00 0.00	312.7 0.0 0.0	96.7 0.0 0.0	11.8 0.0 0.0	18.9 0.0 0.0	2.02 0.00 0.00	300.0 0.0 0.0

3+35 WB (OKAA Type M) Falling Weight Deflectometer Test Result – 07/23/2008

				TTI	MODULUS	ANALYSIS	SYSTE	M (SUMMAS	RY REPORT)			(V	ersion 6	. 0)
District:								Ņ	ODULI RANG	E(psi)			_	
County :						Thicknes	s(in)	Mi	nimum	Maximum	Poisson	Ratio V	alues	
Highway/Road:				Paveme	ent:	4.8	0		60,000	540,000	H1:	V = 0.3	8	
				Base:		8.0	0		10,000	150,000	H2:	V = 0.3	5	
				Subbas	ie:	6.0	0		5,000	150,000	H3:	v = 0.3	5	
				Subgra	ide:	123.1	0 (by DB	)	10,	000	H4:	v = 0.4	0	
Load	Measur	ed Defle	ction (r	nils):				Calculate	ed Moduli v	alues (ksi	): A	bsolute	Dpth to	
Station (1bs)	R1	R2	R3	R4	R5	R6	R7	SURF (E1)	BASE (E2)	SUBB(E3)	SUBG(E4) E	RR/Sens	Bedrock	
366.000 11,994	17.62	12.78	11.41	6.99	4.58	3.24	2.01	267.2	95.1	35.1	13.2	2.46	141.9	
Mean:	17.62	12.78	11.41	6.99	4.58	3.24	2.01	267.2	95.1	35.1	13.2	2.46	141.9	
Std. Dev:	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.0	0.0	0.0	0.00	0.0	
Var Coeff(%):	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.0	0.0	0.0	0.00	0.0	

### 3+66 EB (OKAA Type M) Falling Weight Deflectometer Test Result – 07/23/2008

					TTI M	ODULUS	ANALYSIS	SYSTEM	(SUMMAR	Y REPORT)			(Ve	ersion 6.0)
District:									м	ODULI RANG	E(psi)			_
County :							Thickness	(in)	Mi	nimum	Maximum	Poisson	Ratio Va	lues
Highway/Road:	:				Pavemen	it:	4.80	)		60,000	540,000	H1:	V = 0.38	1
					Base:		8.00	)		10,000	150,000	H2 :	V = 0.35	i
					Subbase		6.00	)		5,000	150,000	H3 :	V = 0.35	i
					Subgrad	le:	134.04	(by DB)		10,	000	H4 :	v = 0.40	)
Lo	bad	Measure	d Deflec	tion (mi	ils):				Calculate	d Moduli v	alues (ksi)	: A	bsolute I	pth to
Station (1h	os)	R1	R2	R3	R4	R5	R6	R7	SURF (E1)	BASE (E2)	SUBB(E3)	SUBG(E4) E	RR/Sens B	edrock
367.000 12,	038	17.81	12.99	11.65	6.98	4.62	3.32	2.00	380.5	60.3	55.4	13.2	2.45	152.8
Mean:	;	17.81	12.99	11.65	6.98	4.62	3.32	2.00	380.5	60.3	55.4	13.2	2.45	152.8
Std. Dev:		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.0	0.0	0.0	0.00	0.0
Var Coeff(%):		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.0	0.0	0.0	0.00	0.0

3+67 EB (OKAA Type M) Falling Weight Deflectometer Test Result – 07/23/2008

				TTI	MODULUS	ANALYSIS	SYSTEM	(SUMMAR	Y REPORT)			(Version 6	5.0)
District: County : Highway/Road:				Paveme: Base: Subbas Subgra	nt: e: de:	Thicknes: 3.5( 8.0) 6.0 282.5(	s(in) 0 0 0(by DB)	M Mi	ODULI RANG nimum 30,000 10,000 5,000 10,	E(psi) Maximum 500,000 150,000 150,000 000	Poisson Hl: H2: H3: H4:	Ratio Values V = 0.38 V = 0.35 V = 0.35 V = 0.40	
Load Station (1bs)	Measur R1	ed Deflec R2	R3 (T	nils): R4	R5	R6	R7	Calculate SURF(E1)	ed Moduli v BASE(E2)	alues (ksi) SUBB(E3)	: A SUBG(E4) E	bsolute Dpth to RR/Sens Bedrock	
408.000 11,983	15.24	10.13	8.75	5.19	3.76	2.78	1.85	238.8	107.5	74.9	21.7	2.45 300.0	
Mean: Std. Dev: Var Coeff(%):	15.24 0.00 0.00	10.13 0.00 0.00	8.75 0.00 0.00	5.19 0.00 0.00	3.76 0.00 0.00	2.78 0.00 0.00	1.85 0.00 0.00	238.8 0.0 0.0	107.5 0.0 0.0	74.9 0.0 0.0	21.7 0.0 0.0	2.45 300.0 0.00 0.0 0.00 0.0	

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# 4+08 WB (ODOT Type A) Falling Weight Deflectometer Test Result – 07/23/2008

					TTI	MODULUS	ANALYSIS	SYSTE	M (SUMMAS	RY REPORT)			(Version	6.0)
District:							Thicknes	- (i-)	) Mi	ODULI RANG	E(psi)	Poirror	Patio Values	
Highway/R	toad:				Pavemer	nt:	5.4	0		50,000	530,000	H1:	v = 0.38	
					Base: Subbase Subgrae	e: de:	8.0 6.0 164.1	o o B(by DB	)	10,000 5,000 10,	150,000 150,000 000	H2 : H3 : H4 :	v = 0.35 v = 0.35 v = 0.40	
Station	Load (1bs)	Measu R1	red Defle R2	R3	nils): R4	R5	R6	R7	Calculate SURF(E1)	ed Moduli v BASE(E2)	alues (ksi) SUBB(E3)	: A SUBG(E4) E	bsolute Dpth to RR/Sens Bedrock	5 C
467.000	12,016	17.13	11.55	10.00	5.62	3.82	2.84	1.81	247.3	48.9	91.0	17.0	2.97 183.6	
Mean: Std. Dev: Var Coeff	(%):	17.13 0.00 0.00	11.55 0.00 0.00	10.00 0.00 0.00	5.62 0.00 0.00	3.82 0.00 0.00	2.84 0.00 0.00	1.81 0.00 0.00	247.3 0.0 0.0	48.9 0.0 0.0	91.0 0.0 0.0	17.0 0.0 0.0	2.97 183.6 0.00 0.0 0.00 0.0	

4+67 EB (ODOT Type A) Falling Weight Deflectometer Test Result – 07/23/2008

					TTI	MODULUS	ANALYSIS	SYSTE	M (SUMMAR	Y REPORT)			(V	Version 6.	0)
District: County : Highway/Ro	oad:				Pavena Base: Subbas Subgra	ent: se: sde:	Thicknes 4.5 8.0 6.0 77.2	s(in) 0 0 5(by DB)	м Мі 4	CODULI RANG nimim 10,000 10,000 5,000 10,	E(psi) Maximum 1,150,000 150,000 150,000 000	Foisson Hl: H2: H3: H4:	Ratio V v = 0.3 v = 0.3 v = 0.3 v = 0.4	Values 8 5 5 0	
Station	Load (lbs)	Measur R1	red Defle R2	R3	nils): R4	RS	R6	R7	Calculate SURF(E1)	d Moduli v BASE(E2)	alues (ksi) SUBB(E3)	): A SUBG(E4) E	bsolute RR/Sens	Dpth to Bedrock	
52.000	12,027	18.89	15.23	12.59	7.59	4.48	2.91	1.76	827.1	26.4	99.4	9.7	1.24	95.7	
Mean: Std. Dev: Var Coeff	(%):	18.89 0.00 0.00	15.23 0.00 0.00	12.59 0.00 0.00	7.59 0.00 0.00	4.48 0.00 0.00	2.91 0.00 0.00	1.76 0.00 0.00	827.1 0.0 0.0	26.4 0.0 0.0	99.4 0.0 0.0	9.7 0.0 0.0	1.24 0.00 0.00	95.8 0.0 0.0	

# 0+52 EB (AASHTO #57) Falling Weight Deflectometer Test Result – 05/21/2009

				TTI	MODULUS	ANALYSIS	SYSTEM	(SUMMAR	Y REPORT)			(1	Version 6.	0)
District:								3	COULI RANG	E(psi)				
County :						Thickness	;(in)	Mi	nimun	Maximum	Poisso	n Ratio V	/alues	
Highway/Road:				Favene	nt:	5.00	)	2	60.000	900.000	Hl	: v = 0.3	8	
3 11				Base		8.00	5		10,000	150,000	H2	v = 0.3	5	
				Subbas	-	6.00			5 000	150,000				
				Subbas		6.00			3,000	150,000				
				subgra	ade:	69.40	(BY DB)		10,	000	H4	: V = 0.4	.0	
Load Station (lbs)	Measu: Rl	red Defle R2	R3	nils): R4	R5	R6	R7	Calculate SURF(E1)	d Moduli v BASE(E2)	alues (ksi) SUBB(E3)	: I SUBG(E4) I	Absolute ERR/Sens	Dpth to Bedrock	
86.000 11,994	20.92	17.57	14.90	9.35	5.58	3.50	2.01	833.3	19.2	75.5	7.4	0.93	88.4	
Mean: Std. Dev:	20.92	17.57	14.90 0.00	9.35	5.58 0.00	3.50	2.01	833.3 0.0	19.2 0.0	75.5 0.0	7.4 0.0	0.93	88.4 0.0	
Var Coeff(%):	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.0	0.0	0.0	0.00	0.0	

0+86 WB (AASHTO #57) Falling Weight Deflectometer Test Result – 05/21/2009

				TTI N	ODULUS	ANALYSIS	SYSTEM	(SUMMAS	RY REPORT)			(V)	ersion 6.0)	
District: County : Highway/Road:				Favener Base: Subbase Subgrad	it: :: le:	Thickness(in) 4.00 8.00 6.00 107.84(by DB)		MODULI RANG Minimum 410,000 10,000 5,000 10,		E(psi) Maximum 1,150,000 150,000 150,000 000	Poisson Hl: H2: H3: H4:	Poisson Ratio Values H1: V = 0.38 H2: V = 0.35 H3: V = 0.35 H4: V = 0.40		
Load Station (lbs)	Measu: Rl	red Defle R2	ection (1 R3	nils): R4	RS	R6	R7	Calculate SURF(E1)	ed Moduli v BASE(E2)	alues (ksi SUBB(E3)	): 3 SUBG(E4) 5	Absolute 1 SRR/Sens 1	Dpth to Bedrock	
127.000 11,972	21.77	17.50	14.07	8.01	4.42	2.83	1.80	1066.7	13.7	141.5	12.2	1.45	125.8	
Mean: Std. Dev: Var Coeff(%):	21.77 0.00 0.00	17.50 0.00 0.00	14.07 0.00 0.00	8.01 0.00 0.00	4.42 0.00 0.00	2.83 0.00 0.00	1.80 0.00 0.00	1066.7 0.0 0.0	13.7 0.0 0.0	141.5 0.0 0.0	12.2 0.0 0.0	1.45 0.00 0.00	125.8 0.0 0.0	

#### 1+27 EB (AASHTO #57) Falling Weight Deflectometer Test Result – 05/21/2009

				TTI I	MODULUS	ANALYSIS	SYSTEM	M (SUMMAR	Y REPORT)			(V	ersion 6.	0)	
District:17 (Bryan					я	COULI RANG	E(psi)								
County :21 (BRAZO		Thickness(in)			Minimum Maximum		Poisson Ratio Values								
Highway/Road: FM28		Favener	nt:	4.50		250,000 870,000		H1: V = 0.38							
				Base:		8.00	0		10,000	150,000	H2 :	v = 0.3	5		
		Subbas	e :	6.00		5,000		150,000	H3: V = 0.35		5				
					Subgrade:		73.16(by DB)		10,000			H4: V = 0.40			
Load	ction (m	uils):				Calculate	Calculated Moduli values (ksi):			Absolute Dpth to					
Station (lbs)	Rl	R2	R3	R4	RS	R6	R7	SURF(E1)	BASE (E2)	SUBB(E3)	SUBG(E4) E	RR/Sens	Bedrock		
161.000 12,027	21.25	17.39	14.27	8.63	5.15	3.29	1.96	764.0	23.8	82.3	8.2	1.21	91.7		
Mean: Std. Dev: Var Coeff(%):	21.25 0.00 0.00	17.39 0.00 0.00	14.27 0.00 0.00	8.63 0.00 0.00	5.15 0.00 0.00	3.29 0.00 0.00	1.96 0.00 0.00	764.0 0.0 0.0	23.8 0.0 0.0	82.3 0.0 0.0	8.2 0.0 0.0	1.21 0.00 0.00	91.7 0.0 0.0		

1+61 WB (AASHTO #57) Falling Weight Deflectometer Test Result – 05/21/2009
						3931 9070	OVOTO	u /ornuuso	V PPDOPT)			/14	avaion 6	a)
									(1 KEFOR(1)					
District:								,	COULI RANG	E(psi)				
County :						Thicknes	s(in)	Mi	inimun	Maximum	Poisson	Ratio V	alues	
Highway/Road:				Pavener	nt:	4.5	0	3	390,000	1,120,000	H1 :	v = 0.3	8	
				Base:		8.0	0		10,000	150,000	H2 :	v = 0.3	5	
				Subbase		6.0	0		5,000	150,000	H3 :	v = 0.3	5	
				Subgrad	de:	79.7	6(by DB	>	10,	000	H4 :	v = 0.4	0	
Load	Measur	ed Defle	etion (	nils):				Calculate	d Moduli v	alues (ksi	): A	bsolute	Doth to	
Station (lbs)	Rl	R2	R3	R4	RS	R6	R7	SURF(E1)	BASE (E2)	SUBB(E3)	SUBG(E4) E	RR/Sens	Bedrock	
227.000 12,060	16.12	13.18	10.86	6.66	4.08	2.69	1.60	965.2	34.3	109.6	10.9	1.20	98.3	
Mean:	16.12	13.18	10.86	6.66	4.08	2.69	1.60	965.2	34.3	109.6	10.9	1.20	98.3	
Std. Dev:	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.0	0.0	0.0	0.00	0.0	
Var Coeff(%):	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.0	0.0	0.0	0.00	0.0	

## 2+27 EB (OKAA Type M) Falling Weight Deflectometer Test Result – 05/21/2009

				TTI	MODULUS	ANALYSIS	SYSTE	M (SUMMAR	RY REPORT)			(V	ersion 6.0	0)
District:17 (Bryan) County :21 (BRAZOS) Highway/Road: FM2818 Load Measured Deflection				Pavens Base: Subbas Subgra	ent: se: ade:	Thicknes: 4.0 8.0 6.0 236.4	s(in) 0 0 5 6 (by DB	у И: 2	(ODULI RANG inimum 250,000 10,000 5,000 10,	E (psi) Maximum 880,000 150,000 150,000 000	Poisson Hl: H2: H3: H4:	Ratio V v = 0.3 v = 0.3 v = 0.3 v = 0.4	alues 8 5 5 0	
Load Station (lbs)	Measur Rl	red Deflec R2	R3	nils): R4	R5	R6	R7	Calculate SURF(E1)	ed Moduli v BASE(E2)	alues (ksi) SUBB(E3)	): 3 SUBG(E4) 5	bsolute RR/Sens	Dpth to Bedrock	
230.000 12,093	15.33	12.17	9.91	6.13	3.85	2.60	1.59	880.0	46.2	65.4	19.1	2.05	254.5 *	
Mean: Std. Dev: Var Coeff(%):	15.33 0.00 0.00	12.17 0.00 0.00	9.91 0.00 0.00	6.13 0.00 0.00	3.85 0.00 0.00	2.60 0.00 0.00	1.59 0.00 0.00	880.0 0.0 0.0	46.2 0.0 0.0	65.4 0.0 0.0	19.1 0.0 0.0	2.05 0.00 0.00	254.5 0.0 0.0	

2+30 WB (OKAA Type M) Falling Weight Deflectometer Test Result – 05/21/2009

				TTI	MODULUS	ANALYSIS	SYSTE	M (SUMMAR	RY REPORT)			(Ve	rsion 6.0)
District:								}	MODULI RANG	E(psi)			
County :						Thicknes	s(in)	M	inimun	Maximum	Poisso	n Ratio Va	lues
Highway/Road:				Favene	nt:	4.5	0	3	350,000	1,050,000	Hl	v = 0.38	
5 1				Base:		8.0	0		10,000	150,000	H2	v = 0.35	
				Subbas		6.0	-		E 000	150,000	22		
				Subbas	de.	103.3	o (bur DB	、 、	3,000	150,000	113	· · · - 0.35	
				Subgra	aue:	103.5	O(DY DB	/	10,	.000		: 0 = 0.40	
Load	Measur	ed Deflea	tion (	ils):				Calculate	ed Moduli v	alues (ksi	):	Absolute D	pth to
Station (lbs)	Rl	R2	R3	R4	RS	R6	R7	SURF(E1)	BASE (E2)	SUBB(E3)	SUBG(E4)	ERR/Sens B	edrock
252.000 12,027 277.000 12,027	14.95 15.38	11.88 12.06	9.82 9.77	6.09 5.78	3.82 3.70	2.65 2.57	1.64 1.67	821.2 735.6	46.1 37.8	99.6 150.0	13.7 13.9	1.17 1.26	123.7 120.0 *
Mean: Std. Dev: Var Coeff(%):	15.17 0.30 2.00	11.97 0.13 1.06	9.80 0.04 0.36	5.94 0.22 3.69	3.76 0.08 2.26	2.61 0.06 2.17	1.66 0.02 1.28	778.4 60.5 7.8	41.9 5.9 14.1	124.8 35.6 28.6	13.8 0.2 1.3	1.21 0.06 5.24	121.8 1.8 1.5

## 2+67 EB (OKAA Type M) Falling Weight Deflectometer Test Result – 05/21/2009

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT)   (Version 6.0)     District:17 (Bryan)   MODULI RANGE (psi)     County :21 (BRAZOS)   Thickness(in)   Minimum   MAXIMUM Poisson Ratio Values     Highway/Road:   FM2818   Favement:   4.00   270,000   910,000   H1: v = 0.38     Base:   8.00   10,000   150,000   H2: v = 0.35   Subbase:   6.00   5,000   150,000   H3: v = 0.35     Subgrade:   157.62 (by DB)   10,000   H4: v = 0.40     Load Measured Deflection (mils):   Calculated Moduli values (ksi):   Absolute Dpth to				
District:17 (Bryan)     MCDULI RANGE (psi)       County :21 (BRAZOS)     Thickness (in)     Minimum     Maximum     Foisson Ratio Values       Highway/Road: FM2818     Favement:     4.00     270,000     910,000     H1: v = 0.38       Base:     8.00     10,000     150,000     H2: v = 0.35       Subbase:     6.00     5,000     150,000     H3: v = 0.35       Subgrade:     157.62 (by DB)     10,000     H4: v = 0.40       Load     Measured Deflection (mils):     Calculated Moduli values (ksi):     Absolute Dpth to		TTI MODULUS ANALYSIS SYSTEM	(SUMMARY REPORT)	(Version 6.0)
County :21 (BRAZOS)     Thickness(in)     Minimum     Maximum     Foisson Ratio Values       Highway/Road: FM281S     Favement:     4.00     270,000     910,000     H1: v = 0.38       Base:     8.00     10,000     150,000     H2: v = 0.35       Subbase:     6.00     5,000     150,000     H3: v = 0.35       Subgrade:     157.62 (by DB)     10,000     H4: v = 0.40       Load     Measured Deflection (mils):     Calculated Moduli values (ksi):     Absolute Dpth to	District:17 (Bryan)		MODULI RANGE (psi)	
Highway/Road: FM2818   Favement:   4.00   270,000   910,000   H1: v = 0.38     Base:   8.00   10,000   150,000   H2: v = 0.35     Subbase:   6.00   5,000   150,000   H3: v = 0.35     Subgrade:   157.62 (by DB)   10,000   H4: v = 0.40     Load   Measured Deflection (mils):   Calculated Moduli values (ksi):   Absolute Dpth to	County :21 (BRAZOS)	Thickness(in)	Minimum Maximum	Poisson Ratio Values
Base:     8.00     10,000     150,000     H2: v = 0.35       Subbase:     6.00     5,000     150,000     H3: v = 0.35       Subgrade:     157.62 (by DB)     10,000     H4: v = 0.40       Load     Measured Deflection (mils):     Calculated Moduli values (ksi):     Absolute Dpth to	Highway/Road: FM2818	Pavement: 4.00	270,000 910,000	H1: V = 0.38
Subbase:     6.00     5,000     150,000     H3: v = 0.35       Subgrade:     157.62 (by DB)     10,000     H4: v = 0.40       Load     Measured Deflection (mils):     Calculated Moduli values (ksi):     Absolute Dpth to	5 1	Base: 8.00	10,000 150,000	H2: v = 0.35
Subgrade: 157.62 (by DB) 10,000 H4: v = 0.40   Load Measured Deflection (mils): Calculated Moduli values (ksi): Absolute Dpth to		Subbase: 6.00	5 000 150 000	$H_{3}$ , $V = 0.35$
Load Measured Deflection (mils): Calculated Moduli values (ksi): Absolute Dpth to		Subgrade: 157, 62 (by DB)	10 000	$H_4$ , $V = 0.40$
Load Measured Deflection (mils): Calculated Moduli values (ksi): Absolute Dpth to		500grade: 157.02(by 50)		
	Load Measured Deflection	o (mils);	Calculated Moduli values (ksi):	Absolute Doth to
Station (lbs) R1 R2 R3 R4 R5 R6 R7 SURF(E1) BASE(E2) SUBB(E3) SUBG(E4) ERR/Sens Bedrock	Station (lbs) R1 R2 R3	R4 R5 R6 R7	SURF(E1) BASE(E2) SUBB(E3) S	UBG(E4) ERR/Sens Bedrock
290.000 12,005 17.43 14.58 12.33 7.90 4.91 3.15 1.81 910.0 54.6 34.5 13.5 2.72 175.6 *	290.000 12,005 17.43 14.58 12.3	33 7.90 4.91 3.15 1.81	910.0 54.6 34.5	13.5 2.72 175.6 *
Mean: 17.43 14.58 12.33 7.90 4.91 3.15 1.81 910.0 54.6 34.5 13.5 2.72 175.6	Mean: 17.43 14.58 12.3	33 7.90 4.91 3.15 1.81	910.0 54.6 34.5	13.5 2.72 175.6
Sea. Dev:     0.00	Var Coeff(%): 0.00 0.00 0.0	0 0.00 0.00 0.00 0.00	0.0 0.0 0.0	0.0 0.00 0.0

2+90 WB (OKAA Type M) Falling Weight Deflectometer Test Result – 05/21/2009

				TTI	MODULUS	ANALYSIS	SYSTEM	(SUMMAR	RY REPORT)			(Ve	ersion 6.0)
District:17 (Bryan) County :21 (BRAZOS) Highway/Road: FM2818 Pavenen Base: Subbase Subgrad Load Measured Deflection (mils):				nt: e: de:	Thicknes 4.0 8.0 6.0 180.9	s(in) 0 0 8(by DB)	н М3 2	CODULI RANG inimin 280,000 10,000 5,000 10,	E (psi) Maximum 930,000 150,000 150,000 000	Poisson Hl: H2: H3: H4:	Ratio Va v = 0.38 v = 0.39 v = 0.39 v = 0.40	alues 5 5 0	
Load Station (lbs)	Measu: Rl	red Defle R2	R3	nils): R4	R5	R6	R7	Calculate SURF(E1)	d Moduli v BASE(E2)	alues (ksi) SUBB(E3)	: A SUBG(E4) E	bsolute I RR/Sens B	opth to Bedrock
336.000 11,961	18.83	15.47	12.87	7.92	4.84	3.17	1.87	930.0	25.7	93.5	13.9	2.90	199.0 *
Mean: Std. Dev: Var Coeff(%):	18.83 0.00 0.00	15.47 0.00 0.00	12.87 0.00 0.00	7.92 0.00 0.00	4.84 0.00 0.00	3.17 0.00 0.00	1.87 0.00 0.00	930.0 0.0 0.0	25.7 0.0 0.0	93.5 0.0 0.0	13.9 0.0 0.0	2.90 0.00 0.00	199.0 0.0 0.0

3+36 WB (OKAA Type M) Falling Weight Deflectometer Test Result – 05/21/2009

				TTI	MODULUS	ANALYSIS	SYSTE	M (SUMMAS	RY REPORT)			(1	Version 6.0	)}
District:									MODULI RANG	3E(psi)				
County :						Thicknes	s(in)	M	inimun	Maximum	Poisso	n Ratio V	Alues	
Highway/Road:				Pavene	nt:	4.5	0	3	340,000	1,030,000	Hl	: v = 0.3	8	
				Base:		8.0	0		10,000	150,000	H2	: V = 0.3	5	
				Subbas	e:	6.0	0		5,000	150,000	H3	: v = 0.3	5	
				Subgra	de:	112.1	7(by DB	)	10,	,000	H4	: V = 0.4	.0	
tood		red Defla						Calculate	ad Maduli .	aluar (kei)		Absolute	Deth to	
Station (lbs)	R1	R2	R3	R4	RS	R6	R7	SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	ERR/Sens	Bedrock	
377.000 11,994	18.10	15.37	13.03	8.39	5.43	3.77	2.26	1030.0	29.2	99.1	9.9	1.02	130.7 *	
Mean:	18.10	15.37	13.03	8.39	5.43	3.77	2.26	1030.0	29.2	99.1	9.9	1.02	130.7	
					0.00	0.00	0.00		0.0		0.0	0.00		
Std. Dev:	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.0	0.0	0.0	0.00	0.0	
Std. Dev: Var Coeff(%):	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.0	0.0	0.0	0.00	0.0	

3+67 EB\_+10 FT (OKAA Type M) Falling Weight Deflectometer Test Result – 05/21/2009

				TTI	MODULUS	ANALYSIS	SYSTE	M (SUMMAS	RY REPORT)			(V	ersion 6.0)
District:17 (Brya County :21 (BRA2 Highway/Road: FM2	n) 205) 1818			Faven Base: Subbas Subgra	ent: se: ade:	Thicknes 3.5 8.0 6.0 282.5	s(in) 0 0 0 0(by DB)	) M: 2	(ODULI RANG inimum 280,000 10,000 5,000 10,	E (psi) Maximum 920,000 150,000 150,000 000	Poisson Hl: H2: H3: H4:	Ratio V v = 0.3 v = 0.3 v = 0.3 v = 0.4	Values 8 5 5 0
Load Station (lbs)	Measu: Rl	red Defle R2	ction (m R3	ils): R4	R5	R6	R7	Calculate SURF(E1)	ed Moduli v BASE(E2)	alues (ksi) SUBB(E3)	): A SUBG(E4) E	bsolute RR/Sens	Dpth to Bedrock
409.000 11,917	14.57	11.16	8.92	5.48	3.72	2.74	1.75	920.0	62.9	81.6	20.0	0.73	300.0 *
Mean: Std. Dev: Var Coeff(%):	14.57 0.00 0.00	11.16 0.00 0.00	8.92 0.00 0.00	5.48 0.00 0.00	3.72 0.00 0.00	2.74 0.00 0.00	1.75 0.00 0.00	920.0 0.0 0.0	62.9 0.0 0.0	81.6 0.0 0.0	20.0 0.0 0.0	0.73 0.00 0.00	300.0 0.0 0.0

4+09 WB (ODOT Type A) Falling Weight Deflectometer Test Result – 05/21/2009

				TTI	MODULUS	ANALYSIS	SYSTE	M (SUMMAR	Y REPORT)			(Version	6.0)
District:								н	ODULI RANG	E(psi)			
County :						Thickness	s(in)	Mi	nimin	Maximum	Poisson	i Ratio Values	
Highway/Road:				Pavene	nt:	5.25	5	3	10,000	990.000	Hl	: V = 0.38	
5 31				Base		B. 0/	2		10,000	150,000	H2	V = 0.35	
				and a second									
				Subbas	ie:	6.00			5,000	150,000	H.3 :	v = 0.35	
				Subgra	de:	173.65	9(by DB	)	10,	000	H4 :	v = 0.40	
Load	Measur	ed Deflec	tion (m	nils):				Calculate	d Moduli v	alues (ksi)	: 3	Absolute Doth to	
Station (lbs)	Rl	R2	R3	R4	RS	R6	R7	SURF(E1)	BASE (E2)	SUBB(E3)	SUBG(E4) B	SRR/Sens Bedrock	
477.000 11,983	12.62	10.07	8.45	5.45	3.73	2.74	1.80	739.6	50.9	148.1	17.2	0.64 192.9	
Mean: Std. Dev:	12.62	10.07	8.45	5.45	3.73	2.74	1.80	739.6	50.9 0.0	148.1 0.0	17.2 0.0	0.64 192.9 0.00 0.0	
Var Coeff(%):	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.0	0.0	0.0	0.00 0.0	

4+67 EB (ODOT Type A) Falling Weight Deflectometer Test Result – 05/21/2009

## APPENDIX – D

Dynamic Cone Penetration Testing



Station No. 0+ 50 EB DCP Index over Depth (M-AASHTO #57, April 2008)



Station No. 2+ 35 WB DCP Index over Depth (OKAA Type M, April 2008)



Station No. 4+ 15 WB DCP Index over Depth (ODOT Type A, April 2008)



Station No. 0+ 51 EB DCP Index over Depth (M-AASHTO #57, July 2008)



Station No. 1+60 WB DCP Index over Depth (M-AASHTO No. 57, July 2008)



Station No. 2+25 EB DCP Index over Depth (OKAA Type M, July 2008)



Station No. 3+ 35 WB DCP Index over Depth (OKAA Type M, July 2008)



Station No. 3+66 EB DCP Index over Depth (OKAA Type M, July 2008)



Station No. 0+52 EB DCP Index over Depth (*M-AASHTO #57*, May 2009)



Station No. 0+86 WB DCP Index over Depth (*M-AASHTO #57*, May 2009)



Station No. 1+27 EB DCP Index over Depth (*M-AASHTO #57*, May 2009)



Station No. 2+27 EB DCP Index over Depth (OKAA Type M, May 2009)



Station No. 2+30 WB DCP Index over Depth (OKAA Type M, May 2009)



Station No. 2+67 WB DCP Index over Depth (OKAA Type M, May 2009)



Station No. 2+67 WB DCP Index over Depth (OKAA Type M, May 2009)



Station No. 4+09 WB DCP Index over Depth (ODOT Type A, May 2009)



Station No. 4+67 EB DCP Index over Depth (ODOT Type A, May 2009

APPENDIX – E

Traffic Count

Time (Hours)		East 3	Bound	(Towa	rds Jer	nkins	Ave	e)		Wes	t Bound	l (Towa	rds As	p Av	e)	
12	Car	Truck	Van	SUV	Bus	5	6	Motor Cycle	Car	Truck	Van	SUV	Bus	5	6	Motor Cycle
Total	523	180	24	312	0	12	0	5	663	158	47	353	24	12	3	3

Traffic Count on September 29, 2008

Traffic Count on December 05, 2008

Time (Hours)		East I	Bound	(Towa	rds Jei	nkins	Ave	e)		Wes	t Bound	l (Towa	rds Asj	p Av	e)	
9	Car	Truck	Van	SUV	Bus	5	6	Motor Cycle	Car	Truck	Van	SUV	Bus	5	6	Motor Cycle
Total	352	95	18	197	0	18	3	1	427	102	18	205	17	16	0	1

## APPENDIX – F

Aggregate Imaging Analysis



















