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Laboratory Characterization of Geogrid-Reinforced Unbound Granular Material for use in Flexible Pavement Structures

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LABORATORY CHARACTERIZATION OF GEOGRID-REINFORCED UNBOUND GRANULAR MATERIAL FOR USE IN FLEXIBLE PAVEMENT STRUCTURES

A Thesis
Submitted to the Graduate Faculty of the Louisiana State University and Agricultural and Mechanical College in partial fulfillment of the requirements for the degree of Master of Science

in

The Department of Civil and Environmental Engineering

By
Gael Souci
B.S., Louisiana State University, 2007
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Rather than being an individual effort, I consider this research work a team effort more than anything. I wish to thank all those people that contributed in one way or another in helping complete this daunting yet rewarding task.

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ABSTRACT

This thesis documents and summarizes research and background information carried on geogrid reinforced base course in pavement design. Research was experimental carried through Repeated Load Triaxial (RLT) tests at the Louisiana Transportation Research Center. The experimental tests were performed to observe the benefit of the geogrid as well as to differentiate between geogrid location, geometry and tensile modulus of the various geogrid. Experiments were also carried to further describe the Shakedown Theory and its use for characterization of base course materials.

The experimental results showed that there was a benefit in placing the geogrid within the aggregate specimen. There were also noticeable differences in performance resulting from the geogrid placement location in the specimen as well as the different tensile strength of the geogrid. The results followed the intuitive expectation the stiffer the geogrid, the lesser the plastic deformation. Geometry had a noticeable effect as well when comparing the bi-axial (BX) geogrid and the tri-axial (TX) geogrid. The experimental results also showed that less deformation was obtained under cyclic loading for geogrid reinforced bases versus unreinforced bases. The results also supported that a change in moisture will yield different permanent strain values in repeated load tests. The same reinforcement trend obtained at optimum moisture content was also transferred for the moisture effect tests. The higher frequency tests with increased number of cycles also produced the same trend. The geogrid with the higher tensile modulus and the new geometry gave the best results.
CHAPTER 1 INTRODUCTION

1.1 Background

Pavement structure is a system designed to support loads, more specifically traffic loads. In the United States of America there are over 2.5 million miles of paved roadway. Of those 2.5 million miles, 50,000 miles are rigid paved roads and the rest is of a low, intermediate or high type of flexible paved roadway (FHWA, 2009). Flexible pavements are made of layers. There is the natural soil layer often called subgrade, sometimes followed by the subbase, then some type of granular base course and finally the asphalt top layer. In this study, the main focus is on the base course layer. The base course layer forms part of the flexible pavement system and therefore it will be important to understand how the system works as a whole.

Understanding the system also means understanding the design process of the system. In pavement design, the main goal is to determine the thickness of pavement required and doing so in the most economical way. In designing a pavement structure, the four main factors to consider are: loading from traffic, types and characteristics of materials used, environmental conditions and failure criteria. In considering traffic loading, axle loads, number of repetitions, contact areas and speed of traffic are the main factors. In the case of materials, the properties of the materials used for construction of the pavement must be specified. Resistance to stresses and strains are material dependent. When taking environmental conditions into consideration, the factors of rain and temperature are the main concerns. Rutting, fatigue cracking and temperature cracking are the main failures present in dealing with flexible pavements.

The base course layer in a pavement structure usually consists of material that is unbound and granular. Unbound granular materials are used in base course layers are used due to their
mechanical and physical properties as well as availability. The base course layer also serves to spread the load from the traffic load down to the subgrade. The base course layer is expected to spread the traffic load in such a way that the subgrade will be able to resist those stresses and remain within allowable limits. Other than dissipating stresses to the subgrade, the base course layer is also expected to resist applied stresses and strains so as to resist permanent deformation within certain limits.

1.2 Problem Statement

There has been an increase from 1989 of 0.7 million miles of urban roadway to almost 1.1 million miles now (FHWA, 2009). The increase in miles of roadway complements the vehicle miles of travel per year which has increased from 2 million to 3 million in the same time frame. Increase in traffic means increase in traffic loading. As mentioned in section 1.1 traffic loading is a major factor in the design of a pavement structure and the base course layer is used to safely transfer traffic loads to the underlying layers. With increased traffic loading, the base course layers have become thicker and thicker. Thicker base course layer means more material and therefore more cost. The geogrid can be used as an alternate to reduce these material costs. Many studies have been carried out to quantify the benefit of using geosynthetic (geotextile/geogrid) reinforcement. Those studies show that the geogrid is a viable alternative to the increasing costs of construction materials. Not only does the geogrid help in reducing material needed, it can also extend the service life of the pavement and provides additional strength to weaker subgrade areas. Weak subgrades usually produce California Bearing Ratio (CBR) values of less than 1.5. It can be considered as a form of soil stabilization and base reinforcement.
1.3 Scope and Objective of Study

The main objective of this study is to characterize the behavior of reinforced and unreinforced unbound granular material used in base course layer. More so, the characterization will be conducted on samples that are reinforced with geogrid. In doing so, this introduces a new set of variables. These variables include:

- Geogrid location
- Geogrid tensile modulus
- Geometry of geogrid
- Effect of moisture of base materials
- Effect of different stress levels
- Effect of number of load cycles

For the variables listed above, unreinforced samples will be presented and compared to samples affected by the different variables.

The aforementioned objectives were achieved through conducting extensive experimental work. The material used was unbound and granular. It was obtained from the Martin Marietta quarry found in Baton Rouge, Louisiana. The material is the 610 Kentucky Limestone which is of a grayish cement color. The repeated load tests were carried using the Material Testing System (MTS) 810 which uses a servo hydraulic loading system. The initial step of this study was to carry physical property tests. These included but not limited to: Standard Proctor compaction, specific gravity, absorption, and sieve analysis tests. In context to the experimental work, we studied the effect of the following variables:
1. Geogrid Location in sample:
   a. Middle of sample
   b. Upper one third of sample
   c. Double layer at upper and lower one third

2. Geogrid Type:
   a. Tensar TX170 (Tri-axial TX1)
   b. Tensar TX160 (Tri-axial TX2)
   c. Tensar BX1200 (Biaxial BX1)
   d. Tensar BX1100 (Biaxial BX2)
   e. Mirafi BasXgrid 11 (Biaxial BX3)

3. Moisture Content:
   a. Optimum Moisture content
   b. +2.5% of optimum
   c. -2.5% of optimum

The motivation for the geogrid used was based from previous research carried by Drs. Murad Abu-Farsakh and Munir Nazzal. The tri-axial geogrid are new to the industry and the Biaxial geogrid studied by Abu-Farsakh et al. (2007) were studied again and used for a basis of comparison. The BX3 geogrid was also studied as it uses a different node construction when compared to the BX1 and 2 geogrids.

The laboratory tests conducted were the repeated load tri-axial (RLT) tests. Two stages were adopted. Single-Stage RLT tests with constant stresses up to 10,000 cycles and multi-stage RLT tests with six stages of different stress levels each of 10,000 cycles. Also using the single stage
stress protocol an analysis of the effect of number of load cycles was studied through increasing the loading frequency. The samples were loaded up to 100,000 load cycles.

1.4 Outline

This thesis consists of five chapters. Chapter one gives is the introductory chapter that provides brief background on the research topic. It also provides the reader with the problem statement, the objectives of the study and the scope of the research to be carried. Chapter two gives a detailed literature review on some key aspects that pertain to the research. Chapter three describes the materials used and the methods used to carry out the experimental work. Chapter four presents the results and findings for this study and finally chapter five consists of the closing remarks of this study.
CHAPTER 2 LITERATURE REVIEW

2.1 Introduction

This chapter presents a survey of the literature found pertaining to the main topic of this thesis. The main focus tends towards the behavior of both unreinforced and geogrid reinforced unbound granular material (UGM) that is used in the construction of base course layer in a pavement structure. The type of loading affecting the base course layer is repeated traffic loading and this will also be surveyed.

The first part of the chapter introduces the various factors that affect the behavior of the UGMs in the base course layer. It also introduces the elastic and plastic behavior of the material under repeated loading. The concept of shakedown theory is then introduced and discussed how it applies to this study. Finally, the behavior of the geogrid in the base course layer is explained and how it pertains to this research.

2.2 Stresses in Base Course Layer

The stresses involved in a material element are governed by normal and shear stresses. In the case of normal stresses, there are three principal stresses depicted as \( \sigma_1, \sigma_2 \) and \( \sigma_3 \). Figure 2.1 illustrates the stresses on an elemental cube of a material. In Figure 2.1 (b), when the element goes through a rotation, it can be proven that for any general state of stress there exists no shear stress on three mutually perpendicular planes. The remaining normal stresses are the principal stresses mentioned above. Principal stresses \( (\sigma_1, \sigma_2 \text{ and } \sigma_3) \) do not depend on the choice of coordinate system.
Stresses in the unbound granular layer (UGL) are induced by the wheel load of a moving vehicle. The dynamic load causes the principal stresses to ‘rotate’ and act in more than one direction. Figure 2.2 illustrates this phenomenon showing that the shear stresses are zero when the wheel load is directly above the element. In the laboratory, single-stage and multi-stage repeated loading triaxial (RLT) tests are the most common test methods used to characterize the UGMs. The RLT tests cannot reproduce the exact conditions in the field, i.e. having a stress reversal in shear and change in direction of principal stresses. This stress reversal is sometimes referred to as principle stress rotation. None the less, the RLT test is considered a good indicator of the material behavior under repeated loading condition. The loading combinations of vertical and horizontal stresses within the base layer can be duplicated using the RLT test. However the RLT for this study cannot reproduce the stress reversal or change direction of the principal stresses. The stresses blueprinted in the RLT test are those found in-situ when the wheel load is right above the area of interest. In this case the stresses applied in a repeated load test (RLT) are $\sigma_1 = \sigma_z$ (vertical stress) and $\sigma_2 = \sigma_3 = \sigma_r$ (horizontal stress) (Lekarp, 1997).
2.3 Behavior of Base Course Layer under Repeated Loading

In its simplest form, stresses are applied to the base course layer in pulses. These stress pulses are generated from the passing over of traffic load. This load occurs during the lifetime of the pavement structure and occurs repeatedly. Previous studies showed that the deformation response can be characterized as resilient (elastic) and residual (plastic) (Lekarp, 1997). The response (elastic and plastic) is a function of the applied stress. Figure 2.3 depicts this response and shows that as the stresses increase, the deformation also increase.
From Figure 2.3, it can be observed that during the lower stress levels the material experiences strain hardening. During this phase, the particles rearrange themselves in a way that causes inter-particle interlock and thus increases stiffness. However, as the stresses increase, the particles start to form shear bands and begin losing stiffness (weakening). The material goes into the strain softening region and slowly as the stresses increase; comes to a state of failure.

UGMs as well as other pavement materials, display an amalgam of resilient and residual strains. The resilient strains are elastic and thus recovered after each load cycle while on the other hand the plastic strains accumulate after each load cycle as there are irrecoverable. The stress-strain relationship developed in a UGM is a non-linear curve and more precisely forms a hysteresis loop. Figure 2.4 illustrates this behavior.
When designing pavements, engineers anticipate that there is more elastic deformation than plastic deformation. Plastic deformation causes irrecoverable ‘damage’ to the structure. Damage to the pavement structure can be seen as rutting or fatigue cracking. This loss of serviceability can be avoided or reduced by characterizing the material. The resilient behavior of UGMs is characterized by the resilient modulus. The Mechanistic Empirical Pavement Design Guide (MEPDG) uses the resilient modulus as the main property when analyzing and designing pavement structures. However, it has been found (Abu-Farsakh et al., 2007) that resilient modulus by itself is not enough to fully understand the behavior of UGMs. There are other factors such as weather and traffic load conditions that can be better characterized using repeated load triaxial test (RLT). More so, previous studies have shown little or no benefit in resilient modulus with the inclusion of geogrid (Nazzal, 2007 and Perkins, 1997) but permanent deformation tests using RLT helps in realizing the benefit. So if geogrid were to be used in the MEPDG, RLT tests would be the best present method for characterization. The RLT test as the name says applied repeated loading to a sample in a triaxial cell. The deformation response of the sample is recorded and the resilient and permanent deformation can be calculated.

2.3.1 Resilient Behavior

Resilient behavior of UGMs under repeated loading is mainly caused by the deformations of individual grains (Werkmeister, 2003). Figure 2.5, shows the response between two particles when no stress is present. As the force between the particles is increased, the size of the contact area increases. The increase in contact area causes friction resistance between each particle to increase. One can also see that the displacement $\Delta \delta$ between each particle decreases also. This decrease in $\Delta \delta$ represents the resilient deformation of the particles.
2.3.2 Permanent Deformation

On the other hand, permanent deformation occurs when the particles re-arrange themselves. Under stress, the particles slide and rotate which is the main cause of the rearrangement. To resist sliding or rotation, the particles must exhibit frictional resistance forces. When breaking and crushing of the particles occur, this means that the contact stresses have exceeded the forces the particles can endure when in contact against each other. Of course each particle can resist a different stress level and this is due to the fact that each particle is different in shape, size, mineralogy, and how much stress is applied to this particular fragment. It can be argued that most of the mechanisms that cause permanent deformation of the UGMs in base
course layer occur during construction (compaction) rather than traffic loading. This is another reason why characterizations of UGMs during RLT tests are of high importance as you want to be able to see how much the construction phase will affect your serviceability.

### 2.4 Factors affecting Resilient and Permanent Deformation Response

There are many factors that affect the deformation behavior (resilient and permanent) of UGMs. The factors deemed most important are described in the sections that immediately follow.

#### 2.4.1 Number of load Cycles

As the number of load cycles (N) increases, most certainly will the permanent strain increase. However, if the loading intensity is not too high, the permanent strain accumulation can be seen to stabilize and comes to a limiting value (Paute et al. 1996). An explanation for this is that the permanent deformation rate per load cycles tends toward zero. Some researchers (Barksdale 1972 and Sweere 1990), reported that there is no limiting value and that strains will keep accumulating as long as there are cycles. In order for stabilization of the permanent strains to occur, Lekarp 1997 found that the stress level needs to be of a low magnitude. Kolisoja 1998 found that specimens loaded would stabilize around 80,000 cycles but then has the potential to become unstable and begin accumulating permanent strain through more load cycles.

#### 2.4.2 Stress State

It has been found that permanent deformation strongly depends on the stress level and increases as the deviator stress increases and confining stress decreases (Werkmeister 2003). Through RLT tests, (Morgan, 1966) observed this same phenomenon; while keeping confining
stress constant, and increasing deviator stress more permanent strain was accumulated and vice versa. Barksdale 1972 also carried out similar tests. In his tests he found that permanent deformation was highly dependent on the applied stress and increased when deviator stress increased and confining stress decreased. Figure 2.6 illustrates Barksdale’s work on the matter.

![Figure 2.6: Influence of Stress Ratio on Permanent Strain in a Granite Gneiss Material (Barksdale, 1972).](image)

In the case of resilient behavior, stress level was found to have a considerable impact. Hicks and Monismith (1971) found that the stress level greatly affected the resilient modulus. They found that increasing confining stress would in turn cause an increase in resilient modulus. On the other hand they also found that the same effect occurred when increasing axial stress but not as high impact as when increasing confining stress. Other researchers (Uzan 1985, Thom and Brown 1989 and Sweere 1990) confirmed the findings of Hicks and Monismith (1971) in that the confining pressure and principal stresses greatly affected the resilient modulus of the unbound granular material.
2.4.3 Moisture Content

Moisture content most certainly affects the behavior of UGMs for both resilient and permanent deformation. For instance, an increase in moisture above the optimum moisture content causes separation between particles and thus less contact area. This also means that an increase in pore pressure leads to a decrease in effective stress which will result in a decrease in shear strength of the material. The loss of shear strength will in turn cause weak spots in the base course layer that will result in irrecoverable deformations such as rutting (Arnold, 2004). If the UGM is saturated, the direct result will be the formation of potholes. The suction between particles is reduced to zero or even negative during saturation. The material then loses all cohesion, de-compacts and thus causes potholes or other related failures. Researchers found that the blending of high levels of saturation and low permeability, leads to low effective stress, high pore pressure and thus low resistance to permanent deformation (Haynes and Yoder 1963, Barksdale 1972, Maree et al. 1982, Thom and Brown 1987, Nazzal et al. 2007). In the case of resilient modulus, researchers (Hicks and Monismith 1971 and Barksdale and Itani 1989) found that there was a significant decrease in resilient modulus with an increase in moisture content. More so, Hicks and Monismith also found that the resilient modulus decreases as the moisture content is increased from its optimum moisture content.

2.4.4 Stress History

In literature, it has been found that stress history is linked to permanent deformation. Smaller permanent strains also occur if the initially applied loads are higher than subsequent loads (Barksdale, 1991). It is interesting to note that during a repeated loading test, the effect of stress history appears as a result of gradual material stiffening. The latter causes a reduction in
the proportion of permanent to resilient strains during subsequent loading cycles (Nazzal, 2007). All in all very limited research has been carried on the effect of stress history. This can be explained because most laboratory test samples are made new and have no previously applied stress.

For the case of resilient response, Hicks (1970) observed that stress history effect was eliminated when roughly one hundred cycles of the same amplitude were applied prior to testing. Allen (1973) found the same thing and suggested that samples be conditioned for 1000 cycles before repeated load tests were carried.

2.4.5 Density

Density is an important factor when considering the permanent deformation development in the UGM. An increased density can improve the resistance to permanent deformation especially under repetitive loading. In other words, a similar stress path in a sample with higher density than one with lower will endure less permanent strain accumulation (Barksdale 1972, 1991, Allen 1973, Marek 1977, Thom and Brown 1988, and Niekerk 2002).

Barksdale, (1972) reported that if a material was compacted at 95% of maximum compaction density (normal proctor) that it yielded more permanent strains than the material compacted at 100% of maximum compaction density. Similarly, Allen, (1973) found that there was a 20% reduction in total plastic deformations in crushed limestone when the specimen density was increased from proctor to modified proctor.
Hicks and Monismith (1971) found that density had a more significant effect when the material was partially crushed material over a completely crushed material. In junction with this finding, they also found that the effect of density decreased with increasing fine contents.

### 2.4.6 Effect of Grading, Fines Content, and Maximum Grain Size

It has been observed that if grading is changed in such a manner that it causes relative density to increase, permanent deformation resistance will also increase and vice versa. Thom and Brown (1988) through RLT tests found that an un-compacted uniformly graded specimen produced less permanent strain over a non-uniformly graded specimen.

Increasing fine contents was found to cause an increase in the level of permanent deformation in RLT tests (Barksdale 1972, 1991). This was also confirmed by Dodds et al. (1999) when 10% fines were added more deformations were observed. Dodds (1999) also found
that the increase in fines caused greater pore water pressure development which in turn reduced the effective stress and thus inducing lower shear strength causing greater failure.

Finally, in 1973 Allen found that angular materials produced smaller plastic deformations when compared to rounded particles. This can be explained due to the fact that angular materials are able to interlock with one another better than rounded particles and in turn causing higher shear resistance.

2.5 Shakedown Theory

In pavement design, one of the main objectives is that the structure remains within permissible limits (rutting) during designed life. More specific for this study is for the UGM in the base course layer to remain within those limits. The limits for the unbound granular layer (UGL) vary based on design purpose but mainly all UGLs should not exhibit any permanent deformation. The system as a whole has very little tolerance for permanent deformation. The mechanism responsible for said permanent deformations are cyclic traffic loading. In conventional geotechnical engineering, limit analysis is used to define the collapse condition of a soil specimen under static loading. Since the loading mechanism is cyclic in pavement, a different limit analysis is studied; the shakedown theory.

Previous studies (Sharp 1984, Paute 1996, Dawson 1999, The 2000) using low stress levels have been able to determine that deformation reaches an asymptote (Figure 2.8). When this asymptotic deformation is reached, any further strains developed will be resilient (recoverable). However, any increase in stress ratio will cause irrecoverable deformations as found by Lekarp (1998). For the purpose of design, this means that there is a maximum permissible load attainable before the elastic deformations turn into permanent deformation. This
point in pavement has been defined as the critical stress level or based off the shakedown theory: the shakedown limit.

![Graph showing Permanent Deformation Behavior at Low Stress Level]

**Figure 2.8: Permanent Deformation Behavior at Low Stress Level**

In the recent years researchers (Arnold 2003, Werkmeister 2003, Nazzal 2007) have proposed a criterion to describe different responses UGMs under cyclic loading in terms of vertical permanent strain rate. These criterions are described below and are classified in three ranges:

- **Plastic Shakedown Range: Range A.** The response shows high strain rates per load cycle. It is only applicable for a finite number of load applications during the initial compaction. Once the compaction period is passed, the permanent strain rate per load cycle decreases to the point where the response is solely elastic. Range A occurs at low stress levels.

- **Plastic Creep Range: Range B.** Initially behavior is similar to range A but after the initial compaction period, the permanent strain rate per cycle either decrease or remain constant. During a RLT test the permanent strain is acceptable but there is some plastic
deformation. This range is dependent on the number of cycles. A large number of cycles could make the specimen go either way: remain within permissible range or fail.

- Incremental Collapse Shakedown Range: *Range C*. There is an initial compaction period but soon after the permanent strain rate barely decreases or not at all and there is continuous accumulation of permanent strain.

Figure 2.9 presents the results of the research carried out by Johnson (1986). He identified four possible shakedown ranges. These are: elastic; elastic shakedown; plastic shakedown and ratcheting. The elastic response is unlikely to occur in UGMs. This is the reason for using the three ranges described earlier as the elastic response is omitted.

![Figure 2.9: Elastic/Plastic Behavior under Repeated Cyclic Load (Johnson, 1986)](image)

In pavement design, Range A or elastic shakedown is the desired range and Range C should never occur. Range B can be acceptable but depends on other factors such as serviceability (life span), amount of traffic load, etc. With proper characterization of the material, the shakedown
concept can be very useful for pavement design. The Engineer will have valuable information of when a particular material will either stabilize in Range A or fail in Range C.

2.6 Geogrid Reinforced Base Course Layer

The first use of geotextiles as we know it came in the 1950’s (Huang and Gao 2004). However, in the 1930’s, a type of cotton woven textile was used in test sections of highways in four states. The first documented case study was that of a structure built on the waterfront in Florida back in 1958 (Huang and Gao, 2004). Eventually in the 1970’s did we see geotextile prominently used in dam constructions in France; Henri Vidal was the first to promote the system of reinforced retaining wall system. The first successful reinforced earth wall in the United States was built in 1972 based off the Vidal reinforced wall system. After extensive research; carried in the late 70’s early 80’s at the Oregon State University, did the use of geosynthetics boom in the USA. As of now there are three main types of geosynthetic products, these are: geogrids, geotextile, and synthetic fibers. Geogrid is the type of reinforcement that pertains mainly to UGMs.

Geogrid is an extruded sheet of polyethylene or polypropylene with apertures punched in a regular pattern (Nazzal, 2007). The simplest manner in which the geogrid works is that the soil aggregates penetrate through those apertures and eventually interlock in them. When the soil interlocks in the geogrid and stress is applied, the stress is transmitted to the rib of the geogrid. More precisely, the stress is transmitted to the longitudinal ribs through the junctions. The transverse and longitudinal ribs meet at the junctions (Nazzal, 2007). This arrangement makes the rib and junction design and strength key factors.
The use of geogrid in UGL in pavement structure usually resulted in reducing the accumulated amount of permanent deformations as well as increasing the service life of pavement structure, as defined by traffic benefit ratio (TBR). The TBR is defined as the ratio of the number of load cycles to achieve a particular rut depth in reinforced section to that of an unreinforced section with identical properties and loading characteristics (Abu-Farsakh et al. 2007). Another factor of improvement is the Base Course Reduction factor (BCR). The BCR is known as the reduction in base course thickness in design induced by the use of geogrid reinforcement. Just like TBR, BCR is defined as the reinforced base thickness divided by the unreinforced base thickness.

2.7 Mechanism of Geogrid Reinforced Base Course

There are three main mechanisms that work in the reinforcement of base course materials using geogrid. These are lateral confinement, tension effect, interlocking and increase in bearing capacity.

2.7.1 Lateral Confinement

The main benefit generated by the geogrid is found in lateral confinement. As a normal load is applied (e.g. Traffic) on the unbound granular layer; its natural tendency is to deform laterally. If the base course material is allowed to deform laterally it will endure irrecoverable strains (permanent deformation) which will result in rutting. If a geogrid is placed within the base course layer, the geogrid will resist the lateral movement in the base course layer and avoid or reduce rutting. Aggregates cannot resist tension. Lateral movement of the material can be seen as a tensile force that is resisted by a geogrid. The geogrid can resist higher tensile strengths than base course materials. The aggregates fall in the geogrid apertures as they begin to deform
laterally. This activates the interlocking mechanism (Figure 2.10) and transmits the tensile stresses in the aggregate to the geogrid which is turn is able to resist much higher tensile stresses.

Figure 2.10: Interlocking Mechanism with Geogrid (Wrigley, 1989).

Another important reinforcement component induced by the geogrid is the distribution of vertical stresses on the subgrade. The geogrid increases the stiffness of the base course material. This increase in stiffness causes less stress to be transmitted to the weaker subgrade. The decreased stress level to the subgrade is due to the fact that the stress to the subgrade is now more widely distributed. Figure 2.11 illustrates two cases when geogrid/geotextile is present and not present in a pavement structure. This is especially true when dealing with weaker subgrades. The reinforcement prevents the loss of geogrid to the subgrade through the application of traffic load. The shear stress transmitted to the subgrade is also reduced. To summarize the four main reinforcement mechanisms under lateral confinement are:

- Prevention of lateral deformations which in turn cause vertical deformations (aggregates are weak in tension; geogrid makes up for this deficit).
• Increase in stiffness of UGM provided sufficient aggregate/geogrid interactions are made.
• Better stress distribution to the subgrade.
• Reduction in shear stress transmitted to subgrade from base course.

Figure 2.11: Improvement of Stress Distribution to Subgrade (Huang, 2004).

2.7.2 Increase of Bearing Capacity

Chen, (2007) found that the inclusion of reinforcement resulted in an increase in ultimate bearing capacity. The reinforcement helps in altering the failure type from a punch failure to a more general failure. The general failure model is preferred as less total deformation in all the pavement layers is observed (Binquet and Lee, 1975).

2.7.3 Tension Membrane Effect

The tension membrane effect is also based from an increase in the tensile modulus of the geogrid. For instance, if the tensile modulus of the geogrid decreases more deformation is required to mobilize the effect. The tension membrane effect is mobilized as a result in subgrade deformation. Generally speaking, a softer subgrade is needed for the tension membrane to take
effect as shown by Barksdale et al. (1989). Therefore, once the deformation occurs in the subgrade the effect of the geogrid is better appreciated as it stretches. This stretch causes the load distribution to be larger and thus limits the overall deformation.

2.8 Factors Affecting Geogrid Benefit

Geogrid geometry is a crucial factor affecting the performance of the geogrid in the pavement structure. The new tri-axial geometry of the TX geogrid can enhance the geogrid benefit. In the bi-axial geogrid the interlocking mechanism causes the tension stresses in the soil to transfer to the longitudinal and transverse ribs which then carry the tension to the junctions. The tension is only carried in only two directions perpendicular to each other at the junction point. In the case of the tri-axial geogrid, the rolling wheel induces reverse normal stresses and shearing stresses in more than one direction. Each junction has six ribs which can each carry a tensile load. Thus, tension stresses carried in the ribs can come in higher levels than can be carried by the bi-axial geogrid.

Another important consideration in using geogrid as a method of reinforcing UGMs in pavement design are the factors that affect the overall benefit provided by the geogrid. Some of these factors include, but are not limited to, are: base course thickness, location of geogrid in UGL, strength of subgrade, and the material properties of the geogrid. Various experimental studies have been conducted to estimate the optimum location to place the geogrid and what thickness of base course to use. For example, for moderate loads a geogrid placed in the middle of a 200mm thick base course layer was found to be optimum (Perkins and Ismeik, 1998). In this study, the geogrid is tested in three locations. These locations are double geogrid (bottom third and top third of a six lift pavement), single geogrid top third and middle. Figure 3.3 (Chapter 3)
illustrates the arrangement of the geogrid in the testing sample. The optimum location to place the geogrid is at the location where the elastic tensile strain is highest.
CHAPTER 3 METHODOLOGY

3.1 Experimental Testing

The experimental testing program for this study was carried out to fulfill the objectives mentioned in chapter one. More so, the experimental work was specifically conducted in such a way to evaluate the effect of the factors that affect the benefit of using geogrid in unbound granular materials. Repeated Load Triaxial (RLT) tests were carried in both single and multi-stage test setups. The work started with the selection of materials, carrying out physical property tests and also establishing a thorough testing factorial to evaluate the performance of geogrid reinforced base course specimens.

3.2 Materials used for this Study

3.2.1 Base Course Materials

All experimental work was conducted using the same base course material. The unbound granular material was obtained from a local Baton Rouge quarry known as Martin Marietta. The aggregate is classified as the Martin Marietta 610 Kentucky Limestone (Figure 3.1). A series of physical property tests were carried to further characterize the material. These include sieve analysis, Standard Proctor compaction, specific gravity, California Bearing Ratio (CBR), and percent absorption. These tests conform to ASTM standards C-136, D-698, C-127, D-1883 respectively. The results of these tests are listed in Chapter 4.
3.2.2 Geogrid

We used five different types of geogrid reinforcement in this study. The geogrid are:

- TX170 (Tri-axial TX1)
- TX160 (Tri-axial TX2)
- BX1200 (Bi-axial BX1)
- BX1100 (Bi-axial BX2)
- BasXgrid 11 (Bi-axial BX3)

There are two main types of geogrid geometry; the triaxial (TX) and the Biaxial (BX, BasXgrid) geogrids. The TX and BX geogrids come from Tensar Earth Technologies while the BasXgrid is from Mirafi. Figure 3.2 illustrates the two distinctive types of geogrid used in this study. Table 3.1 summarizes the physical and mechanical properties of the aforementioned geogrid. The Tensar geogrids are made of a stress resisting polypropylene material while the Mirafi product is made of woven high molecular weight polyester. The BasXgrid is also coated using a polymer.
### Table 3.1: Geogrid Properties

<table>
<thead>
<tr>
<th>Geogrid Reinforcement</th>
<th>Type</th>
<th>Tensile Strength(^a) (kN/m)</th>
<th>Tensile Modulus(^b) (kN/m)</th>
<th>Junction Efficiency (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MD(^c)</td>
<td>XMD(^d)</td>
<td>MD(^c)</td>
<td>XMD(^d)</td>
</tr>
<tr>
<td>TX170</td>
<td>TX1</td>
<td>9.5</td>
<td>475</td>
<td></td>
</tr>
<tr>
<td>TX160</td>
<td>TX2</td>
<td>8.6</td>
<td>430</td>
<td></td>
</tr>
<tr>
<td>BX1200</td>
<td>BX1</td>
<td>6.0</td>
<td>9.0</td>
<td>300</td>
</tr>
<tr>
<td>BX1100</td>
<td>BX2</td>
<td>4.1</td>
<td>6.6</td>
<td>205</td>
</tr>
<tr>
<td>BasXgrid 11</td>
<td>BX3</td>
<td>7.3</td>
<td>7.3</td>
<td>365</td>
</tr>
</tbody>
</table>

\(^a\) At 2% strain, \(^b\) At 2% strain, \(^c\) Machine Direction, \(^d\) Cross Machine Direction.

![Figure 3.2 (a): Biaxial Type Geogrid (b): Tri-Axial Type Geogrid](image)
3.3 Sample Preparation

Following the recommendation of AASHTO T307 the size of the sample was based on the particle size of the material. For untreated, unbound granular material, the testing sample diameter should be at least five times greater than the maximum particle size of the material. In the case of this study, the maximum particle size was 19 mm. The testing setup of 150 mm diameter and 300 mm height was then used. Other studies also recommend using 150 mm diameter and 300 mm for particle size of 19mm as well (NCHRP, 2004).

Based on the AASHTO recommendations, a split mold (Figure 3.4 (b.)) was used for the compaction of the unbound granular material. First of all, the Kentucky Limestone base material was oven-dried. It was found that after remaining twenty-four hours in the oven, the material still had some moisture. This moisture, known as residual moisture content, was found to become almost nil after letting the material air dry for twenty-four hours and in the oven for another twenty-four hours. Once the material was dry, it was placed in a splitter to obtain four homogenous samples. The next step was to add water to a calculated mass of dry material to obtain the desired moisture content. The material was left to absorb the moisture for a minimum of one hour and a maximum of two hours. The material was then divided into six equal quantities; the material is compacted in six lifts to achieve uniform compaction and to maintain at least ±1% of maximum dry density. A sample of at least 1000 grams was saved to measure moisture content of mixture and make sure the sample remained within ±0.5% of target moisture content. The six lifts were compacted using a vibratory compactor as they were placed in the spilt mold. Each lift had a thickness of 50 mm. To control the maximum dry density once a layer was compacted, a measurement was taken from the top of the spilt mold to the top of the
compacted lift. At the end of each compacted lift the smooth surface was lightly roughed out to create some void space to obtain bonding with the next compacted layer. Each sample was enclosed by two latex membranes with thickness of 0.3 mm. The use of two membranes was found to be very important as the first membrane would slightly rip and tear due to compaction. A second membrane was used to seal sample. Figure 3.4 (a.) is a photo of the base with a porous stone.

Figure 3.3 illustrates the arrangement of geogrid used for this study. The geogrid was placed at the desired lift horizontally.

![Figure 3.3: Geogrid Arrangement (Abu-Farsakh et al. 2007)](image)

(a). (b).
3.3 Samples Prepared for Different Moisture Content

The effect of moisture content on permanent deformation of unreinforced and reinforced samples was also carried out in this study. More so, the effect of moisture content was studied with and without reinforcement through both the single-stage and multi-stage repeated load triaxial tests. In all cases the effect of moisture was compared to the optimum moisture content. The sample preparation was conducted out in the same way described in section 3.3 with the difference of having different moisture contents. The two moisture contents are ±2.5% of the optimum moisture content. The -2.5% of optimum moisture content is known as dry of optimum and +2.5% of optimum moisture content is known as the wet of optimum.

3.4 Testing Setup

All repeated load triaxial tests were carried out using the Material Testing System (MTS810) with a closed loop and servo hydraulic system. Figure 3.5 shows the testing
equipment. The applied loads were measured using a ±5000lbf capacity load cell. The load cell is placed inside the testing chamber. This particular setup helps in reducing equipment compliance errors, alignment errors and pressure area errors. The axial deformation was measured using two Linearly Variable Differential Transducers (LVDTs). The two LVDTs were secured to the top plate. The confining pressure was achieved through the use of pressurized air. It was measured using a pressure sensor. The prepared sample was placed on the load cell and secured on to the load through a base plate (Figure 3.4(a.)). The sample was then sealed with the use of o-rings and clamps so that the confining pressure could be applied. Once the sample was safely secured in the pressure chamber (Figure 3.5), it was conditioned to be prepared for the RLT tests. The conditioning testing phase is described in section 3.5.1.

Figure 3.5: MTS 810 Testing Machine with Testing Sample
3.5 Repeated Load Triaxial (RLT) Tests

In order to characterize the permanent deformation behavior of the material the repeated load triaxial tests was performed on unreinforced and geogrid reinforced samples. The RLT tests help in determining the deformation properties of the base course material. The RLT test consists of a haversine-shaped load pulse. The reason for a haversine load pulse is to better simulate the traffic loading conditions. More precisely, the load pulse consists of 0.1 second load duration and a 0.9 second rest period. Figure 3.6 shows the haversine-shaped load pulse. The cyclic and contact loads in Figure 3.6 are described in section 3.5.1. Figure 3.7 shows the haversine load curve during a RLT test. The figure also shows where the resilient and plastic deformations were recorded and used to calculate the corresponding strains. In the RLT tests; applied load, vertical deformations, and confining pressure were recorded. Based off the recorded vertical deformations two types of strains can be calculated. These are elastic and plastic strains. Equations 3.1 and 3.2 define the elastic and plastic strains respectively.

![Figure 3.6: Haversine Load Pulse in RLT Tests (Protocol P07, FHWA)](image)
Figure 3.7: Applied Load and Response Curve for RLT Tests (Werkmeister, 2003).

\[ \varepsilon_r(N) = \frac{\delta_{res}(N)}{L_0(1 - \varepsilon_p(N-1))} \]  

Equation 3.1

\[ \varepsilon_p(N) = \frac{\delta_{per}(N)}{L_0(1 - \varepsilon_p(N-1))} \]  

Equation 3.2

Where:

\( L_0 \) = Sample Original Length
\( \varepsilon_r(N) \) = Resilient Strain at Cycle N
\( \varepsilon_p(N) \) = Permanent Strain at Cycle N
\( \delta_{res}(N) \) = Total Resilient Change in Sample Length at Cycle N
\( \delta_{per}(N) \) = Total Permanent Change in Sample Length at Cycle N

3.5.1 Single-Stage RLT Test-(Standard)

Standard Single-Stage repeated load tests were carried out to fulfill the objectives of this study. The single-stage RLT tests were key to determine the permanent and resilient deformation behavior of the unreinforced and geogrid reinforced testing samples. The single-stage RLT tests
were performed by following the AASHTO T-307 standard. More so, the T-307 standard was followed especially when taking into consideration the condition phase of the sample before testing. Condition consisted of 1,000 cycles applied at a pressure of 14 psi (93 kPa) and a confining stress of 15 psi (103.4 kPa). Conditioning is important as it removes unevenness of the top and bottom layers. It also helps in the initial rearrangement of the aggregates which could cause larger obsolete permanent deformation.

Once the conditioning phase was completed, the sample was tested for 10,000 cycles. The confining pressure was fixed at 3 psi (21 kPa) and the peak cyclic stress applied was 33 psi (230 kPa). Figure 3.6 shows the manner in which the loading stresses were applied. The maximum load in Figure 3.6 is equivalent to the peak cyclic stress mentioned above. As shown in Figure 3.6 the maximum load consists of both the contact load and the cyclic load. The confining pressure was selected based on field measurement and calculation based on various studies; notably Barksdale (1993). The peak cyclic stress was also based on previous finite element studies (e.g., Nazzal, 2007) of induced stresses on base course layer due to vehicular loading as well as field measurements and calculations obtained from literature.

Data collection was carried out through an elaborate system. Loading and vertical deformation values were recorded 512 times per second at load cycles intervals of: 0-10/unit cycle, 10-100 at every 10th cycle, 100-1000 at every 50th cycle, 1000-2000 at every 100th cycle, 2000-3000 at every 200th cycle, 3000-10000 at every 500th cycle. This method of data recording was chosen based on literature review and common sense. Most of the deformations in a single-stage permanent deformation test occur during the first 2000 cycles and this is why most of the data recording occurs during the beginning of the test.
Based on previous studies (Abu-Farsakh et al., 2007), a parameter known as reduction in vertical permanent strain (RPS) was introduced to numerically compare the benefit of the geogrid. To obtain RPS the following equation is used:

\[
\text{RPS} \% = \frac{\text{permanent strain without geogrid} - \text{permanent strain with geogrid}}{\text{permanent strain without geogrid}} \times 100
\]

The testing factorial for the single-stage RLT tests at standard frequency is summarized in Tables 3.3 and 3.4. Table 3.3 presents the single-stage RLT tests at the different moisture contents while Table 3.4 presents the reinforcement location in the testing specimen factorial. For example; from Table 3.3 it can deduced that 3 tests were conducted for TX1 geogrid and from Table 3.4, three tests were carried at the double location in the specimen. Therefore, this yields to a total of 9 tests at the optimum moisture content for the double geogrid setup.

3.5.2 Single-Stage RLT Test-(High Frequency)

The effect of number of cycles on the permanent deformation behavior was investigated by increasing the number of load cycles from 10,000 cycles to 100,000 cycles. The higher frequency RLT tests were performed using the same general protocol as the standard single-stage tests. The main difference occurs in the loading frequency. In the 10,000 cycle single-stage tests, the frequency at which the load is applied is 1 hz. In the case of the 100,000 cycle test this was increased to 2 hz. The higher frequency was chosen to save time as the 100,000 cycle test at 1 hz is very lengthy. The same haversine load pulse was used with the exception of a 0.05s load phase and 0.45s rest period. The stress levels were exactly the same as for the 10,000 cycle test (confining: 3psi, peak cyclic stress: 33psi). There was also a 1000 cycle conditioning phase using the same stress levels (confining: 15psi, peak cyclic 14psi). Data collection was recorded 256
times per second per cycle. The cycle intervals from 0-10,000 was the exact same. However, from 10,000-20,000 data was recorded every 1000 cycle. From 20,000-50,000 data was recorded every 2000th cycle and finally for 50,000-100,000 every 5000th cycle. The testing factorial carried out for the single-stage RLT high frequency tests are listed in Table 3.5. The testing moisture content was kept at optimum moisture content and the location of the geogrid was kept constant at the middle of the specimen.

3.5.3 Multi-Stage RLT Test

Similar to the single-stage testing, multi-stage tests were carried to fulfill the objectives of this study. In the case of multi-stage, the permanent deformation behavior was observed at different stress levels. In this study, there were six stages of 10,000 cycles each. More so, the shakedown limits of the material were determined. The stress levels for each stage are summarized in Table 3.2. Table 3.6 presents the testing factorial conducted to achieve the objectives of this study for the multi-stage RLT tests.

Each stage differed from the previous one due to an increase in \( q/p \) ratio \([q: \text{deviatoric stress } \sigma_1- \sigma_3; p: \text{mean confining pressure } (\sigma_1 + 2\sigma_3)/3]\). In doing so, crossing the static failure line of the sample would be easier to achieve and thus determining the shakedown limits. To increase the ratio, \( p \) was kept constant and \( q \) was increased. Samples were tested with three different values of \( p \): 72, 145 and 198 kPa. Test 1, 2 and 3 represent the different \( p \) values. These different \( p \) value tests help in producing linear curves in the \( p-q \) space and used as a tool to define the boundary between ranges A, B and C.
Table 3.2: Multi-Stage RLT Tests Stress Levels

<table>
<thead>
<tr>
<th>Stage</th>
<th>Test 1</th>
<th></th>
<th>Test 2</th>
<th></th>
<th>Test 3</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$p$ (kPa)</td>
<td>$q$ (kPa)</td>
<td>$p$ (kPa)</td>
<td>$q$ (kPa)</td>
<td>$p$ (kPa)</td>
<td>$q$ (kPa)</td>
</tr>
<tr>
<td>1</td>
<td>72</td>
<td>43</td>
<td>145</td>
<td>136</td>
<td>198</td>
<td>210</td>
</tr>
<tr>
<td>2</td>
<td>72</td>
<td>91</td>
<td>145</td>
<td>183</td>
<td>198</td>
<td>276</td>
</tr>
<tr>
<td>3</td>
<td>72</td>
<td>120</td>
<td>145</td>
<td>229</td>
<td>198</td>
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<td>4</td>
<td>72</td>
<td>155</td>
<td>145</td>
<td>274</td>
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<td>5</td>
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<td>183</td>
<td>145</td>
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<td>6</td>
<td>72</td>
<td>195</td>
<td>145</td>
<td>350</td>
<td>198</td>
<td>510</td>
</tr>
</tbody>
</table>

3.6 Statistical Analysis

Statistical analysis, more precisely analysis of variance (ANOVA) was carried on the RPS (section 3.5.1) for the single-stage RLT tests. ANOVA analysis is performed to test if means of different ‘groups’ are equal or not equal to one another. The main goal of the ANOVA is to test the differences in the means of various groups without increasing type I error rate. Unlike the two-sample t-test, ANOVA is capable of detecting whether or not a treatment is significant with losing power or causing the type I error to inflate. However, when comparing various means, ANOVA is not capable of telling the user which of these means is the different one. For this purpose, post ANOVA techniques were also used. The post ANOVA-LSM (least square means) allows the user to differentiate between the various factor levels and locate where the differences are. As the name implies, the post ANOVA-LSM technique uses the least square means as a basis for analysis. In the post ANOVA-LSM means with differently letter grades are different. Factorial ANOVA is the type of ANOVA that pertains to this study. The factorial
ANOVA is used to test the effects of at least two treatments or factors. More so, ANOVA analysis assumes that the data is normally distributed as well as homogeneity of variance. Statistical Analysis Software (SAS) was used as a tool to carry out the statistical analysis.

3.7 Testing Factorial

Tables 3.3 through 3.6 present the testing factorial established to achieve the aforementioned objectives (chapter 1, section 1.3).

Table 3.3: Testing Factorial for Single-Stage RLT Tests (Moisture Content Dependent)

<table>
<thead>
<tr>
<th></th>
<th>Unreinforced</th>
<th>TX1</th>
<th>TX2</th>
<th>BX1</th>
<th>BX2</th>
<th>BX3</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Optimum Moisture Content</strong></td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td><strong>Wet (+2.5%)</strong></td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td><strong>Dry (-2.5%)</strong></td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
</tbody>
</table>

Table 3.4: Testing Factorial for Single-Stage RLT Tests (Geogrid Location Dependent)

<table>
<thead>
<tr>
<th></th>
<th>TX1</th>
<th>TX2</th>
<th>BX1</th>
<th>BX2</th>
<th>BX3</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Double</strong></td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td><strong>Upper</strong></td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td><strong>Middle</strong></td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
</tbody>
</table>

Table 3.5: Testing Factorial for High Frequency RLT Tests

<table>
<thead>
<tr>
<th></th>
<th>TX1</th>
<th>TX2</th>
<th>BX1</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Optimum Moisture Content</strong></td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
</tbody>
</table>

1: All tests carried at Middle Location
### Table 3.6: Testing Factorial for Multi-Stage RLT Tests

<table>
<thead>
<tr>
<th></th>
<th>Unreinforced</th>
<th>TX1</th>
<th>TX2</th>
<th>BX1</th>
<th>BX2</th>
<th>BX3</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Optimum Moisture</strong></td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Content(^1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Wet (+2.5%)</strong>(^1)</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>n/a</td>
<td>2</td>
</tr>
<tr>
<td><strong>Dry (-2.5%)</strong>(^1)</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>n/a</td>
<td>2</td>
</tr>
</tbody>
</table>

\(^1\): All tests carried at Middle Location
CHAPTER 4 RESULTS AND ANALYSIS

Chapter 4 will present the results and analysis of the experimental work carried for this study. The first part of the chapter contains the results of physical property tests carried on the tested material while the second part shows the repeated loading characterization results and analysis.

4.1 Physical Property Tests

As mentioned in chapter 3 section 3.2.1, physical property tests were carried to further characterize the testing material. Figure 4.1 shows the grain size distribution obtained for the material from the sieve analysis. The medium grain size ($D_{50}$) of the material was found to be 5 mm. the effective size ($D_{10}$) was found to be 0.28 mm. The coefficient of uniformity ($C_u$) and coefficient of curvature ($C_c$) were found to be 24 and 1.97, respectively. Based on these values the classification properties of the material can be determined. The Unified Soil Classification System (USCS) classify this material as gravel well-graded and the American Association of State Highway and Transportation (AASHTO) as an ‘A-1-a’ soil. Table 4.1 presents a summary of the physical property tests carried on the 610 Kentucky Limestone. Figure 4.2 represents the Standard Proctor test graph. The values of maximum dry density and optimum moisture content obtained from the Standard Proctor test analysis are given in Table 4.1. Table 4.1 also presents the results of other conventional tests conducted on 610 Kentucky Limestone base material.
Figure 4.1: Particle Size Distribution

Figure 4.2: Standard Proctor Compaction Test
Table 4.1: Physical Property Test Results

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk Specific Gravity (G&lt;sub&gt;sb&lt;/sub&gt;)</td>
<td>2.59</td>
</tr>
<tr>
<td>% Absorption</td>
<td>1.11 %</td>
</tr>
<tr>
<td>California Bearing Ratio (CBR)</td>
<td>101</td>
</tr>
<tr>
<td>Optimum Moisture Content&lt;sup&gt;1&lt;/sup&gt; (%)</td>
<td>6.6 %</td>
</tr>
<tr>
<td>Maximum Dry Density&lt;sup&gt;1&lt;/sup&gt; (γ&lt;sub&gt;d,max&lt;/sub&gt;)</td>
<td>140.3 lb/ft&lt;sup&gt;3&lt;/sup&gt; (2247 kg/m&lt;sup&gt;3&lt;/sup&gt;)</td>
</tr>
<tr>
<td>Mean Particle Size (D&lt;sub&gt;50&lt;/sub&gt;)</td>
<td>5 mm</td>
</tr>
<tr>
<td>Uniformity Coefficient (C&lt;sub&gt;u&lt;/sub&gt;)</td>
<td>24</td>
</tr>
<tr>
<td>Coefficient of Curvature (C&lt;sub&gt;c&lt;/sub&gt;)</td>
<td>1.97</td>
</tr>
<tr>
<td>USCS Classification</td>
<td>GW</td>
</tr>
<tr>
<td>AASHTO Classification</td>
<td>A-1-a</td>
</tr>
<tr>
<td>Friction Angle (φ)&lt;sup&gt;2&lt;/sup&gt;</td>
<td>49°</td>
</tr>
<tr>
<td>Cohesion Strength&lt;sup&gt;2&lt;/sup&gt;</td>
<td>26kPa</td>
</tr>
</tbody>
</table>

<sup>1</sup>: Standard Proctor Test  
<sup>2</sup>: Monotonic Tri-axial Test

4.2 Single-Stage RLT Tests

Single-stage repeated load triaxial (RLT) tests were carried out on unreinforced and geogrid reinforced test samples. For the reinforced cases, TX1, TX2, BX1, BX2 and BX3 geogrids were used. The reinforcement arrangements were upper one third (upper), middle and double (as described in Figure 3.2).

Figures 4.3 to 4.10 present the curves of the average permanent strain amount versus the number of cycles defined for different RLT cases. Averages consisted of at least three samples with coefficient of variation being equal to or less than 15%. Coefficient of variation is defined as the standard deviation divided by the sample mean. The curves have been arranged to compare the two main factors here; location and geogrid type (geometry). Figures 4.3 to 4.5 show the results of permanent strain while comparing the geogrid location in the specimen factor. Figures 4.6 through 4.10 show the results of the permanent strain but this time comparing the geogrid type (geometry).
Similar to results from previous studies (Werkmeister 2003, Nazzal 2007, Austin 2008), the permanent deformation curve has two distinct stages. In the first stage the material accumulates most of the permanent deformation. This behavior is explained in chapter 2 under section 2.3. The particles re-arrange themselves due to induced stresses which causes the larger initial deformation. During the second stage (secondary stage) the material accumulates a much lower rate of permanent strain; it almost seems like the material reaches a limiting value. It is worthy to note that the biggest benefit generated from the inclusion of the reinforcement is found in the secondary stage. This shows that reinforcement benefits are generated through aggregate properties such as shape, interlocking and particle friction mechanisms.

In figures 4.3 through 4.5 it can be observed that the reinforcement in double arrangement yielded the most favorable results when compared to the unreinforced case. More so, geogrid TX1 always yielded the lowest deformation value and BX2 the most. When looking at figures 4.6 through 4.10, it can be observed that the double arrangement produces the lowest permanent deformation. On the other hand, it is hard to tell which of middle or upper one third reinforcement location is favorable as in some cases middle location yields the least deformation and in others upper one third location. Table 4.2 summarizes the results obtained through Figures 4.3 to 4.10 and presents the mean, standard deviation and coefficient of variation. Table 4.2 summarizes the reinforced cases; in the unreinforced case the average permanent is 2.84% with a coefficient of variation of 13.4%.
Figure 4.3: Permanent Strain Curve for Sample Reinforced at Double Location in Testing Specimen

Figure 4.4: Permanent Strain Curve for Sample Reinforced at Upper One Third Location in Testing Specimen
Figure 4.5: Permanent Strain Curve for Sample Reinforced at Middle Location in Testing Specimen

Figure 4.6: Permanent Strain Curve for Sample Reinforced with Geogrid TX1
Figure 4.7: Permanent Strain Curve for Sample Reinforced with Geogrid TX2

Figure 4.8: Permanent Strain Curve for Sample Reinforced with Geogrid BX1
Figure 4.9: Permanent Strain Curve for Sample Reinforced with Geogrid BX2

Figure 4.10: Permanent Strain Curve for Sample Reinforced with Geogrid BX3
Table 4.2: Summary of Single-Stage Permanent Deformation Values

<table>
<thead>
<tr>
<th></th>
<th>Double</th>
<th></th>
<th>Upper</th>
<th></th>
<th>Middle</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean</td>
<td>SD</td>
<td>COV</td>
<td>Mean</td>
<td>SD</td>
<td>COV</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TX1</td>
<td>1.21</td>
<td>0.176</td>
<td>14.526</td>
<td>1.7</td>
<td>0.130</td>
<td>7.646</td>
</tr>
<tr>
<td></td>
<td>1.75</td>
<td>0.177</td>
<td>10.138</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TX2</td>
<td>1.39</td>
<td>0.251</td>
<td>14.983</td>
<td>1.74</td>
<td>0.041</td>
<td>2.335</td>
</tr>
<tr>
<td></td>
<td>1.9</td>
<td>0.139</td>
<td>7.300</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BX1</td>
<td>1.38</td>
<td>0.078</td>
<td>5.678</td>
<td>1.83</td>
<td>0.208</td>
<td>11.351</td>
</tr>
<tr>
<td></td>
<td>1.9</td>
<td>0.208</td>
<td>10.940</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BX2</td>
<td>1.60</td>
<td>0.136</td>
<td>8.512</td>
<td>2.02</td>
<td>0.164</td>
<td>8.071</td>
</tr>
<tr>
<td></td>
<td>2.08</td>
<td>0.248</td>
<td>11.893</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BX3</td>
<td>1.44</td>
<td>0.123</td>
<td>8.550</td>
<td>1.83</td>
<td>0.148</td>
<td>8.085</td>
</tr>
<tr>
<td></td>
<td>1.88</td>
<td>0.224</td>
<td>11.927</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unrein.</td>
<td>2.84</td>
<td>0.38</td>
<td>13.4</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*SD: Standard Deviation
*COV: Coefficient of Variation

Figures 4.11 through 4.15 represent the mean percentage values of the RPS values at 100, 1000, 3000, 5000 and 10,000 load cycles. Based off the RPS, the biggest benefit generated by a geogrid was 58% at 10,000 load cycles. This was for the TX1 geogrid at double location. From the RPS figures it can be observed that at 100 cycles there is no definite trend. This can be caused by the fact that first 100 cycles are still in the first stage where the accumulation of permanent strain is high. Once this part is over and the mechanism of interlocking has been triggered do we see the most reduction in permanent strain (secondary stage). More so, the reinforcement benefit induced was geogrid geometry dependent and location of geogrid reinforcement in specimen. The TX geogrids (TX1 and TX2) caused the biggest RPS value while the BX2 caused the smallest. It is worthy to note that the RPS value increased as the number of cycles increased. The biggest RPS values were found at the 10,000 load cycle mark. Double location proved to yield the most reduction in permanent strain while the upper one third and middle location were fairly similar.
Figure 4.11: RPS at 100 Load Cycles

Figure 4.12: RPS at 1000 Load Cycles
Figure 4.13: RPS at 3000 Load Cycles

Figure 4.14: RPS at 5000 Load Cycles
The results from the Analysis of Variance (ANOVA) and post ANOVA-LSM are presented in Tables 4.3 to 4.6. The ANOVA analysis was used to evaluate and compare the effects of the various factors affecting the benefit of the use of the reinforcement. The RPS at 100, 1000, 3000, 5000 and 10,000 load cycles was used as the dependent variable for this analysis. The independent variables or effects were:

- Effect of Geogrid Location;
- Effect of Geogrid Type (tensile modulus and geometry);
- Effect of number of load cycles;
- Effect of the interaction between geogrid location and type;
- Effect of the interaction between geogrid type and number of cycles;
• Effect of the interaction between geogrid location and number of cycles.

Table 4.3 shows the results of the ANOVA analysis at a 95% confidence level (α-value of 0.05 or less). In this study the null hypothesis $H_0$ tests if the means are equal to each other. ($H_0: \mu_1=\mu_2=\mu_3$ and so on). To assess if an effect is generated by the factors, the means are compared to each other. Table 4.3 shows that for geogrid type, location and number of load cycles all had an effect on reduction of permanent deformation. In the Pr>F column the effects of location, type and number of cycles each have values smaller than $\alpha=0.05$ and therefore we reject the null hypothesis. When comparing the interactions, the geogrid location had a larger effect than geogrid type when both were compared to number of load cycles. Also, the interaction of geogrid type and geogrid location had a strong effect on the RPS as it produced a small F-value.

Tables 4.4 through 4.7 show the results obtained for the post ANOVA-LSM analysis. Table 4.4 shows the results obtained of the grouping of geogrid location effect. The letter grouping follows the order of the Roman alphabet. Letter A indicates that the double location yielded the most improvement while the middle location provided the least. Table 4.5 shows the grouping of geogrid type effect. Geogrid TX1 proved to produce the most improvement while BX2 provided the least benefit on the RPS. Geogrids TX2, BX1 and BX3 provided very similar improvements and were all closely ranked. Table 4.6 provides the results obtained from the grouping of number of load cycles effect. At the 10,000 load cycles mark is where most of the benefit was appreciated. At 100 cycles the effect on the RPS was the least with the less favorable letter grade. At 3000 cycles the benefit to the RPS is fairly similar to the ones at 5000 and 10,000 load cycles and suggest that the greatest benefit occurred somewhere around that mark. Finally Table 4.7 shows the post ANOVA-LSM obtained for the geogrid location and type interaction. This analysis is particularly interesting as it shows which geogrid at which location yielded the
greatest improvement in permanent strain reduction. As expected from the results in Tables 4.4 and 4.5 geogrid TX1 at double location provided the largest improvement on the RPS. The geogrid type and location that produced the smallest benefit was the BX2 at the middle. Interestingly, the TX2 geogrid at upper location provided better improvement of the RPS than the BX2 at double location.

Table 4.3: ANOVA Analysis Results for RPS

<table>
<thead>
<tr>
<th>Effect</th>
<th>Num DF</th>
<th>Den DF</th>
<th>F Value</th>
<th>Pr &gt; F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>2</td>
<td>32</td>
<td>181.28</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Type</td>
<td>4</td>
<td>32</td>
<td>29.42</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Cycles</td>
<td>4</td>
<td>32</td>
<td>148.26</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Location*Type</td>
<td>8</td>
<td>32</td>
<td>4.30</td>
<td>0.0014</td>
</tr>
<tr>
<td>Type*Cycles</td>
<td>16</td>
<td>32</td>
<td>0.31</td>
<td>0.9916</td>
</tr>
<tr>
<td>Location*Cycles</td>
<td>8</td>
<td>32</td>
<td>0.90</td>
<td>0.5291</td>
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</tbody>
</table>

Table 4.4: Grouping of Geogrid Location Effect on RPS

<table>
<thead>
<tr>
<th>Letter Grouping</th>
<th>Mean</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>44.011</td>
<td>Double</td>
</tr>
<tr>
<td>B</td>
<td>30.809</td>
<td>Upper</td>
</tr>
<tr>
<td>C</td>
<td>26.592</td>
<td>Middle</td>
</tr>
</tbody>
</table>
Table 4.5: Grouping of Geogrid Type Effect on RPS

<table>
<thead>
<tr>
<th>Letter Grouping</th>
<th>Mean</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>40.061</td>
<td>TX1</td>
</tr>
<tr>
<td>B</td>
<td>35.050</td>
<td>TX2</td>
</tr>
<tr>
<td>B</td>
<td>33.903</td>
<td>BX3</td>
</tr>
<tr>
<td>B</td>
<td>33.145</td>
<td>BX1</td>
</tr>
<tr>
<td>C</td>
<td>26.861</td>
<td>BX2</td>
</tr>
</tbody>
</table>

Table 4.6: Grouping of Number of Cycles Effect on RPS

<table>
<thead>
<tr>
<th>Letter Grouping</th>
<th>Mean</th>
<th>Cycles</th>
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<tr>
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<td>39.611</td>
<td>10000</td>
</tr>
<tr>
<td>AB</td>
<td>39.429</td>
<td>5000</td>
</tr>
<tr>
<td>AB</td>
<td>39.263</td>
<td>3000</td>
</tr>
<tr>
<td>B</td>
<td>35.666</td>
<td>1000</td>
</tr>
<tr>
<td>C</td>
<td>15.052</td>
<td>100</td>
</tr>
</tbody>
</table>
Table 4.7: Grouping of Geogrid Location and Type Interaction Effect on RPS

<table>
<thead>
<tr>
<th>Letter Grouping</th>
<th>Mean</th>
<th>Location*Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>52.145</td>
<td>Double TX1</td>
</tr>
<tr>
<td>AB</td>
<td>45.319</td>
<td>Double BX1</td>
</tr>
<tr>
<td>B</td>
<td>43.162</td>
<td>Double BX3</td>
</tr>
<tr>
<td>B</td>
<td>42.341</td>
<td>Double TX2</td>
</tr>
<tr>
<td>C</td>
<td>37.749</td>
<td>Upper TX1</td>
</tr>
<tr>
<td>C</td>
<td>37.088</td>
<td>Double BX2</td>
</tr>
<tr>
<td>CD</td>
<td>36.784</td>
<td>Upper TX2</td>
</tr>
<tr>
<td>D</td>
<td>31.254</td>
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<tr>
<td>E</td>
<td>29.548</td>
<td>Upper BX3</td>
</tr>
<tr>
<td>E</td>
<td>28.998</td>
<td>Middle BX3</td>
</tr>
<tr>
<td>E</td>
<td>27.915</td>
<td>Upper BX1</td>
</tr>
<tr>
<td>EF</td>
<td>26.201</td>
<td>Middle BX1</td>
</tr>
<tr>
<td>EF</td>
<td>25.060</td>
<td>Middle TX2</td>
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<tr>
<td>G</td>
<td>22.049</td>
<td>Upper BX2</td>
</tr>
<tr>
<td>G</td>
<td>21.447</td>
<td>Middle BX2</td>
</tr>
</tbody>
</table>
4.2.1 Single-Stage RLT Tests-(High Frequency)

The effect of number of load cycles was carried using MTS machine through conducting RLT tests at higher frequency, allowing the increase of load cycles to 100,000. The geogrid used were TX1, TX2 and BX1. The arrangement factor was kept constant as the geogrid was placed in the middle location in the specimen for all cases. Figure 4.16 presents the permanent deformation curves obtained for the different tested cases. The same trend is found here as with the more conventional 10,000 load cycles test. The TX1 geogrid yielded the most favorable results with the greatest reduction in permanent strain while the BX1 and TX2 were difficult to set apart. Figure 4.17 present the curve obtained during the 100,000 load cycles at the 10,000 load cycles mark. Compared to the regular 10,000 load cycles test, at the 10,000 load cycle point extracted from the higher frequency 100,000 cycles test there is slightly less accumulated permanent strain. As mentioned in chapter 3 section 3.5.2, the frequency for the 100,000 load cycles tests was 2hz rather than the conventional 1hz. It is interesting to see that the same primary and secondary permanent deformation behavior found in the standard single-stage RLT tests was also present for the higher frequency 100,000 load cycles RLT test. The curves for 100,000 load cycles do not look reaching an asymptote and seem to be slowly but surely continuously accumulating permanent strains. These results confirm the predictions set by Barksdale, (1972) and Sweere, (1990) who reported that there is no limiting value and that the sample will continue accumulating permanent strains as long as there are load cycles applied.
Figure 4.16: Permanent Strain Curve for 100,000 Load Cycles

Figure 4.17: Permanent Strain Curve extracted from High Frequency RLT Tests at 10,000 Load Cycles
Figures 4.18 to 4.22 represent the RPS values diagrams obtained for the 100,000 load cycles tests. Again, the same trend was found here similar to the regular single stage 10,000 load cycles tests. The TX1 geogrid proved to yield the most reduction in RPS at 57.5% at 5,000 load cycles. In comparison to the same arrangement (middle) for the single-stage 10,000 load cycles test this reduction in RPS is higher. Furthermore, the RPS values did not peak at 100,000 load cycles. In the case of TX1 the highest RPS value was found at 5000 cycles. For the TX2 and BX1 geogrids the largest RPS value was at 5000 and 50,000 load cycles respectively.

Figure 4.18: RPS at 1000 Load Cycles
Figure 4.19: RPS at 5000 Load Cycles

Figure 4.20: RPS at 10,000 Load Cycles
Similar ANOVA and post ANOVA-LSM analysis were carried as the ones in section 4.2. As table 4.8 shows, only the effects of geogrid type and number of load cycles was studied since
the geogrid location was kept constant. By looking at the F-value in table 4.2, it can be deduced that the geogrid type had an effect on the RPS while the number of load cycles did not. This is confirmed in the post ANOVA-LSM analysis; tables 4.9 and 4.10. In table 4.9, the geogrid which yielded the largest improvement is TX1. The next to follow was the TX2 geogrid and with similar but slightly less improvement is the BX1 geogrid. Table 4.10 shows the grouping of number of load cycles. All cycles starting from 1000 through to 100,000 yielded the same letter group. However, the statistical analysis software ranked the 50,000 cycle as the location where the biggest improvement was found and 100,000 the second to last. This implies that the benefit peaked close to 50,000 cycles and decreased slightly towards the 100,000 load cycles mark.

Table 4.8: ANOVA Analysis Results of RPS for High Frequency Test

<table>
<thead>
<tr>
<th>Type 3 Tests of Fixed Effects</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effect</td>
</tr>
<tr>
<td>Type</td>
</tr>
<tr>
<td>Cycles</td>
</tr>
</tbody>
</table>

Table 4.9: Grouping of Geogrid Type Effect on RPS for High Frequency Test

<table>
<thead>
<tr>
<th>Letter Grouping</th>
<th>Mean</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>56.366</td>
<td>TX1</td>
</tr>
<tr>
<td>B</td>
<td>38.850</td>
<td>TX2</td>
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<tr>
<td>B</td>
<td>38.124</td>
<td>BX1</td>
</tr>
</tbody>
</table>
Table 4.10: Grouping of Number of Cycles Effect on RPS for High Frequency Test

<table>
<thead>
<tr>
<th>Letter Grouping</th>
<th>Mean</th>
<th>Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>45.685</td>
<td>50000</td>
</tr>
<tr>
<td>A</td>
<td>45.531</td>
<td>10000</td>
</tr>
<tr>
<td>A</td>
<td>45.471</td>
<td>5000</td>
</tr>
<tr>
<td>A</td>
<td>44.336</td>
<td>100000</td>
</tr>
<tr>
<td>A</td>
<td>41.211</td>
<td>1000</td>
</tr>
</tbody>
</table>

4.2.2 Resilient Deformation

Figures 4.23 through 4.27 depict the average resilient strain curves obtained from single-stage RLT tests. These curves were obtained from the same tests carried for the permanent deformation and for geogrids TX1, TX2, BX1, BX2 and BX3 were studied. More so, the same geogrid locations of double, upper and middle were tested. In each case the curve obtained for the unreinforced case is shown as reference. The resilient strain for both the unreinforced and geogrid reinforced samples initially increased through the first couple hundred cycles. Then it slowly decreased to reach an asymptote roughly around the 4000-6000 load cycles range. This is described as a steady resilient response. Through literature it had been explained that this response is caused by increased deviatoric strain in the direction perpendicular to the load application which in turn causes the Poisson’s ratio to slightly decrease. In turn the decreased Poisson’s ratio causes the stiffness to increase and thus causing a decrease in resilient strain. The point where the material reaches a steady resilient response is stress dependent (Nazzal et al. 2007).
Figures 4.23 through 4.27 demonstrate similar results obtained by previous researchers such as Perkins et al. (2004) and Abu-Farsakh et al. (2006). The improvement of the resilient strain by the insertion of the geogrid was found to be minimal. Since resilient behavior of a granular material is based on individual grains (Werkmeister et al. 2002). The benefit generated by the geogrid was not expected. However, it is worth mentioning that there was a slight improvement of the resilient strain response as shown in the figures. The unreinforced sample produced the highest resilient strain and the reinforced samples produced slightly less.

Table 4.11 represents the resilient modulus values obtained for the different single-stage RLT tests. More so, the resilient modulus is known as the composite resilient modulus. It is obtained from the resilient deformation curves when the resilient strain reaches a stabilized value (usually starting at the 5000 load cycle point). Different to the regular resilient modulus test, these values were obtained through one deviatoric stress only. This is explained since the single-stage RLT tests have constant cyclic stress and confining pressure. Similar to the resilient strain characteristics, the resilient modulus does not exhibit a lot of change with the presence of the geogrid. The highest resilient modulus ($M_r$) value obtained was 39.14 ksi for the TX1 double geogrid and the lowest $M_r$ was 32.57 ksi for the BX2 geogrid at middle location. As a reference the $M_r$ value for the unreinforced case was 33.44 ksi. As table 4.11 shows there is no real trend with the geogrid tensile modulus, geometry or arrangement.
Figure 4.23: Resilient Deformation Curve Reinforced with Geogrid TX1

Figure 4.24: Resilient Deformation Curve Reinforced with Geogrid TX2
Figure 4.25: Resilient Deformation Curve Reinforced with Geogrid BX1

Figure 4.26: Resilient Deformation Curve Reinforced with Geogrid BX2
Figure 4.27: Resilient Deformation Curve Reinforced with Geogrid BX3

Table 4.11: Resilient Modulus Values

<table>
<thead>
<tr>
<th></th>
<th>Double (ksi)</th>
<th>Upper (ksi)</th>
<th>Middle (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TX1</td>
<td>39.14</td>
<td>33.73</td>
<td>34.02</td>
</tr>
<tr>
<td>TX2</td>
<td>38.43</td>
<td>37.59</td>
<td>38.98</td>
</tr>
<tr>
<td>BX1</td>
<td>33.69</td>
<td>33.93</td>
<td>33.88</td>
</tr>
<tr>
<td>BX2</td>
<td>38.96</td>
<td>36.09</td>
<td>32.57</td>
</tr>
<tr>
<td>BX3</td>
<td>35.15</td>
<td>34.69</td>
<td>33.68</td>
</tr>
</tbody>
</table>

Unreinforced = 33.44 ksi
4.3 Multi-Stage RLT Tests

Multi-Stage RLT tests were performed on unreinforced and reinforced test samples. The main achievement of the multi-stage RLT tests was to further characterize the deformation behavior of the material at different stress levels and to determine the shakedown limits. The multi-stage RLT tests were also used to observe the behavior of the reinforced sample when compared to the unreinforced case.

Figure 4.28 presents the multi-stage RLT test results as number of cycles against permanent strain. The figure clearly illustrates the six different stress levels and confirms that the permanent strain is dependent on the stress levels. Higher deviatoric stress caused a greater accumulation of plastic deformation. Figure 4.28 also helps to appreciate the benefit of the reinforcement. In the case of the reinforcement, the location of the geogrid was kept constant at middle location. Once again this location was chosen as it is the most likely location found in field conditions. The reinforcement trend followed the one found in the single-stage RLT tests. The TX1 geogrid proved to yield the greatest resistance against permanent deformation and the BX2 the least. It is interesting to note that for all the geogrids except TX1 the difference is made during stages five and six. This could imply that the geogrid type/tensile modulus/geometry is more prevalent during higher stress levels. More so, the effect of the geogrid was not seen until at least the second stress level, which also implies that there, needs to be a certain stress level before the benefit of the geogrid is seen.
Figures 4.29 through 4.32 represent the vertical permanent strain rate per cycle versus permanent strain and confining pressure for unreinforced and reinforced cases. Figures 4.29 and 4.31 represent the permanent strain rate per cycle versus the permanent strain for unreinforced and reinforced cases, respectively. Figure 4.30 and 4.32 show the vertical permanent strain rate per cycle versus confining pressure for both unreinforced and reinforced cases, respectively. All four figures can be used to define the shakedown ranges of A, B and C. Range A behavior in figures 4.29 and 4.31 is visible through the part where the permanent strain rate decreases in a way that stops all further accumulation of permanent strain. It seems that the behavior in range A (plastic shakedown range) reaches an asymptote in the vertical direction or where a final permanent strain value is reached.
Range B can be broken down into two behaviors: primary and secondary stage. For range B behavior (plastic creep shakedown) the behavior of the primary stage is similar to the behavior found in Range A. On the other hand, in the secondary stage, the permanent strain rate accumulation decreases at a smaller rate. In this part, the individual particles are deforming; consisting of distortion, fractures and particle movement (rotating and sliding). The inter-particle damage causes the permanent strain rate to become constant and in turn causing greater accumulation of permanent strain. Eventually the accumulation of deformation will lead to material collapse and reaching the tertiary stage (Nazzal, 2007). In both figures (4.29 and 4.31) the range B behavior is clearly visible especially by following stage 4 stress level. The material initially behaves similarly to range A and after the damage to the particles is endured the permanent strain value begins to increase.

Range C is known as the incremental collapse shakedown range. In range C, the material continuously accumulates incremental permanent strain with each cycle. Therefore, every cycle is causing deformation which in turn causes a horizontal shape in the figures. There is an initial range A and B behavior but it lasts very few loading cycles.

When comparing the unreinforced case versus the reinforced case, figures 4.29 and 4.31 demonstrate the benefits caused by the geogrid. For the unreinforced case only stages 1 and 2 reach the line determining the range A behavior while in the reinforced case stages 1, 2 and 3 reach past the range A line. This shows that the geogrid caused the material to remain longer in range A and thus resist higher stress levels.

Figures 4.30 and 4.32 also present the benefits generated by the geogrid. In this situation, the unreinforced sample has only stage 1 in the range A area while the reinforced case has two
stages. Both stages 5 and 6 are in the range C area and a hint of stage 4 for the unreinforced case is in the range C area.

The horizontal lines in figures 4.29 to 4.32 have the same constant values. The line separating ranges A and B defines the plastic shakedown limit. This line was derived from the description given earlier and using the criteria suggested by Werkmeister, (2003). Werkmeister determined that the plastic shakedown limit line was found at 0.045×10^{-3} strains and used the following equations:

Range A:  \( \varepsilon_{p_{5000}} - \varepsilon_{p_{3000}} < 0.045 \times 10^{-3} \)  \( \text{Equation 4.1} \)

Range B:  \( \varepsilon_{p_{5000}} - \varepsilon_{p_{3000}} > 0.045 \times 10^{-3} \)  \( \text{Equation 4.2} \)

Where:

\( \varepsilon_{p_{5000}} \times 10^{-3} = \) accumulated permanent strain at 5000 load cycles

\( \varepsilon_{p_{3000}} \times 10^{-3} = \) accumulated permanent strain at 3000 load cycles

The line separating ranges B and C is known as the plastic creep limit. This limit was defined using the description given earlier and criteria suggested by Werkmeister, (2003). Researchers found that this line was situated at 0.4×10^{-3} strains. The following equations were used to compute the location of the plastic creep limit line:

Range B:  \( \varepsilon_{p_{1_{5000}}} - \varepsilon_{p_{1_{3000}}} < 0.4 \times 10^{-3} \)  \( \text{Equation 4.3} \)

Range C:  \( \varepsilon_{p_{1_{5000}}} - \varepsilon_{p_{1_{3000}}} > 0.4 \times 10^{-3} \)  \( \text{Equation 4.4} \)
In the case of this study the shakedown plastic shakedown limit was found to exactly fit the line given by equations 4.1 and 4.2. However, the line describing the plastic creep limit from equations 4.3 and 4.4 were found not to be suitable for this study. The plastic creep limit was found suitable at $1.1 \times 10^{-4}$ strains.

It was found possible to simply classify the material response into the three shakedown ranges using the permanent strain rate per cycle curves. This also helps complimenting the equations 4.1 through 4.4.

- Range A: Permanent Strain Rate decreasing with load cycles;
- Range B: Constant permanent strain rate with load cycles;
- Range C: Permanent strain rate increasing with load cycles.

Once the shakedown limits were determined, the plastic shakedown limit and plastic creep limit can be represented in the $p-q$ stress space. The limits were assumed to be linear and plotted against the static shear strength line. The samples were tested at three maximum mean stresses of $p = 72\text{kPa}$, $145\text{kPa}$ and $198\text{kPa}$. Using the limits defined in Figures 4.29 through 4.32 the boundary between shakedown ranges A and B was found as the point where the highest loading stage/stress (1, 2 etc) crossed the range A-B boundary. Using the three maximum mean stresses, three points where obtained and thus a best fit straight line was produced. The same procedure was adopted for the plastic creep limit but this time the lowest loading stage/stress that crossed the range B-C boundary was used. Reinforcement yielded a benefit in Figures 4.29 to 4.32 by having more stages in range A for example.
Figure 4.29: Multi-Stage Strain Rate versus Permanent Strain with Shakedown Limits (Unreinforced)

Figure 4.30: Multi-Stage Strain Rate versus Confining Pressure with Shakedown Limits (Unreinforced)
Figure 4.31: Multi-Stage Strain Rate versus Permanent Strain with Shakedown Limits Reinforced with TX1 Geogrid

Figure 4.32: Multi-Stage Strain Rate versus Confining Pressure with Shakedown Limits Reinforced with TX1 Geogrid
Figure 4.33 represents the shakedown limits for the tested material obtained at optimum moisture content plotted in the $p$-$q$ space. Figures 4.34 through 4.38 present shakedown limits in $p$-$q$ stress space for the samples reinforced with TX1, TX2, BX1, BX2 and BX3 geogrids. The calculation of shakedown limits can help further characterize any material. Different materials (reinforced materials also) will have different limits and thus can be ranked by the use of the shakedown theory in a manner that helps predict the materials performance in a pavement structure. For pavement structures, the behavior of the material in range A was deemed favorable as in this range the material behaved elastically after reaching a stabilization period. Range C is not suitable for pavement structures as the material exhibits irrecoverable strains with each additional load cycle. The material in range B is dependent on number of load cycles. The more load cycles applied the more likely permanent deformation will occur and eventually reach failure.

![Figure 4.33: Shakedown Limits for Limestone Unreinforced](image-url)
Figure 4.34: Shakedown Limits for Limestone Reinforced with TX1 Geogrid

Figure 4.35: Shakedown Limits for Limestone Reinforced with TX2 Geogrid
Figure 4.36: Shakedown Limits for Limestone Reinforced with BX1 Geogrid

Figure 4.37: Shakedown Limits for Limestone Reinforced with BX2 Geogrid
4.4 Effect of Moisture Content on Permanent Deformation

As mentioned in chapter 3 section 3.4.1 the effect of moisture content on permanent deformation of unreinforced and reinforced samples was studied. In the case of the single-stage RLT tests, the reinforcement location factor was kept constant; the geogrid was placed at the middle location of testing samples. The middle location did not yield the biggest benefit in permanent strain reduction but it is the most practical location to mimic the field conditions and for constructability purposes. The geogrid types used to study effect of moisture content were TX1, TX2, BX1 and BX3.

4.4.1 Single-Stage RLT

Figures 4.35 and 4.36 show the permanent strain curves obtained for the different samples prepared at wet (+2.5%) and dry (-2.5%) of optimum respectively. Both figures are
compared to the unreinforced case at optimum moisture content as a basis for reference. For both wet and dry of optimum, geogrid reinforced samples TX1, the geogrid with largest tensile modulus, produced the least permanent strain when compared to TX1 and BX1. Also, the benefit of the geogrid was more appreciated at the optimum moisture condition. More deformation was accumulated for the samples prepared at wet of optimum when compared to those prepared at the dry of optimum. This confirms the findings of Arnold (2004) which was cited in chapter 2 section 2.4.3.

Figures 4.37 and 4.38 present the RPS values obtained from the single-stage RLT tests prepared at wet (+2.5%) and dry (-2.5%) of optimum respectively. For the case of the wet of optimum (+2.5%) specimen, the RPS at 100 cycles yielded negative values. This means that the sample in the unreinforced case yielded better initial resistance against permanent deformation. This phenomenon can be explained due to the fact that in the first few cycles the material is re-arranging and trying to stabilize and thus the geogrid mechanism is not yet triggered. It can also be explained due to the fact that the elevated moisture content (+2.5%) causes separation between the particles causing more cycles until the effect of the reinforcement is observed. However at the 1000 load cycle mark the geogrid reinforcement mechanisms are stabilized and the RPS value are positive. The largest RPS value obtained for the sample prepared at wet of optimum (+2.5%) was found to be at 31%. This was for the case of the TX1 geogrid reinforcement at the 10,000 load cycle point. Figure 4.38 presents the dry of optimum (-2.5%) RPS values for load cycles at 100, 1000, 3000, 5000 and 10,000. The geogrid reinforcement TX1 at the 10,000 load cycle mark produced by far the largest reduction in permanent strain value at slightly less than 71%. The next largest RPS value from a different geogrid was 41% for the TX2 geogrid at the 10,000 load cycle point.
Figure 4.39: Permanent Strain Curve for Sample at Wet of Optimum (+2.5%)

Figure 4.40: Permanent Strain Curve for Sample at Dry of Optimum (-2.5%)
Figure 4.41: RPS for Single-Stage RLT Tests at Wet of Optimum (+2.5%) 

Figure 4.42: RPS for Single-Stage RLT Tests at Dry of Optimum (-2.5%)
4.4.2 Multi-Stage RLT

The effect of moisture content on permanent deformation of geogrid reinforced samples was also studied using the multi-stage RLT tests. In these tests, the geogrid studied are the TX1, TX2, BX1 and BX3. The geogrid reinforcement was placed at the middle location in the testing specimen. Figures 4.43 and 4.44 present the multi-stage RLT permanent deformation curves obtained for the specimens prepared at wet and dry of optimum respectively. In Figure 4.43 the unreinforced specimen at wet of optimum (+2.5%) only lasted until stage 5 stress levels. This resulted in a permanent strain value of 7.2% within 31,000 load cycles. The geogrid reinforcement benefit followed the trend of the stiffest geogrid yielding the largest resistance against permanent deformation; TX1 geogrid reinforcement produced the most favorable results for both for samples prepared at wet (+2.5%) and dry (-2.5%) of optimum. From Figures 4.43 and 4.44 the benefit generated by the geogrid reinforcement is clearly visible, however when comparing the different geogrids the difference between them is not clearly defined. The result of this makes it hard to observe the benefits gained in the p-q stress pace. More tests at different p and q stress levels would need to be conducted to visualize the geogrid reinforcement benefit in the p-q stress space. Figures 4.46 and 4.48 present the shakedown limits obtained at wet and dry of optimum respectively in the p-q stress space. When compared to each other, the sample prepared at wet of optimum (+2.5%) produces a plastic creep line that is further down and right of the static failure line. This indicates that the wet of optimum sample resists smaller applied stresses and will reach the plastic failure region sooner that the sample prepared at dry of optimum (-2.5%). Figures 4.45 and 4.47 present the multi-stage shakedown limits for the specimen prepared at wet (+2.5%) and dry (-2.5%) of optimum respectively.
Figure 4.43: Multi-Stage RLT Permanent Strain Wet of Optimum (+2.5%)  

Figure 4.44: Multi-Stage RLT Permanent Strain Dry of Optimum (-2.5%)
Figure 4.45: Multi-Stage Strain Rate versus Permanent Strain with Shakedown Limits for Specimen at Wet of Optimum (+2.5%)
Figure 4.47: Multi-Stage Strain Rate versus Permanent Strain with Shakedown Limits for Specimen at Dry of Optimum (-2.5%)

Figure 4.48: Shakedown Limit for Unreinforced Specimen at Dry of Optimum (-2.5%)
CHAPTER 5 SUMMARY, CONCLUSION AND RECOMMENDATIONS

5.1 Summary

Throughout this research study, the use of geogrids as a reinforcement mechanism has been discussed and recognized. This study was carried out to assess the benefits generated by using geogrid reinforcement in the base course layer in a pavement structure. The objectives of this study were achieved through conducting experimental testing program to investigate the behavior of the material in unreinforced and geogrid reinforced base course material. First, physical property tests were conducted to characterize the tested base course material. Then, two types of RLT tests were performed; single-stage and multi-stage. The RLT tests were used to characterize the permanent deformation behavior of the unreinforced and geogrid reinforced material through simulated conditions that are as close as possible to those encountered in the field.

Five different geogrids that have different mechanical properties were used in this study. Of the five geogrids two of them have triangular shaped geometry (triaxial geogrids) while the other three have rectangular shaped geometry (biaxial geogrids). The geogrid factors studied here were: geogrid type and geogrid location in the testing sample. The effects of moisture content, stress levels and number of load cycles on permanent deformation of unreinforced and geogrid reinforced samples were also studied.
5.2 Conclusion

Listed below are the conclusions drawn from this study:

- The inclusion of geogrid reinforcement helped in reducing the accumulation of permanent deformation in the RLT tests. This was demonstrated in both the single-stage and multi-stage RLT tests using the different geogrids.

- Of the two TX geogrids the TX1 geogrid performed consistently better than other four geogrids. Of the three BX geogrids, BX1 performed the best and BX2 provided the least benefit. BX3 performed in between the two with a slight bias on the BX1 side. This conclusion was drawn from the single-stage RLT tests.

- Placing the geogrid in the double location yielded the largest improvement regardless of geogrid type. The upper location and middle location provided closer improvements with the upper location being slightly better.

- Of the factors studied (geogrid location and type), geogrid location in the sample proved to have the greatest influence on the reduction of permanent strain.

- In the 100,000 cycles (higher frequency) single-stage RLT test, there was no limiting value to the permanent deformation of all tested specimens and they kept accumulating permanent strain.

- The shakedown theory was successfully implemented in this study as a means to characterize the deformation properties of unbound granular materials.

- Using reinforcement in the multi-stage RLT proved to generate a benefit as a factor of the different stress levels. The TX1 geogrid caused the base course material to resist a higher stress level in the range A shakedown area and thus potentially giving a pavement structure additional resistance to traffic loads before failure.
• Geogrid reinforcing did not show meaningful improvements in the resilient deformation or resilient modulus. There was no logical trend with geogrid tensile modulus or geometry.

• The effect of moisture on unreinforced and geogrid reinforced specimens was noticed in both single-stage and multi-stage RLT tests. The moisture caused change to the state of stress. Geogrid improvement was noted but with not very high magnitude.

5.3 Recommendations

1. When comparing the BX geogrids with the TX geogrids, the reason for the different behavior was unclear. It was hard to assess the effect of geometry on the benefit generated in reducing permanent deformation. More tests should be carried to study the effect of geometry on geogrid reinforcement mechanisms.

2. The TX geogrid claims 100% junction efficiency; this was difficult to assess. More research effort is needed to investigate this unique property. This could be studied together with geogrid-aggregate interlocking.

3. The relationship between the aggregate size and geogrid aperture size should also be further investigated.

4. Further research effort is needed for implementing the shakedown concepts of reinforced versus unreinforced samples for potential use in pavement design.

5. Further research is needed to study the influence of other factors such as geogrid location in the specimen for the shakedown concept. The shakedown limits values should also be further studied with reinforced samples.
6. Further research work is recommended for defining the shakedown limit values that can be applied to a wider range of aggregate and reinforcement materials for use in pavement structures.

7. Large scale tests are recommended to mobilize the pull-out mechanism of geogrid/aggregate interface and its influence in increasing resistance to permanent deformation.

8. More studies are needed to relate the geogrid strength properties and geometry to the improvement in permanent deformation of reinforced samples.

9. More studies are needed to relate the laboratory behavior of reinforced samples to real time field performance of geogrid reinforced base layer in a pavement structure.
REFERENCES


Notes from Experimental Statistics Courses, EXST 7005, 7015., *Statistical Inference I and II*. Louisiana State University, Baton Rouge, Louisiana.


VITA

Gael Souci was born in April 1984 in Reduit, Mauritius to Philippe and Jacqueline Souci. In 2002, Gael Souci graduated from the International Baccalaureate program carried at Le Bocage International School. In January of 2004 he started his bachelor’s degree in civil engineering at Louisiana State University, Baton Rouge and received his degree in 2007. During his undergraduate studies, Gael met his wife Kara Mihlon Souci who was also studying civil engineering at Louisiana State University. Gael was a competitive swimmer and represented his country in International Swimming competitions as well as with the Louisiana State University varsity swimming team. Gael will graduate with a Master of Science degree in civil engineering in December 2009.