

INVESTIGATING THE REINFORCEMENT EFFECT OF GEOGRIDS IN FLEXIBLE PAVEMENTS

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Bingye Han
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DEDICATION

To my parents

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ABSTRACT

Geogrids have been increasingly used to reinforce granular base course to improve the performance of flexible pavements. In this study, different interfacial behaviors of geogrid-reinforced aggregate were investigated through laboratory experiments. Influence factors affecting the reinforcement effect were analyzed, and reinforcement mechanisms were revealed.

Firstly, the repeated load triaxial (RLT) test was conducted to investigate the vertical resilient deformation behavior of the reinforced aggregate. The result show that the RLT test was not effective in evaluating the reinforcement effect. To further analyze the lateral resilient deformation behavior of the reinforced aggregate, a large-scaled cyclic shear test was introduced. The result show that this method could effectively quantify the reinforcement effect of geogrids along with a term, resilient interface shear modulus.

Secondly, the interfacial shear behavior of the reinforced aggregate was studied by a large direct shear test. The relative size of aggregate to geogrid aperture had a significant effect on the mechanical properties of the reinforced aggregate and interface failure modes. A guideline to optimize the combination of geogrids and aggregates was proposed based on the results.

Thirdly, a modified loaded wheel tester was employed to investigate the permanent deformation behavior of the reinforced aggregate. Critical influence factors affecting the reinforcement effect were analyzed. A better reinforcement effect could be obtained by increasing the portion of effective aggregates, increasing the tensile strength of geogrids. Triaxial geogrids performed better in reinforcing aggregate than biaxial geogrids when their tensile strength were similar. The tensile strength of geogrids had much significant effects on the reinforcement compared to the aperture geometry.

Last, the resilient and permanent deformation behaviors of reinforced granular base were further studied through a full-scale accelerated pavement testing to verify the laboratory results. The test program included performance testing, response testing and pavement trench. The result demonstrated that an enough permanent deformation was needed for mobilizing geogrids to constrain the lateral movement of granular particles,

increase the modulus of base course, and improve the stress distribution on subgrade. For a pavement system consisting of a thin surface and base layer, a proper placement position of geogrids was at the base-subgrade interface.

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CHAPTER ONE
INTRODUCTION

1.1 Research background

1.1.1 Geogrid in pavement design

A traditional flexible pavement is usually composed of three distinct layers: asphalt mixture surface course, granular base course, and soil subgrade. The permanent deformation (rutting) will be produced on the flexible pavement under repeated traffic loads, which not only affects the driving comfort, but also causes safety hazards to drivers. As a functional supporting layer in pavement structure, the granular base course plays a significant role in pavement systems. The results of the AASHO road test show that in the flexible pavement, the permanent deformation caused by the asphalt mixture surface course, granular base course (including subbase course), and subgrade accounts for 32%, 59%, and 9% of the total permanent deformation of the entire pavement, respectively (1). It is found that the permanent deformation generated by the granular base course contributes more than half of that of the entire pavement structure, and this proportion is further increased as the thickness of the asphalt pavement decreases (2). The granular base course in a high quality can effectively dissipate the stresses imposed by vehicles to the underlying subgrade (3). Therefore, limiting the permanent deformation of the granular base course is particularly important for the permanent deformation of flexible pavements control to meet the basic requirements of the flexible pavement design.

The unbound granular materials (UGMs) usually constitute the granular base course. Although the UGMs usually have certain shear strength to resist deformation, they have no tensile strength and, hence, the granular particles at the bottom of the base layer could move laterally under repetitive traffic loading, weakening the base-subgrade interface, which causes a gradual deterioration in pavement systems (4; 5). The deterioration could be due to the migration of subgrade fine particles into the base layer and the penetration of base course materials into subgrade (6; 7). In order to improve the tensile properties of granular base course, geosynthetic materials which have good tensile properties are introduced to place at the tensile deformation zone of the base course to restrict the lateral movement of the UGMs and increase the stability of the granular base course. Therefore, reinforcing the

granular base course by geogrids is regarded as an effective way to improve the pavement performance.

Geosynthetics have been increasingly used as construction materials in civil engineering projects such as pavements, retaining walls, landfills, etc. Today, there are eight main types of geosynthetic products available in the market, including geotextiles, geogrids, geonets, geomembranes, geosynthetic clay liners, geopipes, geofoams, and geocomposites (8). Each product is designed to solve a range of civil engineering problems. Geogrids are one of the most often used geosynthetic products by highway agencies in the pavement engineering. Placed at the base-subgrade interface or within the base course, geogrids have been widely used for base reinforcement and subgrade stabilization in flexible pavements since 1970s. Numerous laboratory and field tests have confirmed the benefits of using geogrids as reinforcement in pavement systems, including improving the stress distribution of the base course (9), increasing pavement-bearing capacity (10; 11), decreasing rutting deformations (12-14), reducing base-soil contamination (15-17), lowering the design thickness of the base course (18-23), controlling crack propagation (24-26), and prolonging pavement service life (27-35).

Geogrids are mainly made of polypropylene, polyethylene, polyester or coated polyester materials, and made by a series of standard manufactural procedures consisting of heating, plasticizing, extruding, flattening, perforating, directional drawing, and cooling etc. to form a unique mesh-like planar product. Compared with the other types of geosynthetic products, geogrids possess a larger tensile strength and rigidity to withstand more tensile stresses. Geogrids are usually divided into uniaxial, biaxial and triaxial geogrids according to the direction of resistance they provide in the plane, as shown in Figure 1.1. The uniaxial geogrids are commonly used in the retaining walls, slopes and embankments, while the biaxial and triaxial geogrids are usually used for base reinforcement and subgrade stabilization, and the latter two geogrids are the main research objects in the proposed doctoral study.

1.1.2 Assessment methods for evaluating geogrid reinforcement in flexible pavements

Geogrids are believed to realize these benefits through three main mechanisms, including

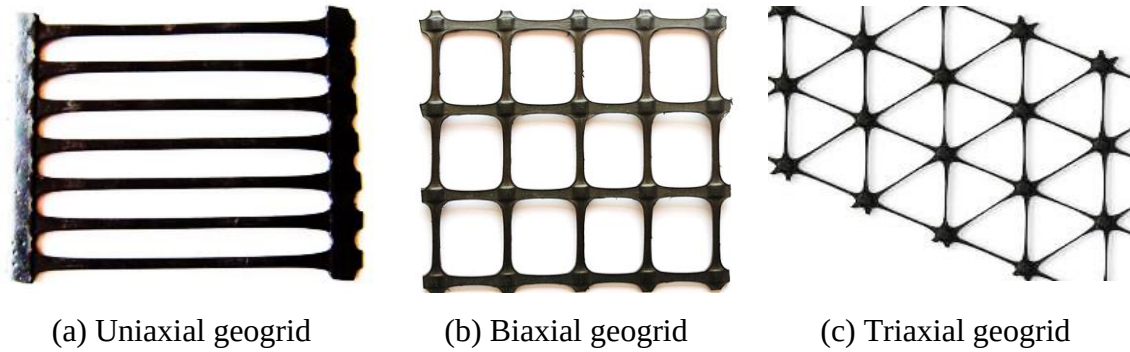


Figure 1.1. Types of geogrids

lateral restraint, increased bearing capacity, and tensioned membrane effect (36-38). Interaction between geogrids and aggregates under repeated traffic loading plays a significant role in implementing these mechanisms, so quantification of the interaction has been considered as an essential problem to be solved in the geogrid-reinforced flexible pavement design (39). In the past four decades, lots of researchers utilized different test methods to evaluate the geogrid-aggregate interaction, and these methods are mainly divided into material tests for reinforced granular material and structural tests for reinforced granular base course according to the research object.

1.1.2.1 Reinforced granular material

Recently, several test methods have been proposed to quantify the geogrid-aggregate interaction governing the performance of geogrid-reinforced aggregate. These tests include the cyclic triaxial test, monotonic pullout test, direct shear test, bending stiffness test, push test, bender element test, modified loaded wheel test.

The cyclic triaxial test is a commonly used approach to characterize the geogrid-aggregate interaction under cyclic loading. Yang and Han (40) believe that the geogrid can provide additional lateral restraint stress on aggregates, which can limit the lateral movement of aggregates under cyclic loading and improve the resilient modulus of aggregates and reduce the permanent deformation of aggregates. Rahman et al. (41) confirmed this conclusion in their cyclic triaxial tests on construction and demolition

materials. Gu et al. (42) used the cyclic triaxial test to evaluate the effects of geogrid reinforcement in UGMs, and their test results indicated that geogrids increase both the vertical and horizontal resilient moduli, and reduce the permanent deformation of the reinforced UGMs as well. However, several other researchers' cyclic triaxial results show that the permanent deformation of reinforced aggregates is obviously lower than that of unreinforced aggregates, but the resilient modulus does not increase significantly (43-46). Therefore, the cyclic triaxial test could be considered effective in evaluating the geogrid effect in terms of permanent deformation, but controversial in characterizing the reinforcement in terms of resilient modulus.

The geogrid and its surrounding granular base materials usually undergo a certain relative displacement under repeated traffic loads, resulting in a shearing occurring at the geogrid-aggregate interface. This shearing resistance can limit the lateral movement of aggregate to form an effective interlocking in the geogrid apertures, which can reinforce the entire pavement structure. The reinforcement effect of geogrids on aggregate depends on the degree of shearing resistance between the geogrids and aggregate (22; 47; 48). Pullout and direct shear tests are the other two common methods to measure this shearing resistance (49-55). On one hand, the relative displacement between geosynthetic inclusion and aggregates is small when geogrids are placed in the aggregate base course (56), but the relative displacement in above-mentioned pullout and direct shear test are too large to simulate the real condition. On the other hand, the load applied by these two methods on the geogrid-interface is a monotonic force, which is different from that induced by the repeated wheel loads.

In addition to these traditional methods, several scholars creatively proposed different new methods to describe the geogrid-aggregate interaction in reinforced aggregate. Sprague et al. (57) used a bending stiffness test to quantify the reinforcement effect of geogrids. The geogrid was sandwiched into aggregates and the reinforced aggregate was confined by a flexible membrane. A uniform vacuum pressure was applied to the sandwiched sample to measure the deflection of the reinforced aggregate for bending stiffness calculation. The results show that the bending stiffness of reinforced aggregate is higher than that of unreinforced aggregate, and the increase percentage has a good

correlation with the traffic benefit ratio (TBR) measured by the field geogrid-reinforced flexible pavement. Matys and Baslik (58) proposed a push test to evaluate the interlocking effect of geogrids on the aggregates. Aggregates were confined in a cone and pushed along the geogrid to form a displacement between aggregates and the geogrid. The push force was recorded for the reinforcement effect comparison. Byun and Tutumluer (59) used a bender element to measure the shear modulus around the geogrid in a cyclic triaxial test. Two sets of bender elements which can emit and receive shear waves were attached on the cylinder sample to record the shear wave velocity. Results show that the shear modulus of reinforced aggregate near the geogrid is significantly larger than that of unreinforced aggregate, and the calculated shear modulus can effectively quantify the interaction between geogrids and aggregates. However, the repeated moving wheel load cannot be simulated in above-mentioned tests. Therefore, a more convenient and practical test method should be proposed to evaluate the interaction between geogrids and aggregates.

1.1.2.2 Reinforced granular base course

The overall strength and stability of reinforced granular base course in the flexible pavement will be improved due to the inclusion of geogrids in the granular materials. The mechanical behavior of the reinforced granular base course is greatly different from that of the conventional unreinforced granular base course because of the interaction between geogrids and aggregate. Unlike the reinforced granular materials, researchers often utilize large-scaled test methods to investigate the reinforcement effect of geogrids in the flexible pavement system, including cyclic plate loading test, field tracking test, accelerated pavement testing (APT), and falling weight deflectometer (FWD) test.

Cyclic plate loading test is a commonly used methods to evaluate the geogrid confinement effects on reinforced aggregate base course in the pavement structure. Halim (60) conducted a group of cyclic plate loading tests to study the reinforcement effect of the reinforced aggregate base course on the flexible pavement structure, and found that the reinforced pavement structure can withstand more loadings when the pavement achieved a pre-determined rut depth of 20 mm. Based on Halim's results, Carroll et al. (61) further improved the test, and proposed an equivalent thickness conversion diagram of the

reinforced aggregate base course and the unreinforced aggregate base course as shown in Figure 1.2. The inflection point represents the minimum thickness required for the reinforced aggregate base.

To better understand the geogrid reinforcement mechanisms, Haas et al. (62) built a reinforced and unreinforced pavement structure in a large-scale tank with a dimension of $4.5 \text{ m} \times 1.8 \text{ m} \times 0.9 \text{ m}$. The pavement was loaded by the cyclic loading plate, and surface permanent deformation, strain of the geogrids, and the stress on the top surface of the subgrade was measured. The test results show that reinforced base course can significantly reduce the surface permanent deformation. When the permanent deformation is large, the reinforcement is strengthened by the tensioned membrane effect, and when the permanent deformation is small, the reinforcement is realized by the lateral constraint effect. The base course thickness of the reinforced pavement structure could be reduced by 25 - 50 % when it reached to a similar permanent deformation in the unreinforced pavement structure. In addition, they found that the position of the geogrid in flexible pavements is a critical factor affecting the reinforcement. They believed that the geogrid should be placed at the base-subgrade interface for a thinner base course, but the geogrid should be put at the middle of

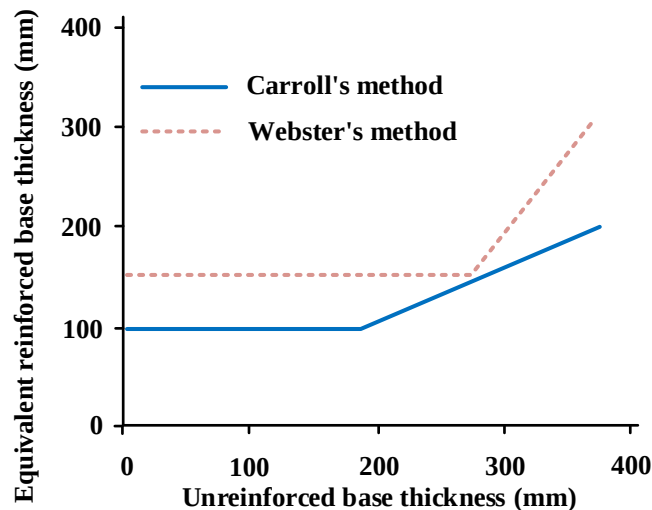


Figure 1.2. Design chart for base thickness proposed by Carroll et al. (61) and Webster (63)

the base course for a thicker base course. Al-Qadi et al. (64) used the cyclic plate loading test to compare the reinforcement effect of geogrids and geotextiles in pavement systems in terms of the surface permanent deformation. The results show that the permanent deformation of the pavement and its developing rate could be significantly lowered by the introduction of geogrids or geotextiles in the pavement. The mechanisms of these two geosynthetic product is different due to their properties. The geotextile can effectively separate the base and subgrade to cut off their mutual migration of granular particles into each layer, and reinforcement of geogrids in pavement is realized by their lateral restrain effect on granular particles.

Montanelli et al. (65) evaluated the contribution of geogrids to the structural bearing capacity of different flexible pavements based on the results from cyclic plate loading test. The geogrid was placed at the base-subgrade interface, and the California bearing ratio (CBR) values of the subgrade are in the range of 1 to 18. The structural number (SN) is an important parameter for the AASHTO pavement design method (66). The researchers proposed a term named layer coefficient ratio (G) as a multiplication factor for calculating the SN. G is the ratio of the layer coefficient of the reinforced base course to that of the unreinforced base course, as shown in Eq. 1.1:

$$SN = a_1 D_1 + a_2 D_2 G \quad (\text{Eq. 1.1})$$

where, a_1 and a_2 are layer coefficients of structural bearing capacity between the pavement structural layers, and D_1 and D_2 are thicknesses of corresponding layers. The thickness of the reinforced granular base is related to the value of G , and the result show that the value of G is between 1.5 and 2, indicating that the designed thickness of the reinforced base course is lower than that of the unreinforced base course and can be determined by Eq. 1.2:

$$D_2 = \frac{SN - a_1 D_1}{D_2 G} \quad (\text{Eq. 1.2})$$

Chen et al. (9) also employed the cyclic plate loading test to evaluate the reinforcement effect of the indoor test sections, and analyzed the mechanical response of reinforced sections including stress and strain under the cyclic loadings. The result show that the bearing capacity, rigidity and rutting resistance of the reinforced sections are significantly improved compared with the control unreinforced sections. The installation of geogrids in the base course can transfer the stress to a wider area within base course and

subgrade, so that the permanent deformation of the base course and subgrade in the reinforced section accounts for a lower proportion of the permanent deformation of the entire pavement structure than that in an unreinforced section.

Unlike the cyclic loading test, the accelerated pavement testing (APT) can more accurately simulate the wheel load applied on the pavement structure. Moghaddas-Nejad and Small (12) built a 1/4-size reinforced and unreinforced pavement in a test pit, and used APT facility to evaluate the reinforcement effects of geogrids on the pavement structure. The test results show that placing geogrids in the base course can significantly reduce the rutting of the pavement structure. The reduction of rutting stems from: 1) the constraint of the geogrid on the aggregate, 2) the plate effect between base course and geogrid. Tang (67) used a small-scaled APT facility, MMLS3, to evaluate the ability of different geogrid products to improve the rutting resistance of the flexible pavement structure. The test results show that the existence of the geogrid can improve the stress distribution in the base course, and decrease the stress applying on the top surface of the subgrade.

Field trafficking test is another excellent method for evaluating benefits of the reinforced aggregate base course. Researchers usually build full-scaled pavements and used APT testing or actual traffic load to test the pavement to quantify the reinforcement effects of geogrids, and investigate the mechanical behavior of the reinforced pavement structure. In order to study the reinforcement effect of geosynthetics in the base course, Chan et al. (22) paved 12 different reinforced thin flexible pavements, and utilized uniaxial and multi-axial APT facility to load the pavements. The stress and strain of geogrids in the base course were recorded to evaluate the reinforcement effect of geosynthetics in different pavement structures. The result show that the reinforcement effect depends largely on the thickness of the base course, the quality of aggregate, and the placement position of the reinforcement. The stiffness of the geogrids used in their study is lower than that of the geotextiles, but the reinforcement effect of the geogrids is still proved better than that of geotextiles in base course. If a pre-rutting could be made on the reinforced granular base just after the base course construction, the rutting resistance of the reinforced structure could be further improved. However, the pre-tensioning of the geosynthetics in the

pavement construction does not significantly improve the long-term performance of the reinforced flexible pavement.

Fannin and Sigurdsson (68) built a test road in the field, and the test road was divided into five sections, one of which was the control unreinforced section, and the other four sections were different geogrid-reinforced sections. In the field test, the whole test road was loaded by a standard axle-loaded truck, and the rut dept of the pavement structure and the strain of the geogrids under different loading passes were obtained for reinforcement comparison. The test result show that the overall rutting resistance of the reinforced pavement is effectively improved, and the reinforcement effect of the geogrids on granular base course mainly depends on the interlocking effect between geogrids and aggregate, and the stiffness of the geogrids as well.

FWD test is also employed to test the performance of the reinforced pavement. Collins et al. (69) conducted a 12-year FWD tracking test on one reinforced road. The test road was divided into one unreinforced section and four reinforced sections, and the reinforced sections considered the influence of different base course thickness. The test result show that when the pavement was open to the public, the modulus of base course of the unreinforced section did not have an obvious increase during its service period. However, the modulus of the thinner base course of the reinforced section increased with the increase in service time, but the modulus of the thicker base course of the reinforced section did not increase obviously. At the same time, some longitudinal cracks were appeared in the unreinforced section, while no obvious cracks and ruts were found in the reinforced section, indicating that the reinforced granular base course also improved the fatigue resistance of the pavement.

Cox et al. (70) conducted on-site cyclic plate loading tests on 16 completed reinforced and unreinforced actual road sections in Arkansas. FWD test were also conducted after cyclic plate loading test. The results show that after 100,000 cyclic loadings, there is no significant difference between reinforced sections and unreinforced sections in terms of the pavement permanent deformation and base resilient modulus which is back-calculated based on the deflections basins. They concluded that when the surface permanent deformation of the pavement is small, the geogrids will not activate the reinforcement.

They further conjectured that the reinforcement effect of the geogrids could be achieved when there is a large permanent deformation to cause a large strain in the geogrids, but this argument has not been confirmed by field measurements.

1.1.3 Summary

Through the summary of the existing research methods, it is found that the interaction between geogrids and aggregate plays a significant role in the base reinforcement. The interaction could affect the mechanical behavior of the reinforced unbound granular materials (UGMs), which in turn affects the corresponding mechanical behaviors of the flexible pavement with the reinforced granular base course. Researchers used different methods to study the mechanical properties and mechanical behavior of the reinforced UGMs and reinforced granular base, and these methods are summarized in Table 1.1.

Among these test methods, the direct shear, pullout, repeated load triaxial test are employed to characterize the mechanical behavior of the reinforced UGMs and analyze the interaction mechanisms between geogrids and aggregate. These test methods are efficient, cost-effective, time-saving, and repeatable. However, the sizes of the equipment used in these studies are small, so the number of geogrid grids contained in the specimens limited, which cannot simulate the actual interaction between geogrids and aggregate. In addition, the loads applied to the specimen are quite different to the loads applied to pavements, so the mechanical response of the specimens in these tests are different from that of actual road. In terms of cyclic plate loading test, field trafficking test and accelerated pavement testing, although these tests can better simulate the actual loading condition for the geogrid-reinforce pavement, these methods are not widely used as routine test approaches since they are time-consuming, labor-consuming, and costly. Also, the facilities are only available in limited universities and research institutes (71-73). Therefore, a more convenient and practical test method should be proposed to evaluate the interaction between geogrids and aggregates, and the test method could be used to quantify the interaction between geogrids and aggregate, select appropriate geogrids for different base course, analyze the reinforcement mechanisms, and evaluate the reinforcement effect of geogrids on pavements.

Table 1.1. Summary of test methods on geogrid-reinforced flexible pavements

Subject	Method	Load type	Test performance	Characterized mechanical behavior
Reinforced UGMs	Direct shear	Monotonic	Interaction degree between geogrids and aggregate	Interfacial shear behavior at aggregate-geogrid interface
	Pullout	Monotonic	Interaction degree between geogrids and aggregate	Interfacial shear behavior at aggregate-geogrid interface
	Triaxial shear	Monotonic	Shear capacity	Shear behavior of reinforced UGMs
	Repeated load triaxial	Cyclic	Resilient modulus, Rut depth	Resilient and permanent deformation behavior of reinforced UGMs
Reinforced granular base	Field trafficking	Traffic	Long-term performance	Permanent deformation behavior of reinforced granular base
	APT	Traffic	Overall structural performance	Permanent deformation behavior of reinforced granular base
	FWD	Dynamic	Resilient modulus	Resilient deformation behavior of reinforced granular base
	Cyclic plate loading	Static/cyclic	Mechanical response	Resilient and permanent deformation behavior of reinforced granular base

In addition, the reinforcement effects of geogrids on the flexible pavement system are closely related to several critical influence factors, and these factors can be mainly divided into three categories, including geogrid properties (geogrid type, aperture size, tensile strength, junction strength etc.), granular material properties (aggregate type, compaction degree, water content, particle size etc.), and the matching properties of the two materials (the relative size of geogrid aperture to aggregate, and the geogrid placement position etc.). The stress transfer mechanisms between geogrids and aggregate depends on these factors, and therefore, to clarify the impacts of these factors on reinforcement effect is a basis for the geogrid-reinforced flexible pavement design. Although numerous studies have confirmed that the use geogrids in base course can improve the performance of flexible pavements. However, the following three problems need to be solved before the widespread application of the geogrid-reinforced flexible pavements.

1) The interfacial behavior and the interaction mechanisms between geogrids and aggregate is not clear. The mechanical behavior of the reinforced granular base course is quite different from that of the traditional granular base course. However, most studies focus on evaluating the reinforcement effect of geogrids on the flexible pavement, but the studies on the interaction between geogrids and aggregate are limited.

2) There is a lack of a test method that can simulate the actual stress state of the reinforced granular base course and effectively evaluate the degree of interaction between geogrids and aggregates. Understanding the aggregate-geogrid interaction is particularly important for the geogrid-reinforced pavement design, but there is no standard method for quantifying the interaction.

3) The studies on the factors influencing the reinforcement effect are insufficient. The reinforcement effect of geogrids in flexible pavements is related to many factors, and the current studies cannot quantitatively relate the properties of UGMs and geogrids to the performance of the geogrid-reinforced flexible pavements, which cannot provide theoretical guidance for the design of the geogrid-reinforced flexible pavements.

1.2 Objective and scope

The overall objectives of the proposed doctoral study are to 1) investigate the interfacial behavior of geogrid reinforced granular base through studying interactions between geogrids and aggregates; 2) quantify the reinforcement effect of geogrids on reinforced aggregate and reinforced aggregate base course; 3) identify the critical influence factors affecting the reinforcement effects of geogrids for exploring what type of geogrids can be combined with what type of base to maximize the reinforcement.

The scope of the research includes laboratory characterization of interfacial behavior between geogrids and aggregates through large-scaled shear, resilient modulus, and laboratory scaled loaded wheel tests. Different influence factors affecting the reinforcement effects will be investigated. To validate the laboratory scaled results, a full-scaled accelerated pavement test will be considered. Figure 1.3 presents a flow chart of this study.

1.3 Structure of dissertation

The dissertation is divided into six chapters. Chapter 1 provides background and literature review for the studies presented herein. Chapter 2 presents a new test method, the cyclic shear test, to quantify the reinforcement effect of geogrids and characterize the aggregate-geogrid interface resilient deformation behavior. Chapter 3 investigates the particle size effects on the shear behavior of the aggregate-geogrid interface by large-scaled direct shear test. Chapter 4 analyzes critical factors affecting the reinforcement of geogrids on aggregate base and describe the permanent deformation behavior of the reinforced granular materials by a small-scaled loaded wheel tester. Chapter 5 presents a full-scaled accelerated pavement testing to validate the laboratory scaled results. The final chapter, chapter 6, presents an outline of the conclusions as well as recommendations for future research.

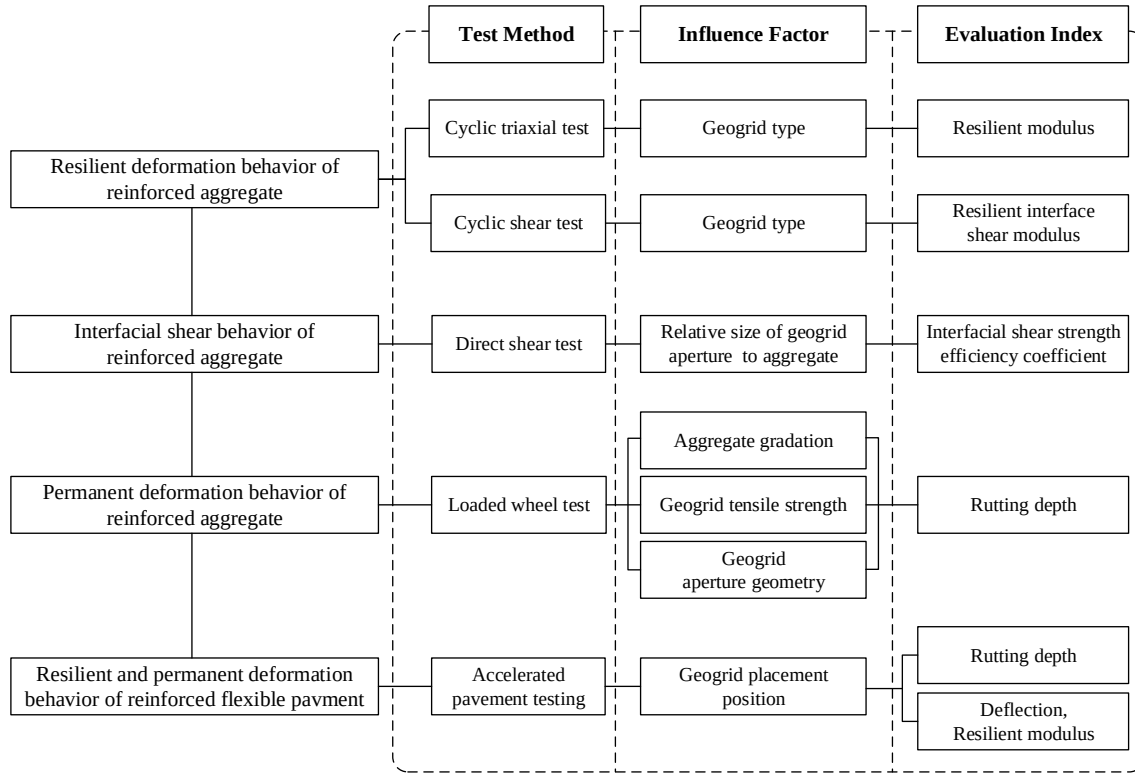


Figure 1.3. Flow chart of the proposed research

CHAPTER TWO
QUANTIFYING THE REINFORCEMENT EFFECTS OF GEOGRIDS
IN UNBOUND GRANULAR BASE MATERIALS

The contents of this chapter were originally published in the two following journals.

Han, B., Ling, J., Shu, X., Gong, H., Sun, Y., Polaczyk, A. P. & Huang, B. (2018). "Resilient interface shear modulus for characterizing shear properties of pavement base materials." *Journal of Materials in Civil Engineering*, 30(12): 04018333.

Han, B., Ling, J., Shu, X., Boudreau, R. L., Song, W., Hu, W. & Huang, B. (2019). "Quantifying the effects of geogrid reinforcement in unbound granular base." *Geotextiles and Geomembranes*, 47(3): 469-376.

Bingye Han's contribution was conducting all literature review, data analysis, and writing the text contained in the manuscript. Dr. Baoshan Huang provided guidance throughout the research process as well as editorial assistance.

2.1 Abstract

This chapter presents an effort to quantify the effects of geogrid reinforcement in unbound granular base through laboratory testing. Two laboratory tests, the large-scale cyclic shear test and the repeated load triaxial (RLT) test, were employed. The test protocol of the cyclic shear test was developed by modifying that for the RLT test. The cyclic shear test was performed by applying a series of cyclic shear stresses to the geogrid-aggregate interface under different normal stresses. Two different types of geogrids were used as reinforcement in an unbound granular material. Resilient modulus (M_R) from the repeated load triaxial test and a term called resilient interface shear modulus (G_i) from the cyclic shear test were used to characterize the effects of geogrid reinforcement in unbound granular base, respectively. The results of the RLT test showed that the inclusion of geogrid had a negligible effect on the resilient modulus, indicating that the RLT test may not be effective in evaluating the geogrid reinforcement in unbound granular materials. Compared to the RLT test, the cyclic shear test showed a great potential in identifying the effects of geogrid reinforcement, with an obvious improvement in the degree of interlocking between geogrids and aggregates.

2.2 Introduction

A number of field and experimental studies have demonstrated the benefits of using geogrids in base reinforcement, including improved pavement performance, extended pavement service life, and lowered construction and maintenance costs (29; 31; 62; 64; 74; 75). However, geogrid reinforcement is usually not incorporated in the design method for flexible pavements, including the latest mechanistic-empirical pavement design guide (MEPDG). Most of the design methods for geogrid-reinforced pavement are based on limited test results, which limits the applications of geogrid reinforcement in pavement base. Therefore, characterization of geogrid-reinforced unbound granular materials (UGMs) and quantification of geogrid reinforcement is a key step for incorporating geogrid reinforcement into MEPDG (76).

The interfacial interaction between geogrids and aggregates plays a significant role in base reinforcement. Many researchers have employed different methods to test and quantify the reinforcement effects, including direct shear test (77-79), pull-out test (80-84), cyclic loading plate test (28; 85-87), and repeated load triaxial test (RLT) (46; 88-90). Among these tests, the RLT test is a widely used method for characterizing the resilient and permanent stress-strain behavior of geogrid-reinforced UGMs.

Geogrids could create an additional confining stress to aggregate particles to restrict their lateral movement under repeated loads to improve the overall strength and stability of UGM, and this, in turn, may increase its resilient modulus and reduce the permanent deformation of the reinforced system under repeated loads. Rahman et al. found that the inclusion of geogrids in construction and demolition (C&D) materials can improve the resilient modulus and decrease the permanent deformation of the geogrid-reinforced C&D materials (41). Gu et al. used the RLT tests to evaluate the effects of geogrid reinforcement in UGM, and their test results indicated that geogrids increase both the vertical and horizontal resilient moduli and reduce the permanent deformation of the reinforced UGMs (42). Siekmeier and Casanova demonstrated through the Particle Flow Code 3D (PFC3D) simulations that the resilient modulus of geogrid-reinforced aggregate is increased compared to unreinforced aggregate, and this effect can be expressed using a term called geogrid gain factor (91). However, for the several researchers who have conducted RLT

tests, although they concluded that the inclusion of geogrids has a considerable effect in reducing the permanent deformation, geogrids have a negligible effect in increasing the resilient modulus of reinforced UGMs (43-45). Yang and Han developed an analytical model for the reinforced UGMs and reported that the increase in the resilient modulus of granular materials is not substantial compared to the reduction in the permanent deformation (40). Therefore, the RTL test could be considered effective in evaluating the geogrid effect in terms of permanent deformation, but it may not be effective in characterizing the reinforcement in terms of resilient modulus because consistent resilient modulus results could not be obtained from different researchers.

Therefore, a more effective test method for quantifying geogrid reinforcement with a more reasonable parameter is more desirable to characterize the stress-strain behavior of geogrid-reinforced base materials. A stiffness parameter called resilient interface shear modulus was developed by Cuelho and Perkins to quantify the interlocking between geogrid and aggregate at the geogrid-aggregate interface (92). The resilient interface shear modulus G_i is defined as the cyclic shear stress applied at interface divided by the relative displacement between aggregate and geogrids. Tang employed a direct shear test to test G_i and used it in his material model (67). However, the monotonic shear loads applied at the geogrid-aggregate interface in the direct shear test cannot simulate actual loads and the relatively large shear displacements were not representative of those in actual pavements. A laboratory test method is needed to better simulate the actual loading and to characterize the effects of geogrid reinforcement under small shear displacements.

2.3 Objective and scope

The objective of this chapter is to utilize a large-scale cyclic shear test to quantify the effects of geogrid reinforcement in unbound granular base materials, and investigate the cyclic shear properties of the geogrid-aggregate interface. In this study, the cyclic shear test protocol was derived from the RLT test to apply cyclic shear loading under different normal stresses. Different interfaces including unreinforced aggregate-aggregate, biaxial geogrid-reinforced aggregate, and triaxial geogrid-reinforced aggregate interfaces were

tested to validate the repeatability and feasibility of the test method. A traditional RLT test was also conducted in this chapter for comparison purpose.

2.4 Methodology

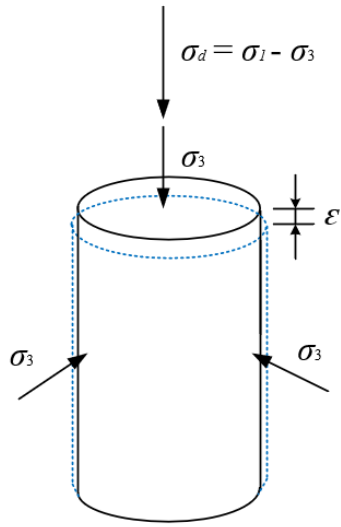
The stiffness of unbound granular materials is usually characterized by their resilient modulus, M_R , and it is an important input parameter for the mechanistic-empirical pavement design. M_R is defined as the repeated deviator stress σ_d divided by the axial recoverable strain ε as shown in Figure 2.1 (a). The repeated load triaxial test is usually conducted to determine M_R by applying a group of loading combinations at a range of confining stress σ_3 and cyclic axial stress σ_1 , and measuring the recoverable deformation caused by these loading combinations. The resilient modulus is a function of axial deviator stress and confining stress. A constitutive model can be regressed based on the measured data and then used to predict M_R at any levels of stresses. Two commonly used constitutive models are given as follows:

$$\text{SHRP model: } M_R = k_1 (\sigma_c)^{k_2} (\sigma_3)^{k_3} \quad (\text{Eq. 2.1})$$

$$\text{MEPDG model: } M_R = k_1 \sigma_{atm} \left(\frac{\theta}{\sigma_{atm}} \right)^{k_2} \left(\frac{\tau_{oct}}{\sigma_{atm}} \right)^{k_3} \quad (\text{Eq. 2.2})$$

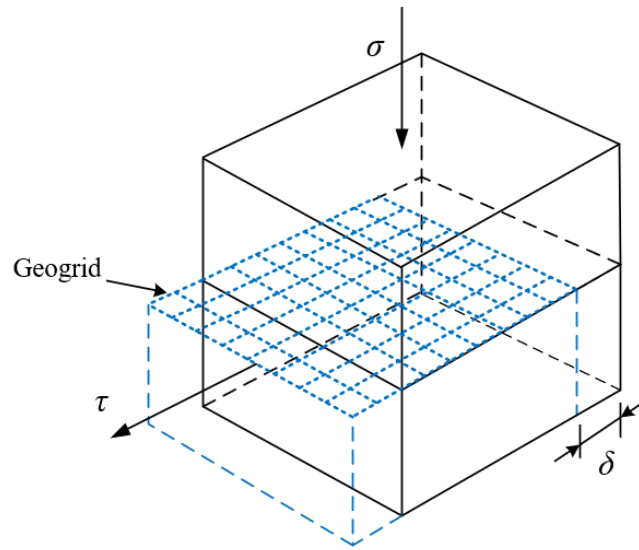
where k_1 , k_2 , k_3 are constants of constitutive models; σ_c is cyclic axial stress; σ_3 is confining stress, θ is bulk stress and estimated as $\sigma_c + 2\sigma_3$; σ_{atm} is atmospheric pressure; τ_{oct} is octahedral shear stress and estimated as $\sqrt{2}/3 \times \sigma_d$; and σ_d is deviator stress and estimated as $\sigma_1 - \sigma_3$.

As mentioned earlier, the resilient interface shear modulus G_i , has been proposed to characterize the interlocking between geogrids and aggregate at the geogrid-aggregate interface. G_i is defined as a cyclic shear stress τ applied to the geogrid-aggregate interface divided by the relative displacement δ between geogrid and aggregate, as shown in Figure 2.1 (b). The dashed blue grids represent geogrids. The resilient interface shear modulus is determined through a cyclic shear test and the loading scenarios consist of 15 different loading sequences at a variety of normal stresses and cyclic shear stresses. The value of G_i was averaged from the last ten cycles for each loading sequence. Like the resilient modulus,



$$M_R = \sigma_d / \varepsilon$$

(a) repeated load triaxial test



$$G_i = \tau / \delta$$

(b) cyclic shear test

Figure 2.1. Test layout

G_i is also a function of normal stress and cyclic shear stress. The SHRP and Universal models can be modified and used to predict G_i as follows:

$$G_i = k_1 (\tau)^{k_2} (\sigma)^{k_3} \quad (\text{Eq. 2.3})$$

$$G_i = k_1 P_a \left(\frac{\sigma}{\sigma_{atm}} \right)^{k_2} \left(\frac{\tau}{\sigma_{atm}} + 1 \right)^{k_3} \quad (\text{Eq. 2.4})$$

where k_1 , k_2 , k_3 are constants of revised models; σ_{atm} is atmospheric pressure; P_a is the atmospheric pressure divided by a unit length (101.3 kPa/m); cyclic axial stress σ_c and confining stress σ_3 in Eq. 2.1 is replaced by cyclic shear stress τ and normal stress σ in Eq. 2.3, respectively; Similarly, bulk stress θ and octahedral stress τ_{oct} is also replaced by cyclic shear stress τ and normal stress σ in Eq. 2.4, respectively.

2.5 Materials and experiment

2.5.1 Materials

2.5.1.1 Base course material

Two types of base-course materials, labeled A and B, were tested in this study, and the particle-size distribution of the two base materials are shown in Figure 2.2. Material A is single-sized crushed limestone aggregate (19 – 25 mm), and its specific gravity is 2.791. The aggregate particles in this size could interact effectively with the geogrid used in this study based on the selection criteria (77). Material B is one commonly used aggregate base material in Tennessee, which satisfies the Gradation D of the Tennessee Department of Transportation (TDOT) standard specification. The material B is classified as A-1-a and GW according to AASHTO M 145-08 (93) and ASTM D 2487-11 (94), respectively. The material has a maximum dry density of 2150 kg/m³ and an optimum moisture content of 7.3%, as determined by the modified Proctor test. Both materials were used in the cyclic shear test. Material A, the single-sized aggregate, was used to check the repeatability of cyclic shear test method, whereas Material B, the typical Tennessee base-course material, was used to test the feasibility of the method. However, only material B was used in the repeated load triaxial test.

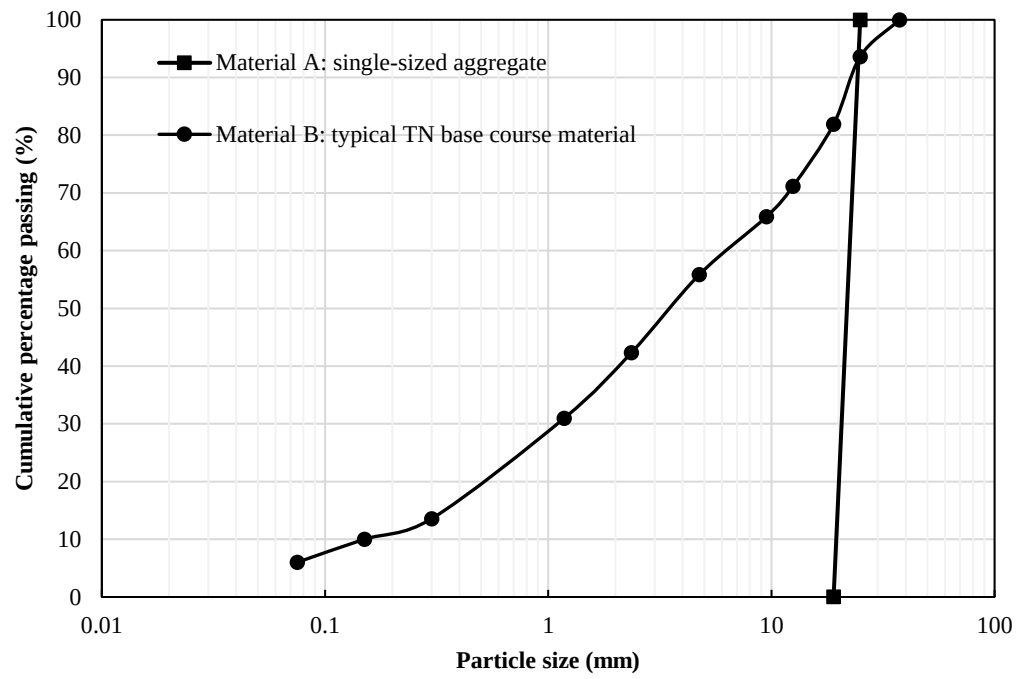


Figure 2.2. Particle-size distribution curve of base materials

2.5.1.2 Geogrids

Two types of geogrids, labelled as BX and TX, with different shapes were tested in this study (Figure 2.3). TX is a type of triaxial geogrid with triangular apertures, and BX is a type of biaxial geogrid with rectangular apertures. Their physical and mechanical properties are summarized in Table 2.1.

2.5.2 Test methods

2.5.2.1 Repeated load triaxial test

The resilient modulus was determined by the repeated load triaxial (RLT) test to simulate traffic loads on both unreinforced aggregate and reinforced aggregate in accordance with AASHTO T307 (95). The RLT tests were conducted by using Instron 8502 servo-hydraulic dynamic testing system. The capacity of the load cell used was 25 kN, and the axial displacement was measured as an average value from two linearly variable differential transducers (LVDT) placed between the top platen and the base of the load cell. In this test, the unreinforced/reinforced specimen was subjected to a total of 15 loading sequences consisting of different static confining stresses and cyclic deviatoric stresses, as shown in Table 2.2. The cyclic axial load consisted of a 0.1s haversine-shaped load pulse and a 0.9s

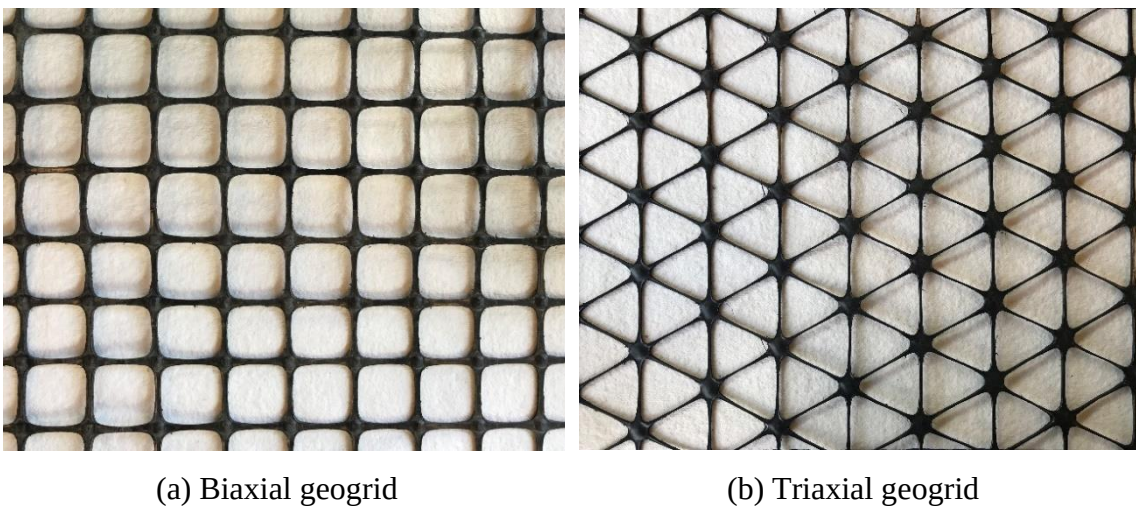


Figure 2.3. Geogrids used for testing

Table 2.1. Properties of geogrids used in tests

Property	Geogrids	
	BX	TX
Aperture size (mm)	32 × 31	46 × 46 × 46
Tensile strength at 5% (kN/m)	30	-
Radial stiffness at 0.5% (kN/m)	-	365
Junction efficiency (%)	90	95

resting load pulse in the loading program to better simulate the traffic loads. The dimension of the sample was selected based on the particle size of the base course material. The base course material B, the typical Tennessee base-course material was compacted in six layers at a water content of 4.5% to achieve a uniform degree of compaction at around 92%. Figure 2.4 shows the preparation of specimens and the test setup. A bronze stone plate was placed at the bottom and top of the specimen. Two layers of rubber membrane were used to confine the sample and two O-rings were installed to the bottom and top loading cap. The confining stress was achieved by a pneumatic pressure. Figure 2.5 schematically shows unreinforced, biaxial geogrid reinforced and triaxial reinforced specimens tested in this study. Both the biaxial geogrid and triaxial geogrids were cut into a specific pattern to fit across the 150-mm diameter of the cross section of specimen, and then were placed at the mid-height of the specimen. As shown in Figure 2.5, the number of closed apertures was only 8 and 13 for the biaxial and triaxial geogrids, respectively.

2.5.2.2 Cyclic shear test

In the resilient modulus test, a piece of biaxial or triaxial geogrids cut to fit the 150-mm diameter may only contain limited intact apertures, which makes it difficult to evaluate the geogrid reinforcement. Therefore, a large-scale shear box was employed to accommodate more grids in the test. The shear box consisted of a moveable bottom box with a dimension of 65 cm × 50 cm × 18 cm and a stationary top box with a dimension of 50 cm × 50 cm × 18 cm as shown in Figure 2.6. The confining loads and cyclic loads were

Table 2.2. Testing sequences for base materials (95)

Sequence No.	Confining stress, σ_3 (kPa)	Max. Axial Stress, σ_{max} (kPa)	Cyclic Stress, σ_{cyclic} (kPa)	Constant Stress, $0.1\sigma_{max}$ (kPa)	No. of Load Applications
0-Conditioning	103.4	103.4	93.1	10.3	1000
1	20.7	20.7	18.6	2.1	100
2	20.7	41.4	37.3	4.1	100
3	20.7	62.1	55.9	6.2	100
4	34.5	34.5	31.0	3.5	100
5	34.5	68.9	62.0	6.9	100
6	34.5	103.4	93.1	10.3	100
7	68.9	68.9	62.0	6.9	100
8	68.9	137.9	124.1	13.8	100
9	68.9	206.8	186.1	20.7	100
10	103.4	68.9	62.0	6.9	100
11	103.4	103.4	93.1	10.3	100
12	103.4	206.8	186.1	20.7	100
13	137.9	103.4	93.1	10.3	100
14	137.9	137.9	124.1	13.8	100
15	137.9	275.8	248.2	27.6	100



(a) Specimen in mold

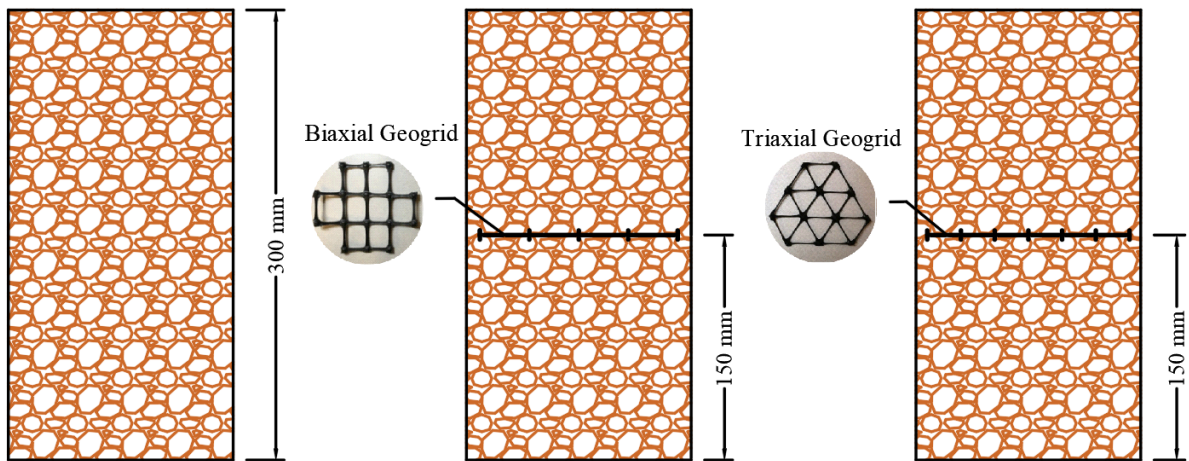


(b) Prepared Specimen



(c) RLT test

Figure 2.4. Preparation and testing of specimen in RLT test



(a) Unreinforced case

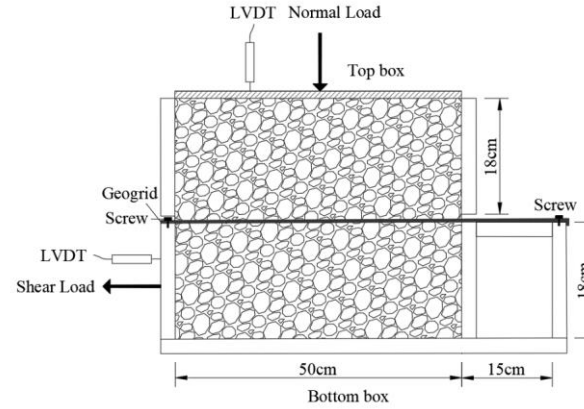
(b) BX reinforced case

(c) TX reinforced case

Figure 2.5. Unreinforced and reinforced specimens



(a) Photo of the shear box



(b) Schematic of the shear box

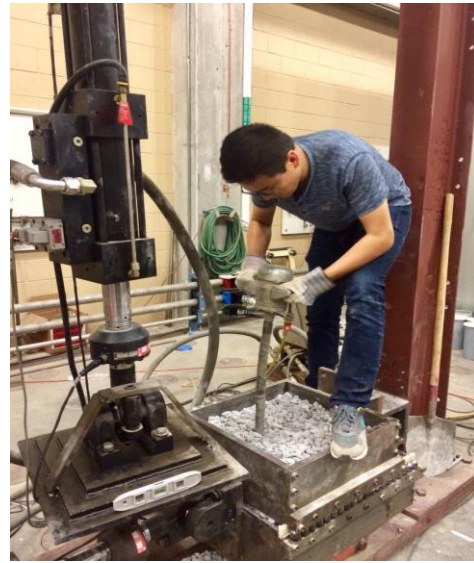
Figure 2.6. Overview of the large-scaled shear box

applied through two same servo-hydraulic material testing system (MTS) actuators. The capacity of the load cells used in horizontal and vertical directions were both 100 kN, and the shear displacement could be measured through its built-in LVDT with an accuracy of 0.001mm. An MTS controller recorded the maximum and minimum shear load and displacement through LVDTs in each cycle, and these data were used to calculate and analyze the interaction properties of the geogrid-aggregate interface. The ambient temperature and humidity were kept constant to reduce the effect of environment on the experiment.

Both two base course materials were compacted in six lifts, as shown in Figure 2.7 (a), but the compaction method for Materials A and B was different. Material A, the single-sized aggregate, was air-dried and densified by a concrete vibrator, and each layer was vibrated 40 times to ensure that the percentage of voids between aggregate particles could be kept at 42 – 45 % [Figure 2.7 (b)]. Material B was compacted by a 4.9-kg hammer falling freely from a 45-cm height at the optimum moisture. The materials with a specific mass was filled and compacted until the thickness of each layer reached the target line of the box to keep the compaction degree of aggregate near 92% [Figure 2.7 (c)]. After compacting the bottom box, a large piece of geogrids with a dimension of 71 cm × 48.5 cm was installed at the top surface of the bottom box. The geogrid was screwed to the bottom



(a) Target lines in the shear box



(b) Material A compaction



(c) Material B compaction



(d) Placement of triaxial geogrid

Figure 2.7. Base course materials set-up

box at the front and rear sides to ensure that the geogrid could move with the bottom box and be mobilized during testing [Figure 2.7 (d)].

The test protocol for the cyclic shear test was adapted from AASHTO T307 test (95). The vertical normal stress was considered as confining stress, and the cyclic shear loads were directly applied to the geogrid-aggregate interface. Like the protocol of the resilient modulus test, different cyclic stresses were applied at the geogrid-aggregate interface at different normal stresses, as shown in Table 2.3. The symbol TX-A and TX-B represents the interface of base material A and B reinforced with the same triaxial geogrid, respectively. In this test protocol, a total of 31 loading sequences were conducted in a complete test process, including one conditioning sequence and 30 routine loading sequences. The conditioning sequence was aimed to reduce the inherent anomalies of the compacted aggregate, and the remaining 30 loading sequences were equally divided into six groups by the ratio (τ_t/τ_f) of a total shear stress (τ_t) to its failure shear stress (τ_f). The failure shear stress was determined from a failure line which were estimated based on results from a series of direct shear tests, and then the total shear stress was determined based on the percentage of its corresponding failure shear stress at a given normal stress. For material B, the typical Tennessee base course material, the ratios were selected as 13%, 20%, 33%, 47%, 75% and 100% for group 1 to group 6, respectively. These breakage values could make the shear stresses represent more typical load levels in the actual situation. The seating stress for each group was selected as 3% of the failure shear stress. The cyclic shear stress was applied between the seating stress and total shear stress as shown in Table 2.3. Five levels of normal stress were applied in each group, and these levels of normal stress were to represent the actual normal stresses on the potential shear failure interface of the unbound granular base course materials. For material A, the single-sized aggregate, both the levels of normal stress and ratios ($\tau_t/\tau_f = 20\%, 36\%, 53\%, 70\%, 86\%, 100\%$) were higher than material B because of its higher failure envelope lines as shown in Figure 2.10, but the principle of the cyclic shear test is the same to material B. The cyclic load consisted of a 0.2s haversine-shaped loading and unloading pulse and a 0.8s rest period in according with ASTM D7499/D7499M-09 (96). The normal stress was kept constant in each load sequence. For the first two relatively low normal stresses in each

Table 2.3. Test sequence for cyclic shear test

Group	Sq.	Normal Stress (kPa)			Seating Stress (kPa)			Cyclic Shear Stress (kPa)			Total Shear Stress (kPa)			No. of Reps
		TX-A	TX-B	BX-B	TX-A	TX-B	BX-B	TX-A	TX-B	BX-B	TX-A	TX-B	BX-B	
0	Cond.	100.0	77.6	77.6	4.9	3.7	3.4	76.8	37.4	33.8	81.7	40.8	37.2	1000
1	1	50.0	31.0	31.0	3.6	2.1	2.0	16.4	7.1	7.0	20.0	9.3	9.0	300
	2	75.0	51.7	51.7	4.2	2.8	2.5	19.4	9.4	8.9	23.6	12.2	11.4	300
	3	100.0	77.6	77.6	4.9	3.7	3.1	22.4	12.2	11.2	27.3	15.9	14.3	100
	4	125.0	103.4	103.4	5.6	4.5	4.1	25.4	15.0	13.5	31.0	19.6	17.7	100
	5	150.0	137.9	137.9	6.2	5.6	4.6	28.4	18.8	16.6	34.6	24.5	21.3	100
2	6	50.0	31.0	31.0	3.6	2.1	2.0	36.2	11.9	11.8	39.8	14.1	13.7	300
	7	75.0	51.7	51.7	4.2	2.8	2.5	42.9	15.7	14.9	47.1	18.5	17.3	300
	8	100.0	77.6	77.6	4.9	3.7	3.1	49.5	20.4	18.7	54.4	24.1	21.9	100
	9	125.0	103.4	103.4	5.6	4.5	4.1	56.2	25.1	22.6	61.8	29.6	26.7	100
	10	150.0	137.9	137.9	6.2	5.6	4.6	62.9	31.4	27.7	69.1	37.1	32.4	100
3	11	50.0	31.0	31.0	3.6	2.1	2.0	56.2	21.7	21.4	59.7	23.9	23.4	300
	12	75.0	51.7	51.7	4.2	2.8	2.5	66.5	28.6	27.0	70.7	31.4	29.5	300
	13	100.0	77.6	77.6	4.9	3.7	3.1	76.8	37.2	34.1	81.7	40.8	37.2	100
	14	125.0	103.4	103.4	5.6	4.5	4.1	87.2	45.7	41.1	92.7	50.3	45.2	100
	15	150.0	137.9	137.9	6.2	5.6	4.6	97.5	57.2	50.5	103.7	62.8	55.1	100

Table 2.3. Continued

Group	Sq.	Normal Stress (kPa)			Seating Stress (kPa)			Cyclic Shear Stress (kPa)			Total Shear Stress (kPa)			No. of Reps
		TX-A	TX-B	BX-B	TX-A	TX-B	BX-B	TX-A	TX-B	BX-B	TX-A	TX-B	BX-B	
4	16	50.0	31.0	31.0	3.6	2.1	2.0	76.1	31.7	31.3	79.7	33.9	33.3	300
	17	75.0	51.7	51.7	4.2	2.8	2.5	90.1	41.7	39.5	94.4	44.6	42.0	300
	18	100.0	77.6	77.6	4.9	3.7	3.1	104.1	54.3	49.8	109.0	57.9	52.9	100
	19	125.0	103.4	103.4	5.6	4.5	4.1	118.1	66.8	60.0	123.7	71.3	64.2	100
	20	150.0	137.9	137.9	6.2	5.6	4.6	132.1	83.5	73.7	138.4	89.2	78.4	100
5	21	50.0	31.0	31.0	3.6	2.1	2.0	95.9	51.6	50.9	99.5	53.7	52.9	300
	22	75.0	51.7	51.7	4.2	2.8	2.5	113.6	67.9	64.2	117.9	70.7	66.7	300
	23	100.0	77.6	77.6	4.9	3.7	3.1	131.3	88.3	80.9	136.2	91.9	84.1	100
	24	125.0	103.4	103.4	5.6	4.5	4.1	148.9	108.6	97.6	154.5	113.2	101.8	100
	25	150.0	137.9	137.9	6.2	5.6	4.6	166.6	135.8	119.9	172.8	141.5	124.5	100
6	26	50.0	31.0	31.0	3.6	2.1	2.0	115.9	71.4	70.5	119.5	73.6	72.4	300
	27	75.0	51.7	51.7	4.2	2.8	2.5	137.2	94.0	89.0	141.5	96.8	91.5	300
	28	100.0	77.6	77.6	4.9	3.7	3.1	158.6	122.3	112.1	163.5	125.9	115.2	100
	29	125.0	103.4	103.4	5.6	4.5	4.1	179.9	150.5	135.2	185.5	155.0	139.3	100
	30	150.0	137.9	137.9	6.2	5.6	4.6	201.2	188.1	166.0	207.5	193.8	170.7	100

NOTE: Sq. = Sequence; Cond. = Conditioning; No. = Number

group, 300 shear cycles were applied to the samples to ensure a stable resilient behavior. For the other relatively higher normal stresses, 100 shear cycles were conducted.

2.6 Results and analysis

2.6.1 Repeated load triaxial test

The resilient modulus test results for unreinforced aggregate (NG), biaxial geogrid reinforced aggregate (BX), and triaxial geogrid reinforced aggregate (TX) are shown in Figure 2.8. It was found that M_R was dependent on both confining stress and axial stress. The value of M_R for the three interfaces increased with an increase of confining stresses and axial stresses. Although the M_R value of reinforced aggregate was a slightly higher than that of the unreinforced aggregate, the test results did not demonstrate that the incorporation of geogrids had a significant effect in increasing the resilient modulus of UGM. Also, the reinforcement effect could not be differentiated between different types of geogrids through the resilient modulus test. The results from this study were consistent with those from other researchers' studies (43; 45), which have reported that inclusion of geogrid does not show a significant effect on the resilient modulus of UGM. This may be attributed to the fact that the resilient modulus test does not generate axial or lateral strains large enough to mobilize the geogrid and hence geogrid reinforcement at the mid-height interface.

Two different constitutive models, the two-parameter model (SHRP) and the three-parameter model (MEPDG) were used to fit the M_R results because both models could reflect the effects of confining stresses and deviator stresses well (97). Table 2.4 presents the statistical regressed constants of these two models based on the test data. It can be found that both models were able to predict the resilient modulus of reinforced and unreinforced unbound granular materials with a high coefficient of determination (R^2). Figure 2.9 compares the predicted resilient moduli of NG, TX and BX. The values of resilient modulus were calculated from the MEPDG model, and with the parameters listed in Table 2.4. Figure 2.9 shows that both the biaxial and triaxial geogrids had a negligible effect on the resilient modulus at different stress levels throughout all the testing.

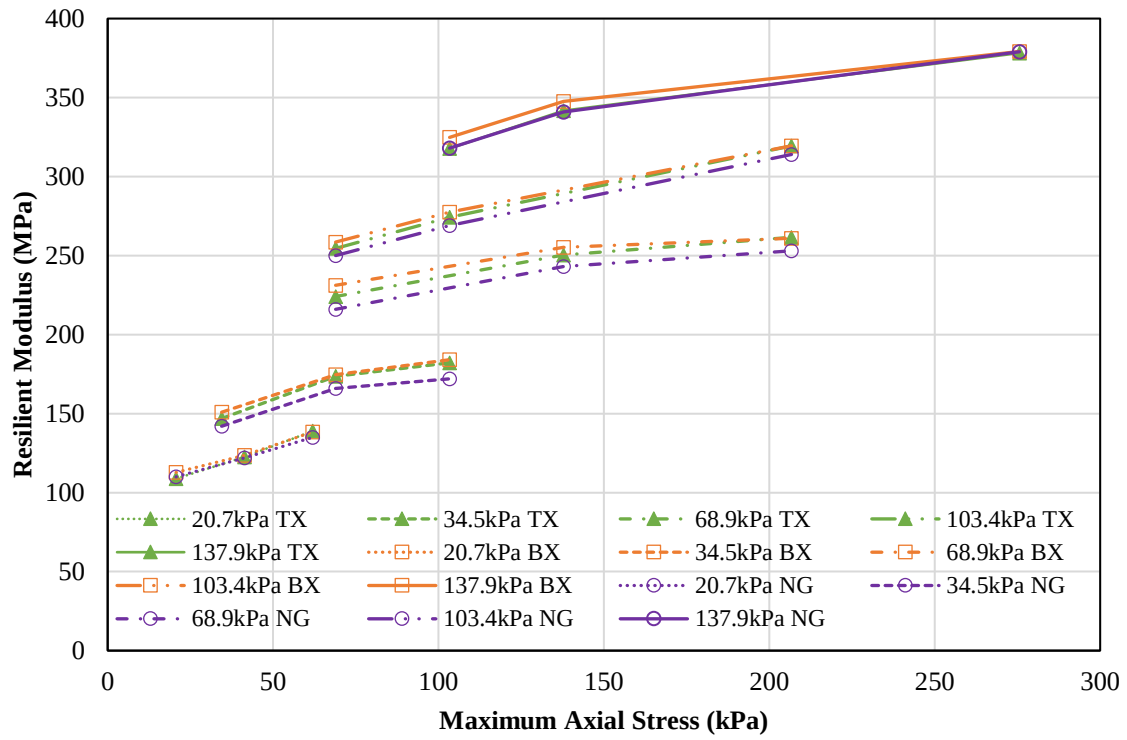


Figure 2.8. Resilient moduli of NG, BX and TX

Table 2.4. Summary of model parameters for RLT tests

Model	Specimen	k_1	k_2	k_3	R^2
SHRP	NG	19,099	0.170	0.409	0.996
	BX	19,907	0.169	0.410	0.997
	TX	19,815	0.182	0.393	0.997
MEPDG	NG	1,416	0.593	-0.136	0.991
	BX	1,479	0.595	-0.149	0.994
	TX	1,452	0.586	-0.113	0.995

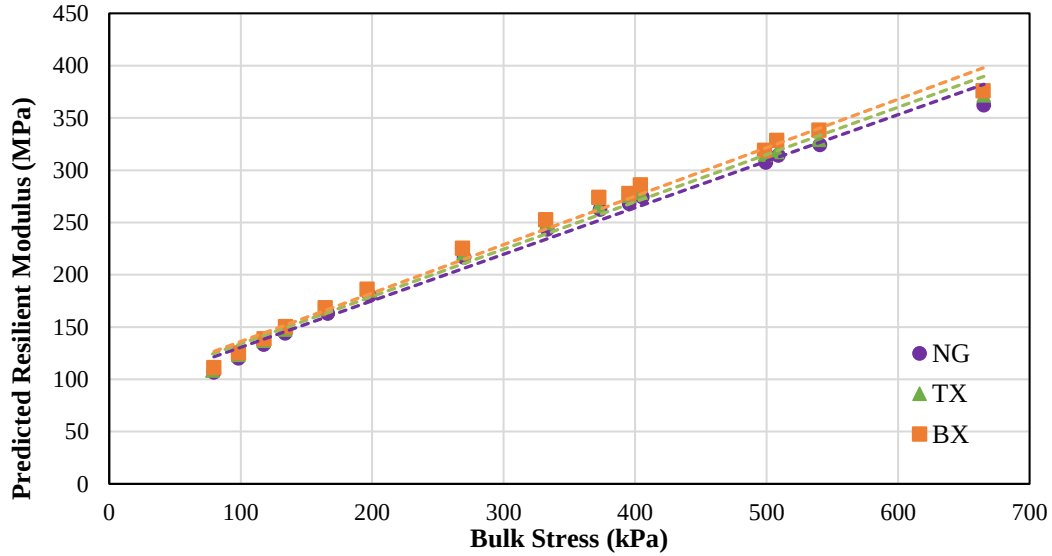
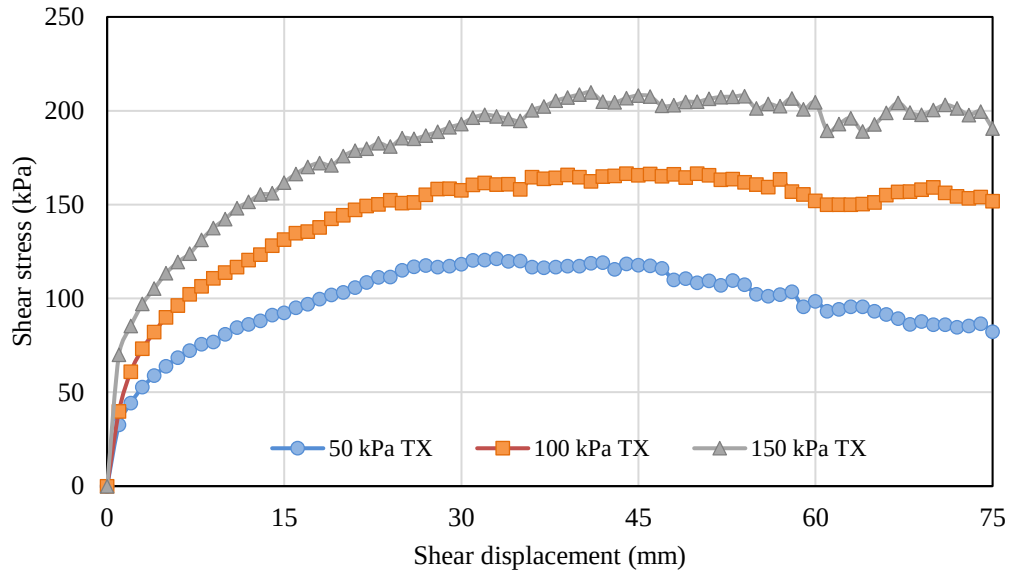


Figure 2.9. Bulk stress versus M_R plots for unreinforced/reinforced aggregate

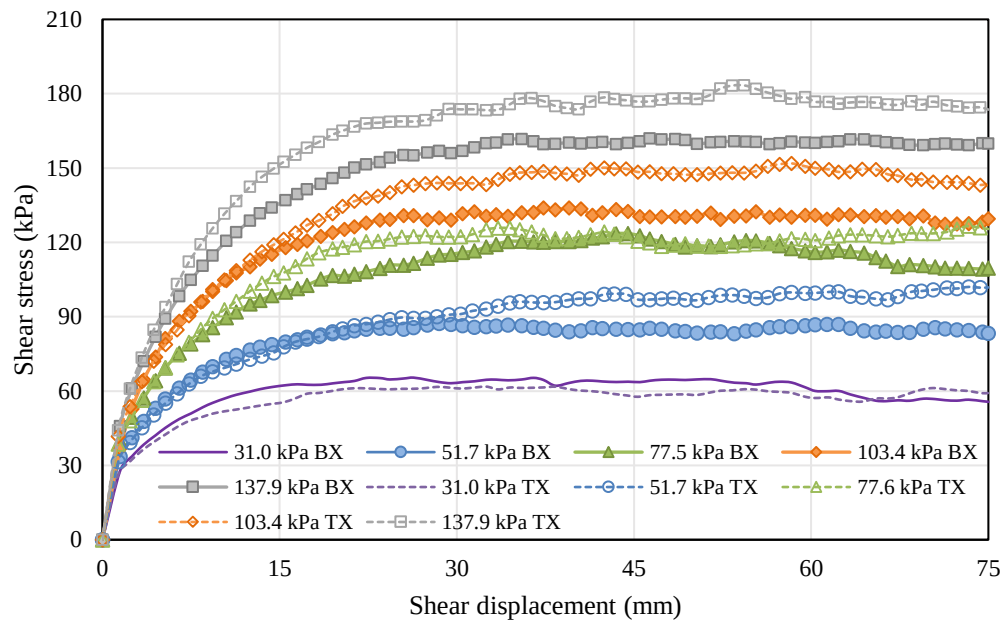
2.6.2 Cyclic shear test

2.6.2.1 Shear stress versus shear displacement curves and peak shear-strength envelopes

The direct shear test of the geogrid-aggregate interface is the basis of the cyclic shear test. The levels of cyclic shear stress were determined based on the results of the direct shear test and selected as specific percentages of the failure stress according to the shear strength envelope at given normal stresses. The direct shear tests were also conducted at different normal stresses with a constant shear rate of 1 mm/min in according with ASTM D5321/D5321M-17 (98). Each test was stopped when the shear displacement reached 75 mm that corresponds to a horizontal strain of 15%. Figure 2.10 shows the shear stress versus shear displacement curves of geogrid-aggregate interfaces at different normal stresses. Figure 2.10 (a) shows the curve of triaxial geogrid reinforced material A, and Figure 2.10 (b) shows that of biaxial/triaxial geogrid reinforced material B. It was found that the shear stress gradually rose to a peak shear stress and then remained nearly constant until the end of the tests. At different normal stress levels, it is obvious that at the same shear displacement, the shear stress increased with the increase of the normal stress. The



(a) Biaxial-geogrid-reinforced material A



(b) Biaxial/triaxial geogrid-reinforced material B

Figure 2.10. Shear stress versus shear displacement curves

Mohr-Coulomb failure envelope lines of geogrid-aggregate interfaces were determined from the direct shear results at different normal stresses. Figure 2.11 shows a good linear relationship between the normal stress and shear stress. Three trendlines were linearly regressed based on the test results for triaxial geogrid reinforced material A, and biaxial/triaxial geogrid reinforced material B with a relatively high coefficient of determination R^2 . As mentioned above, the resulting trendlines could be used to determine the levels of cyclic shear stress in cyclic shear test protocols, resulting in three sets of parameters of protocols for TX-A, TX-B, and BX-B.

2.6.2.2 Resilient interface shear modulus

The shear resistance behaviour of the geogrid-aggregate interface could be expressed in terms of the resilient interface shear modulus, G_i , as follows:

$$G_i = \frac{\tau_{\max} - \tau_{\min}}{\delta_{\max} - \delta_{\min}} \times 10^{-3} \quad (\text{Eq. 2.5})$$

where:

G_i is the resilient interface shear modulus, MPa/m

$\tau_{\max}$, $\tau_{\min}$ is the maximum and minimum shear stress for a given load cycle, kPa

$\delta_{\max}$, $\delta_{\min}$ is the maximum and minimum relative displacement between geogrid and aggregate for a given load cycle, m

As shown in Figure 2.12, the cyclic shear stress was applied between the total shear stress (maximum shear stress) and the seating shear stress (minimum shear stress), along with a maximum and minimum relative displacement, respectively. The resilient interface shear modulus is an average value from the last ten cycles for each loading sequence. The resilient interface shear modulus for each cycle was calculated through the Eq. 2.5, using the $\tau_{\max}$, $\tau_{\min}$, $\delta_{\max}$ and $\delta_{\min}$ recorded from the last ten load cycles for each loading sequence. Figure 2.12 illustrates the determination of resilient interface shear modulus. The area at the interface subjected to shearing decreased with the increase in shear displacement and the reduction in shear area was accounted for by deducting the shear displacement from the original box length. The difference between the original box length and reduced box length was the relative displacement between top box and bottom box.

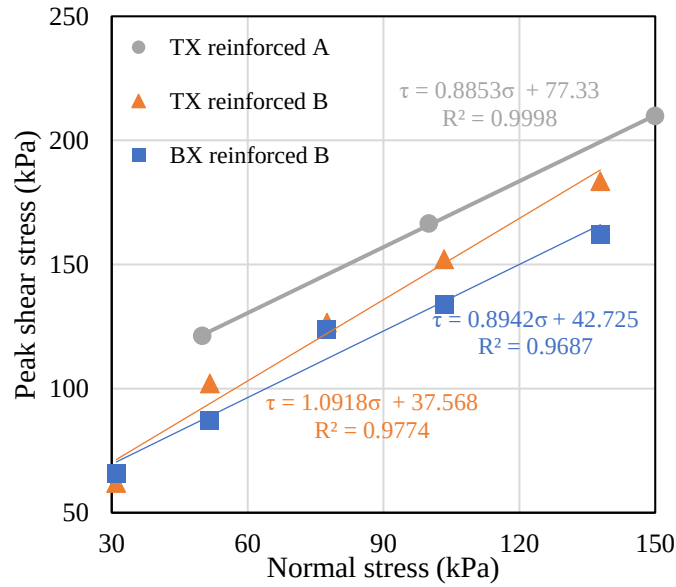


Figure 2.11. Peak shear strength envelopes of geogrid-reinforced aggregate

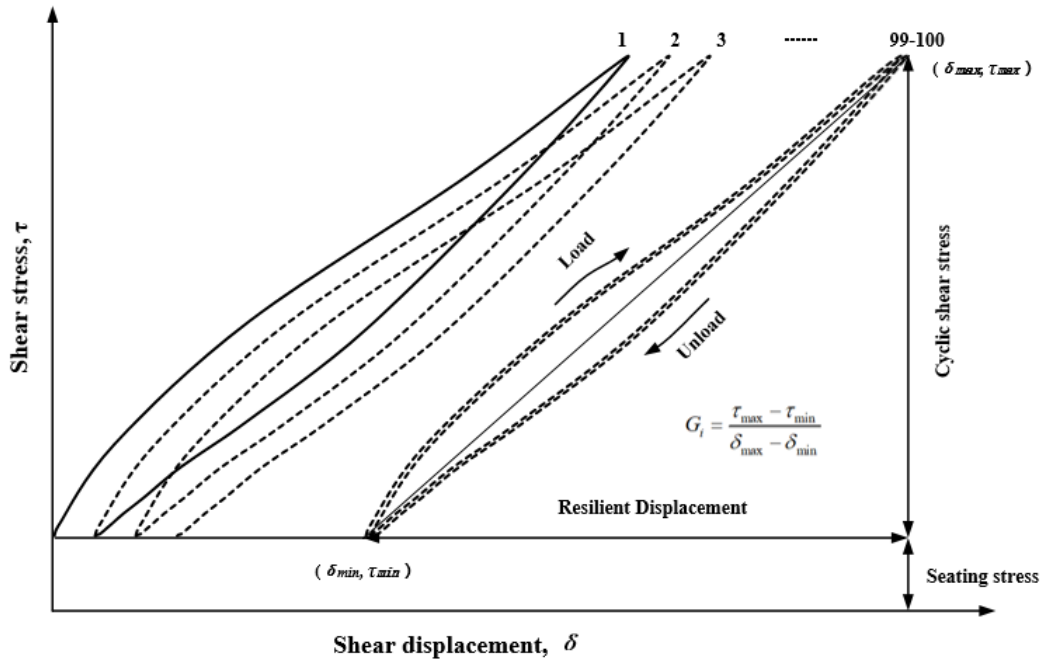


Figure 2.12. Illustration of interface shear modulus calculation

Figure 2.13 shows the shear stress versus shear displacement curves of a biaxial geogrid reinforced aggregate of material B from last ten cycles for each loading sequence from 1 to 30. The curves exhibited a stable resilient behaviour in last ten cycles for each loading sequence, and the stresses and displacements were used for determining the resilient interface shear modulus for each loading sequence.

2.6.2.3 Repeatability of the cyclic shear test

Compared to material B used in this study, material A, the single-sized aggregate was easier to prepare. A small portion of particles was broken into small particles during the direct shear or cyclic shear test. The broken aggregate particles were sieved out and replaced with new 19 – 25 mm particles after each test to eliminate the potential influence of particle size change on the interactions between geogrid and particles. Three repeated tests were conducted following the cyclic shear test protocol as shown in Table 2.3 on the same geogrid-aggregate interface: the single-sized aggregate reinforced with the triaxial geogrid. There is a total of 30 data points in each test, and the results of these three tests are summarized in Table 2.5. The Tukey-Honestly significant differences test was

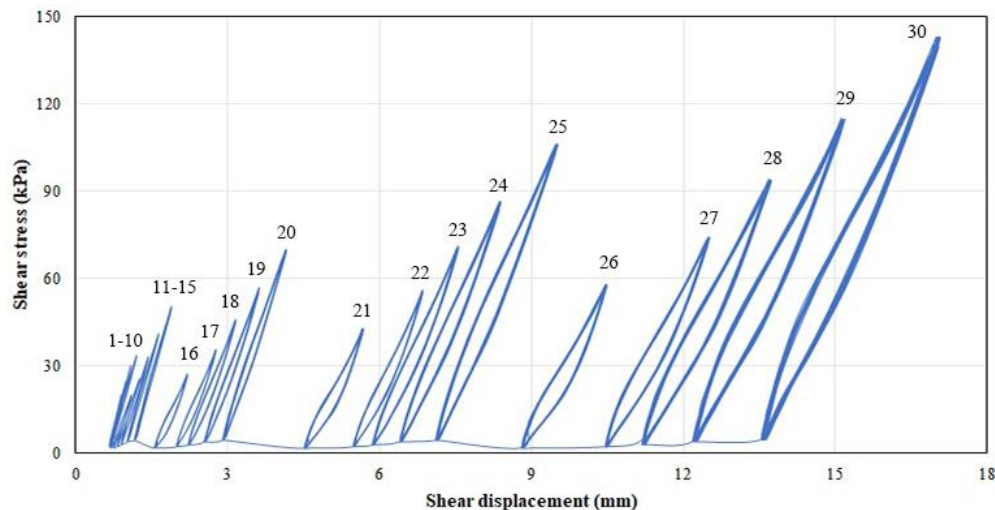


Figure 2.13. Shear stress versus shear displacement curves of last ten cycles for each sequence of biaxial geogrid reinforced material B

Table 2.5. Comparison of resilient interface shear modulus (G_i) between three tests

Total Shear Stress (kPa)	Normal Stress (kPa)	G_i			Total Shear Stress (kPa)	Normal Stress (kPa)	G_i		
		1	2	3			1	2	3
20.0	50	82.7	85.1	83.3	109.0	100	59.0	62.5	60.1
39.8	50	67.6	74.5	70.1	136.2	100	55.7	57.8	55.5
59.7	50	62.7	61.4	61.7	163.5	100	53.2	56.5	53.6
79.7	50	56.0	58.7	57.1	31.0	125	91.0	96.1	88.6
99.5	50	52.1	53.6	52.2	61.8	125	74.4	78.0	74.3
119.5	50	49.1	51.3	48.4	92.7	125	67.0	68.4	66.8
23.6	75	87.2	93.7	87.3	123.7	125	59.5	62.1	60.5
47.1	75	70.3	76.6	71.5	154.5	125	56.0	57.9	56.7
70.7	75	64.9	65.5	64.9	185.5	125	54.6	55.9	54.8
94.4	75	58.1	60.9	58.7	34.6	150	93.0	97.4	90.2
117.9	75	55.1	55.9	55.2	69.1	150	76.9	79.9	75.3
141.5	75	52.7	54.9	52.6	103.7	150	67.4	68.8	66.8
27.3	100	89.7	97.1	89.1	138.4	150	59.9	62.5	61.2
54.4	100	72.3	76.7	73.4	172.8	150	56.6	58.9	56.6
81.7	100	66.5	67.3	65.9	207.5	150	54.9	56.7	55.1

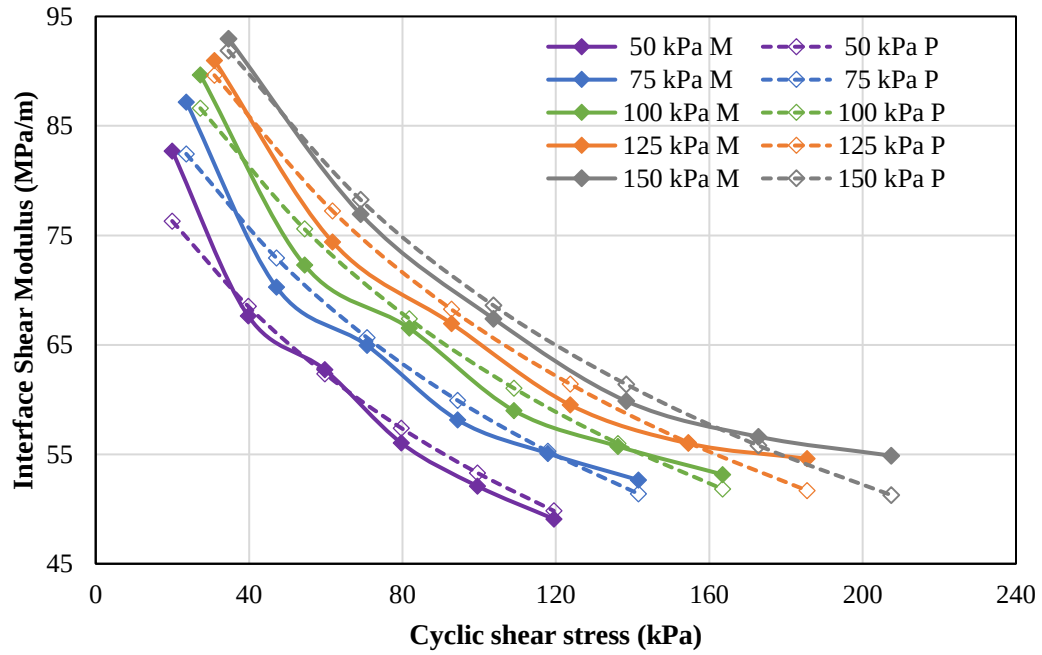
conducted to identify potential variation among these three tests. As indicated in the Table 2.6, all three confidence intervals (95%) contained 0, and all p-values were larger than 0.05, which indicated that there were no significant differences between these three tests.

2.6.2.4 Feasibility of the cyclic shear test

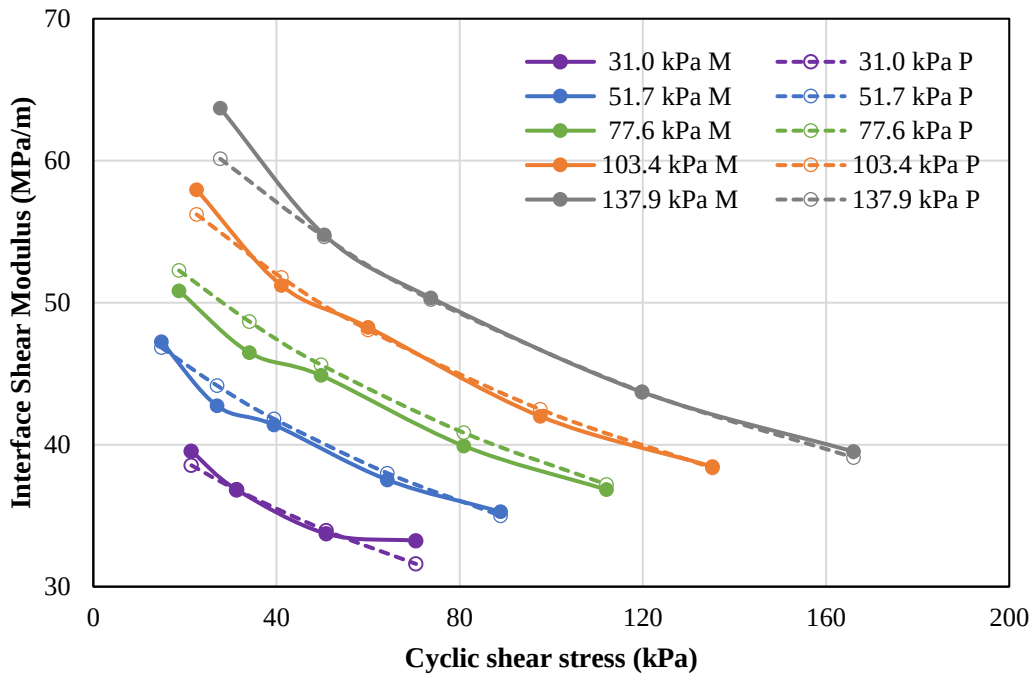
In this study, a total of four interfaces were tested with the cyclic shear test, including triaxial geogrid reinforced material A (TX-A), biaxial geogrid reinforced material B (BX-B), triaxial geogrid reinforced material B (TX-B), and material B unreinforced with any geogrids (NG-B). Figure 2.14 shows the results of the resilient interface shear modulus for these four interfaces. The solid lines represent the measured resilient interface shear moduli labelled as M , while the dashed lines represent the predicted resilient interface shear moduli. Figure 2.14 (a) through Figure 2.14 (d) show that G_i was dependent on the normal stress and cyclic shear stress. Generally, G_i increased as the normal stress increased and decreased as the cyclic shear stress increased. It was found that G_i of reinforced aggregate were closely related to the base course material type. For materials A and B reinforced with the same triaxial geogrid, G_i of TX-A was larger than that of TX-B at the same combination of normal and cyclic shear stress. The results of reinforcement effect of geogrid could be characterized by the cyclic shear test. Compared to unreinforced material B, G_i of both reinforced aggregates (BX-B, TX-B) was larger than that of unreinforced aggregate at a low cyclic shear stress, but the difference became small with increasing cyclic shear stress. For dense-graded material B, compared to the interface in aggregate-aggregate, the interface in both biaxial geogrid and triaxial geogrid reinforced aggregates showed an

Table 2.6. Multiple comparisons between three repeated cyclic shear tests

Comparison	Mean difference	95% Confidence Interval		p-value
		Lower bound	Upper bound	
Test2-Test1	2.88733	-5.1152	10.8899	0.667
Test3-Test1	0.04367	-7.9589	8.0462	1.000
Test3-Test2	-2.84367	-10.8462	5.1589	0.675

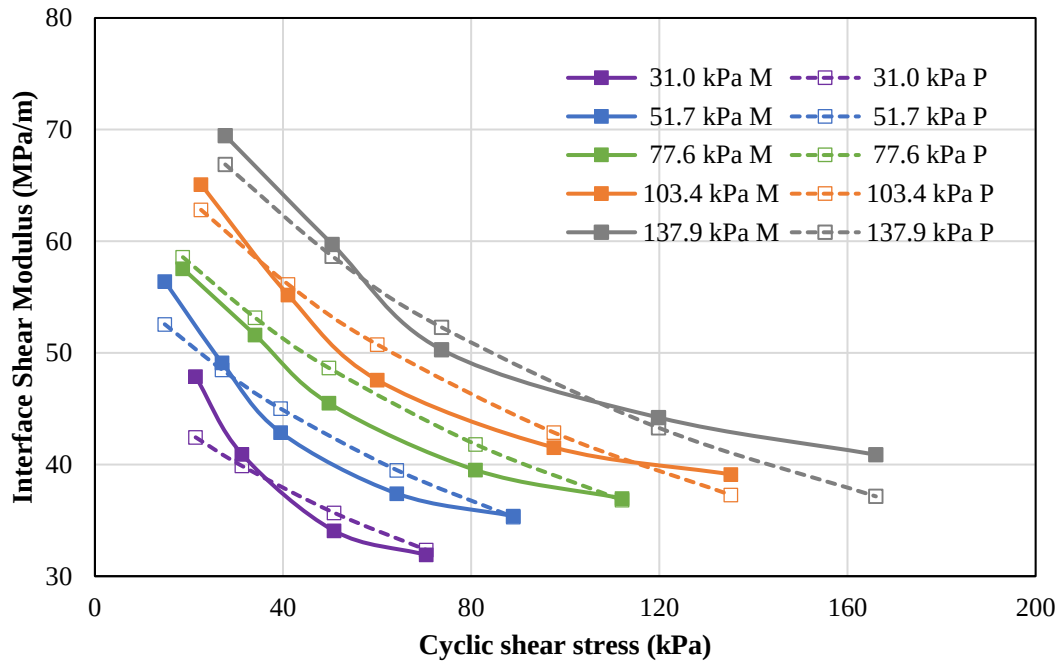


(a) TX-A-1

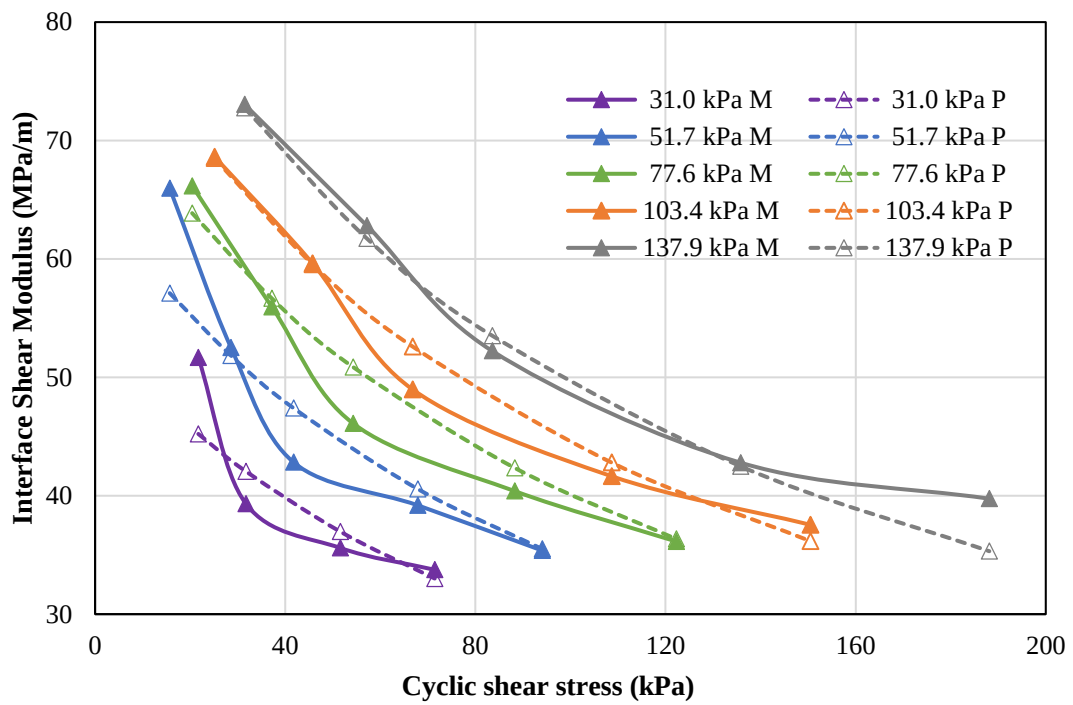


(b) NG-B

Figure 2.14. Resilient interface shear modulus versus cyclic shear stress: (a) TX-A-1, (b) NG-B, (b) BX-B, (c) TX-B.



(c) BX-B



(d) TX-B

Figure 2.14 Continued

obvious inflation points on the curves of cyclic shear stress versus interface shear modulus curves as shown in Figure 2.14 (c) and Figure 2.14 (d) when the ratio of total shear stress to failure shear stress (τ_t/τ_f) was around 0.5, indicating a better interaction between geogrid and aggregate with a higher G_i when the $\tau_t/\tau_f < 0.5$.

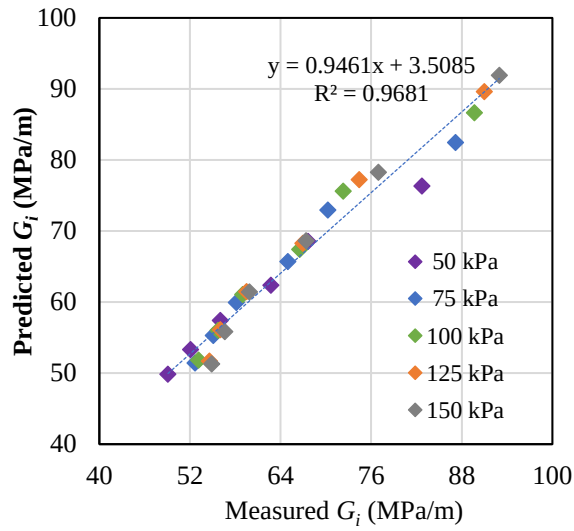
2.6.2.5 Three-parameter model of resilient interface shear modulus

The modified MEPDG model as shown in Eq. 2.4 was used to accommodate the effects of both confining stress and shear stress. The regression parameters for each sample are summarized in Table 2.7. According to the cyclic shear test protocol presented in Table 2.3, there were 30 measured G_i values from a cyclic shear test. However, normal stresses and cyclic shear stresses applied to material B in loading sequence 1 to 6 were relatively low compared to other loading sequences, resulting in a very small displacement in this sequence. The magnitude of this small displacement was close to the accuracy of the LVDT, causing a relatively large deviation or measurement error. Therefore, these six measured G_i were excluded in the regression.

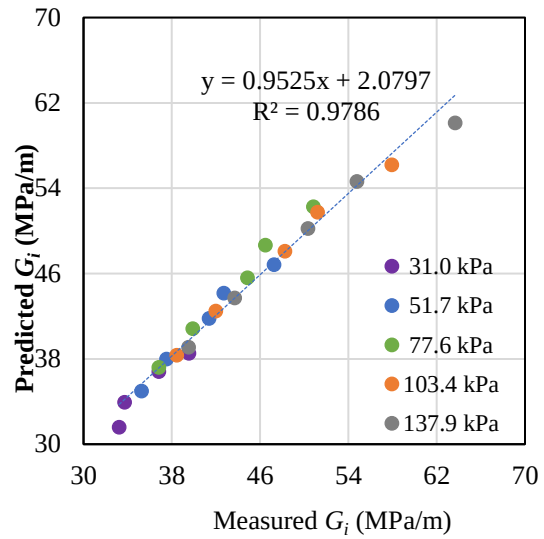
Figure 2.15 compares the measured and predicted moduli for four interfaces, including TX-A, NG-B, BX-B and TX-B. It was found that the measured G_i had a good correlation with the predicted G_i , which meant the modified three-parameter model was proved to be capable of characterizing G_i very well.

Table 2.7. Three parameters in the model for resilient interface shear modulus

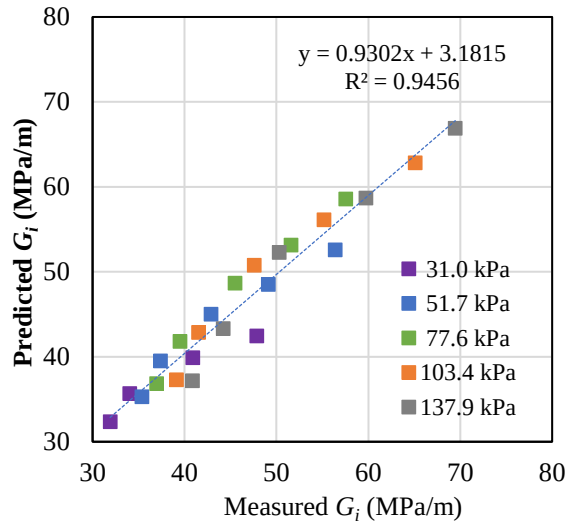
Sample	lgk_1	k_1	k_2	k_3	R^2
TX-A-1	3.007	1016.249	0.243	-0.711	0.9681
TX-A-2	3.034	1081.434	0.249	-0.742	0.9692
TX-A-3	3.002	1004.616	0.231	-0.694	0.9661
NG-B	2.793	620.869	0.318	-0.591	0.9786
BX-B	2.860	724.436	0.332	-0.807	0.9456
TX-B	2.916	824.138	0.366	-0.927	0.9326



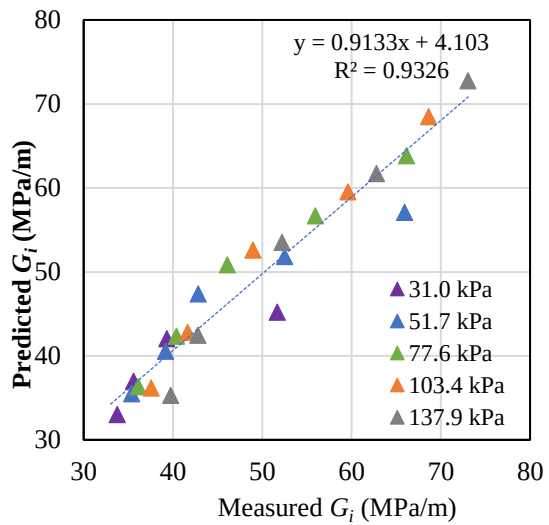
(a) TX-A-1



(b) NG-B



(c) BX-B



(d) TX-B

Figure 2.15. Predicted resilient interface shear modulus versus measured resilient interface shear modulus

Figure 2.16 shows the relationship between the resilient interface shear modulus and the confining normal stress for three different interfaces, including NG-B, BX-B and TX-B. Eq. 2.4 and the constants were used to plot the curves in Figure 2.16. The cyclic shear stress in Eq. 2.4 was assumed to be 20 kPa. It can be clearly observed that resilient moduli of both BX and TX were significantly larger than that of NG.

2.6.3 Discussions

In this study, the effects of geogrids reinforcement in UGM was first evaluated through a repeated load triaxial test. However, the inclusion of geogrids did not show a tangible increase in resilient modulus. Unlike the RLT test, the cyclic shear test could identify the effects of geogrid reinforcement by improving the interlocking between aggregate particles and geogrids. The two laboratory test methods used in this study had a lot in common, such as specimens were subjected to a group of loading sequences with different loading combinations of confining stress and cyclic axial stress or cyclic shear stress. Both tests were conducted in a similar loading pattern and the parameters from both tests (M_R and G_i) were stress-dependent. M_R was a function of confining stress and deviator stress and G_i a function of vertical normal stress and horizontal cyclic shear stress. The original M_R models

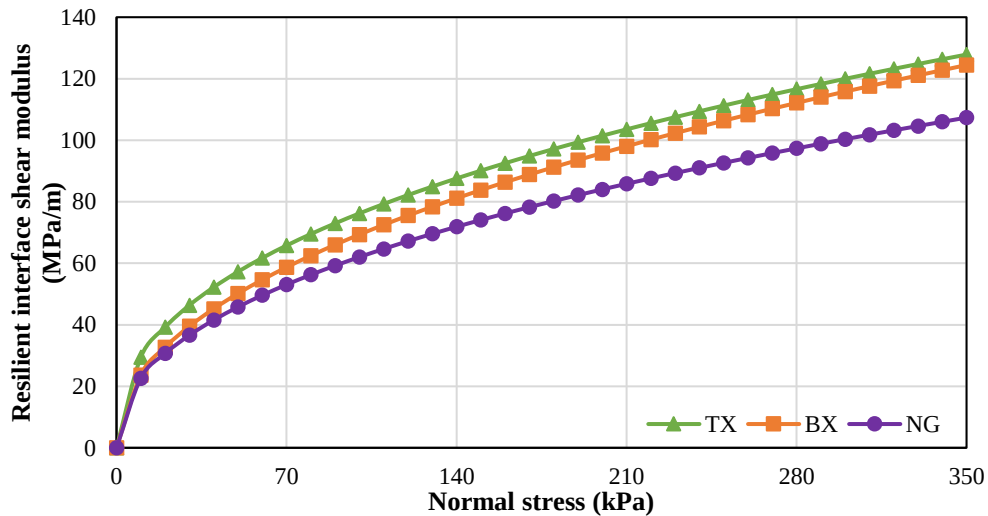


Figure 2.16. Resilient interface shear modulus versus normal stress for TX, BX and NG

could be modified and used for G_i . However, the two tests were different when they were used to quantify the effect of geogrid reinforcement in unbound base materials. The cyclic shear test showed a better potential for quantifying the reinforcing effect of geogrids in a base material. In the RLT test, geogrids were unable to provide a lateral confinement to the unbound granular material due to the limitation of sample size. Compared to the RLT test, the number of grids used in the cyclic shear test was much higher (180 and 240 for the biaxial and triaxial geogrids, respectively), which could provide a more effective lateral confinement to aggregate particles. The resilient modulus test was performed at relatively low stress levels and the test was considered a nondestructive test. The maximum vertical resilient deformation in the repeated load triaxial test was only 0.199 mm (BX loading sequence 15), and the lateral deformation of particles was only 0.07 mm if Poisson's ratio was assumed to be 0.35 for the granular material. The magnitude of the lateral deformation was not large enough to activate or mobilize geogrid reinforcement. Therefore, the geogrid showed a negligible effect on the stiffness of UGM in this test. However, the minimum relative displacement between geogrids and particles in the top box in the cyclic shear test could reach 0.685 mm (BX loading sequence 1), which was directly applied to the particles and geogrids and could activate the lateral confinement. Therefore, the cyclic shear test seemed more promising in quantifying the geogrid reinforcement than the resilient modulus test.

2.7 Conclusions

In this study, a series of RLT tests and cyclic shear tests were conducted on unreinforced and reinforced aggregates to quantify geogrid reinforcement in base course. Two different types of geogrids were used as reinforcement in an unbound granular material. The test protocol for the cyclic shear test was obtained by modifying that for the RLT test. Like the resilient modulus (M_R) from the RLT test for evaluating the stiffness of reinforced/unreinforced unbound granular materials, a term called resilient interface shear modulus (G_i) was employed to quantify the interlocking between geogrids and aggregate in the cyclic shear test. Based on the laboratory results, conclusions could be drawn from the study as follows:

The results of RLT tests showed that the inclusion of geogrids did not have a significant effect on the resilient behavior of reinforced specimens compared to unreinforced specimens. Thus, the RLT test may not be effective in evaluating the effects of geogrid reinforcement in unbound granular materials.

Duplicate tests through triaxial geogrid reinforced aggregate of material A showed a good repeatability of cyclic test protocol, and both of biaxial and triaxial geogrid reinforcement effect could be characterized by the cyclic shear test through geogrid-reinforced aggregate of material B. Based on the laboratory test results of the unreinforced and reinforced aggregates, the cyclic shear test exhibited its effectiveness in characterizing geogrid reinforcement. The presence of geogrids strengthened the interlocking between aggregate particles and geogrids and the degree of the interlocking could be quantified with the cyclic shear test.

Like the stress dependency of the resilient modulus, the resilient interface shear modulus was a function of confining normal stress and cyclic shear stress. The value of G_i increased with an increase in normal stress and decreased with an increase in cyclic shear stress. A NCHRP three-parameter model was modified to predict the resilient interface shear modulus with a high reliability.

In this study, two laboratory test methods were compared for their capability for quantifying geogrid reinforcement in granular materials. Results showed that the cyclic shear test along with the resilient interface shear modulus showed a great potential for characterizing the geogrid-reinforced granular base material. Further studies are still needed to explore various factors affecting geogrid reinforcement and to incorporate G_i as a material input in the design of geogrid-reinforced pavements.

2.8 Acknowledgments

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CHAPTER THREE
LABORATORY INVESTIGATION OF PARTICLE SIZE EFFECTS
ON THE SHEAR BEHAVIOR OF AGGREGATE-GEOGRID
INTERFACE

A version of this chapter was originally published by Bingye Han, Jianming Ling, Xiang Shu, Hongren Gong and Baoshan Huang:

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Bingye Han's contribution was conducting all literature review, data analysis, and writing the text contained in the manuscript. Dr. Baoshan Huang provided guidance throughout the research process as well as editorial assistance.

3.1 Abstract

As an effective method to explore the interaction mechanism between aggregate particles and geogrids, a large-scale direct shear test was employed to study the shear behavior of aggregate-geogrid interface and the effects of particle size on the interfacial shear strength. Two types of geogrids with rectangular or triangular aperture and three single-sized coarse aggregates, 12.5 - 19 mm, 19 - 25 mm, 25 - 37.5 mm, were tested in this study. Multiple parameters were employed to investigate the reinforcement of geogrids in aggregate, including peak shear strength, residual shear strength, vertical displacement and interfacial shear strength efficiency coefficient, α . Results show that the values of α ranged from 0.78 to 1.01, and the aggregate-geogrid interface of AGG2 achieved the highest α among the three aggregates, indicating that the 19 - 25 mm aggregate had the best interlocking effect reinforced with geogrids used in this study. The combinations of geogrids and aggregate exhibited different interaction mechanisms and failure modes. Based on the test results, the method was improved for determining the appropriate geogrid aperture size for an aggregate gradation. Effective interlock could be achieved when the ratio of the equivalent aperture size to particle diameter was 1.30 - 1.71 for biaxial geogrids and 1.08 - 1.43 for triaxial geogrids.

3.2 Introduction

Geogrids consist of longitudinal and transverse ribs with relatively large apertures, and the ribs of geogrids can confine aggregate and constrain the lateral movement of aggregate to

improve pavement performance (20). The movement of aggregate on geogrid surface or in the apertures of geogrid plays an important role in the interaction between the two mechanisms. However, the interaction is complicated and influenced by many factors including geogrid properties, aggregate properties and loading conditions (99). Different methods have been used to understand their interaction mechanisms, including pull-out test (100-102), direct shear test (55; 103), plane strain test (104; 105), and torsional ring shear test (106; 107). Among these methods, the direct shear test, especially the large-scale direct shear test, is one of the most commonly used methods to investigate the interface behavior between geogrids and aggregate. Many studies using direct shear test can be found in the literature (108-114).

The geogrid reinforcement effect largely depends on the interactions between the geogrids and aggregate. Gu et al. (115) investigated the effect of geogrid-aggregate shear coefficient on the mechanical reinforcement. Aggregate particles are likely to be trapped into apertures if the ratio of the aperture size to the aggregate particle size, A/D , is appropriate, resulting in an effective interlocking between aggregate and geogrids. Sarsby (116) revealed that the interaction between sand and geogrids can be maximized when the ratio of the minimum width of geogrid aperture to the average particle size, D_{50} , is larger than 3.5. Athanasopoulos (117) showed that the apparent interface friction angle can reach its peak value when the ratio of geotextile aperture size to the average sand particle size is approximately 1.60. Brown et al. (118) reported that the ratio of geogrid aperture size to the nominal size of ballast should be 1.4 based on the settlement behavior of geogrid-reinforced ballast. Indraratna et al. (119) concluded that the optimum ratio of geogrid aperture size to the average ballast size is 1.20 based on the interface shear properties. However, Tang et al. (110) and Liu et al. (109) reported that the shear behavior of the soil-geogrid interface and the aperture size are not closely related.

Although limited researches have been conducted to determine the optimum ratio of geogrid aperture size to aggregate particle size, their conclusions are different or even contradictory. Most of researchers use the average particle size, D_{50} , as a particle indicator to determine the proper ratio of aperture size of geogrids to particle size. However, the use of D_{50} may not be reasonable because the D_{50} is merely a general index of gradation. In

fact, particles in the size of D_{50} that can interlock with geogrids may not exist, or if particles in this size exist, the interlocking may come from aggregate particles instead of D_{50} . Also, the percentage of too coarse or too fine aggregate particles could significantly affect D_{50} , but they contribute little to interlocking between aggregate and geogrids. Therefore, to determine the optimum ratio of A/D, single-sized aggregate is more appropriate. In addition, triaxial geogrids have been successfully used to reinforce different unbounded granular materials. The mechanical reinforcement of biaxial and triaxial geogrids was investigated and compared in other studies (120; 121). However, limited research has been conducted on the optimum ratio of A/D. Therefore, more research efforts are needed to gain a better understanding when triaxial geogrids are used.

3.3 Objective and scope

The objective of this chapter was to utilize the large-scale direct shear tester to explore the effect of particle size on the shear behavior of aggregate-geogrid interface, including both biaxial and triaxial geogrids. The findings from the study can serve as a basis for development of a practical guideline on the recommendation on the optimum A/D ratios. In this study, three single-sized aggregates and two types of geogrids with different aperture shapes were tested to determine the optimum ratio of aperture size to particle size to maximize the benefits of geogrid reinforcement in aggregate. Three possible modes of aggregate-geogrid interface failure were proposed based on the results.

3.4 Materials and test methods

3.4.1 Materials

The aggregate used in the study was crushed limestone with angular to sub-angular particles. To investigate the influence of particle size on the shear behavior of aggregate-geogrid interface, the aggregate was sieved and divided into three single-sized aggregate particles, AGG1 (12.5 - 19 mm), AGG2 (19 - 25 mm), AGG3 (25 - 37.5 mm). Its specific gravity was 2.754, 2.791 and 2.733 for AGG1, AGG2 and AGG3, respectively. The geogrids were extruded biaxial polypropylene geogrids and punched polypropylene triaxial geogrids. Their mechanical and physical-chemical properties are listed in Table 3.1.

Table 3.1. Mechanical and physical properties of geogrids

Geogrid	Biaxial	Triaxial
Tensile strength at 5% (kN/m)	30	-
Radial stiffness at 0.5% (kN/m)	-	270
Junction efficiency (%)	90	95
Aperture size (mm)	32 × 31	46 × 46 × 46

3.4.2 Direct shear device

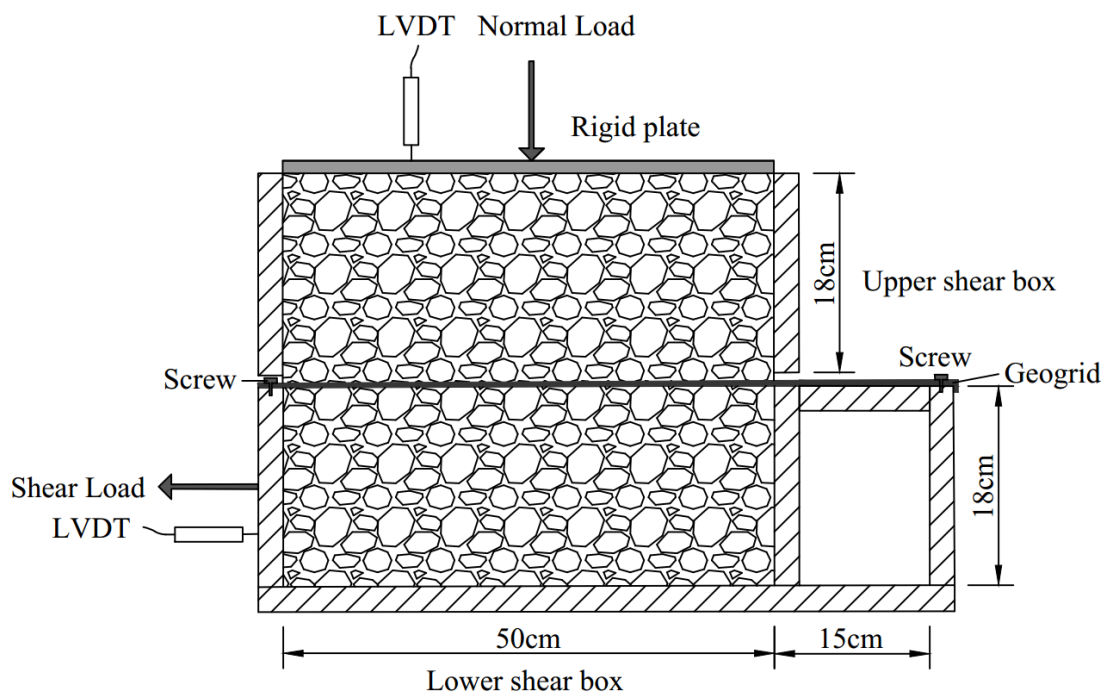
A large-scale direct shear box consisted of a fixed upper box and a moveable lower box, and their dimensions were 50 cm × 50 cm × 18 cm and 65 cm × 50 cm × 18 cm, respectively, as shown in Figure 3.1. During the testing, shear area decreased during the shearing, and a shearing force of up to 100 kN could be applied. A normal stress was also applied through a rigid square plate to the aggregate with a capacity of up to 100 kN. The two actuators could apply either displacement-controlled or load-controlled static or cyclic shear force. The horizontal and vertical displacements were measured automatically by two Linear Variable Differential Transformers (LVDT). The maximum measurements of horizontal and vertical actuators were 127mm and 254mm, respectively. To reduce the effect of temperature on the experiment, room temperature was kept constant at 20 ± 1 °C.

3.4.3 Test procedures

The aggregates were air dried and compacted by a concrete electric vibrator in four layers, and the number of vibration and compaction for each layer was 40 times to ensure that the relative dry density of aggregate was kept at 55 - 58 %. The size of geogrids installed at the interface was 71 cm × 48.5 cm, and the geogrids were fixed with steel bars and screws at the front and rear sides of lower box to ensure that the geogrids could be mobilized during the testing. The direct shear test was conducted at different normal stresses of 50, 100 and 150 kPa with a constant shear rate of 1 mm/min in according with ASTM D 5321(122). These normal stresses were selected according to the base stress levels in the



(a) Photo of the direct shear box



(b) Schematic of the direct shear box

Figure 3.1. Overview of the large-scale direct shear box

pavement and previous studies (109; 113; 123). The test was completed when the horizontal shear displacement reached 100 mm. Table 3.2 presents the variables considered in the study.

To ensure the accuracy and consistency of the direct shear test, four pre-tests were finished under the same experiment conditions to check the reproducibility. AGG1 particles were used in the pre-test at a normal stress of 50 kPa. The results of shear stress versus shear displacement curves are shown in Figure 3.2 and descriptive statistic parameters of the results are presented in Table 3.3, including peak shear stress τ_p , horizontal displacement at peak shear stress Δ_h , and residual shear stress at the end of the test τ_R . The coefficients of variation of the peak shear stress reported in other researchers' results were 28% and 4.8%, respectively (123; 124). Compared to their coefficients of variation, the results of the current study were considered repeatable and accurate.

3.5 Results and discussion

The results of the large-size direct shear tests include shear stress versus shear displacement curves, vertical displacement versus shear displacement curves, peak shear strengths and residual shear strengths of aggregate and aggregate-geogrids, and interfacial shear strength efficiency coefficients for different aggregates.

3.5.1 Shear stress-shear displacement curves

Figures 3.3 through 3.5 show the shear stress-shear displacement curves and vertical displacement-shear displacement curves for different interfaces of reinforced and

Table 3.2. Direct shear test program

Test type	Soil sample	Normal stress/kPa
Without geogrid (N)	AGG1, AGG2, AGG3	50,100,150
With biaxial geogrid (R)	AGG1, AGG2, AGG3	50,100,150
With triaxial geogrid (T)	AGG1, AGG2, AGG3	50,100,150

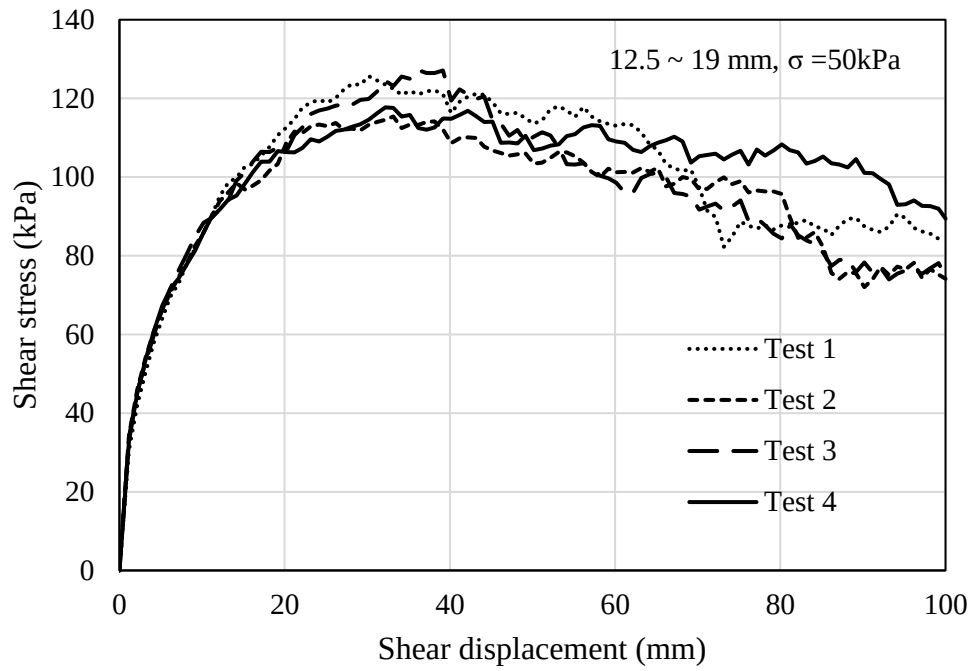


Figure 3.2. Reproducibility of the test results

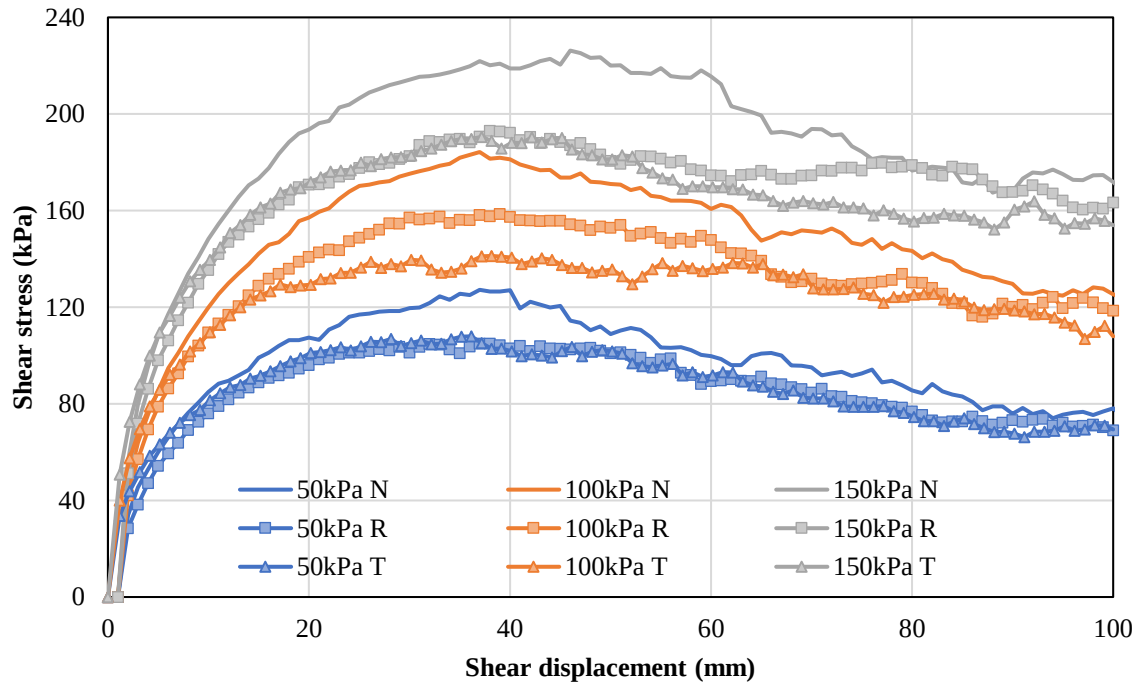
Table 3.3. Descriptive statistics of the results

Test	τ_p (kPa)	Δ_h (mm)	τ_R (kPa)
Test 1	125.65	30	84.34
Test 2	115.41	33	74.17
Test 3	127.20	36	74.60
Test 4	117.73	32	89.40
Mean	121.50	33	80.63
Std. Deviation	5.8	2.5	7.5
Coefficient of variation (%)	4.8	7.6	9.3

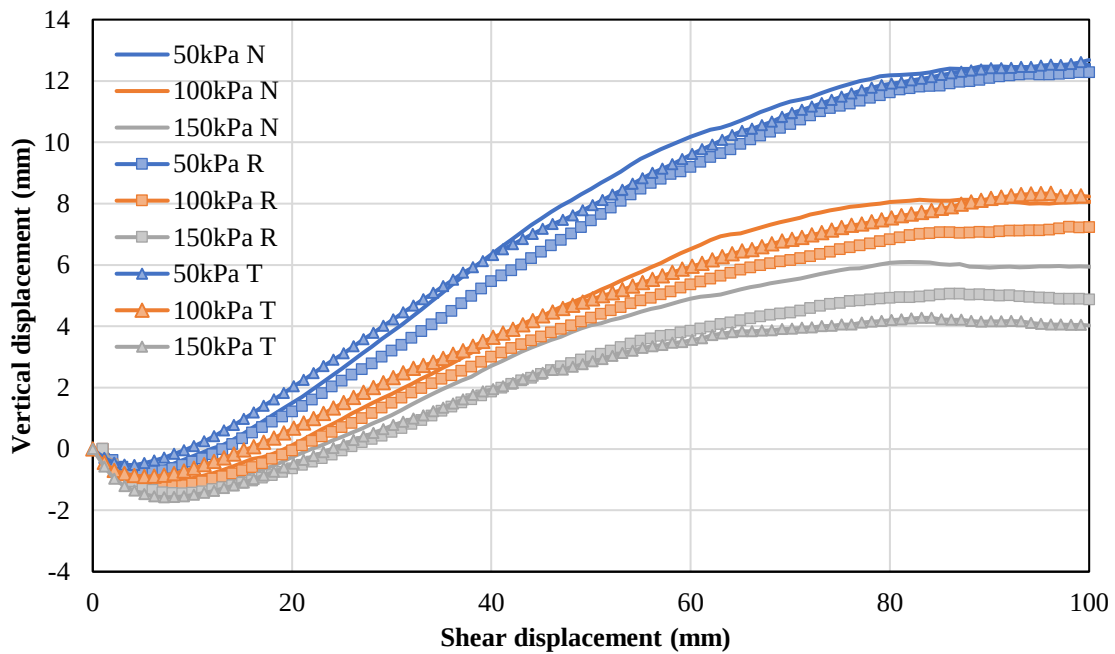
unreinforced aggregate, including the aggregate-aggregate interface (N), the aggregate-biaxial geogrid interface (R), and the aggregate-triaxial geogrid interface (T). To ensure the repeatability of the tests, two replicates were tested for the unreinforced and reinforced aggregates at each stress state. Figure 3.3 shows the case of AGG1 (12.5 - 19 mm). It was found that the shear stress increased with the increase of the normal stress. The shear stress gradually increased to a peak shear stress during the testing, and then decreased to a residual stress at the end of the test. Both the peak interfacial shear stress of aggregate-biaxial geogrids and aggregate-triaxial geogrids were lower than that of unreinforced aggregate. In Figure 3.3, a positive vertical displacement represents a dilation while a negative vertical displacement means a contraction. The results show all the samples experienced a contraction at the initial stage, then followed by a continuous dilation until the end of the test. In general, the vertical displacement of unreinforced aggregate was a little higher than that of aggregate reinforced by biaxial geogrids and aggregate reinforced by triaxial geogrids because the unreinforced aggregate was denser than the aggregate reinforced by geogrids near the shearing plane, leading to a higher dilation and a larger vertical displacement. The addition of geogrids made the reinforced aggregate looser than unreinforced aggregate. It was also obvious that the initial contraction during the testing increased with the increase in normal stress.

Figure 3.4 shows the plots of shear stress and vertical displacement versus shear displacement for AGG2 (19 - 25 mm). The peak shear stresses of unreinforced aggregate and biaxial geogrid reinforced aggregate were close, which were slightly larger than that of triaxial geogrid reinforced aggregate at 50 kPa and 100 kPa. It was also noted that this difference became smaller at 150 kPa. On the other hand, the change in vertical displacement of AGG2 was different from that of AGG1. The vertical displacements of reinforced aggregate interfaces including biaxial geogrids reinforced aggregate and triaxial geogrids reinforced aggregate were higher than that of the unreinforced aggregate, and the difference in vertical displacement were obvious at high normal stresses (100 kPa and 150 kPa), indicating that the reinforced aggregate behaved more like a dense material, thus resulting in a strong interaction between AGG2 particles and geogrids.

Figure 3.5 shows the plots of shear stress and vertical displacement versus shear

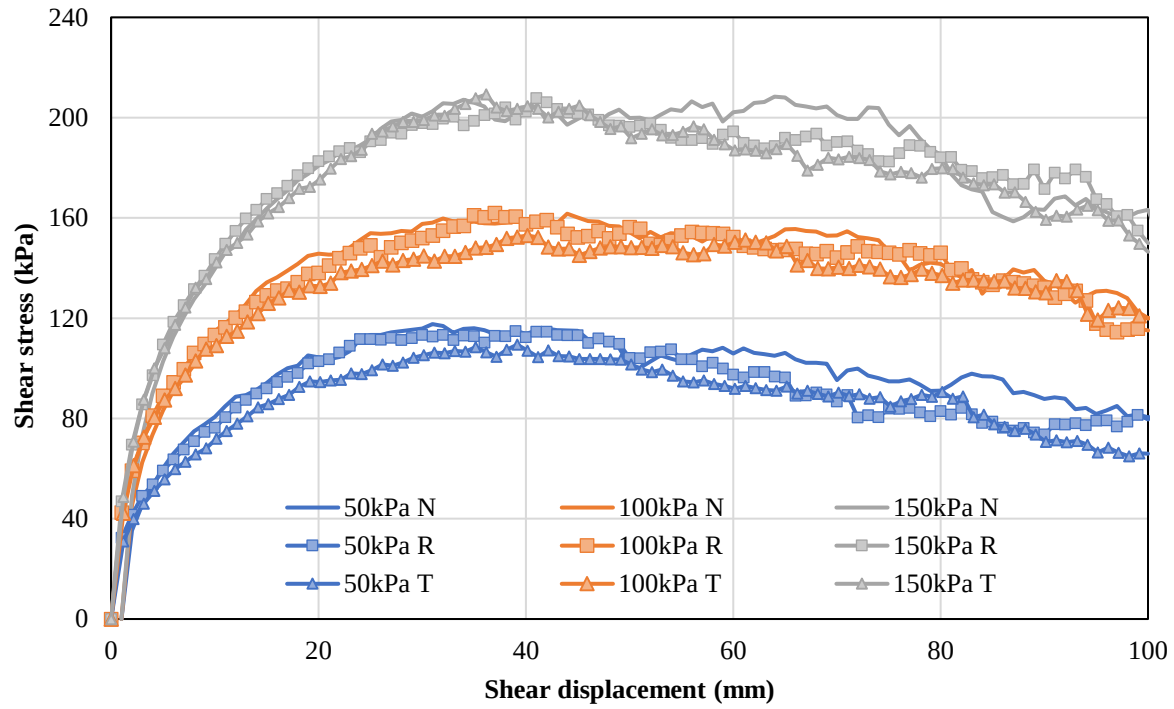


(a) Shear stress versus shear displacement curves

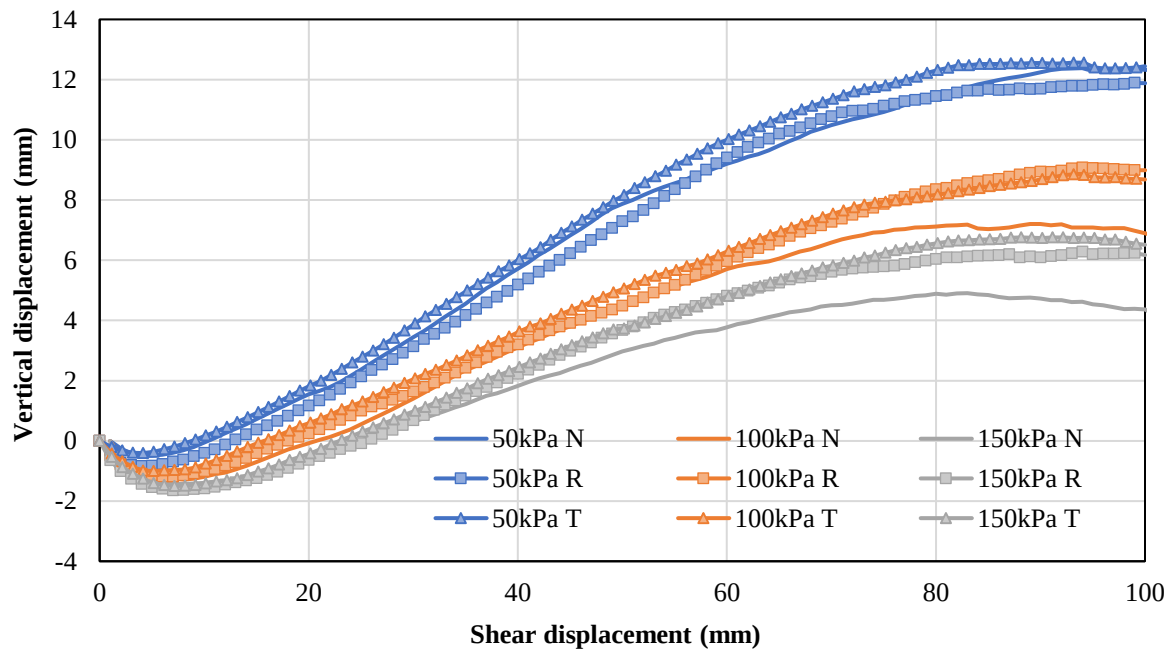


(b) Vertical displacement versus shear displacement curves

Figure 3.3. Plots of shear stress and vertical displacement versus shear displacement for different interfaces of 12.5 - 19 mm aggregate

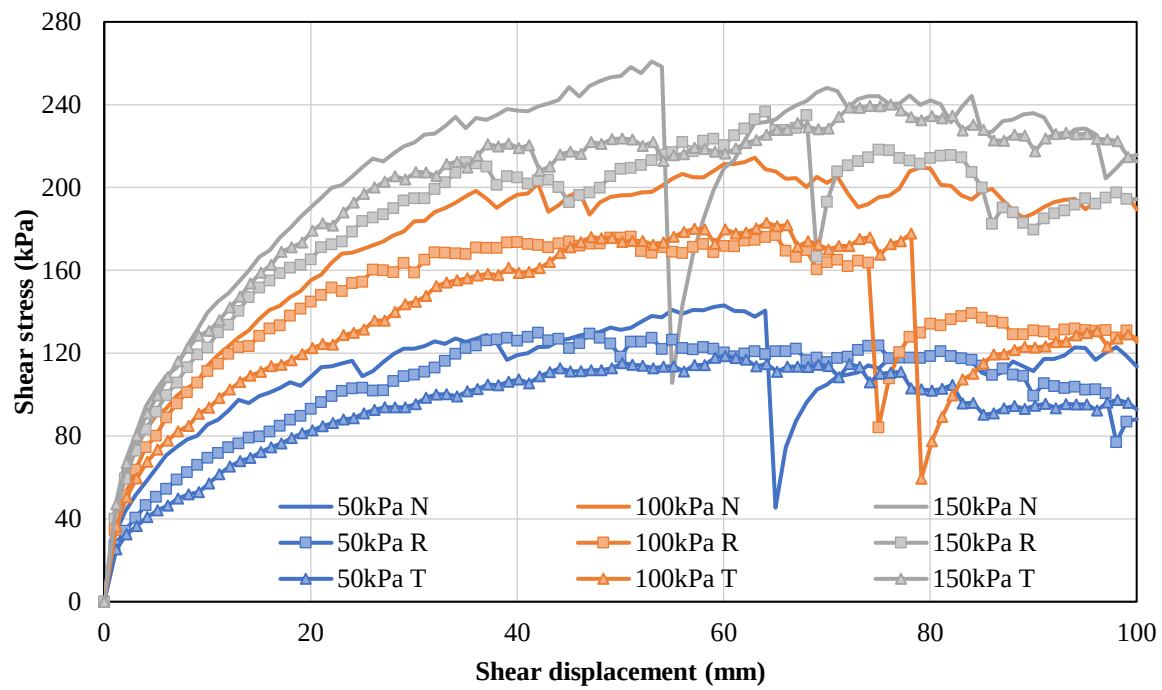


(a) Shear stress versus shear displacement curves

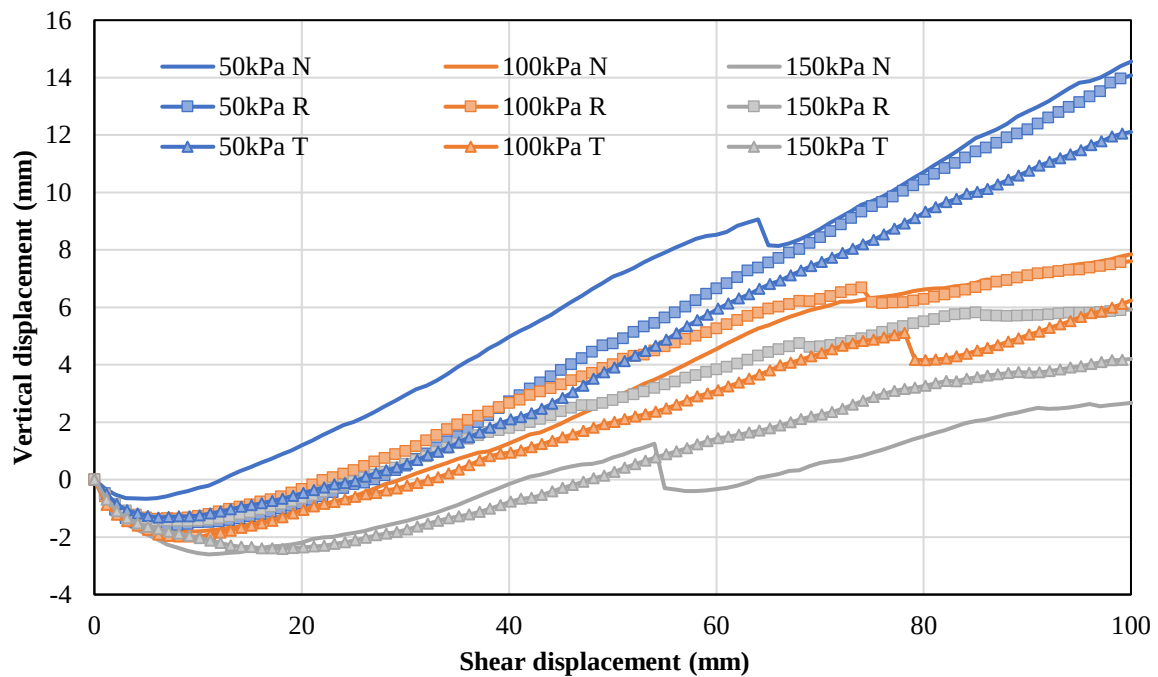


(b) Vertical displacement versus shear displacement curves

Figure 3.4. Plots of shear stress and vertical displacement versus shear displacement for different interfaces of 19 - 25 mm aggregate



(a) Shear stress versus shear displacement curves



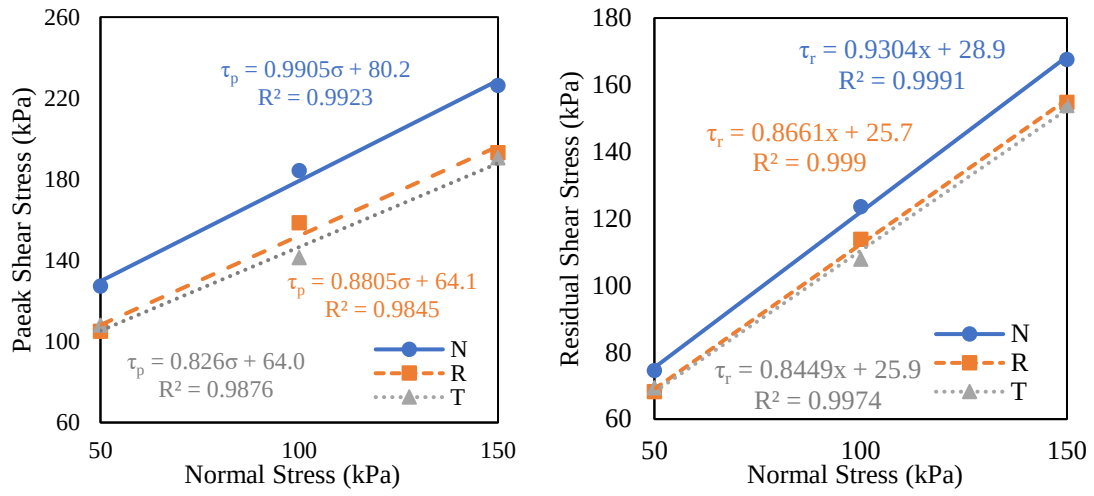
(b) Vertical displacement versus shear displacement curves

Figure 3.5. Plots of shear stress and vertical displacement versus shear displacement for different interfaces of 25 - 37.5 mm aggregate

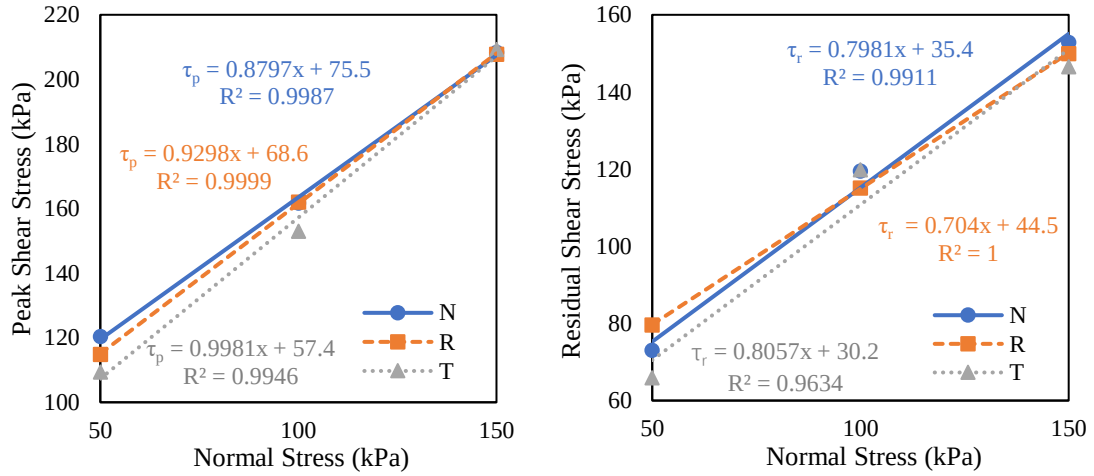
displacement for AGG3 (25 - 37.5 mm). There existed some abrupt changes in both shear stress and vertical displacement, and these sudden drops did seem random because they occurred at different interfaces and different normal stresses. This may be caused by the fact that particles in AGG3 were easier to move during the testing because of the minimum number of particle point-to-point contacts and the largest shearing force the particles experienced among the three aggregates. The movement of a single AGG3 aggregate particle like sliding and rotation had more significant effects on the shear interface than on that in AGG1 or AGG2. Therefore, the sudden drops were more obvious in the AGG3. The presence of sudden change made the comparison of vertical displacements more complex and difficult at different interfaces. However, the results of shear stress versus shear displacement still clearly show that peak shear stress of reinforced aggregate interface was lower than that of unreinforced aggregate and the shear stress versus shear displacement curves of reinforced aggregate interface were smoother.

3.5.2 Peak and residual shear strength envelopes

Figure 3.6 shows the Mohr-Coulomb failure envelope lines of peak and residual shear stresses. The peak shear stress represents the best mobilization of geogrids reinforcing aggregates, while the residual shear stress represents the worst mobilization of geogrids interacting with aggregates. During the shearing stage, the peak shear stress and the residual stress are relevant. In Figure 3.6, test results show a good linear relationship between peak shear stress τ_p or residual shear stress τ_r and normal stress σ with a relatively high correlation coefficient R^2 ($R^2 \geq 0.9845$). One exception was the relationship between the residual shear stress and normal stress in AGG3 with a relatively low R^2 of 0.927. This may be caused by abrupt particles breakages. The interfacial shear strength parameters, apparent cohesion c and internal friction angle ϕ , are summarized in Table 3.4. Figures 3.6 (a) and (c) show that the apparent cohesion of aggregate-aggregate interface was obviously higher than that of aggregate-biaxial geogrids and aggregate-triaxial geogrids in both peak shear stress and residual stress for AGG1 and AGG3. However, it is also found in Figure 3.6 (b) that the difference in apparent cohesion was not as obvious as that in AGG2. Regarding friction angles of different interfaces of various aggregates, friction angle of

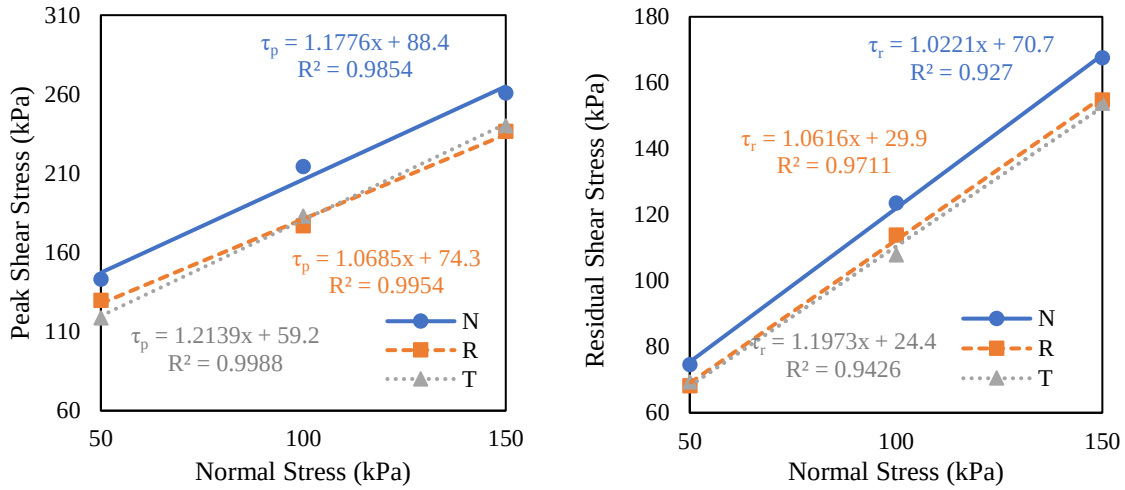


(a) 12.5 - 19 mm



(b) 19 - 25 mm

Figure 3.6. Peak and residual shear stress envelopes for different aggregate particle size from large-scale direct shear tests



(c) 25 - 37.5 mm

Figure 3.6. Continued

Table 3.4. Peak and residual parameters for aggregate with and without geogrids

Material	Geogrid	Apparent cohesion (c) (kPa)		Internal friction angle (ϕ) (°)	
		Peak	Residual	Peak	Residual
AGG1	—	80.2	28.9	44.7	42.9
AGG1	Biaxial	64.1	25.7	41.4	40.9
AGG1	Triaxial	64.0	25.9	39.6	40.2
AGG2	—	75.5	35.4	41.3	38.6
AGG2	Biaxial	68.6	44.5	42.9	35.1
AGG2	Triaxial	57.4	30.2	44.9	38.9
AGG3	—	88.4	70.7	49.7	45.6
AGG3	Biaxial	74.3	29.4	46.9	46.7
AGG3	Triaxial	59.2	24.4	50.5	50.1

unreinforced aggregate interface was larger than that of reinforced aggregate in AGG1. However, this trend did not appear to be true in AGG2 or AGG3 with relatively large particles because the peak or residual shear stress was easily affected by crushed particles during testing.

3.5.3 Interfacial shear strength efficiency coefficient

The interfacial interaction could be compared by an interfacial shear strength efficiency coefficient α , which is defined as the ratio of the shear strength of reinforced interface to that of unreinforced interface (Eq. 3.1):

$$\alpha = \tau_{aggregate-geogrid} / \tau_{aggregate-aggregate} \quad (\text{Eq. 3.1})$$

where, $\tau_{aggregate-geogrid}$ is the interfacial shear strength of aggregate with geogrids, and $\tau_{aggregate-aggregate}$ is the interfacial shear strength of aggregate without geogrids. A higher α means a higher efficiency caused by geogrid reinforcement. Table 3.5 summarizes the peak and residual interfacial shear strength efficiency coefficient for AGG1, AGG2 and AGG3 at the normal stresses of 50kPa, 100kPa and 150kPa. The average α was the average value of three values at different normal stresses. The peak aggregate-biaxial geogrid interfacial shear strength efficiency coefficient for AGG1, AGG2 and AGG3 was 0.85, 0.98 and 0.88, respectively while the peak aggregate-triaxial geogrid interface efficiency coefficient for AGG1, AGG2 and AGG3 was 0.82, 0.95 and 0.87, respectively. The results

Table 3.5. Comparison of peak and residual shear strength efficiency coefficients

	Peak						Residual					
	AGG1		AGG2		AGG3		AGG1		AGG2		AGG3	
	R	T	R	T	R	T	R	T	R	T	R	T
50kPa	0.83	0.85	0.95	0.91	0.91	0.83	0.91	0.93	1.09	0.90	0.78	0.82
100kPa	0.86	0.77	1.00	0.95	0.81	0.85	0.92	0.87	0.96	1.00	0.66	0.67
150kPa	0.85	0.84	1.00	1.00	0.91	0.92	0.92	0.92	0.98	0.96	0.90	0.99
Average α	0.85	0.82	0.98	0.95	0.88	0.87	0.92	0.91	1.01	0.95	0.78	0.82

show that the average peak interface efficiency coefficients of AGG2 with either aggregate-biaxial geogrids or aggregate-triaxial geogrids were the largest among the three aggregates gradations. Similar results are found in the average residual interfacial shear strength. Additionally, α of aggregate-triaxial geogrids was lower than that of aggregate-biaxial geogrids, indicating that the stiffness of geogrid was a significant factor for the interaction mechanisms. Stiff geogrids showed a stronger reinforcing effect than soft geogrids.

3.5.4 Role of aggregate particle size on interfacial shear strength

Based on the test results, it was found that the shear stress versus shear displacement curves were different among three different aggregates. Jewell et al. (125) reported that the shear resistance of soil-geogrid interface consists of two components: the internal shear resistance of interlocked particles in the openings of geogrids and the shear resistance between particles and geogrid surface. When geogrids are placed at the interface, the previous total resistance of internal shear resistance of interlocked particles is divided into these two resistance components. The relative size of aggregate particles to geogrid apertures have a significant influence on the proportions of these two shear resistance components. The magnitude of the internal shear resistance of aggregate in the openings depends on the degree of aggregate interlocking, while the magnitude of the shear resistance between aggregate and geogrid surface depends largely on the contact area of particles and geogrid surface. Usually, the shear resistance between aggregate and smooth geogrid surface is lower than the internal shear resistance of interlocked aggregate since the peak friction angle of aggregate-geogrids is less than the peak friction angle of limestone aggregate particles (109; 119; 125).

For AGG3 (25 - 37.5 mm), compared to the aperture size of geogrids (32×31 mm for biaxial geogrids or 46×46×46 mm for triaxial geogrids), most aggregate particles were too large to pass through the geogrid and to be interlocked in the geogrid openings. In this case, geogrids behaved more like geotextiles or geomembranes. On one hand, a significant proportion of original aggregate-aggregate contacts were replaced with aggregate-geogrid contacts. Therefore, most of the internal shear resistance of interlocked particles was shifted into a lower shear resistance between particles and smooth geogrid surface. On the

other hand, the internal shear resistance of interlocked particles in the openings was largely reduced because the ribs of geogrid could not provide effective lateral confinement for the aggregate particles, leading to a significant loss of interlock of aggregate particles at the interface as shown in Figure 3.7. After the shear test, aggregates were sieved again to determine the weight loss during testing. Insufficient interlocking could also be confirmed by the weight loss results. The weight loss percentage was 4.58% and 3.11% at 100 kPa normal stress for unreinforced and biaxial geogrids reinforced aggregates, respectively. Due to the stronger interlocking between particles, more interlocked particles in unreinforced aggregate group experienced a high stress, leading to more particles being crushed during testing, as shown in Figure 3.7 (a). However, as shown in Figure 3.7 (b), particles A and B in reinforced aggregate group could slip freely along the geogrid surface when the particles experienced a high stress. Therefore, the interfacial shear strength of aggregate-geogrids was lower than that of unreinforced aggregate, causing a lower value of interfacial shear strength efficiency coefficient α , which was in the range of 0.78 - 0.88 as presented in Table 3.5.

For AGG2 (19 - 25 mm), the particle sizes were close to the aperture size of geogrids, and almost every one or two particles could get stuck in the opening as particle A, B and C shown in Figure 3.8 (b) because the ribs of geogrids could constrain the free movement of

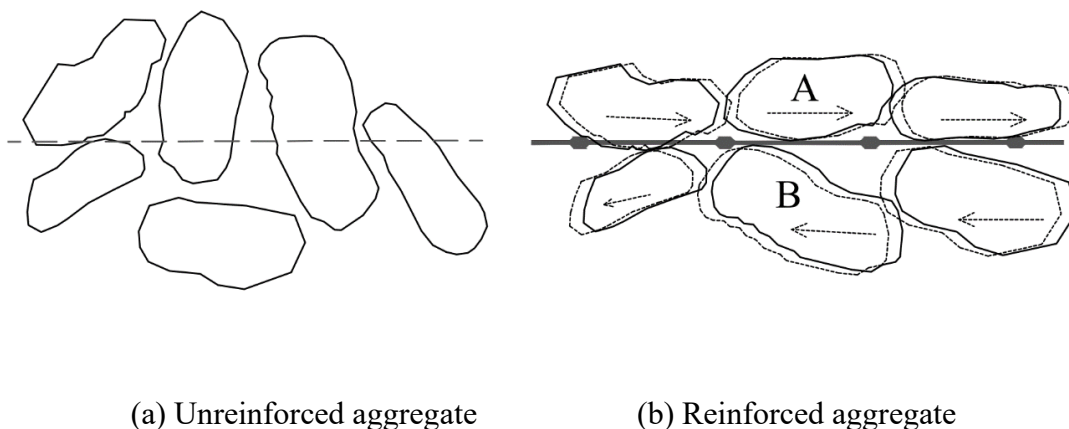


Figure 3.7. Interlocking mechanism of AGG3 (25 - 37.5 mm)

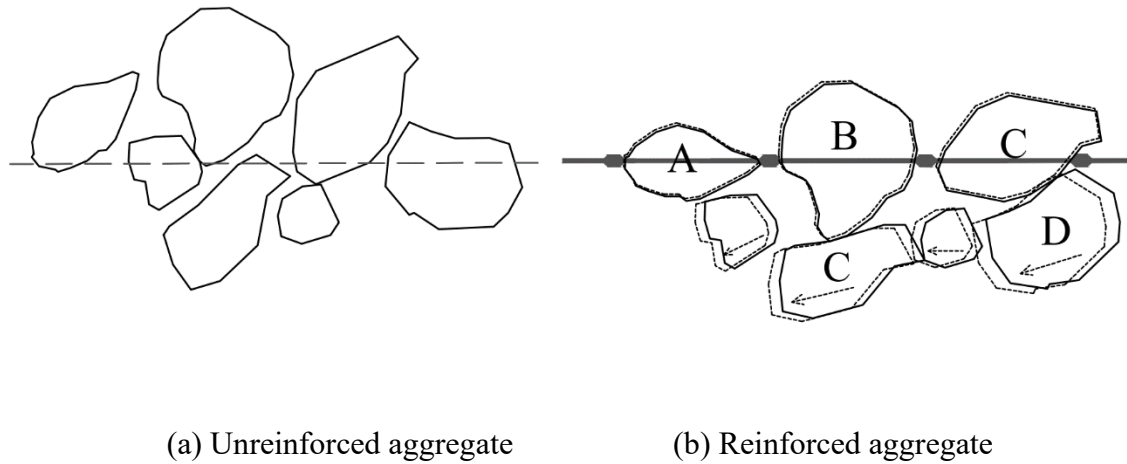


Figure 3.8. Interlocking mechanism of AGG2 (19 - 25 mm)

aggregate when particles were trapped in the openings. In this situation, the trapped aggregate particles still had a good interlock with their neighboring particles near the aggregate-geogrid interface. This could be confirmed from the weight loss test results. The weight loss percentage was 3.82% and 4.07% at 100kPa normal stress for unreinforced and biaxial reinforced aggregates, respectively, which indicated both particles at the unreinforced and reinforced interfaces experienced a high shear stress during the testing. Therefore, the interlocking degree of unreinforced and reinforced aggregates were statistically similar, or the interlocking degree of reinforced aggregates was even slightly improved, indicating that particles trapped in geogrid apertures interacted with geogrid ribs and other particles more positively than that in AGG3.

Positive interactions between particles and geogrid was also verified by the large deformation or even breakage of geogrid ribs during the shear test as shown in Figure 3.9. In addition, if particles were constrained, like those in Figure 3.8, the contact area between aggregate particles and smooth geogrid surface would not increase largely, leading to an insignificant proportion of shear resistance between particles and smooth geogrid surface. In this case, although the shear resistance of the aggregate-geogrid interface also consisted of a large fraction of the shear resistance of aggregate particles and a small fraction of shear



Figure 3.9. Deformation and breakage of biaxial geogrids in AGG2 (19 - 25 mm)

resistance between particles and geogrid surface, and this increased interlocking could counteract the negative effect of shear resistance between particles and geogrid surface. Therefore, the interfacial shear strength of aggregate-geogrid was basically equal to that of unreinforced aggregate, and the interfacial peak and residual shear strength efficiency coefficients of aggregate-biaxial geogrids and aggregate-triaxial geogrids were close to 1: the values of α_{peak} and α_{residual} of aggregate-biaxial geogrids were 0.98 and 1.01, and both α_{peak} and α_{residual} of aggregate-triaxial geogrids were 0.95.

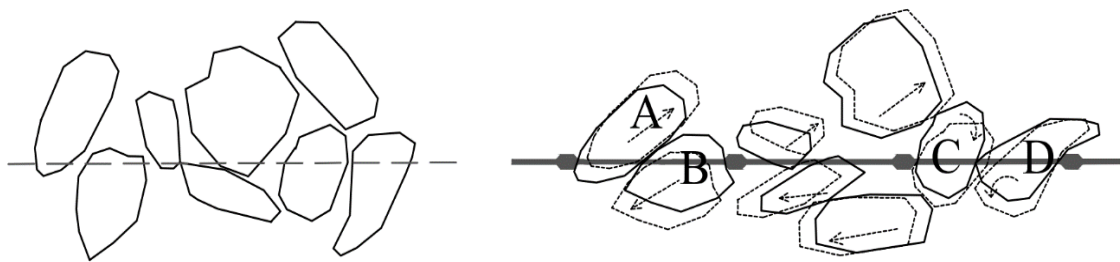
For AGG1 (12.5 - 19 mm), particles were small enough to pass through the apertures, and even three or four particles could pass together through apertures. However, the degree of particle interlocking was not improved, because the particles in the openings were not firmly confined with neighboring particles and geogrids. The loss of interlock was also confirmed by the weight loss results. The weight loss percentage was 6.25% and 4.11% at 100 kPa normal stress for unreinforced and biaxial geogrids reinforced aggregate, respectively. In the unreinforced aggregate, the strong interlocked particles experienced a high stress, and hence more particles were crushed into small particles. However, in the

reinforced aggregate, fewer particles were crushed because during the testing. As shown in Figure 3.10, particles A and B would slide against each other, or particles C and D would rotate reversely under a high stress. These all would cause a weak interlock between particles in the openings.

After the shear test, the deformation of the geogrids was not obvious compared to that in AGG2, indicating a weak interlocking at the interface. In addition, the relatively larger shear resistance of aggregate was replaced partly with the relatively smaller shear resistance between aggregate and smooth geogrid surface. Although the contact area of aggregate-geogrid of AGG1 was not as large as that of AGG3, it also had a negative effect on the shear resistance. Therefore, the peak and residual shear stress of aggregate-geogrid interfaces for both geogrids showed a lower value than those of aggregate-aggregate interface, resulting in low values of interfacial shear strength efficiency coefficient: 0.82 - 0.92.

3.5.5 Optimization of the ratio of equivalent aperture size to particle size

Researchers usually use a ratio of the equivalent aperture size to average aggregate particle size, A/D_{50} , to investigate aggregate particle size or aperture size effect on the interfacial



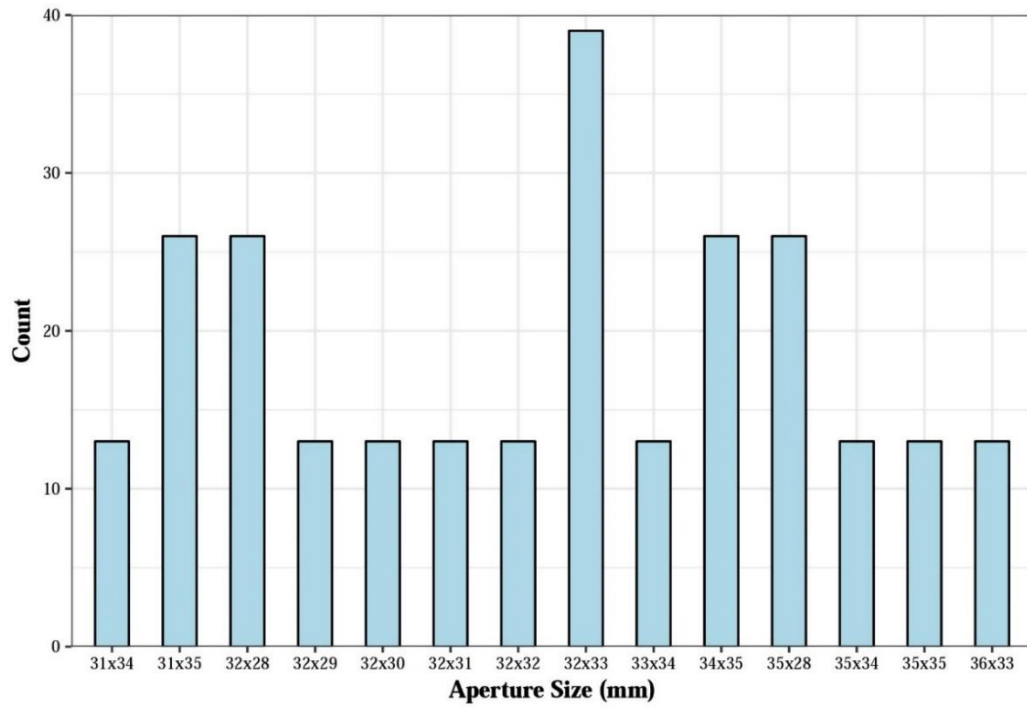
(a) Unreinforced aggregate

(b) Reinforced aggregate

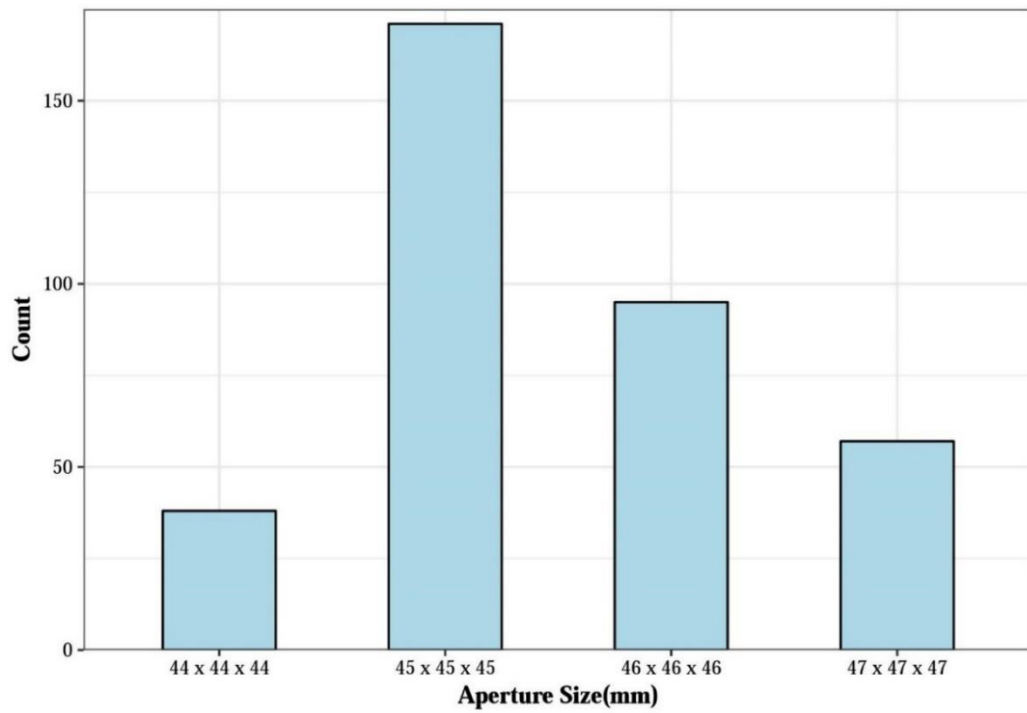
Figure 3.10. Interlocking mechanism of AGG1 (12.5 - 19 mm)

shear strength (55; 109; 119). The square root of the aperture area and the diameter of the inscribed circle were selected as the equivalent aperture size of biaxial and triaxial geogrids in this study, respectively. However, the size of aperture of each geogrid is not always consistent with the geogrid specification provided by the manufacturer, or even there exists an obvious difference between different apertures. Therefore, a piece of 71 cm by 48.5cm biaxial geogrid with 260 apertures and a piece of 75 cm by 47 cm triaxial geogrid with 361 apertures were used to determine the difference in aperture size. The aperture specification of biaxial geogrid provided by the manufacturer shows that each aperture area was 992 mm^2 ($32 \text{ mm} \times 31 \text{ mm}$). However, the minimum and maximum aperture areas were 896 mm^2 ($32 \text{ mm} \times 28 \text{ mm}$) and 1225 mm^2 ($35 \text{ mm} \times 35 \text{ mm}$), respectively. Figure 3.11 (a) shows the distribution of the areas of 260 apertures for biaxial geogrid. Based on the statistic descriptive parameter, the mean size of aperture was found to be $33 \text{ mm} \times 32 \text{ mm}$ instead of $32 \text{ mm} \times 31 \text{ mm}$. Therefore, the equivalent aperture size of the biaxial geogrid was selected as the square root of mean aperture area with a value of 32.5 mm. The aperture area difference of triaxial geogrid was not statistically significant, as presented in Figure 3.11 (b). The minimum and maximum aperture dimensions were $44 \text{ mm} \times 44 \text{ mm} \times 44 \text{ mm}$ and $47 \text{ mm} \times 47 \text{ mm} \times 47 \text{ mm}$. Based on the distribution percentage results, 47 mm was selected as the edge length of triangle instead of 46 mm provided by the manufacturer. Therefore, the equivalent aperture size of the triaxial geogrid was determined as 27.1 mm.

The use of D_{50} to characterize the effect of aggregate particle size on the shear behavior of aggregate-geogrid interface is questionable because D_{50} varies with the portions of all particle sizes and some sizes (e.g., too large or too small sizes) may not have an effective impact on the interlocking between aggregate and geogrids. Excessive fine particle or coarse particle percentage can significantly change the gradation curve of aggregate. However, too coarse or fine particles contribute little to interlocking between aggregate and geogrids. Therefore, single-sized particle was used in this study to determine the optimum ratio of equivalent aperture size to aggregate particle size. For biaxial geogrids, AGG2 (19 - 25 mm) showed the highest interfacial shear strength efficiency coefficient of the peak and residual shear resistances. Therefore, when A/D was in the range of 1.30 - 1.71, the interlocking in the apertures could be improved and the contact area between



(a) Biaxial geogrids



(b) Triaxial geogrids

Figure 3.11. Histogram of aperture size

particles and geogrids could be minimized, resulting in a strong interaction between particles and geogrids. For triaxial geogrids, similar results were found for AGG2 and the optimum ratio of A/D was in the range of 1.08 - 1.43. Although the aperture sizes of the biaxial and triaxial geogrids were similar (992 mm² and 956 mm²), their optimum ratio ranges were different. Brown et al. (118) and Indraratna et al. (119) reported 1.40 D₅₀ and 1.20 D₅₀ for biaxial geogrids, and the gradation of ballast used in their studies was closer to a single-sized aggregate, which shows that the optimum ratio range obtained from the current study was in general agreement with the results found in the literature.

3.6 Conclusions

In this study, a large-scale direct shear test was conducted to investigate the effect of particle size on the shear behavior of aggregate-geogrid interface. Three different single-sized aggregates, AGG1 (12.5 - 19 mm), AGG2 (19 - 25 mm) and AGG3 (25 - 37.5 mm) reinforced with two types of geogrids, biaxial and triaxial, were compared in terms of peak as well as residual shear stress, vertical displacement. The interfacial shear strength efficiency coefficient, α , was also determined based on the failure envelope lines of unreinforced and reinforced interfaces at different normal stress. The values of α ranged from 0.78 to 1.01 for the three aggregates used in the study and the aggregate-geogrid interface of AGG2 achieved the highest α among the three aggregates. It was found that the envelope line of unreinforced and reinforced AGG2 did not show significant difference. However, for AGG1 and AGG3, the envelope line of reinforced aggregate showed an obvious downward trend.

Three possible modes of aggregate-geogrid interface failure were proposed based on the results. The overall shear resistance of was aggregate-geogrid interface composed of two parts: 1) the shear resistance of interlocked particles in the openings of geogrids; 2) the shear resistance between particles and geogrid surface. Particle size had a significant influence on the magnitude and percentage composition of these two shear resistances. Too large particles like AGG3 would cause a loss of interlock at the interface and a relatively large aggregate-geogrid contact area, while too small particles may lead to a poor

interlocking within openings of geogrids. A stronger interaction would be generated only when the particles could pass through the geogrid and were confined in the geogrids.

It was found that AGG2 reinforced with both geogrids had better reinforcement effects than AGG1 and AGG3. For the biaxial geogrids, when the ratio of equivalent aperture size to aggregate particle size (A/D) was in the range of 1.30 - 1.71, the interlocking in the apertures could be improved and the contact area between particles and geogrids could be minimized. For the triaxial geogrids, the optimum ratio of A/D was in the range of 1.08 - 1.43.

This chapter presented the behavior of aggregate-geogrid interfaces through a series of direct shear tests. Further study was needed to investigate the behavior of dense gradation aggregate and determine the percentage of specific particle size (such as AGG2) in a dense gradation, because only particles of the specific sizes could be reinforced with geogrid effectively.

3.7 Acknowledgements

The first author would like to acknowledge China Scholarship Council for the financial support (No. 201506260031).

CHAPTER FOUR
LABORATORY INVESTIGATION OF CRITICAL FACTORS
AFFECTING GEOGRID REINFORCEMENT ON AGGREGATE
USING LOADED WHEEL TESTER

A version of this chapter is in the process of being submitted for publication by Bingye Han, Pawel Polaczyk, Yuetan Ma, Miaomiao Zhang, Xiaoqi Lu, Fulu Wei and Baoshan Huang:

“Laboratory investigation of critical factors affecting geogrid reinforcement on aggregate base using loaded wheel tester.” Submitted to *Journal of Transportation Engineering Part B: Pavements*.

Bingye Han’s contribution was conducting all literature review, data analysis, and writing the text contained in the manuscript. Dr. Baoshan Huang provided guidance throughout the research process as well as editorial assistance.

4.1 Abstract

A modified loaded wheel tester, Asphalt Pavement Analyzer (APA), was employed to evaluate the reinforcement effect of geogrids on unbound granular base course materials by applying simulated repeated wheel loads on the compacted reinforced specimen. Three indicators, traffic benefit ratio (TBR), rutting reduction rate (RRR), and rate of deformation (ROD), were used to characterize the permanent deformation behavior of reinforced aggregate with the number of loading repetitions. Three critical influence factors, the gradation of aggregate base, the tensile strength of geogrids, and the aperture geometry of geogrids, were analyzed for the reinforcement effect. The results show that inclusion of geogrids in unbound granular base materials could significantly improve their resistance to rutting. A better reinforcement effect could be obtained by increasing the portion of effective aggregate interacting positively with geogrids, increasing the tensile strength of geogrids. The reinforcement effect of triaxial geogrids was better than that of biaxial geogrids with a similar tensile strength. The tensile strength of geogrid had much significant effects on the reinforcement as compared to the aperture geometry.

4.2 Introduction

Geogrids have been widely used for base reinforcement and subgrade stabilization in flexible pavements since 1970s (6; 126). Placed at the base-subgrade interface or within the base course, geogrids could improve the pavement performance due to the benefits of

decreasing rutting deformations, increasing pavement-bearing capacity, controlling crack propagation, reducing base course thickness, and prolonging pavement service life (3; 11; 21; 24; 29; 62; 127; 128). Interaction between geogrids and aggregates under repeated traffic loading plays a significant role in implementing these mechanisms, and it is influenced by lots of factors, including geogrid properties (geogrid type, aperture size, tensile strength, junction strength etc.), aggregate properties (aggregate type, compaction degree, water content, particle size etc.), and the matching properties of geogrid and aggregate (relative size of geogrid aperture to aggregate, geogrid placement position etc.).

Several above-mentioned critical influence factors were considered in pullout and direct shear tests, and their impacts on the reinforcement effect were also evaluated (49-55). However, these two methods cannot simulate traffic load, so the shear mode in these methods is different from that under repeated wheel loading. In order to simulate the repeated traffic loading condition, Cuelho and Perkins (2005) explored the reinforced aggregate by cyclic pullout test to identify the impacts of different geogrids on reinforcement effects (92). However, test results show that the reliability of the method is not good, and there is a large dispersion between the measured and predicted results in some reinforced aggregate. Researcher also used the cyclic triaxial test to analyze the influences of geogrid and aggregate properties on the reinforcement effect. Yang and Han (2012) believe that the geogrid can provide additional lateral restraint stress on aggregates, which can improve the resilient modulus of aggregates (40). Rahman et al. (2013) confirmed this conclusion through a group of cyclic triaxial tests on construction and demolition materials (41). However, several other researchers' results show that the resilient modulus does not increase with the inclusion of different geogrids (43-46). Therefore, the cyclic triaxial test cannot effectively quantify the interaction between geogrids and aggregates, nor can it evaluate the impacts from different influence factors on the reinforcement effect. Cyclic plate loading tests, Falling Weight Deflectometer (FWD) tests, accelerated pavement tests, and field trafficking tests are excellent for evaluating the geogrid confinement effects on aggregate base course in the flexible pavement system, but the influence factors considered in the test are limited since the test methods are time-consuming, labor-consuming, and costly. Also, the facilities are only available in limited

universities and research institutes. To effectively quantify the reinforcement effect of geogrids and comprehensively evaluate the influence factors affecting the reinforcement effect, a readily available laboratory test is needed (48; 69; 70; 129-133).

Han et al. (2011) proposed a test method involving the use of a commonly available loaded wheel tester, Asphalt Pavement Analyzer (APA), to evaluate the benefits of geosynthetic reinforcement in aggregate base course (74). Wu et al. (2015) further validated the reliability of this method by comparing with a cyclic plate loading test (75). The modified APA test has proven to be an effective and fast way to investigate the critical influence factors which can affect the reinforcement effect of geogrids on aggregate and identify the importance of each factor. However, both of research groups focused on verifying the effectiveness and practicality of this method, thus limited experiments were conducted on investigating the critical influence factors. In terms of the design method for the geogrid-reinforced pavement, the essential work is to explore what combination of geogrids and aggregate can maximize the reinforcement effect. However, the current design method is based on engineering experiences, and several critical influence factors are not clearly identified for pavement design (39). Based on the results from previous laboratory and field tests, three critical influence factors, the gradation of aggregate base, the tensile strength of geogrids, and aperture geometry of geogrids, play an important role in controlling the performance of the geogrid-reinforced pavement.

4.3 Objective and scope

The objective of the present study was to investigate and quantify the above-mentioned three influencing factors under repeated traffic load to further guide the geogrid-reinforced pavement design. To achieve this objective, the laboratory scaled loaded wheel tester, APA and testing protocols established by Han et al. (2011) and Wu et al. (2015) were considered with four types of geogrids with different tensile strength and aperture shape combining with two types of aggregate gradations (74; 75).

4.4 Test program

4.4.1 Test equipment

The standard APA was modified to describe the permanent deformation of reinforced aggregate in this study. The equipment consisted of three sets of detachable rubber hoses and steel wheels, as shown in Figure 4.1. The rubber hoses could be inflated to simulate a tire pressure in the range of 0 - 827 kPa for the pavement. The steel wheels were groove-shaped, and they could move back and forth along the rubber hoses, interacting with the rubber hoses to simulate moving loads applied on pavements. The loading test could be run in a chamber with a controllable temperature and humidity to eliminate the impact of different environmental factors on the test result. The device was equipped with a built-in intelligent operation and data acquisition system. Several significant test parameters, chamber temperature, rubber hose pressure, wheel load, loading cycles, loading frequency, could be controlled precisely by test programs. The vertical deformation was the most important indicator for the geogrid reinforcement evaluation in this study. Unlike traditional APA devices manually measuring the vertical deformation, this APA device used in the study was intelligent and could automatically record the deformation data for a specimen surface along the wheel path since each axle of the steel wheel was provided with a displacement sensor. There are five positions for deformation measurement along the wheel path, and their average was used to plot the development of permanent deformation with number of loading cycles. The total time for a complete rutting test was 2 hours and 18 minutes (8,000 cycles). The original molds for APA were three separated boxes to contain asphalt mix samples, and the space of each box was too small for reinforced aggregate. Therefore, a modified aluminum test box with a dimension of 533 mm × 380 mm × 115 mm was manufactured to hold base course materials as shown in Figure 4.1.

4.4.2 Materials

4.4.2.1 Geogrid

Four different types of geogrids, GG1, GG2, GG3, and GG4, were selected in this study (Figure 4.2). GG1 and GG2 were biaxial geogrids, and their openings were squares with



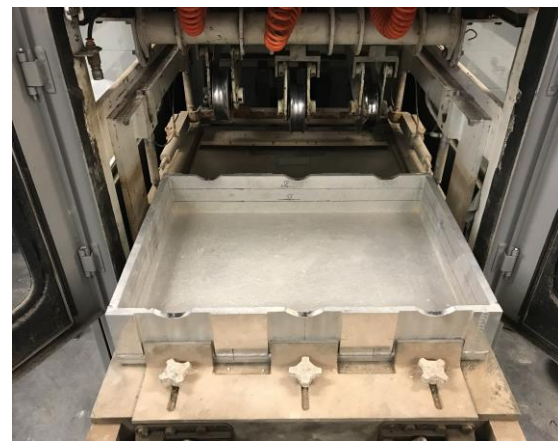
(a) Tester chamber and platform



(b) Loading wheels and rubber hoses



(c) Traditional asphalt mix mold



(d) Modified base materials mold

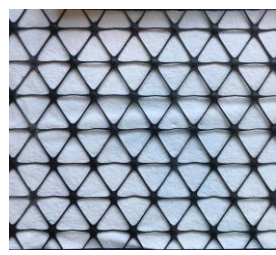
Figure 4.1. Photo of asphalt pavement analyzer (APA)



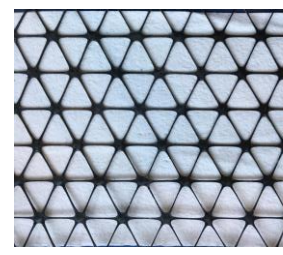
(a) GG1



(b) GG2



(c) GG3



(d) GG4

Figure 4.2. Geogrids used for testing

a side length of 25 mm. GG3 and GG4 were triaxial geogrids, and their openings are equilateral triangles with a side length of 46 mm. The diameter of the inscribed circle of this equilateral triangle was 26 mm, which was close to the side length of the biaxial geogrids. GG2, GG3 and GG4 were punched-drawn polypropylene geogrids, which possessed relatively large tensile strength and thick ribs. GG1 was woven polyethylene geogrid with a relatively small tensile strength. The fundamental properties of the four geogrids provided by the manufacturer are summarized in Table 4.1.

4.4.2.2 Base course material

The base course material used in the study was unbound granular materials consisting of aggregates passing different sizes. It is found that the interaction between geogrids and aggregates could be maximized if the ratio of the geogrid aperture size to the aggregate size, A/D , is appropriate. According to the previous study, the appropriate A/D ranges for biaxial and triaxial geogrids were 1.30 - 1.71 and 1.08 - 1.43, respectively (Han, Ling, Shu, Gong and Huang, 2018). Aggregates in this range were called the effective aggregates. To analysis the particle size effect of aggregates on the geogrid reinforcement, two types of base course material with different gradations, dense-graded aggregates and open-graded aggregates, were investigated in the study as shown in Figure 4.3 (a). The particle size distributions of the two aggregates are shown in Figure 4.3 (b). The dense-graded aggregate was one commonly used aggregate base material in Tennessee, and was classified as gravel well-graded (GW) according to ASTM D2487. The open-graded aggregate was adjusted based on the dense-graded aggregate, and was classified as gravel poor-graded (GP), which increased the portion of the effective aggregates. Based on the above-mentioned conclusion, the optimum ranges of the effective aggregates for biaxial and triaxial geogrids used in this study were 14.6 - 19.2 mm and 18.2 - 24.1 mm, respectively. However, to eliminate the size effect of the aluminum box on the test results, the minimum dimension of the box, the height, should be more than five times than the maximum particle size of the base course material. Therefore, the large aggregates (> 19 mm) in both dense- and open-graded materials were removed. In combination with the mesh size commonly used, the final range of the effective aggregates for both the biaxial and triaxial geogrids was 12.5 - 19 mm.

Table 4.1. Mechanical and physical properties of geogrids

Property	Geogrid			
	GG1	GG2	GG3	GG4
Aperture shape	Square	Square	Triangle	Triangle
<u>Aperture size</u> (mm) MD × CMD	25 × 25	25 × 25	46 × 46 × 46	46 × 46 × 46
<u>Tensile strength</u> ^{a,b} (kN/m) MD / CMD	8 / 8	14 / 14	8.6 / 8.6	9.5 / 9.5
<u>Tensile modulus</u> ^{a,b} (kN/m) MD / CMD	400 / 400	700 / 700	430 / 430	475 / 475
Manufacturing method	Punched-drawn	Woven	Punched-drawn	Punched-drawn
Material	Polyethylene	Polypropylene	Polypropylene	Polypropylene

^a at 2% strain;

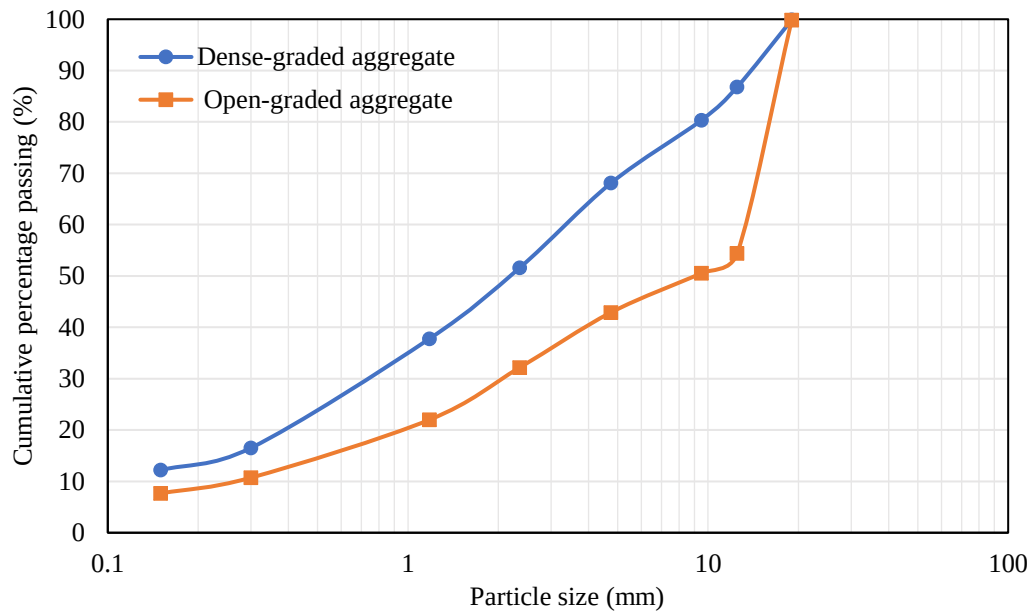
^b MD = Machine direction; CMD= Cross machine direction.



(a) Dense-graded aggregate



(b) Open-graded aggregate



(c) Particle size distribution of base course materials

Figure 4.3. Base course materials used for testing

4.4.3 Test setup and procedures

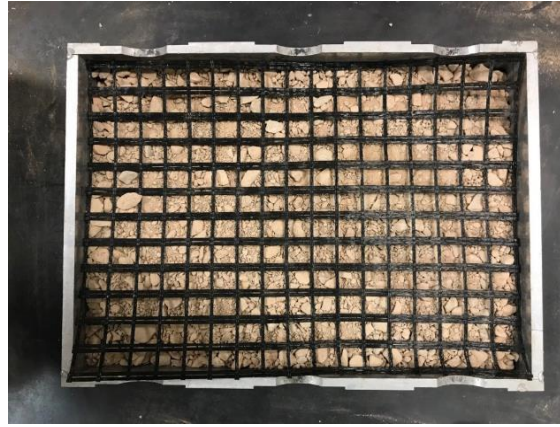
In this study, the effect of moisture content on geogrid-aggregate was not considered, so all samples were prepared in a dry condition. In order to achieve a uniform compaction degree, the aggregates were compacted in four layers. In each layer, the mass of aggregates was predetermined, and this amount of aggregates were compacted manually to the target line on the box to achieve a target density (80% relative density). Trial tests showed that the placement position of the geogrid had a significant effect on geogrid reinforcement in aggregates. If the geogrid was placed 35 mm below the base surface, the reinforcement effect was not obvious. If the geogrid was buried shallowly, the geogrid could be easily exposed to the air during the loading. In this study, the geogrid was placed 25 mm below the base course surface. The main procedures of the test setup are present in Figure 4.4. After the three layers of aggregates were compacted, the geogrid was placed on the top of the third layer of the granular materials, then the last layer of the granular materials was filled and compacted. After all aggregates were compacted, the specimen was ready for APA testing. The APA testing was conducted for 8,000 loading cycles at a frequency of 1 Hz. The wheel load used for the test was 356 N, and the rubber hose pressure was set as 552 kPa in accordance with the parameters of Han et al. (74).

4.4.4 Repeatability tests

Three repeated tests were conducted firstly on the same unreinforced dense-graded aggregates to evaluate the repeatability of test results. The equipment itself automatically recorded the deformation of the aggregates in each load cycle, resulting in a large amount of collected data when the testing finished. In order to display the data more clearly, only the deformations at specific number of load cycles were selected. The development curve of the rut depths with the number of load cycles and statistical parameters for the three repeated tests are presented in Figure 4.5. The statistical analysis showed that the three curves were similar. The Tukey-Honestly index was used to identify and quantify the potential difference between the three repeated tests. The 95% confidence interval was [-0.4084, 0.0250] which contained 0, and the p-value was 0.086 which was greater than 0.05,



(a) Base course materials compaction



(b) Geogrid placement



(c) Prepared specimen



(d) APA testing

Figure 4.4. Preparation and testing of the specimen in APA testing

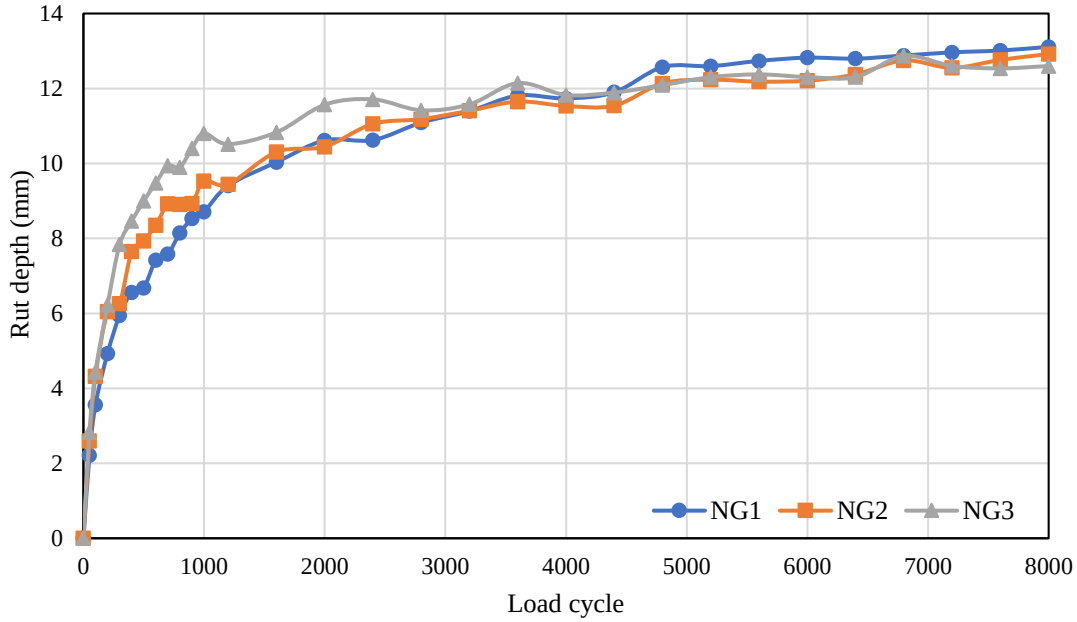


Figure 4.5. Repeatability test results for unreinforced samples

indicating that there was no significant difference between the three tests. Therefore, the test results had a good repeatability.

4.4.5 Evaluation indicator

To evaluate benefits of geogrid reinforcement on the rutting resistance of the reinforced aggregate, three indicators, Rutting Reduction Ratio (RRR), Traffic Benefit Ratio (TBR), and Ratio of Deflection (ROD), were employed to characterize the permanent deformation of the reinforced and unreinforced aggregate.

4.4.5.1 Rutting Reduction Ratio (RRR)

RRR is defined as the ratio of the rut depth of the reinforced aggregate to that of the unreinforced aggregate at the same number of load cycles, and can be determined by the Eq. 4.1. The specimen with a lower RRR value usually represented a better rutting resistance performance.

$$RRR = \frac{d_{reinforced}}{d_{unreinforced}} \quad (\text{Eq. 4.1})$$

where, $d_{reinforced}$ and $d_{unreinforced}$ are the rut depth of the reinforced and unreinforced aggregate at a specific load cycle, respectively. The cumulative rut depth at the terminal cycle (8,000) was used to calculate the RRR.

4.4.5.2 Traffic Benefit Ratio (TBR)

TBR is defined as the ratio of the required number of load cycles of the reinforced aggregate to that of the unreinforced aggregate when the same rut depth is reached in both specimens, as shown in Eq. 4.2.

$$TBR = \frac{N_{reinforced}}{N_{unreinforced}} \quad (\text{Eq. 4.2})$$

where, $N_{reinforced}$ and $N_{unreinforced}$ are the number of load cycles for the reinforced and unreinforced aggregated, respectively. The threshold of rut depth used to calculate the TBR is selected at which the rut depth tended to be stable. TBR is used to reflect the additional load cycles when the geogrid is used to reinforce the base course material, and a larger TBR value usually represents a better reinforcement effect.

4.4.5.3 Rate of deformation (ROD)

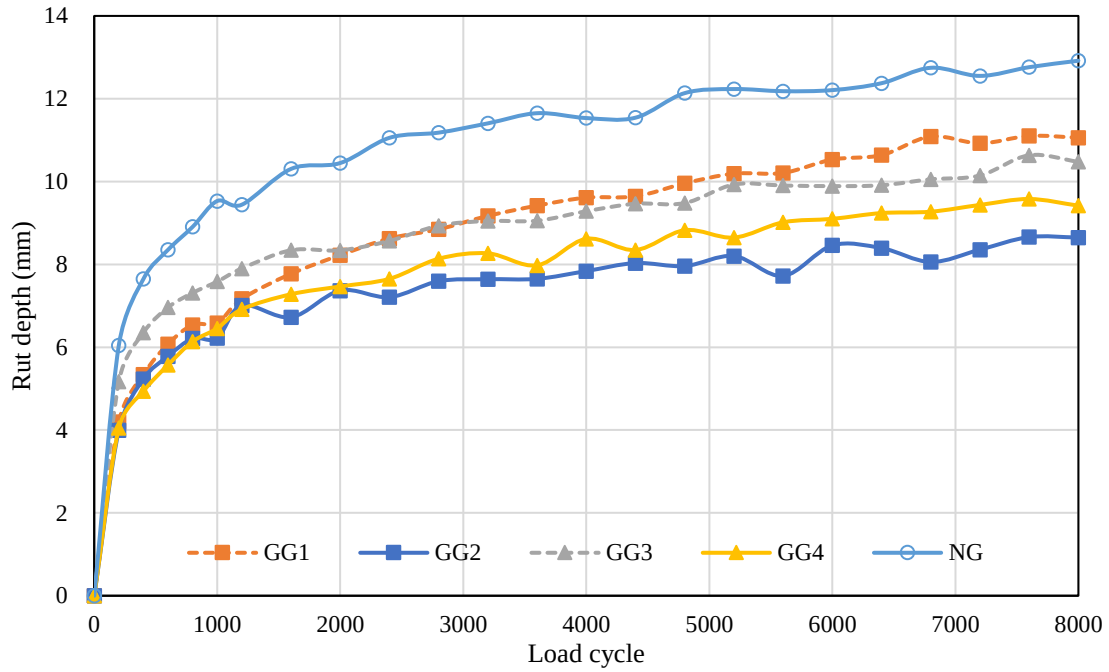
ROD is the development rate of permanent deformation, and can be calculated by the Eq. 4.3. ROD can reflect the rut resistance of the reinforced and unreinforced aggregate, and a lower ROD value means a stronger rut resistance for a specimen.

$$ROD = \frac{d_{n+1} - d_n}{t_{n+1} - t_n} \quad (\text{Eq. 4.3})$$

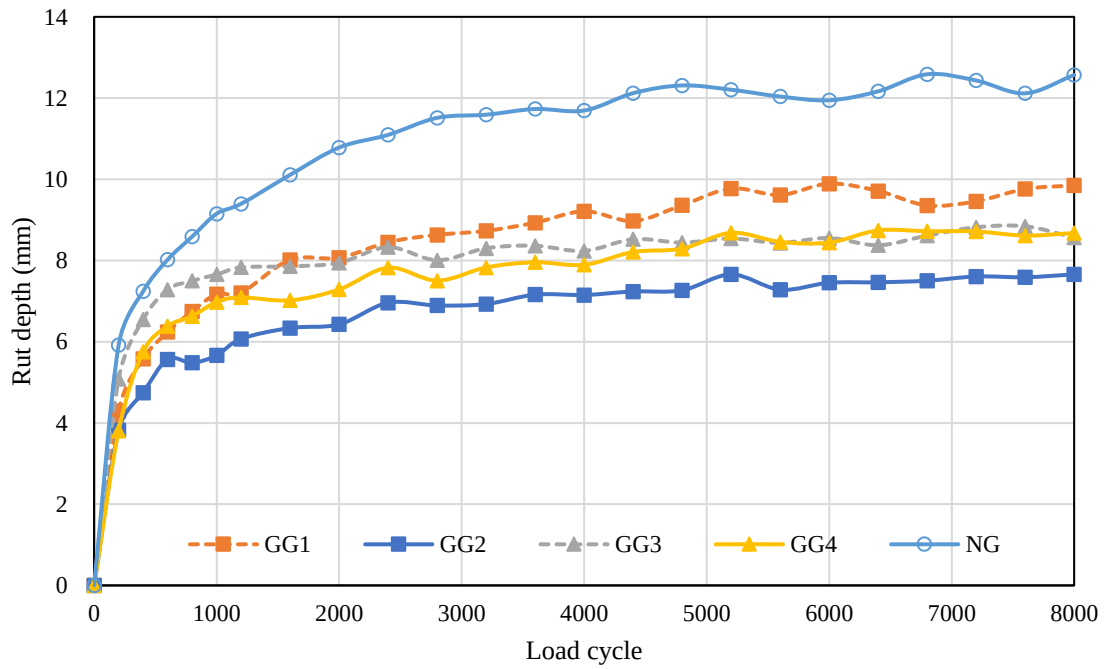
where, d_n and t_n are the permanent deformation of the specimen and the testing time of the nth cycle, respectively.

4.5 Test results and analysis

The data acquisition system automatically recorded lots of permanent deformation data during the numerous load cycles. To facilitate the analysis, the study selected several permanent deformations at specific load cycles. Figure 4.6 presents the results of dense- and open-graded aggregate in unreinforced and reinforced conditions. The RRR, TBR and ROD values were calculated based on the rut depth of the unreinforced and reinforced



(a) Dense-graded aggregate



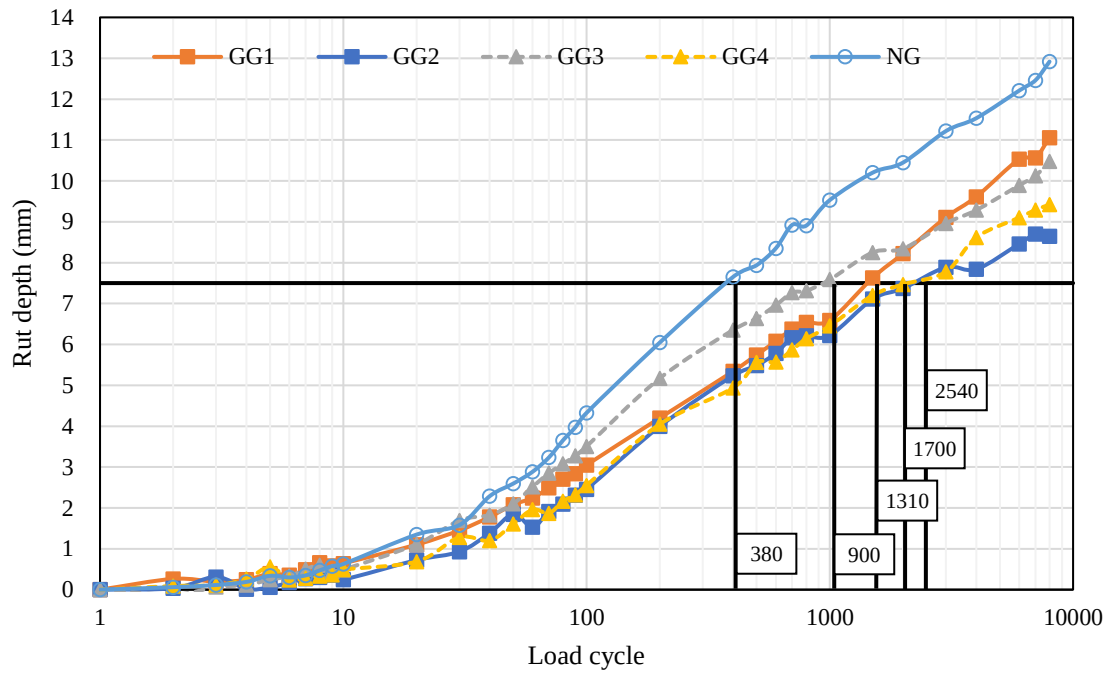
(b) Open-graded aggregate

Figure 4.6. Rut depth results from APA testing

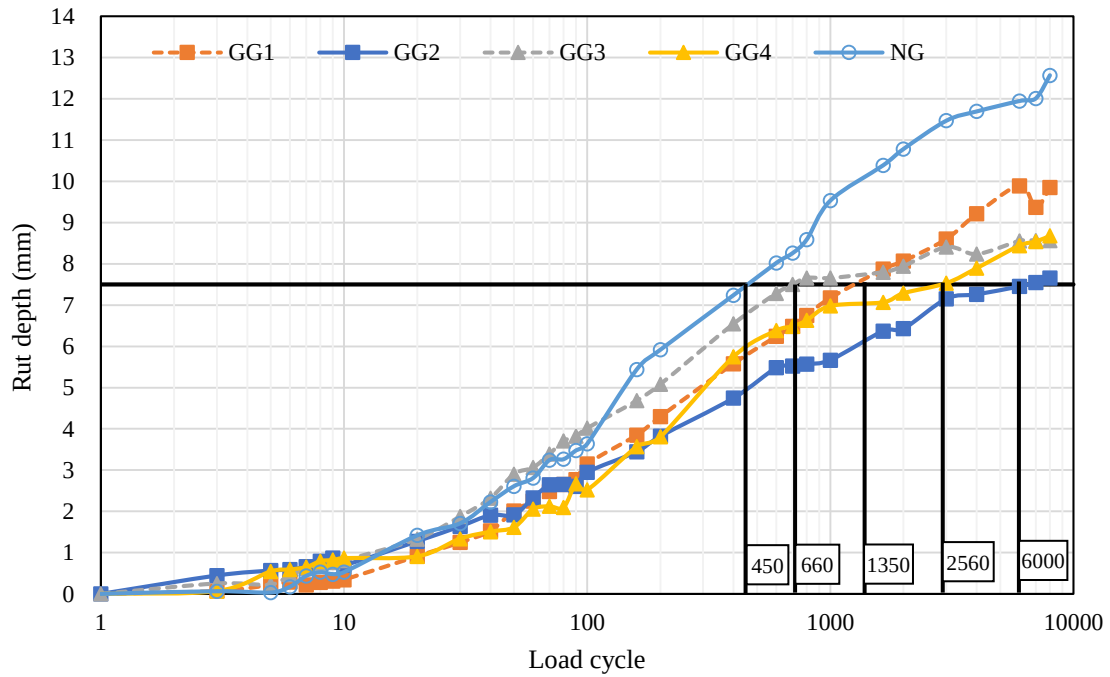
aggregate to quantify the reinforcement effect of the geogrid. The results show that the rut depth in both unreinforced and reinforced aggregate grew gradually as the number of load cycle increased. In the first 2,000 load cycles, the rut depth showed a rapid growth trend, but the growth rate slowed down obviously when the rut depth reached a certain amount of accumulated rutting after a certain number of load cycles.

Figure 4.7 is an illustration diagram for TBR calculation for each reinforced specimen in both dense- and open-graded aggregate based on the APA testing results. It can be seen from Figure 4.7 that the threshold selection of rut depth had a great influence on the results of the TBR. To facilitate comparison, the rut depth (7.5 mm) at the end of the test in GG2-reinforced aggregate was selected as the threshold in both dense- and open-graded aggregate. According to the predetermined threshold, the number of load cycles in each specimen was determined when the rut depth reached to the threshold, and the corresponding load cycles were marked in Figure 4.7. The TBRs were calculated based on Eq. 4.2, and the calculation results are presented in Table 4.2. These TBR values can be used to characterize the permanent deformation of reinforced aggregate and evaluate different influence factors affecting the reinforcement effect of geosynthetic inclusions on the base course material.

Figure 4.8 shows the vertical permanent deformation rate of unreinforced and reinforced aggregate. It can be found from the test results that the deformation rate of geogrid-reinforced aggregate was significantly smaller than that of unreinforced aggregate. This trend was found throughout the loading process, and the gap was more apparent in the early stages of the testing. The deformation rate gradually decreased as the number of load cycles increased. The deformation rate among all the reinforced specimens tended to be consistent and stabilized at a small level at the end of the testing, and the deformation rate of reinforced specimen was still lower than that of unreinforced specimen at the end of the test. In the first 2,000 load cycles, the deformation of all specimens increased rapidly and tended to a certain amount of deformation. After 2,000 load cycles, it can be clearly found that the order of ROD of all specimens from large to small was $GG1 > GG3 > GG4 > GG2$ in both dense- and open-graded aggregate. Placing geogrids into aggregate could improve

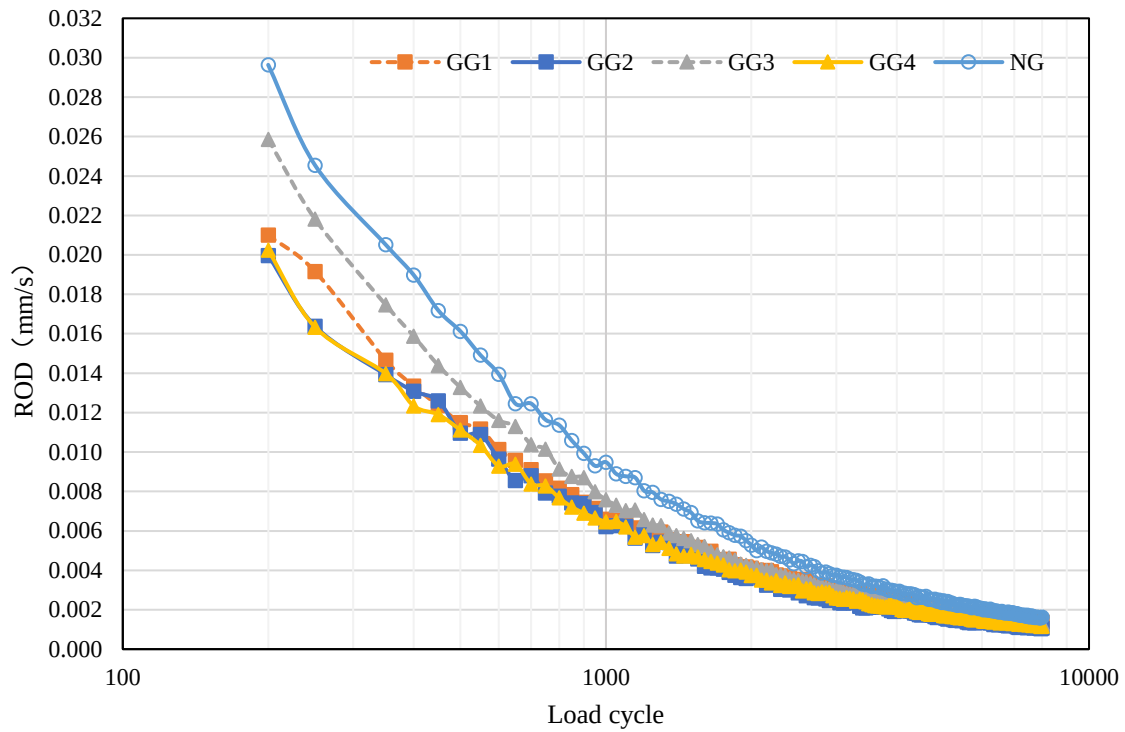


(a) Dense-graded aggregate

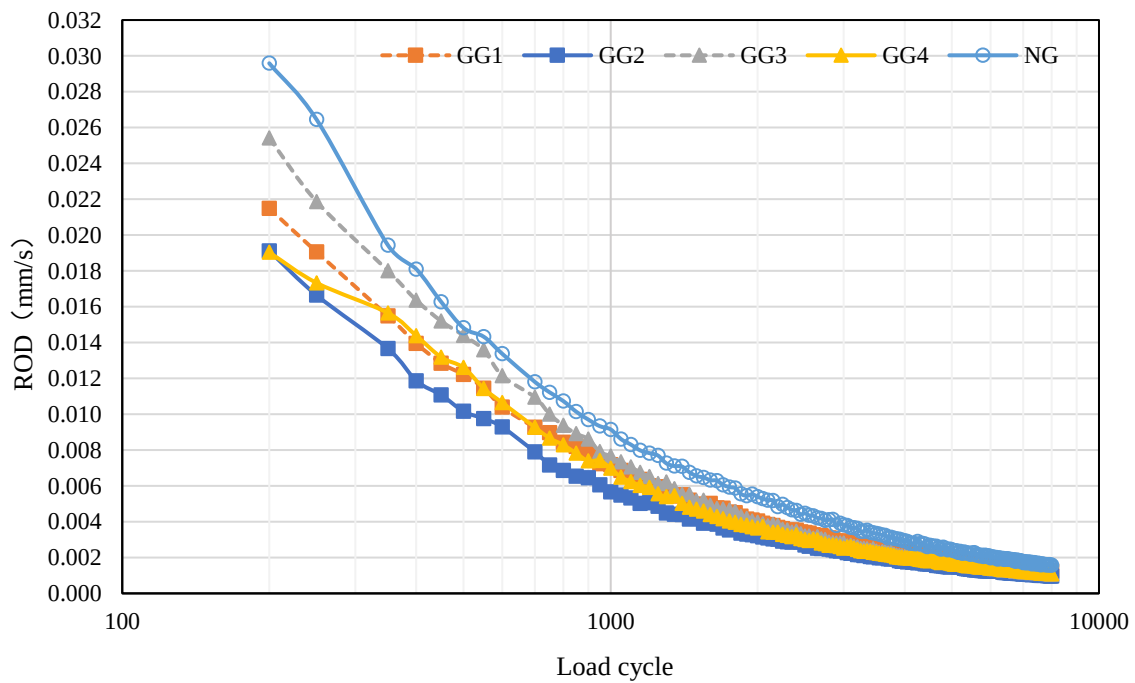


(b) Open-graded aggregate

Figure 4.7. Illustration diagram for TBR calculation and result



(a) Dense-graded aggregate



(b) Open-graded aggregate

Figure 4.8. Rate of deformation results from APA testing

the anti-rutting performance of base course materials, and different geogrids interacting with the two types of aggregate could exhibited different reinforcement effect.

Table 4.2 summarizes the results of the APA testing, including the results of terminal rut depth, RRR, TBR and ROD. Both values of the rut depth, RRR and ROD at the terminal cycle (the 8,000th cycle) were used for comparing the reinforcement effect. The results show that the TBR were generally consistent with RRR and ROD in the reinforcement effect evaluation. The specimen with a higher TBR usually possessed a lower RRR and ROD, indicating a better reinforcement of geogrids on base course materials. However, compared with rut depth, RRR and ROD obtained at the end of the testing, TBR had a higher volatility since TBR was strongly correlated with the selected threshold of rut depths. Therefore, when evaluating the reinforcement effect of geogrids, the three indicators, RRR, TBR, and ROD, should be considered comprehensively. Compared to the non-geogrid reinforced aggregate (NG), all the reinforced aggregate showed a noticeable improvement in rutting resistance, and all geogrids showed a better reinforcement in the specially designed open-graded aggregate. The results from dense-graded aggregate were generally in agreement with results from the open-graded aggregate, and the order of reinforcement effect of these four geogrids from good to bad was GG2 > GG4 > GG3 > GG1. It was worthwhile to pointed out that although the short-term reinforcement effect of GG1 was better than that of GG3 due to the higher TBR of GG1, the long-term reinforcement effect of GG3 was better because the rut depth and ROD of GG1-reinforced aggregate were lower than those of GG3-reinforced aggregate after 2,000 load cycles.

4.5.1 Effect of aggregate base gradation

It can be found from Figure 4.9 that the rut depth of unreinforced dense-graded aggregate at the terminal load cycle was close to that of unreinforced open-graded aggregate. However, for the same geogrid used to reinforce this granular base course material, the rut depth of the reinforced open-graded aggregate was significantly lower than that of the reinforced dense-graded aggregate. The rutting resistance of each specimen from good to bad was ranked in the following order: reinforced open-graded aggregate > reinforced dense-graded aggregate > unreinforced open-graded aggregate $\approx$ unreinforced dense-

Table 4.2. APA testing results

Gradation	Specimen	Evaluation indicator			
		Rut depth (mm)	RRR	TBR	ROD (10 ⁻³ mm/s)
Dense	NG	12.92	1.00	1.00	1.62
	GG1	11.06	0.86	3.45	1.38
	GG2	8.65	0.67	6.68	1.08
	GG3	10.48	0.81	2.37	1.31
	GG4	9.42	0.73	4.47	1.18
Open	NG	12.57	1.00	1.00	1.57
	GG1	9.85	0.78	3.00	1.23
	GG2	7.66	0.61	13.33	0.96
	GG3	8.57	0.68	1.47	1.08
	GG4	8.68	0.69	5.69	1.08



(a) GG2-reinforced dense-graded aggregate

(b) GG2-reinforced open-graded aggregate

Figure 4.9. Reinforced aggregate after testing

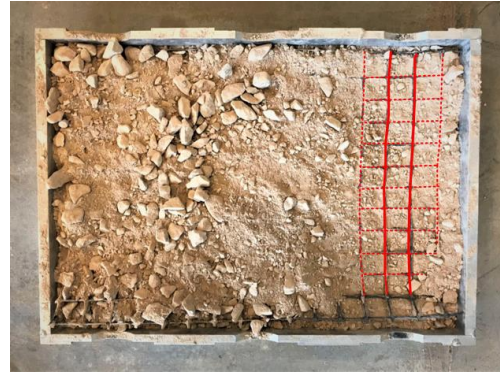
graded aggregate. This order was confirmed in all types of geogrids used in this study. When the four geogrids, GG1, GG2, GG3 and GG4, were used to reinforce the dense-graded aggregate, the accumulated rut depth of corresponding reinforced specimen at the end of the testing was 11.06 mm, 8.65 mm, 10.48 mm and 9.42 mm, respectively. The rut depths lowered to 9.85 mm, 7.66 mm, 8.57 mm and 8.68 mm when the geogrids were used to reinforce the open-graded aggregate. Both the biaxial and triaxial geogrids exhibited a better rutting resistance performance in the open-graded aggregate. The portion of the effective aggregate was different in dense- and open-graded aggregate, as shown in Figure 4.2 (c). The effective aggregate whose diameter was in the range of 12.5 - 19 mm accounted for 13% of all granular material mass in the dense-graded aggregate. The mass percentage of the effective aggregate increased to 53% in the open-graded aggregate. The results demonstrated that the relative size of the geogrid aperture size to aggregate size had a significant impact on the reinforcement effect of geogrids. Increasing the portion of effective aggregate of the granular base course materials would strengthen the interaction between geogrids and aggregates, which could further improve the rutting resistance of the reinforced aggregate.

4.5.2 Effect of geogrid tensile strength

The test results present that the geogrid tensile strength was another important influence factor affecting the rutting resistance of the reinforced aggregate. In both biaxial and triaxial geogrids, the specimen reinforced by the geogrid with a higher tensile strength showed a better anti-rutting ability. The tensile strength of biaxial geogrid GG2 and triaxial geogrid GG4 was higher than that of biaxial geogrid GG1 and triaxial geogrid GG3, respectively. The APA testing results show that the rut depth of GG2-reinforced specimen was lower than that of GG1-reinforced specimen in both dense- and open-graded aggregate. Similar conclusion was obtained in the GG3- and GG4-reinforced specimens. Figure 4.10 exhibits the deformation of GG1 and GG2 in the reinforced dense-graded aggregate after testing. GG1 bore a considerable part of the tensile stress transmitted from aggregates during the loading process. The geogrid itself experienced a large deformation after a long-term repeated load cycles due to its relatively small tensile strength. In such a condition,



(a) Deformation of GG1



(b) Deformation of GG2

Figure 4.10. Deformation of biaxial geogrids in dense-graded aggregate

the lateral confinement effect of the geogrid on aggregates was limited, so aggregates would still experience a relatively large lateral movement. Compared to GG1, GG2 had a higher tensile strength and could withstand more tensile stress, which could effectively limit the lateral movement of aggregates. Therefore, the anti-rutting ability of GG2-reinforced aggregate was better than that of GG1 reinforced aggregate. For the triaxial geogrids, similar results were also found in the dense-graded aggregate, but the difference was not obvious in the open-graded aggregate. In the early stage of test for open-graded aggregate, the rut depth of the GG3-reinforced aggregate was much larger than that of the GG4-reinforced aggregate, but the difference gradually became smaller as the number of load cycles continued to increase. It was worthwhile to point out that the GG2-reinforced aggregate performed best in resisting rutting among all reinforced aggregate, which indicated that the geogrid tensile strength should be considered as higher priority factor contributing to the reinforcement compared to the aperture geometry of the geogrid.

4.5.3 Effect of geogrid aperture geometry

The geogrid aperture geometry was also a significant factor which can impact its reinforcement effect. The tensile strength of biaxial geogrid GG1 and triaxial geogrid GG3 was similar, as shown in Table 4.1. The tensile strength of GG1 and GG3 at 2% strain was 8 kN/m and 8.6 kN/m, respectively. The test results illustrate that in both dense- and open-

graded aggregate, the rut depth of GG3-reinforced aggregate was lower than that of GG1-reinforced aggregate, indicating that the reinforcement of the triaxial geogrid was better than that of the biaxial geogrid when their tensile strengths were similar. Dong et al. (2011) believed that biaxial geogrids were subjected to tensile forces from different directions, and the tensile strength of geogrids was not uniform along geogrid ribs (134). The tensile strength along the machine direction was higher than that along the cross-machine direction. Compare with biaxial geogrids, triaxial geogrids had a more stable grid structure and the tensile stress could be distributed uniformly along all directions. The geogrid placed in base course was often subjected to stress from various directions, so the use of triaxial geogrids was beneficial to strengthen the interlocking effect between the aggregates, improve the interaction between geogrid and aggregates, and improve the stress distribution of the geogrid itself, resulting in a more pronounced reinforcement effect than the biaxial geogrid. It is also found that the interaction degree between triaxial geogrids and aggregates was higher than that between biaxial geogrids and aggregates at the same stress condition from the cyclic shear test results (5). Several researchers have also confirmed this conclusion (135; 136).

4.6 Conclusion

In this study, laboratory experiments were conducted to evaluate the reinforcement effects of geogrids in different granular base course materials by using a modified loaded wheel tester, APA. Three indicators, traffic benefit ratio (TBR), rutting reduction rate (RRR), and rate of deformation (ROD), were used to characterize the permanent deformation behavior of the reinforced aggregate with number of load repetitions. Influence factors like aggregate gradation, geogrid tensile strength and geogrid aperture geometry were analyzed for the reinforcement effect. Based on the laboratory results, conclusions could be drawn from the study as follows:

The modified APA testing was able to effectively characterize the permanent deformation behavior of geogrid-reinforced base course materials with the number of load cycles, and reasonably distinguish the effect of geogrid-aggregate interaction among all the

geogrids investigated in this study. The multiple test results show that this newly developed method had a good repeatability.

Placing geogrids in both of dense- and open-graded aggregate could significantly improve the ability of aggregate to resist permanent deformation. The test results show that the deformation rate of geogrid-reinforced aggregate was significantly smaller than that of unreinforced aggregate. The accumulated rut depth of reinforced aggregate at the terminal cycle (8,000th) was 0.61 – 0.86 times the rut depth of the unreinforced aggregate. It can be found from the TBR results that the required load cycle for reinforced aggregate was 1.74 – 13.33 times the number of the unreinforced aggregate, indicating that reinforced aggregate base could carry more loadings and prolong the pavement service life.

Comparing different reinforced aggregate specimens, it is found that the relative size of the geogrid aperture size to aggregate size had a significant impact on the reinforcement effect of geogrids. Increasing the portion of the effective aggregate which was able to interact positively with geogrids could strengthen the reinforcement effect of geogrids. The geogrid tensile strength was also an important influence factor affecting the reinforcement effect of geogrids. The geogrid with higher tensile strength could better restrain the lateral movement of aggregates in the geogrid aperture, strengthen their interlocking effect, and improve their resistance to permanent deformation. In addition, triaxial geogrids performed better in reinforcing aggregate than biaxial geogrids when the tensile strength of these two geogrids were equivalent, but it was still worthwhile to point out that geogrid tensile strength should be considered as a higher priority factor compared to the aperture geometry of geogrids when selecting geogrids.

4.7 Acknowledgements

The authors would like to thank and acknowledge the Tennessee Department of Transportation for their financial support. The authors also would like to thank Dr. Jianming Ling from Tongji University and Dr. Hao Wu from Central South University for their excellent technical support during this study.

CHAPTER FIVE
ACCELERATED PAVEMENT TESTING TO EVALUATE THE
GEOGRID REINFORCEMENT IN FLEXIBLE PAVEMENTS

A version of this chapter is in the process of being submitted for publication by Bingye Han, Pawel Polaczyk, Rong Ma, Yuetan Ma, Hongren Gong, Fulu Wei and Baoshan Huang: “Accelerated pavement testing to evaluate the geogrid reinforcement in flexible pavement structures” Submitted to *Transportation Research Record*.

Bingye Han’s contribution was conducting all literature review, data analysis, and writing the text contained in the manuscript. Dr. Baoshan Huang provided guidance throughout the research process as well as editorial assistance.

5.1 Abstract

This chapter presents an accelerated pavement testing (APT) to evaluate the reinforcement effect of geogrids on the performance of flexible pavements. A full-scale conventional three-layer flexible pavement structure was constructed, and was divided into one unreinforced section and two reinforced sections with base reinforced by geogrids placed at different depth of the base course. The testing program was divided into three parts: performance testing, response testing and pavement trench. The performance testing recorded the development of surface permanent deformation with number of loadings. The falling weight deflectometer (FWD) was employed to investigate the deflection and modulus of the flexible pavement in the response testing. The excavation of the pavement was to further analyze the reinforcement mechanisms. The test results demonstrated the benefits of incorporating geogrids in base course could reduce the permanent deformation and improve the modulus of the base course. Compared with the unreinforced section, the accumulated permanent deformation in the two reinforced sections decreased significantly with a drop of 13-37%, and the back-calculated modulus of reinforced base increased by 58-78% after APT testing. It was possible that an enough permanent deformation was needed for mobilization of geogrids to constrain the lateral movement of granular particles, to increase the overall structural behavior of the reinforced base course, to improve the stress distribution on subgrade. These interaction mechanisms were also confirmed in the pavement trench. For a pavement system consisting of a thin surface and base layer, a proper placement position of geogrids was at the base-subgrade interface.

5.2 Introduction

The use of geogrid as reinforcement in pavement construction could be traced back to 1970s. Since then, numerous laboratory and field studies have affirmed the benefits of using geogrids as reinforcement in pavement structure, including reducing the surface rutting, extending pavement service life, and decreasing the base course thickness (29; 75). With these benefits being widely recognized, several studies on geogrid-reinforced pavement has been emphasized on understanding mechanisms of reinforcement (72; 128; 137; 138). Three main mechanisms, including lateral confinement, bearing capacity improvement and tensioned membrane effect are believed to improve the mechanical performance of the base course (38; 62). Lateral confinement restricts horizontal movement of granular particles under repeated moving loading, which causes an interlocking between granular particles within geogrid apertures to strength friction at geogrid-aggregate interface (25; 77). When placed in the base course, geogrids may increase the modulus of the base course to improve the overall bearing capacity of the pavement systems (139). The tensioned membrane effect could improve the stresses imposed on the subgrade through the base (70).

Although many researchers incorporated different kinds of instrumentations in full-scale or field studies to validate the interaction mechanisms of geogrid and granular particles under traffic and/or environmental conditions (30; 31; 68; 131). However, most of these studies focus on reporting the rutting profile with number of loadings, rather than providing detailed knowledge on the interaction mechanisms between geogrid and granular particles. In addition, a proper placement of geogrid in base course is also a hot topic of discussion among researchers. Broms (1977) reported that geogrids can maximize the reinforcement effect when they are placed in the middle of the base layer (140). Barksdale et al. (1989) supported this opinion for a thin pavement with a low-quality granular base, but they found that the optimum location is the bottom of base layer for a pavement constructed over a soft subgrade (29). Chan et al. suggested that geogrids should be placed as close as possible to the surface layer in the base to reduce rutting (22). Al-Qadi et al. (2006) recommended that the optimum location of geogrid is at the bottom of the base for a low volume pavement constructed over a weak subgrade and at the upper one-third of

the base for a low/medium volume pavement (141).

Compared to the field test, accelerated pavement testing (APT) is an efficient method to evaluate the reinforcement effect of geogrids on pavement performance at a lower cost during a shorter time period. Meanwhile, the environmental conditions can be also controlled in the APT testing. Therefore, this approach has become a reliable tool to measure mechanical response of the pavement under repeated moving loadings. To develop a methodology for geogrid-reinforced flexible pavement, it is important to understand the reinforcement effects of incorporating geogrids on the deflection of pavement and the modulus of granular base. Therefore, FWD (Falling Weight Deflectometer) test was conducted to analysis the effects of geogrids on the deflection and the modulus of base course.

5.3 Objective and scope

The main objectives of this chapter were to quantify the reinforcement effect of geogrid in the base course, to understand the interaction mechanisms between granular particles and geogrids, and to identify the optimum placement position for geogrids in the base course. To achieve these objectives, three full-scale flexible pavement test sections, one unreinforced section and two reinforced sections, were constructed. The sections were exposed to APT loading test to analysis the rutting profile of the pavement with number of loadings, to investigate the change of modulus in the base course by the FWD test, and to understand reinforcement mechanisms through the pavement excavation.

5.4 Testing program

5.4.1 Test equipment

A full-scale accelerated pavement loading system (APLS) was employed to apply wheel loads on test sections. The equipment is 8.5 m long $\times$ 2.6 m wide $\times$ 2.4 m high, as shown in Figure 5.1 (a). Figure 5.1 (b) shows a photo about the loading dual-wheel assembly. The diameter and width of each wheel is 107.55 cm and 25.2 cm, respectively. The wheel path generated by the facility is 5.5 m. In this study, the equipment applied a bidirectional trafficking to the test sections with a running speed of 1000 passes per hour or 3.6 seconds



(a) Photo about APLS



(b) Loading dual-wheel assembly

Figure 5.1. APLS accelerated loading device

per pass. To make it easier to get the rutting profile with number of loading cycles in a short time, the load was selected at a higher stress level of 80 kN which was twice the American standard axle load. The contact pressure of the tire on pavement was reached to 1.36 MPa. The entire accelerated loading test was carried out in the laboratory, and the indoor temperature and humidity were kept constant to reduce the influence of environment conditions on the test results.

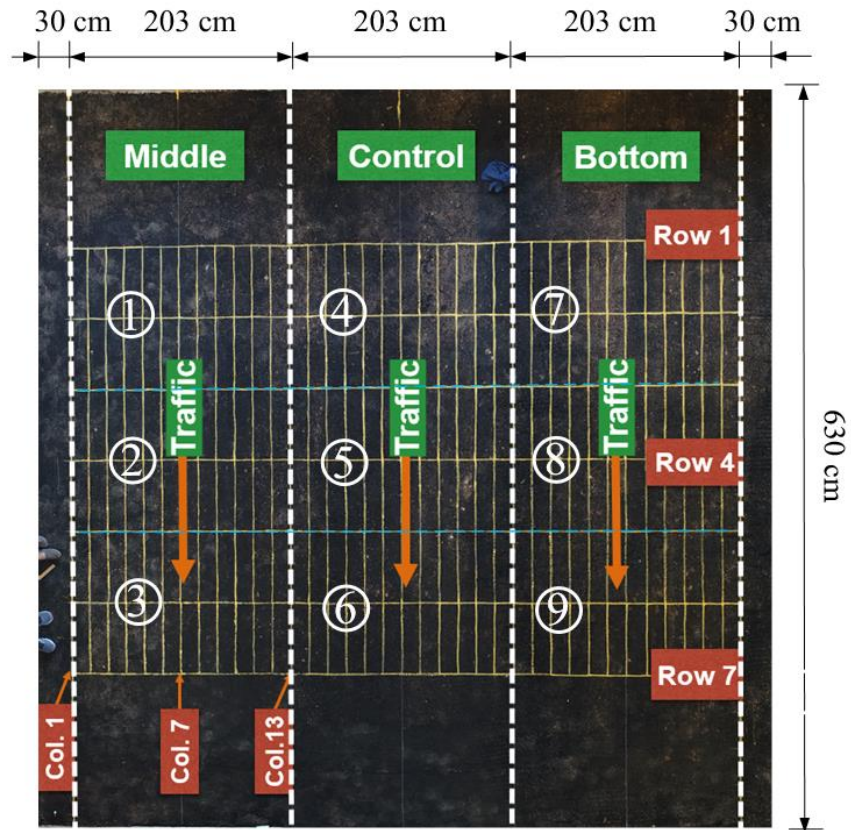
5.4.2 Test sections

The full-scale pavement structure was built in a concrete pit at the University of Tennessee, and the dimension of the pit was 6.30 m long $\times$ 6.70 m wide $\times$ 2.44 m high. To reduce the boundary effect caused by the pit walls, a 30 cm wide spacing was put between the two edge lanes and the concrete walls. The whole pavement was divided into three test sections, and each test lane was 6.30 m long and 2.03 m wide. Figure 5.2 (a) depicts an aerial photography about the pavement. Figure 5.2 (b) shows a cross section of the three test sections. The pavement consisted of three layers, including an 8cm thick hot mixture asphalt (HMA) surface course, 20 cm thick granular base course, and 155 cm thick subgrade. Section B and Section M were reinforced sections, and geogrid was placed at the bottom and middle of the base course layer in these two sections, respectively. Section C was an unreinforced section, and was considered as control section. Each lane was further divided into 3 zones, 7 rows and 13 columns as shown in Figure 5.2 (a), resulting in a total of 9 areas for FWD test and 273 profile measurement points for permanent deformation measurement. The line spacing and column spacing was 61 cm and 15 cm, respectively.

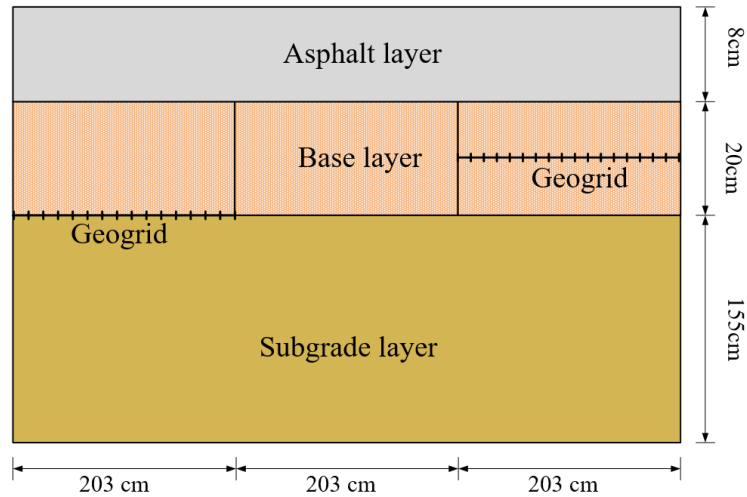
5.4.3 Materials

5.4.3.1 Subgrade

A weak soil was selected for subgrade construction in this study since several researchers found that it is easier to obtain the reinforcement effect of geogrids when the pavement structure was constructed over the soil with a lower California bearing ratio (CBR) (18; 48; 62). Figure 5.3 (a) shows the relationship between CBR and soil moisture content. It is found that the subgrade soil decreased significantly with the increase of moisture content.



(a) Test sections layout (plan view)



(b) Schematic diagram of pavement structure

Figure 5.2. Pavement plane layout and structure combination

The results of nuclear densimeter show that moisture contents of in-situ subgrade were mostly distributed in the range of 17 - 20%, so the corresponding values of CBR were in the range of 2 - 7%. The gradation of subgrade is shown in Figure 5.3 (b). The liquid and plastic limit were 21 and 35, respectively. The soil was categorized as A-7-6 according to the American Association of State Highway and Transportation Officials (AASHTO) or CL according to United Soil Classification System (USCS) classification system. The soil had a maximum dry density of 1710 kg/m^3 and an optimum moisture content of 16.5%.

5.4.3.2 Base course material

Unbound granular materials were used in base course construction. The particle size distribution of the materials is shown in Figure 5.3 (b), which met the specification of Gradation D base materials required by the Tennessee Department of Transportation (TDOT) (142). The maximum dry density and optimum moisture content was 2150 kg/m^3 and 7.2%, respectively. The material was classified as A-1-a according to AASHTO or GW according to USCS.

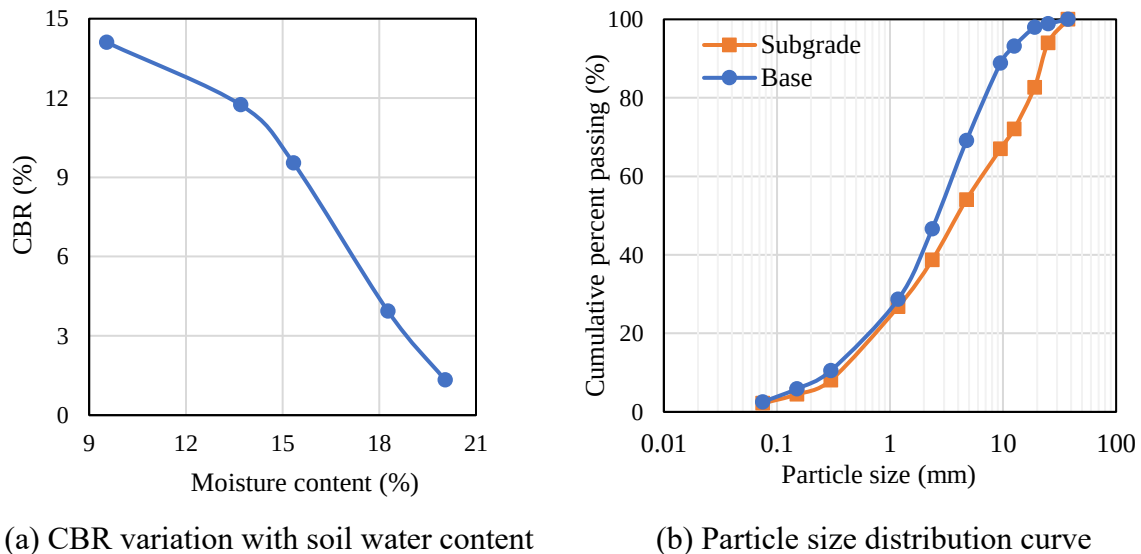


Figure 5.3. CBR and gradation curve

5.4.3.3 Hot mix asphalt (HMA) concrete

Local AC16 was selected for the construction of the surface wearing course. The maximum theoretical density and optimum oil stone of asphalt mixture were 2577 kg/m³ and 4.8%, respectively. The binder used in the study was PG64-22.

5.4.4.4 Geogrid

A polypropylene biaxial geogrid was used to reinforce the pavement. The mechanical performance parameters are shown in Table 5.1. The length and width of the geogrid laid on the site are equivalent to the length (6.5 m) and width (2 m) of each test lane.

5.5 Test Results and analysis

5.5.1 Surface permanent deformation

The total number of loading repetitions for each test section was terminated at 100,000 passes. The surface permanent deformation was the difference between the elevation of each profile measurement point after specific number of loadings and the initial elevation of the corresponding profile measurement point, and it was the sum of the permanent deformation of the surface layer, the base layer and the subgrade. The specific number of loadings was selected at various loading repetitions (1k, 3k, 6k, 10k, 15k, 20k, 30k, 40k,

Table 5.1. Properties of geogrids used in tests

Property	Geogrid
Aperture size (mm)	32×31
Tensile strength at 2% (kN/m) MD/CMD ^a	10.5/10.5
Tensile stiffness at 2% (kN/m) MD/CMD ^a	525/525
Junction efficiency (%)	90

^a MD = machine direction; CMD = cross machine direction.

60k, 80k and 100k) to get the development of surface permanent deformation for each lane. Figure 5.4 (a) illustrates the distribution of accumulated surface permanent deformation for three test sections when the APT test was finished. It is found that the width affected by the wheel loads was 80 cm, so the width of each test section was enough to eliminate the boundary effects between the two edge lanes and the concrete wall, and interference effects between every two adjacent lane as well.

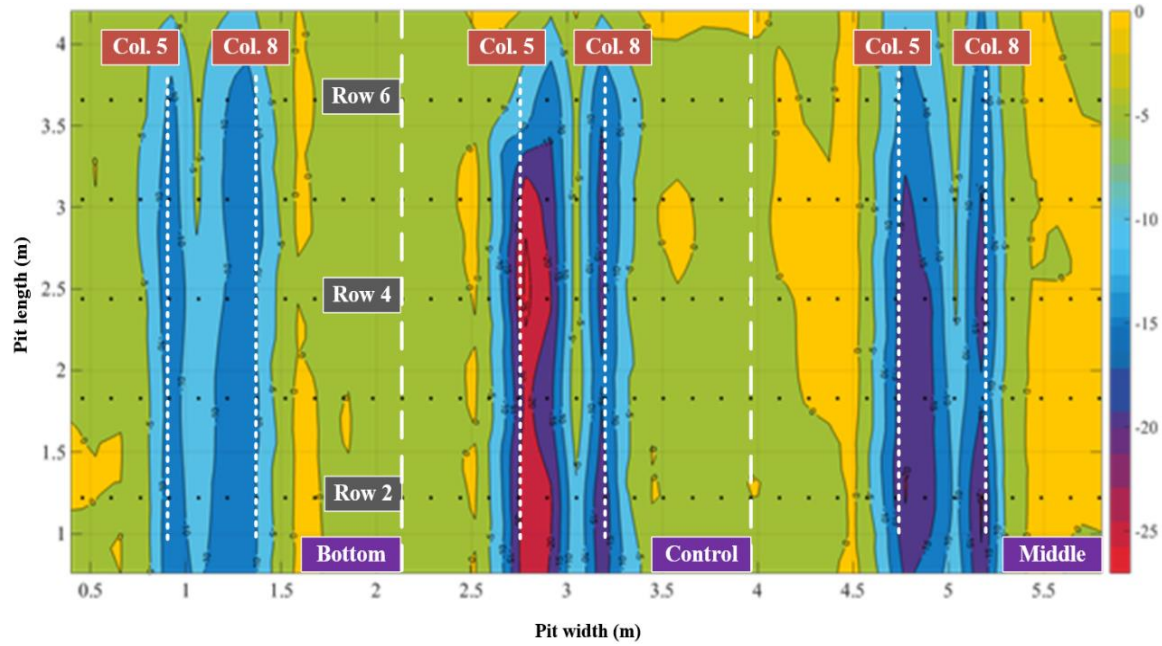
During the accelerated loading testing, to ensure the stable operation of the loading instrument, the actual loading speed was lower than the design speed, so the actual wheel path was less than the design wheel path, leading to the sixth and seventh rows not being fully loaded. Therefore, the data got from these two rows were omitted for the subsequent analysis. Though each test lane was divided into 13 columns, the field result showed that only columns 5, 6, 8 or 9 were affected by the wheel loads. Therefore, the data from these columns were used for the longitudinal profile analysis. To further analyze the surface permanent deformation behavior under the moving wheel load, the development curves of the surface permanent deformation were illustrated in both transverse and longitudinal profiles.

5.5.1.1 Surface permanent deformation in transverse profile

In order to compare the development of surface permanent deformation in different test sections, the fourth row located in the center of the lane was selected as a representative row to show the results. Figure 5.4 (b) shows elevation changes of the transverse profile in the Control, Middle, and Bottom sections under different loading cycles. All curves in the three test sections showed a similar “W” shape. Compared with the unreinforced section, the two reinforced sections showed a smaller surface permanent deformation under the same loading conditions. The order of the surface permanent deformation at 100k repetitions was Control (24.6 mm) > Middle (18.3 mm) > Bottom (14.0 mm).

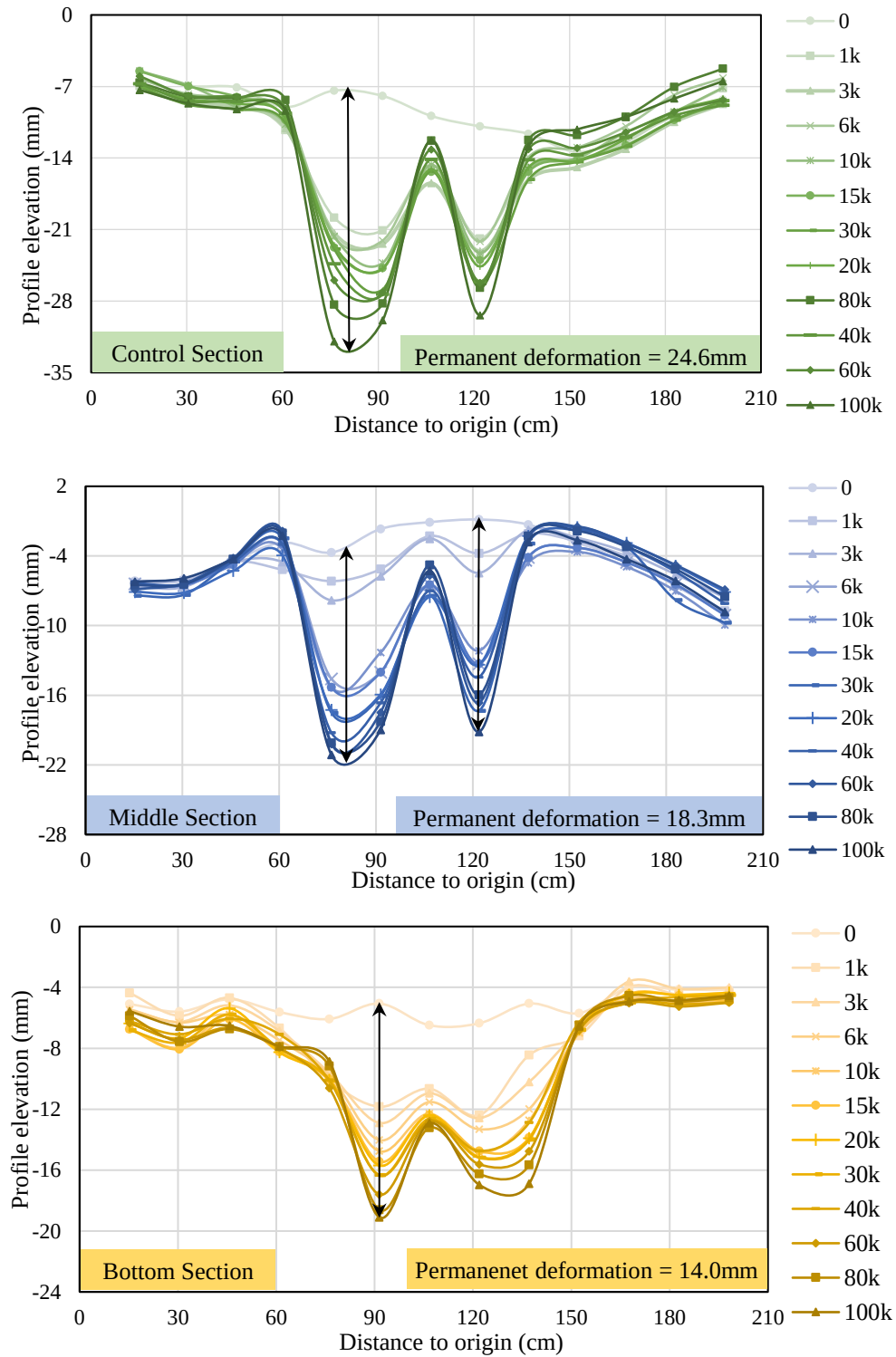
5.5.1.2 Surface permanent deformation in longitudinal profile

The fifth column in each test section was selected as a typical column to show the development of surface permanent deformation in the longitudinal profile, as shown in Figure 5.4 (c). There were five profile elevation measurement points in each column, and



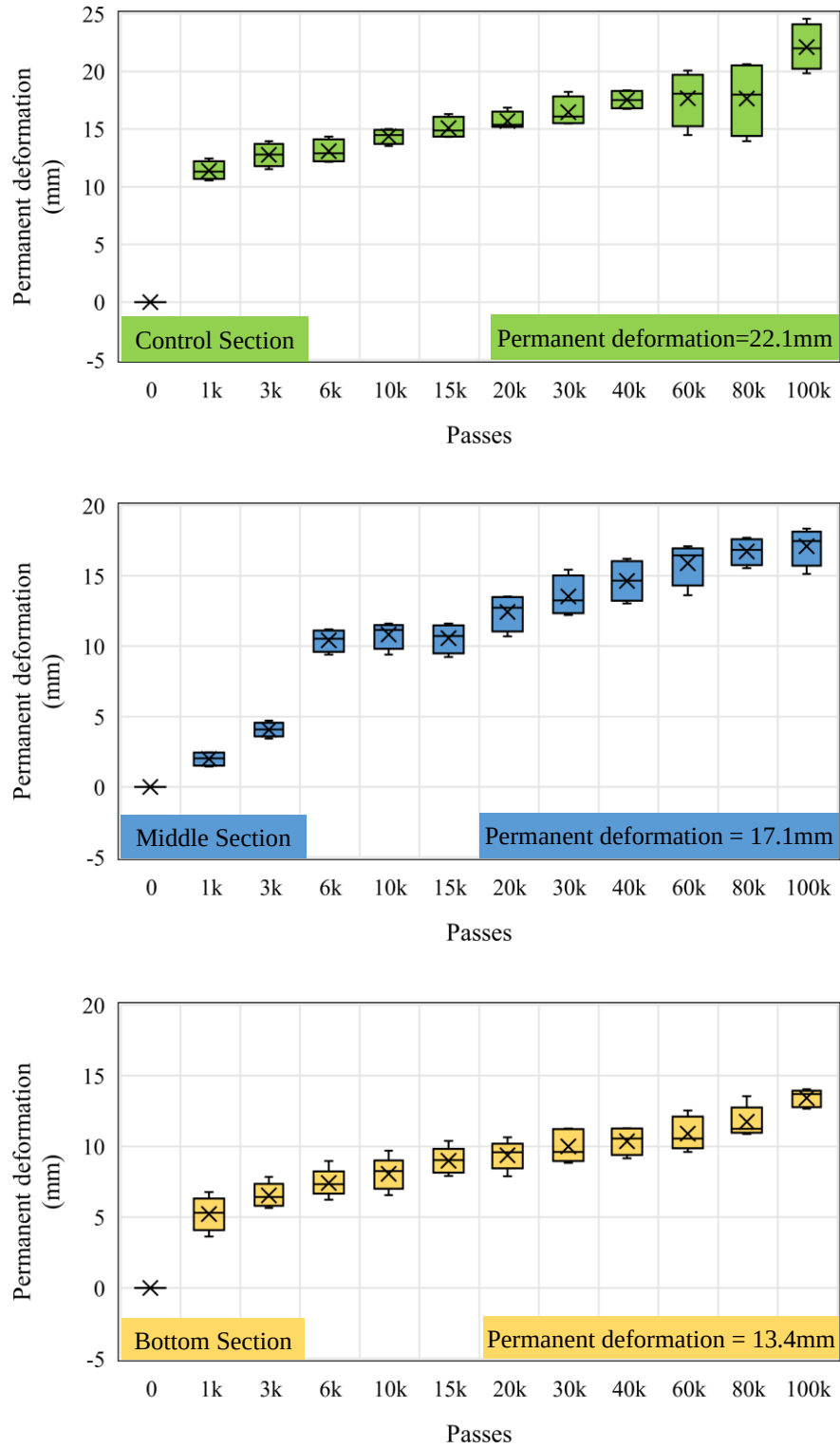
(a) Distribution of accumulated surface permanent deformation

Figure 5.4. Surface permanent deformation in the three test sections



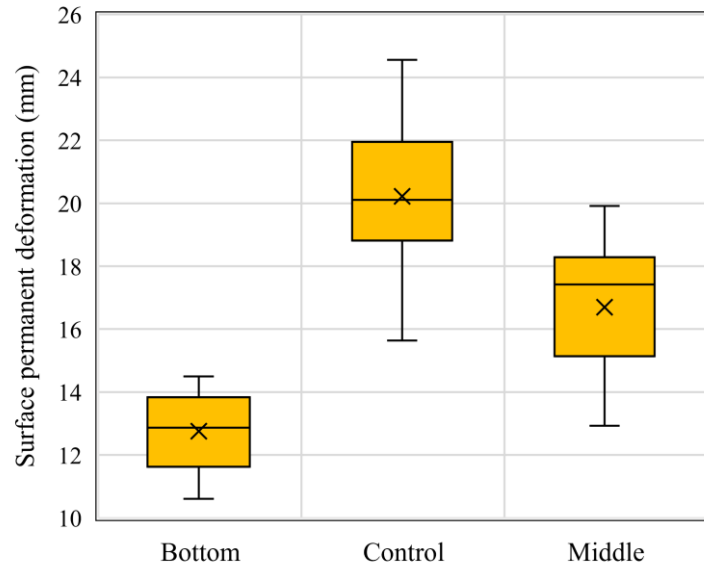
(b) Transverse profile

Figure 5.4. Continued

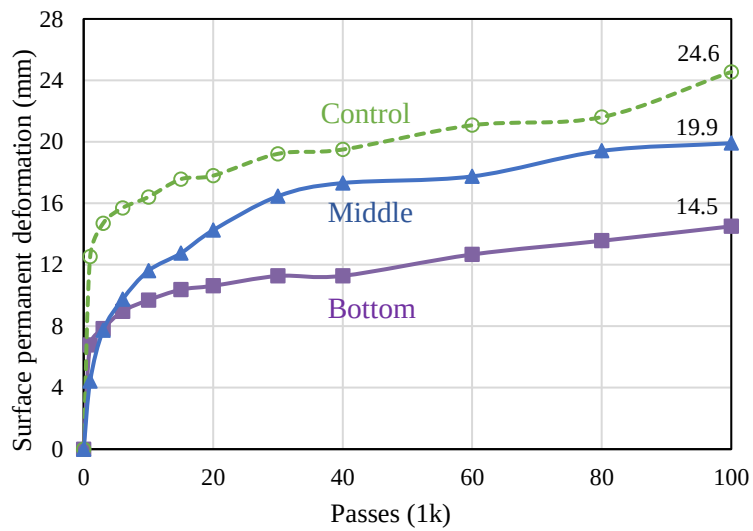


(c) Longitudinal profile

Figure 5.4. Continued



(d) Accumulated surface permanent deformation after 100,000 passes



(e) The development of the maximum surface permanent deformation

Figure 5.4. Continued

all data obtained from these points were used to show the development of surface permanent deformation. The deformations in the longitudinal profile were present in a box-plot form. The box-plot contained maximum, minimum, average, median, upper and lower quartile values of the surface permanent deformations obtained from each measurement point, with a more comprehensive mathematical representation. The result show that in the unreinforced Control section, the surface permanent deformation increased significantly in the first 1,000 loading cycles, and the surface permanent deformation at this point accounted for 51.6% of the total accumulated surface permanent deformation. In the other two reinforced sections, the ratios were 11.6% for Middle section and 38.9% for Bottom section, indicating that the placement of geogrid in the base course could effectively delay the growth rate of the surface permanent deformation.

5.5.1.3 The development of surface permanent deformation

The accumulated surface permanent deformation for the three sections is present in Figure 5.4 (d). The average deformations of the Bottom, Control, and Middle section were 12.8 mm, 20.2 mm, and 16.7 mm, respectively. Compared with the Control section, the surface permanent deformation was reduced by 17.3% and 36.6% when the geogrid was placed at the middle and bottom of the base course. The result indicated that reinforced base course could significantly reduce the surface permanent deformation of the pavement system, and a better reinforcement effect was obtained in the bottom group.

Figure 5.4 (e) shows that the development of maximum surface permanent deformation with number of loading repetitions for the three test sections. It can be found that all the test sections experienced an initial compaction stage (loading repetitions < 10,000 passes), and a medium-term stable development stage (loading repetitions 10,000 - 60,000 passes). Unlike the reinforced sections, the surface permanent deformation increased again rapidly in the late loading stages (loading repetitions 60,000 - 80,000 passes), undergoing a damage stage. However, this phenomenon did not show in the two reinforced sections, indicating that placing geogrid in the base course could not only delay the growth rate of surface permanent deformation, but extend the service life of pavement as well.

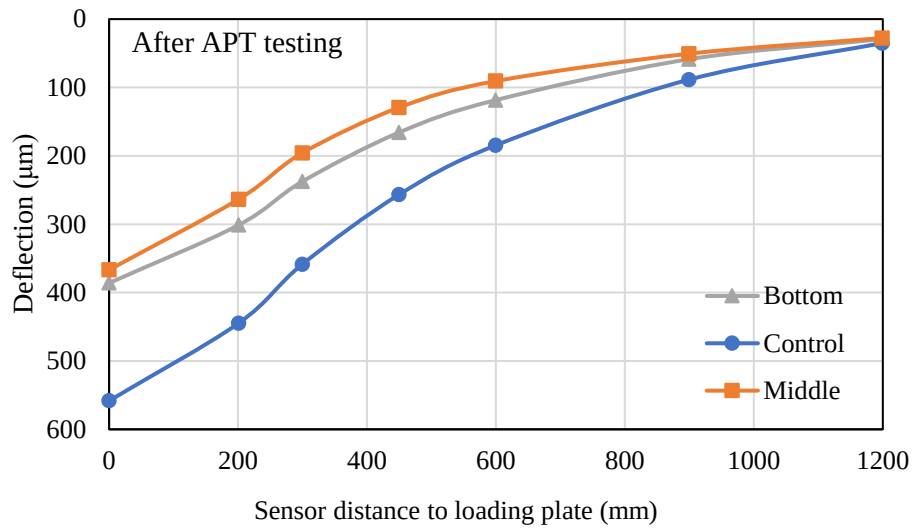
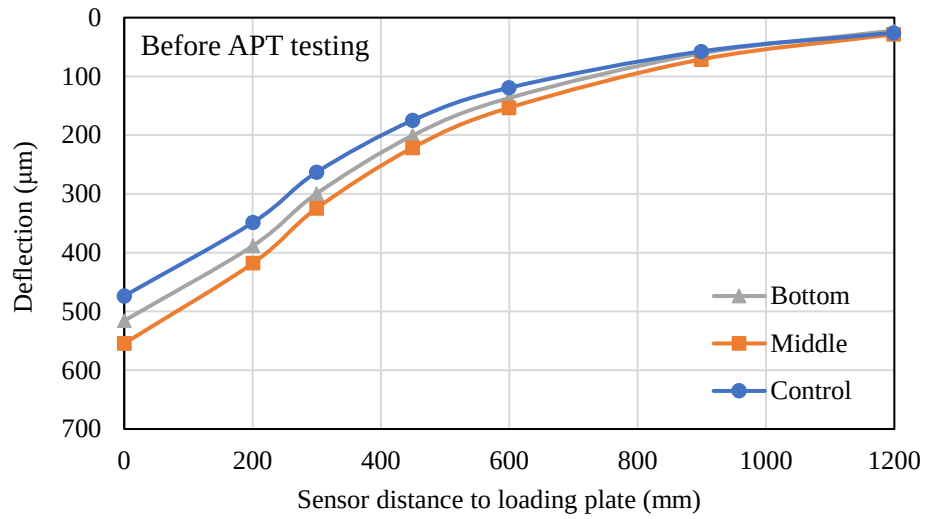
5.5.2 FWD testing

5.5.2.1 Deflection basin

FWD is a device to simulate deflections of the pavement caused by a moving load, and FWD test is a common method to quantify the resilient modulus of pavement layers (143). As shown in Figure 5.2 (a), the three test sections were divided into nine FWD test zones, and the test locations were selected on the wheel paths of the nine test zones. FWD tests were conducted before and after APT testing on the specific locations to check the reinforcement effect of the geogrid on the modulus of the base layer. The equipment used in this study was Dynatest 8002. The FWD generated a dynamic load pulse to the pavement through a loading plate with a diameter of 30 cm. The peak value of the dynamic load was 40 kN, which represents one dual-wheel assembly tire load, and the load duration of was 40 ms. The instrument had seven sensors (D_0 , D_{200} , D_{300} , D_{450} , D_{600} , D_{900} , and D_{1200}) installed along the long frame. The subscripts represent the distance (in mm) from the center of the applied load to these sensors. The deflection data measured by these points could form a deflection basin as shown in Figure 5.5 (a). Figure 5.5 (a) presents the deflection basins for the three test sections before and after APT testing. The values of deflection were the average values from the nine tests. The pavement structure was in a good condition without any deformations before APT testing, but obvious permanent deformation was generated on the wheel paths after APT testing. Before the APT testing, the deflection basins for the three test sections were similar, and the difference of the deflection at the center of the loading was small, indicating that the addition of geogrid did not show reinforcement on the base course. However, the deflection basins became different after APT testing when the pavement structure was permanently deformed. The values of the deflection in Middle and Bottom sections at the center of the load were significantly lower than that in the Control section. It was possible that an obvious permanent deformation was needed for the mobilization of geogrids in increasing the overall stiffness of the pavement structure and reducing the deflections.

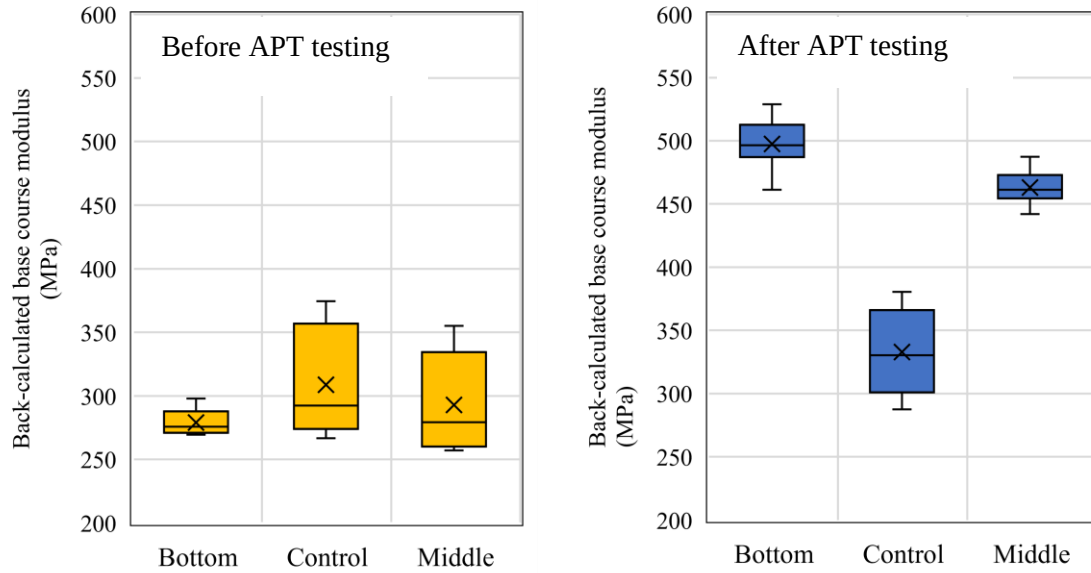
5.5.2.2 Estimation of modulus for base course

The measured data of the deflection was a linear elastic response of each layer of pavement



(a) FWD deflection basin before and after APT testing

Figure 5.5. FWD testing results



(b) Estimation of modulus for base course

Figure 5.5. Continued

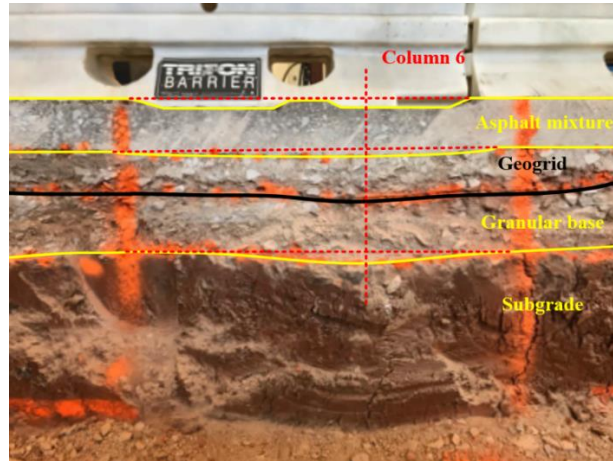
structure, and the data could be used to estimate the elastic modulus of each layer (130). The deflection basins generated by FWD tests were used to back-calculate the modulus for base course to evaluate the effectiveness of the geogrid in reinforcing the pavement. In this study, an analytical software MODULUS 6.1 developed by the Texas Transportation Institute and the Texas Department of Transportation was used for the modulus back-calculation. The modulus back-calculation was a relatively complicated and difficult work, and the purpose of the back-calculation was to get the best fit between the theoretical deflection basins and the on-site measured deflection basin. In order to simplify the back-calculation process and evaluate the reinforcement of the geogrid, only the modulus of the base course was back-calculated since the geogrid was placed in the base course layer, which meant the modulus of the other layers was assumed the same in all three test sections. The pavement consisted of three layers, including HMA surface layer, base course layer and subgrade soil layer. According to the laboratory results, the modulus of the asphalt mixture was 3998 MPa when the loading frequency was 10 Hz under a temperature of 20 °C. The modulus of the subgrade soil was selected as 69 MPa. The modulus of the base

course was set in a range of 69 - 1034 MPa, which meant the back-calculated values were in this range. The Poisson's ratio of the asphalt mixture, granular base and subgrade was 0.38, 0.35 and 0.40, respectively.

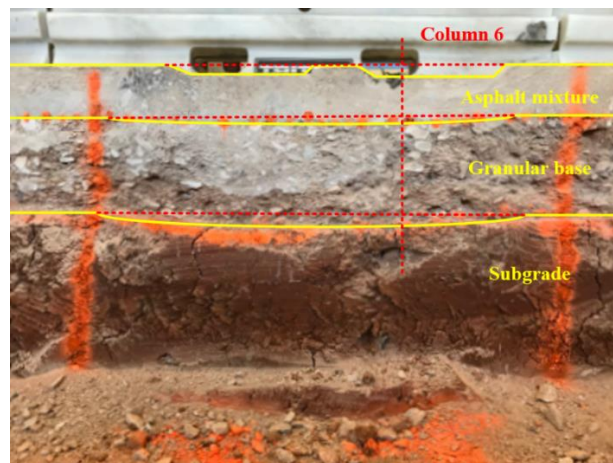
Figure 5.5 (b) shows the estimation results of modulus for the base course in the three test sections before and after APT testing. Before the testing, the back-calculate result of the modulus in the Bottom, Control and Middle section were 279 MPa, 309 MPa, and 293 MPa, respectively. The difference of the modulus in the two reinforced sections and unreinforced section was small, which was also in line with the deflection basins as shown in Figure 5.5 (a). There were no deformations in the pavement structure before the APT testing, so the geogrid was not mobilized. However, as the number of load repetitions increased, the permanent deformation in the pavement structure increased gradually, resulting in an effective mobilization in the geogrid. In such conditions, the geogrid could restrain the lateral movement of the granular particles under repeated loads to form an effective reinforcement in the base course, which was helpful in delaying the grow rate of permanent deformation and reducing the accumulated surface permanent deformation of the pavement structure. Therefore, the modulus of the reinforced sections strengthened due to the benefits of the geogrid reinforcement. The back-calculated values of the modulus in the Bottom, Control and Middle sections were 497 MPa, 333 MPa, and 463 MPa, respectively. Each layer in the pavement structure experienced a further compaction because of a lower compaction degree in each layer during the pavement construction. Therefore, compared with the modulus before APT testing, the modulus after APT testing increased by 78%, 7% and 58% in Bottom, Control, and Middle sections, respectively. The results also illustrated that the geogrid placed at the bottom of the base layer could achieve a better reinforcement for a thin pavement structure.

5.5.3 Pavement trenches

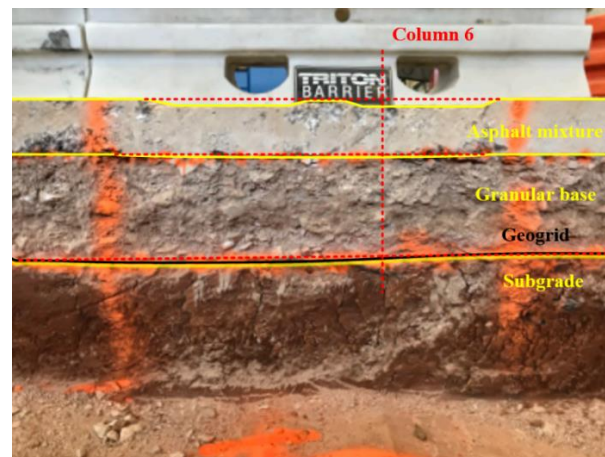
After all the tests were completed, the pavement structure was trenched at the 4th row to further quantify the reinforcement effect of geogrid as shown in Figure 5.6. The permanent deformation of each layer in the pavement structure at 5th column of each test section was measured, as shown in Table 5.2. The results demonstrate that the overall permanent



(a) Middle section



(b) Control section



(c) Bottom section

Figure 5.6. Trench cut in the three sections

Table 5.2. Permanent deformation of each layer in the three sections

Layer	Control		Bottom			Middle		
	δ_i (mm)	δ_i/δ_t (%)	δ_i (mm)	δ_i/δ_t (%)	Decrease (%)	δ_i (mm)	δ_i/δ_t (%)	Decrease (%)
Surface	2.6	11.7	1.6	13.5	38.3	2.0	11.6	21.9
Base	10.6	48.3	6.3	54.0	40.5	8.9	51.4	16.2
Subgrade	8.8	40.0	3.8	32.4	56.8	6.4	37.1	27.0
Total	22.0	100	11.7	100	46.8	17.3	100	21.2

deformation in the reinforced sections was lower than that in the unreinforced section, and the permanent deformation of each layer (surface, base, and subgrade layer) in reinforced pavement was lower than that of corresponding layer in unreinforced pavement to a certain degree. Placement of geogrid in base course was able to improve the stress conditions for each layer of the pavement structure.

It is found from Table 5.2 that the proportion of the permanent deformation of the base course to the total permanent deformation of the pavement structure was the largest among three test sections. This proportion continued to increase when geogrids were placed in the base course, and the proportions were more than half in the Bottom and Middle sections. However, the proportion of the permanent deformation of the subgrade to the total permanent deformation was reduced in the two reinforced sections. In the Control section, the proportion was 40.0%, but this proportion decreased to 32.4% in the Bottom section and 37.1% in the Middle section. In such conditions, the stress which should be borne by the subgrade was more borne by the base course. On one hand, the resilient modulus of the subgrade was lower than that of the base course, so the resistance to deformation of the subgrade was weaker than that of the base course. On the other hand, geogrids placed in the base course mobilized when an obvious permanent deformation produced in the base course. Geogrids could restrain the lateral movement of particles and mobilized their tensioned membrane effect to improve the modulus of the base course and reduce the permanent deformation occurred in the base course. The variation of deformation

proportions in each layer indicated that geogrids could improve the stress distribution and reduce the stress concentration on the top surface of the subgrade, resulting in a reduction in the permanent deformation of each layer.

5.6 Conclusions

In this study, an accelerated pavement testing was carried out on full-scale sections to evaluate the benefits of incorporating geogrids in the base course to improve the performance of pavement systems. The evaluations were conducted by a comparison of results from one unreinforced section and two reinforced sections with base reinforced by geogrids put in the different positions of the base course. To quantify the contribution of geogrids to the long-term performance of the pavement, FWD test was carried out over the three test sections before and after APT testing. The pavement structure was also excavated after APT testing to understand the interaction mechanisms between geogrids and granular particles. Based on the test results, the following conclusions were drawn:

Compared with the Control section, the accumulated surface permanent deformation of the Middle and Bottom section was reduced by 13% and 37%, respectively. In the first 1000 loading cycles, the surface permanent deformation of each test section increased the most. The surface permanent deformation at the moment in Control section accounted for 51.6% of the final accumulated surface permanent deformation, while the Middle section and Bottom section accounted for 11.6% and 38.9%, respectively. The results showed that the reinforced base course could not only effectively delay the growth rate of the surface permanent deformation, but also ultimately reduce the accumulated surface permanent deformation of the pavement structure as well.

In terms of the deflection basin of the pavement structure and the back-calculated modulus of the base course, it can be found that these two indicators in reinforced test sections showed no significant difference compared with unreinforced test before APT testing. However, when the repeated traffic loads caused a significant vertical permanent deformation on the pavement structure, the deflection basin of the Middle and Bottom sections was significantly lower than that of the Control section, and the corresponding modulus of the base course increased by 58% and 78%, respectively. It was possible that

an enough permanent deformation was needed for geogrid mobilization to reinforce the pavement.

The proportion of the permanent deformation of each layer to the total permanent deformation of the pavement structure changed significantly when geogrids were placed in the base course. The variation of deformation proportions in each layer indicated that geogrids could improve the stress distribution and reduce the stress concentration on the top surface of the subgrade, resulting in a reduction in the permanent deformation of each layer.

For a pavement system consisting of a thin surface and base layer constructed over a soft subgrade, the geogrid placed at the bottom of the base course had a better reinforcement effect than placing at the middle of the base course.

5.7 Acknowledgement

The authors would like to thank and acknowledge the Tennessee Department of Transportation for their pavement construction and financial support.

CHAPTER SIX
CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

1) The repeated load triaxial (RLT) test and cyclic shear test were conducted on unreinforced and reinforced aggregate to characterize the interfacial resilient deformation behavior of the aggregate-geogrid interface from the vertical and horizontal directions, respectively. The reinforcement effects of geogrids on aggregates were quantified and the interaction mechanisms between geogrids and aggregate were analyzed in the resilient deformation stage under cyclic loads.

(1) The results of the RLT tests showed that the inclusion of geogrids did not have a significant effect on the resilient behavior of reinforced specimens compared to unreinforced specimens. Thus, the RLT test may not be effective in evaluating the reinforcement effects of geogrid in unbound granular materials.

(2) The presence of geogrids strengthened the interlocking between aggregate particles and geogrids and the degree of the interlocking could be effectively quantified by the cyclic shear test with a term called the resilient interface shear modulus (G_i), and duplicate tests showed a good repeatability for this method.

(3) Like the stress dependency of the resilient modulus, the resilient interface shear modulus was a function of confining normal stress and cyclic shear stress. The value of G_i increased with an increase in normal stress and decreased with an increase in cyclic shear stress. A NCHRP three-parameter model was modified to predict the resilient interface shear modulus with a high reliability.

2) A large-scale direct shear test was employed to investigate the effect of particle size on the shear behavior of aggregate-geogrid interface and the failure modes of the reinforced aggregate.

(1) Compared with the unreinforced aggregate, several mechanical indicators including interfacial shear strength, residual shear stress, vertical displacement, shear strength envelope and the interfacial shear strength efficiency coefficient were significant changed with the inclusion of geogrids in aggregate.

(2) Three possible modes of aggregate-geogrid interface failure were proposed based on the results. The overall shear resistance of aggregate-geogrid interface composed of two parts: a) the shear resistance of interlocked particles in the openings of geogrids; b) the shear resistance between particles and geogrid surface. Particle size had a significant influence on the magnitude and percentage composition of these two shear resistances. Too large particles would cause a loss of interlock at the interface and a relatively large aggregate-geogrid contact area, while too small particles may lead to a poor interlocking within openings of geogrids. A stronger interaction would be generated only when the particles could pass through the geogrid and were confined in the geogrids.

(3) There is a reasonable match standard between geogrids and aggregates. For biaxial geogrids, when the ratio of equivalent aperture size to aggregate particle size (A/D) was in the range of 1.30 - 1.71, the interlocking in the apertures could be improved and the contact area between particles and geogrids could be minimized. For triaxial geogrids, the optimum ratio of A/D was in the range of 1.08 - 1.43.

3) Laboratory experiments were conducted to evaluate the reinforcement effects of geogrids in different granular base course materials by using a modified loaded wheel tester, APA. Three indicators, traffic benefit ratio (TBR), rutting reduction rate (RRR), and rate of deformation (ROD), were used to characterize the permanent deformation behavior of the reinforced aggregate with number of load repetitions. Influence factors like aggregate gradation, geogrid tensile strength and geogrid aperture geometry were analyzed for the reinforcement effect.

(1) The modified APA testing was able to effectively characterize the permanent deformation behavior of geogrid-reinforced base course, and reasonably distinguish the effect of geogrid-aggregate interaction among all the geogrids investigated in this study. The multiple test results show that this newly developed method had a good repeatability.

(2) Placing geogrids in both of dense- and open-graded aggregate could significantly improve the ability of aggregate to resist permanent deformation, with a lower RRR, ROD and a higher TBR, indicating that reinforced aggregate base could carry more loadings and prolong the pavement service life.

(3) A better reinforcement effect could be obtained by increasing portion of the effective aggregate, increasing the tensile strength of geogrids. Triaxial geogrids performed better in reinforcing aggregate than biaxial geogrids when their tensile strength were similar. The tensile strength of geogrids had much significant effects on the reinforcement compared to the aperture geometry.

4) An accelerated pavement testing was carried out on full-scale sections to verify the results from laboratory test. The reinforcement effect evaluation was conducted by rutting comparisons from one unreinforced section and two reinforced sections with base reinforced by geogrids put in the different positions of the base course. To quantify the contribution of geogrids to the long-term performance of the pavement, FWD test was carried out over the three test sections before and after APT testing. The pavement was also excavated after APT testing to understand the interaction mechanisms between geogrids and granular particles.

(1) The APT result show that the reinforced base course could not only effectively delay the growth rate of the surface permanent deformation, but also ultimately reduce the accumulated surface permanent deformation of the pavement structure as well.

(2) The FWD result demonstrate that an enough permanent deformation was needed for mobilizing geogrids to constrain the lateral movement of granular particles and increase the modulus of base course.

(3) The pavement trench show that the proportion of the permanent deformation of each layer to the total permanent deformation of the pavement structure changed significantly when geogrids were placed in the base course. The variation of deformation proportions in each layer indicated that geogrids could improve the stress distribution and reduce the stress concentration on the top surface of the subgrade, resulting in a reduction in the permanent deformation of each layer.

(4) For a pavement system consisting of a thin surface and base layer constructed over a soft subgrade, the geogrid placed at the bottom of the base course had a better reinforcement effect than placing at the middle of the base course.

6.2 Recommendations

Although some research results have been obtained in this dissertation, there are still many problems to be solved since the problem about mechanical behavior of the reinforced flexible pavements is complicated. In order to meet the requirements of the engineering projects, and establish a complete design method for geogrid-reinforced flexible pavements, the following recommendations are provided here for further research work:

(1) The modelling of the reinforced granular base course can help to analyze the mechanical behavior of the system composed of reinforced materials in different stress environments, and can more comprehensively analyze the impacts of different influencing factors on the reinforcement effect, for correcting and guiding the design of geogrid-reinforced base. Therefore, numerical simulation work about geogrid-reinforced flexible pavements should be conducted.

(2) Geogrids are subjected to tension force in the reinforced granular base, and they will exhibit an obvious creep behavior due to their internal material properties. Therefore, in future research, the creep behavior of geogrids and its effect on the reinforcement mechanism should be considered.

(3) Currently, the design of flexible pavement of reinforced granular base is still in a semi-quantitative and semi-experienced level. How to quantify and reasonably introduce the reinforcement effect into the current flexible pavement design method is an important issue to be solved in the future.

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APPENDIX

FWD deflection basin data

Table 6.1. Deflection data of different areas in the test pit (before APT)

Group	Position (Row-Col.)	Measurements for different deflection sensors (μm)							Load /kN
		D ₀	D ₂₀₀	D ₃₀₀	D ₄₅₀	D ₆₀₀	D ₉₀₀	D ₁₂₀₀	
Bottom	2-5	561	426	331	222	151	68	23	41.43
Bottom	2-5	535	405	315	212	145	64	23	39.97
Bottom	2-5	531	404	314	211	145	64	22	39.88
Bottom	4-5	496	367	281	187	128	57	21	40.07
Bottom	4-5	487	363	278	186	127	57	21	40.07
Bottom	4-5	485	362	278	186	127	57	22	40.17
Bottom	6-5	526	396	305	204	139	61	23	40.75
Bottom	6-5	511	385	298	198	135	60	21	40.02
Bottom	6-5	510	385	297	199	135	60	23	40.02
Control	2-5	500	370	277	183	125	61	28	40.26
Control	2-5	490	364	274	181	124	60	28	40.26
Control	2-5	484	361	272	180	123	61	28	40.12
Control	4-5	416	291	214	141	97	50	29	39.73
Control	4-5	410	288	213	141	97	51	27	40.02
Control	4-5	408	287	212	141	97	51	27	40.21
Control	6-5	531	401	309	207	141	62	23	40.80
Control	6-5	512	387	298	199	135	60	22	40.02
Control	6-5	512	387	299	199	136	60	22	40.17
Middle	2-5	557	425	330	222	152	67	23	39.83
Middle	2-5	554	422	327	221	151	67	23	39.78
Middle	2-5	553	421	328	221	151	67	23	39.83
Middle	4-5	563	416	322	222	156	75	34	39.88
Middle	4-5	552	412	321	221	155	75	34	39.83
Middle	4-5	548	410	320	221	155	75	34	39.97
Middle	6-5	537	408	320	225	162	81	32	40.12
Middle	6-5	526	401	317	223	161	80	31	40.12
Middle	6-5	522	401	317	224	162	81	32	40.36

Table 6.2. Deflection data of different areas in the test pit (after APT)

Group	Position (Row-Col.)	Measurements for different deflection sensors (μm)							Load /kN
		D ₀	D ₂₀₀	D ₃₀₀	D ₄₅₀	D ₆₀₀	D ₉₀₀	D ₁₂₀₀	
Bottom	2-5	409	320	251	174	124	62	29	40.31
Bottom	2-5	392	307	241	167	119	60	28	39.29
Bottom	2-5	400	314	247	170	123	62	30	40.02
Bottom	4-5	390	302	237	168	120	59	31	44.90
Bottom	4-5	376	292	229	163	117	57	30	40.70
Bottom	4-5	372	289	227	161	116	57	29	40.56
Bottom	6-5	390	302	237	168	120	59	31	40.17
Bottom	6-5	376	292	229	163	117	57	30	40.07
Bottom	6-5	372	289	227	161	116	57	29	40.02
Control	2-5	563	451	361	258	186	91	36	42.36
Control	2-5	556	452	362	261	189	91	35	40.26
Control	2-5	577	466	374	270	196	94	36	40.56
Control	4-5	550	448	367	263	190	92	39	40.51
Control	4-5	534	435	356	256	185	89	36	39.19
Control	4-5	528	432	354	254	183	89	35	41.00
Control	6-5	574	435	346	246	174	82	36	40.45
Control	6-5	573	438	351	250	178	83	33	40.90
Control	6-5	569	446	355	253	182	86	31	40.45
Middle	2-5	394	285	219	149	102	59	44	40.26
Middle	2-5	370	271	207	138	97	52	30	39.53
Middle	2-5	370	273	208	138	98	46	18	40.21
Middle	4-5	360	255	186	121	85	49	25	40.61
Middle	4-5	346	246	179	117	82	47	25	39.97
Middle	4-5	361	258	188	123	86	50	26	39.83
Middle	6-5	367	263	192	125	88	50	26	39.77
Middle	6-5	367	261	192	126	89	51	28	39.87
Middle	6-5	365	261	193	127	90	51	26	40.02

VITA

Bingye Han was born in Changzhi, Shanxi Province, China in 1992. He received a bachelor's Degree in Road and Railway Engineering from Chang'an University, China in 2012. In August 2016, he joined the Department of Civil and Environmental Engineering in the University of Tennessee at Knoxville (UTK) to pursue the Ph.D. degree. During his stay in UTK, he extended his research area to pavement materials.