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EVALUATING THE PERFORMANCE OF GEOSYNTHETIC REINFORCED PAVEMENTS OVER WEAK SUBGRADE

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ABSTRACT

In this paper, the benefits of using geosynthetics to reinforce/stabilize base aggregate layer/subgrade in road structures was evaluated using both laboratory cyclic plate load testing (CPLT) and full-scale moving wheel load testing. A total of nine cyclic plate load test sections and six full-scale test lane sections were constructed for this purpose. A variety of instrumentations were installed to measure the load-associated and the environment-associated road structure responses and performance. The results of both the laboratory CPLT and full-scale moving wheel load testing clearly demonstrate the benefits of using geosynthetics for reinforcement/ stabilization of road structures in terms of reducing the permanent deformation and extending the service life of road structures. The results also suggested that the improvement in the performance of pavement sections is in the same order with the increase in the tensile modulus of geosynthetics. Better performance was observed when the geogrid layer was placed at the upper one third of the base aggregate layer compared to other locations. The inclusion of geogrid helps to redistribute the applied load to a wider area, resulting in less accumulated permanent deformation in the subgrade.

1. INTRODUCTION

Weak subgrades are a common problem in road construction. Whether it is a temporary access road or a permanent road built over a weak subgrade, a large deformation of the subgrade can lead to deterioration of the paved or unpaved surface. The concept of using geosynthetics as reinforcement in roadway construction started in the 1970s. During the past four decades or so, many studies have been performed to investigate the behavior of geosynthetic reinforced paved/unpaved roads (e.g., Perkins 2002, Berg et al. 2000, Perkins et al. 2009, Abu-Farsakh and Nazzal 2009, Nazzal et al. 2010). The results of experimental studies reported in the literature showed that the geosynthetics can extend the service life of paved/unpaved roads (e.g., Al-Qadi et al. 1997, Perkins 1999, 2002, Cancelli and Montanelli 1999, Wasage et al. 2004, Hufenus et al. 2006, Chen et al. 2009, Abu-Farsakh and Chen 2011), reduce the thickness of base course layer for a given service life (e.g., Cancelli and Montanelli 1999, Montanelli et al. 1997, Abu-Farsakh and Nazzal 2009, Chen and Abu-Farsakh, 2012), and delay the development of rutting (e.g., Moghaddas-Nejad and Small 1996, Kinney et al. 1998).

The degree of improvement in pavement performance with the inclusion of geosynthetics depends on many factors, including the strength of subgrade (e.g., Cancelli and Montanelli 1999, Chen et al. 2009, Abu-Farsakh and Nazzal 2009), types of geosynthetics (e.g., Al-Qadi et al. 1997), tensile modulus of geosynthetics (Cancelli and Montanelli 1999, Abu-Farsakh and Nazzal 2009, Nazzal et al. 2010), location of geogrid (e.g., Al-Qadi et al. 2008, Abu-Farsakh and Nazzal 2009, Abu-Farsakh and Chen 2011), and thickness of base layer (e.g., Collin et al. 1996; Kinney et al. 1998).

Due to the soft nature of the subgrade soil and the presence of high ground water table, cement or lime are typically used to treat/stabilize the subgrade soil in the state of Louisiana. Geosynthetics can offer a potentially economical solution for reinforcing/stabilizing roads built over soft subgrades. Aimed to evaluate the potential benefits of using geosynthetics in road construction, two sets of tests were conducted in this study: laboratory cyclic plate load testing on flexible pavement sections constructed inside a test box and moving wheel load testing on full-scale field test lane sections.

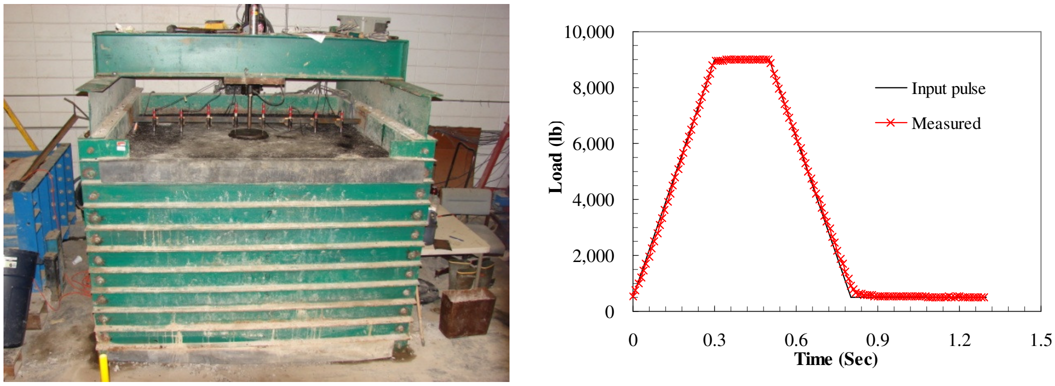
2. TESTING PROGRAM

2.1 Laboratory Cyclic Plate Load Testing

A total of nine tests were conducted on different pavement sections: two unreinforced sections, four reinforced sections with one geogrid layer placed at the base/subgrade interface, two reinforced sections with one geogrid layer placed at the middle of the base layer, and another reinforced section with one geogrid layer placed at the upper one third of the base layer.

2.1.1 Test Facility

A steel box was constructed having inside dimensions of 2.0 m (length) × 2.0 m (width) × 1.7 m (height) (Figure 1a). A hydraulic actuator, which has a force rating of 100 kN and a dynamic stroke of 152.4 mm, was placed between two I beams of the crosshead. The cyclic load was applied through a steel rod that fits into a concave-shaped hole on the loading plate that sits on the surface of the HMA (hot mix asphalt) layer. The loading plate was a 25.4 mm thick steel plate with 305 mm in diameter. The maximum applied load in tests was 40 kN (9000 lbs), which results in a loading pressure of 550 kPa (80 psi) and simulates dual wheels under an equivalent 80 kN single axle load. The load pulse, as shown in Figure 1b, had a linear load that increases from 2.2 kN (500 lbs) to 40 kN (9000 lbs) in 0.3 second, followed by a 0.2 second period where the load was held constant at 40 kN, followed by a linear load decrease to 2.2 kN over a 0.3 second period, then followed by a 0.5 second period of 2.2 kN before the next loading cycle. This load pulse results in a frequency of 0.67 Hz.



(a) (b)

Figure 1. Laboratory test box, hydraulic actuator, reaction system, and load pulse

2.1.2 Pavement Layer Materials

The HMA used in the construction of the pavement test sections was 19.0 mm design level 2 (i.e. 10^6<ESALs<10^7, ESAL stands for equivalent single axle load) superpave mixture currently in use in the state of Louisiana. The optimum asphalt content (PG76-22M) was 4.1 percent. The theoretical maximum specific gravity of the HMA was 2.508.

Kentucky crushed limestone was used in the base course layer for all test sections. The maximum dry density, as determined by standard Proctor test, is 2,247 kg/m^3 at an optimum moisture content of 6.6%. This crushed limestone is classified as GW and A-1-a according to the Unified Soil Classification System (USCS) and the American Association of State Highway and Transportation Officials (AASHTO) classification system, respectively.

The subgrade consisted of silty clay, having a liquid limit (LL) of 31 and a plasticity index (PI) of 15. This soil contains 72% silt and 19% clay. The maximum dry density of the soil is 1,670 kg/m^3, with an optimum moisture content of 18.75%, as determined by the Standard Proctor test. The silty clay soil was classified as CL according to the USCS and A-6 according to the AASHTO classification system.

Four different geogrids, GG1, GG2, GG3, and GG4, were used to reinforce the base layer in the pavement test sections. These geogrids have a punched structure and are made from polypropylene with different geometries. The physical and mechanical properties of these geogrids, as provided by the manufacturers, are listed in Table 1.

Table 1. Properties of geosynthetics

Reinforcement	Aperture Shape	T^a, kN/m		J^b, kN/m		Aperture Stability kg-cm/deg	Aperture Size, mm
		MD^c	CD^d	MD^c	CD^d		
GG1 biaxial geogrid	□	4.1	6.6	205	330	3.2	25×33
GG2 biaxial geogrid		6.0	9.0	300	450	6.5	25×33
GG3 triaxial geogrid	△	8.6^e		430^e		3.6	40×40×40
GG4 triaxial geogrid		9.5^e		475^e		7.8	40×40×40
GG5 triaxial geogrid		7.5^e		375^e		-	40×40×40
GT geotextile	-	7.0	26.3	350	1313	-	-

[a] Tensile Strength (at 2% strain) (in accordance with ASTM D6637 for GG1 and ISO 10319:1996 for GG2), [b] Tensile Modulus (at 2% strain), [c] Machine Direction, [d] Cross Machine Direction, [e] Radial Direction

2.1.3 Test Section Preparation

The silty clay subgrade was first placed and compacted in lifts inside a steel test box. The thickness of each lift was 152 mm. The target dry density and water content of subgrade was 1,600 kg/m^3 and 22 percent, respectively, to achieve a weak subgrade soil of CBR = 0.5 percent. After the completion of subgrade preparation, the instrumentations, including pressure cells, piezometers, and soil strain gages, were then installed. The base course layer was prepared by placing the crushed limestone in 152 mm thick lifts, mixing with the desired water, and then compacting to the predetermined height. The target dry density and water content of the base course layer were 2,210 kg/m^3 (i.e. 98% degree of compaction) and 6.0 percent, respectively. The surface layer was consequently prepared by placing prime coat on the top of base layer, followed by placing cold-mix asphalt concrete along the sides of the box with a width of 314 mm. The remaining center area of the test box (1,372 mm wide square) was left for the HMA. The cold mix asphalt at the boundary area of two mixes was covered with tack coat. The HMA was obtained from a local asphalt plant. It was placed in the oven to age for about 4 hours at a temperature of 150ºC. Once the mixture has reached the proper compaction temperature, it was spread over the reserved central area in the test box, raked level, and immediately compacted to the predetermined height using the jack hammer. Figure 2 depicts a typical layout of flexible pavement section with geogrid base reinforcement, including geometric parameters and layout of instrumentations used in this study.

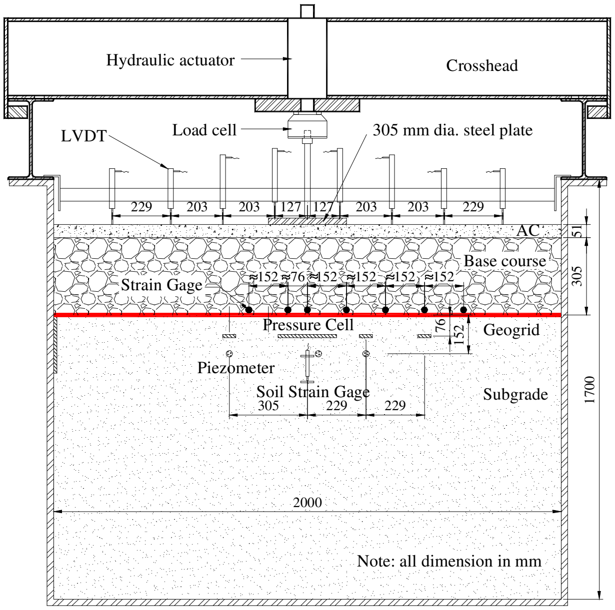
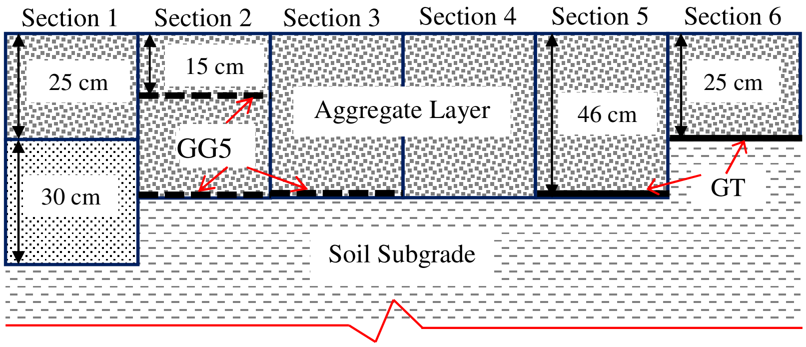


Figure 2 Schematic of indoor test box for cyclic plate load testing

2.2 Full Scale Moving Wheel Load Testing

A total of six test lane sections were constructed over natural soft soil foundations using normal highway construction equipment and procedures, at an outdoor site located at the Pavement Research Facility (PRF) of Louisiana Department of Transportation and Development (LA DOTD) in Port Allen, LA. Each test section is 24 m (80ft) long and 4 m (13 ft) wide. Figure 3a shows the cross sections of the six unpaved test sections. The wheel load testing was first conducted on the unpaved test sections up to 2000 passes or 25 mm (1 inch) rut, whichever comes first (phase 1). The section will be fixed, levelled, and paved using 75 mm (3 inch) HMA and re-loaded again (phase 2) using moving wheels to a rut depth of 19 mm (0.75 inch).

A full-scale accelerated load facility (ALF) was used to apply the moving wheel loads on the six unpaved test lane sections. Figure 3b shows a picture of the ALF with an insert of the dual-wheel assembly. ALF is a testing device that applies unidirectional trafficking to the test sections with a nominal speed of 16.8 km/h (10.5 mph) or 350 passes per hour. ALF has a due-tire axle consisting of two Michelin XZE-model truck tires and applies a 43.4-kN (9,750-lb) axle load, approximately representing half of the standard axle load, the 80-kN (18-kip) single-axle load. ALF has the capacity of applying a normally- distributed wander covering a transverse distance of 76 cm (30 in) to simulate live traffic pattern. The wheel path generated by ALF is about 12 m (40 ft).



(a) Cross sections of the six unpaved test lane sections

(b) Accelerated load facility

Figure 3 Moving wheel load testing

The natural subgrade soil in the study is classified as heavy clay (CL) per USCS or A-7-6 according to the AASHTO classification system. The soil has a PI of 55% with 96.6% passing the #200 sieve. The optimum moisture content is 29.5% at a maximum dry density of 1,305 kg/m^3. The aggregate used for base course construction is Mexican crushed limestone classified as GW or A-1-a. A modified Proctor test of the aggregate yields an optimum moisture content of 9.4% and maximum dry density of 2,066 kg/m^3.

Two geosynthetic products, a triaxial geogrid and a high-strength woven geotextile, are selected for this study and are herein designated as GG5 and GT. The triaxial geogrid was made by means of punching and drawing polypropylene (PP) sheets. The geotextile was made from high-tenacity polypropylene filaments that are formed into weaves. The physical and mechanical properties of GG5 and GT as reported by the manufactures are also presented in Table 1.

The test sections were instrumented by a variety of sensors, including pressure cells to measure the total vertical stresses at the top of the subgrade, piezometers to measure the excess pore water pressure in the subgrade, LVDTs to measure the total deformation of the subgrade, potentiometers to measure the strain at the mid-height of the base aggregate layer, thermocouples to measure temperature variation, and time-domain reflectometers (TDR) to measure moisture variation.

The nuclear density gauge, light falling weight deflectometer (LFWD), Geogauge, and dynamic cone penetrometer (DCP) were used to measure the in-place properties for construction control of subgrade and base course layers. For the subgrade layer, the dry densities varied from 1,044 to 1,192 kg/m^3, with moisture contents ranging from 44 to 52%. The corresponding LFWD moduli, geogauge moduli, and DCP index (DCPI) were in the range of 3.5 to 6 MPa, 20 to 25 MPa, and 150 to 200 mm/blow (for upper 0.5 m), respectively. For the base course layer, the dry densities varied from 1,925 to 2,065 kg/m^3, with moisture contents ranging from 6 to 10%.

The corresponding LFWD moduli, geogauge moduli, and DCPI were in the range of 50 to 70 MPa, 140 to 230 MPa, and 7.5 to 9.5 mm/blow, respectively.

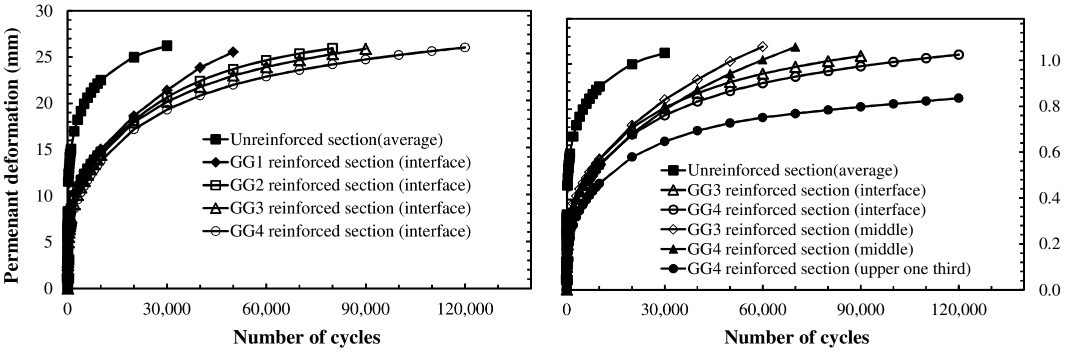
3. TEST RESULTS AND ANALYSIS

3.1 Laboratory Cyclic Plate Load Testing

The development of rut depth (permanent deformation) with number of load cycles is graphically presented in Figure 4a for different types of geogrids placed at the base/subgrade interface, and Figure 4b for GG3 and GG4 geogrids placed at different locations. The results show the trend that the permanent deformation accumulated with the increase in the number of load cycles. The rut depth increase was fast at the early stage of the loading cycles (phase I); however, the rate of increase in rut depth decreased with the increase of the number of load cycles (phase II). By comparing the reinforced sections with the unreinforced sections, one can realize the significant benefit of geogrid base reinforcement on reducing the rut of pavement sections. For example, at 30,000 loading cycles, the total deformation decreased from 26.2 mm for the average of unreinforced sections to 16.4 mm for geogrid reinforced section with GG4 geogrid placed at the upper one third of the base course layer.

Figure 4a shows that the pavement section with GG2 biaxial geogrid base reinforcement with higher tensile modulus performed better than that with GG1 biaxial geogrid reinforcement of lower tensile modulus. The same observation was also observed for GG3 and GG4 triaxial geogrids. Overall, the improvement in the performance of pavement sections is directly proportional to the increase in tensile modulus of the geogrid reinforcement.

Figure 4b shows that the pavement section with geogrid placed at the upper one third of the base coarse layer has the best performance; followed by the pavement section with geogrid placed at the subgrade/base interface; and then by the pavement section with geogrid placed at the middle of base layer. The better performance of the geogrid placed at the upper one third of the base layer indicates that this location is more efficient in reducing the lateral movement of aggregates and provides confinement in the proximity of high vertical and shear stresses in the base layer (Al-Qadi et al., 2008). In addition, the geogrid placed at the upper one third location can help in redistributing the load at a shallow depth to a wider area than those placed at other locations.

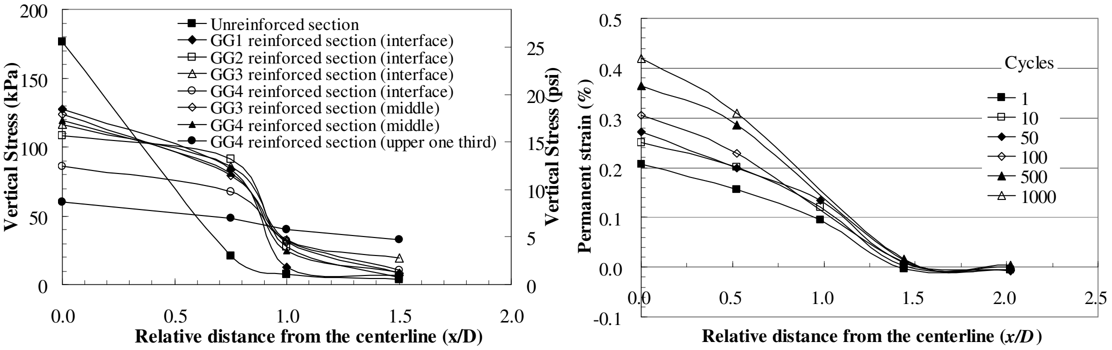


(a) Effect of types of geogrid (b) Effect of locations

Figure 4 Accumulated surface permanent deformations for laboratory cyclic plate load testing

The vertical stress distribution at 76 mm below the subgrade/base interface and along the centreline of the loading plate at the end of 30,000 cycles for both geogrid reinforced and unreinforced sections are shown in Figure 5a. The stresses measured were the total vertical stresses induced by the peak load during each cycle, and the stresses induced by the weight of the soil are not included. The figure shows that the load was redistributed to a wider area in the reinforced test sections, resulting in an improved stress distribution on top of the subgrade layer when geogrids were installed. The magnitude of vertical stress was decreased directly below the plate and increased slightly away from the plate in the reinforced test sections compared to the unreinforced sections.

The typical variations of strains measured along the centerline of the geogrids for different load cycles are presented in Figure 5b. The data demonstrate that significant permanent tensile strains were developed in geogrids, which are believed to restrain the lateral movement of the base course aggregates and hence provide lateral confinement. The highest tensile strains were measured directly beneath the center of the loading plate, where the maximum lateral movement of the base courses were expected to occur, and became almost negligible at a certain distance. The maximum measured strains in geogrids were well below the failure strain for geogrids.



(a) Vertical stress distribution (b) Strain distribution

Figure 5 Stress distribution at the top of subgrade and strain distribution along the geogrid

3.2 Full-scale Moving Wheel Load Testing

Figure 6a presents the results of accumulation of the total permanent deformation with the number of wheel passes for the six unpaved test lane sections. The total permanent deformation for each test lane section shown in Figure 4 is the average of the measurements taken at the different locations along the wheel path in each test section. The control sections, Section 1 and Section 4, exhibited significantly greater total permanent deformation than the reinforced sections under the same number of wheel load passes, indicating the benefits of mobilizing geosynthetics in reducing the permanent deformation in unpaved roads built over soft subgrade soil.

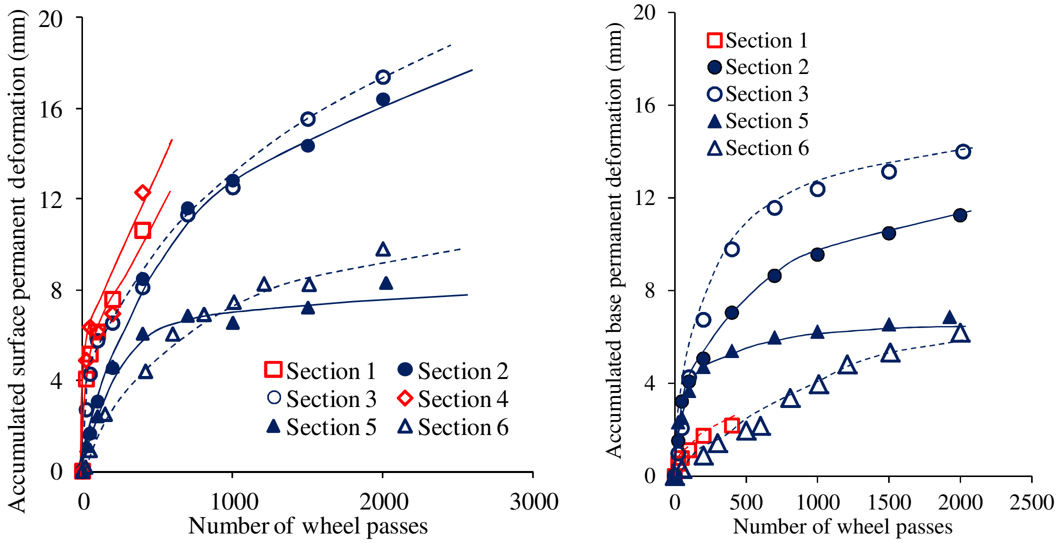
Between Sections 2 and 3, Section 2 reinforced with two layers of geogrids experienced less permanent deformation at the end of the testing. Furthermore, there is less deformation at the early stage of the traffic in Section 2, which is most likely due to the early mobilization of the geogrid layer installed at the upper one third of the base aggregate layer in Section 2. The two control sections, Section 1 and Section 4 showed almost similar performance in terms of the total permanent deformation.

Compared to the test sections (Sections 2 and 3) that are reinforced with triaxial geogrids (GG), the test sections (Sections 5 and 6) with high-strength geotextile (GT), showed less permanent deformations. However, the reader should be very careful before judging the performance of the different geosynthetic reinforced sections. In addition to the presence of geosynthetics or geosynthetic types, there are other factors that can affect the performance of the unpaved test sections in terms of resisting the surface rutting. The deformations needed to mobilize the triaxial geogrids might be different than the high-strength geotextile. The variations in construction such as degree of compaction and subgrade conditions may significantly affect the pavement performance.

Figure 6b presents the derived permanent deformation of the aggregate base layer for each test section except for Section 4 due to the potentiometer malfunction. Sections 2 and 3 reinforced by GG5 geogrid exhibited somehow more permanent deformations in the aggregate base layer than Sections 5 and 6 reinforced with GT. Compared to Section 3, Section 2 with two geogrid layers showed significantly less base aggregate deformation. Although Section 1 showed a higher permanent deformation at the same number of wheel passes, the deformation of aggregate layer in Section 1 is relatively small, indicating that the underlying sand embankment may significantly contribute to the overall permanent deformation.

Foil strain gauges were installed on opposite surfaces at each location to measure the tensile strains developed in geosynthetics with the repetitions of traffic loads. The measurements from strain gauges on opposite surfaces were averaged to account for the flexural bending effect. Figure 7 shows the geosynthetic tensile strains measured at the centerline of the test sections. No reliable results from strain measurements were obtained in Section 5 due to the loss of strain gauges.

Among the geosynthetics installed at the subgrade - aggregate base interface, the geotextile in Section 6 showed higher tensile strain at the end of the loading. Between the two layers of geogrids in Section 2, the geogrid installed at the upper one-third of the aggregate layer developed tensile strains that are more than twice higher than the geogrid installed at the subgrade-aggregate base interface. Overall, Figure 6 demonstrates that the geosynthetics were all mobilized at the end of the pre-rut phase 1 accelerated loading.



(a) On surface

(b) In base aggregate layer

Figure 6 Accumulated permanent deformation

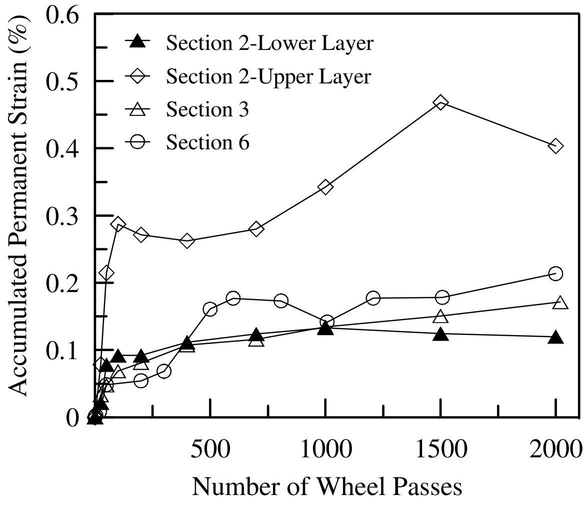


Figure 7 Strains developed in geosynthetics at the center of test sections

4. CONCLUSIONS

Due to the soft nature of subsurface soil and the presence of high water table in the state of Louisiana, roadways are often have to be built over weak subgrade soils, which create many design and construction challenges to state engineers. As an alternative to the traditional method of treating/stabilizing the soft soil with lime or cement, geosynthetic materials were considered and evaluated for stabilizing/reinforcing the subgrade/base layers in roads built over soft subgrades. Two testing programs, laboratory cyclic plate load testing (CPLT) on flexible pavement sections constructed inside a test box and full-scale moving wheel load testing on field test lane sections, were conducted for this purpose. Results from both the laboratory CPLT and the full-scale field testing programs demonstrated the benefits of using geosynthetic for reinforcement/stabilization of road structures in terms of significantly reducing the total permanent deformation/surface rutting and extending the service life of road structures.

The results of laboratory CPLT also showed that the improvement in the performance of the different pavement sections is in the same order with the increase in the tensile modulus of geosynthetics. Better performance was observed when the geogrid layer was placed at the upper one third of the base aggregate layer than that when the geogrid was placed at the base-subgrade interface or at the middle of base layer. The inclusion of geogrid for base reinforcement results in redistributing the applied surface load to a wider area, which results in less accumulated permanent deformation in the subgrade; i.e., the lower maximum vertical stress distribution on top of the subgrade leads to lower permanent vertical strain in the subgrade layer.

Likely due to slightly stiffness variations in the subgrade and base aggregate layers, the two full -scale test lane sections with geotextile showed less permanent deformations than the sections with geogrids.

5. ACKNOWLEDGEMENTS

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