

## ABSTRACT

LENG, JIANJUN. Characteristics and Behavior of Geogrid-Reinforced Aggregate under Cyclic Load. (under the direction of Dr. Mohammed A. Gabr.)

The objective of this study is to investigate the behavior of reinforced unpaved structure under cyclic load through laboratory testing, finite element and theoretical analyses. Main focus of research was on such behavior with degradation of aggregate base layer. Fourteen laboratory large-scale cyclic load plate tests were conducted on unpaved structure sections with two base course thicknesses and several geosynthetic reinforcements placed between base layer and subgrade. Results indicated that reinforcement improved stress distribution transferred to the subgrade, and decreased degradation of base course and surface deformation accumulation. Stiffer geogrids showed better stress attenuation effect and reduced plastic surface deformation as compared with lower modulus geogrids. Degradation was related to base layer thickness and base layer/geogrid interaction. The degradation and permanent surface deformation were correlated to geogrid torsional stiffness. Performance of geogrid-reinforced test sections was simulated using the FEM program ABAQUS. FEM results indicated that geogrid reinforcement can provide lateral confinement at the bottom of the base layer by improving interface shear resistance and increasing mean stress at the bottom of the base layer. The effect of geogrid reinforcement was also shown to reduce surface deformation, improve stress distribution on subgrade layer, and reduce strain induced at the bottom of the base layer due to lateral spread. As ABC thickness decreased, or the elastic modulus

ratio decreased, the benefit due to geogrid reinforcement becomes more apparent. In general, geogrid with higher tensile modulus and better interface friction coefficient enhanced the reinforcement effects. A new unpaved road design model was developed on the basis of geogrid reinforcement mechanisms, degradation of base course, and mobilization of subgrade bearing capacity. Required base course thicknesses calculated using the proposed method compared favorably with results of the field tests reported by Fannin and Sigurdsson (1996).

**CHARACTERISTICS AND BEHAVIOR OF GEOGRID-  
REINFORCED AGGREGATE UNDER CYCLIC LOAD**

By

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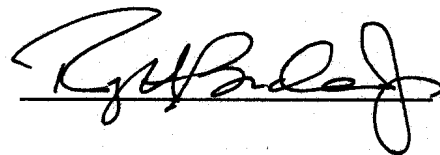
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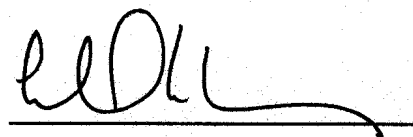
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Chair of Advisory Committee







## BIOGRAPHY

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## **Chapter 1 INTRODUCTION**

### **1.1 Background**

Geosynthetic materials are increasingly being used as reinforcement in earthwork construction such as embankment and roadway systems. The rapid development of geosynthetic reinforcement technology has been accompanied by somewhat slower development of methods of analysis and design. One potential application of geosynthetic reinforcement is its use in paved and unpaved roads. Such use has been expanding in the past two decades, with this trend expected to continue into the future.

According to National Transportation Statistics 2000, there were 1.554 million miles of unpaved road in 1996, which is 39.5% of total 3.934 million miles of public road and street in the United States. In addition, there are 1.066 million miles of low and intermediate type paved road. Low-type here means that the asphalt thickness is less than one inch, and intermediate type means an asphalt thickness between one and seven inches. Unpaved roads and low-type paved roads are usually used for low volume traffic and serve as access roads. Low volume roads play a very important role in rural economy, resource industries (forest, mining, and energy) and transportation for military purposes. When unpaved roads and low-type paved roads are built on soft foundation soils, large deformations can occur, which increase maintenance cost and lead to interruption of traffic service. In general, deterioration of unpaved and paved roads is faster than road replacement. The increasing material and construction costs, and stringent environmental protection requirements make it important to explore alternative construction methods with longer service life but at the same time cost efficient.

The use of geosynthetics in these types of structures may provide such alternative. In these applications, major functions of the geosynthetic materials include filtration, separation, and reinforcement (Koerner, 1994). Geosynthetics provide tensile reinforcement through frictional interaction with base course materials, thereby reducing applied stresses on the subgrade and preventing rutting caused by subgrade overstress. By improving the performances of the roadway structure, geosynthetic inclusions can help increase the service life of the system, or decrease the base course thickness such that a

roadway of equal service life is constructed. Benefits of reducing base course thickness are realized if the cost of the geosynthetic is less than the cost of the reduced base course material, and construction associated with a reduced base thickness (such as excavation, relocation of utilities, and purchase of right-of-way). Geosynthetic reinforcement is particularly attractive in areas where quality gravel sources are scarce, in urban areas where these resources have become depleted, or in environmentally sensitive areas where the siting of gravel quarries is not permitted. In general, benefits derived from the reinforcement function are dependent on the amount of system deformation allowed. Compared with paved roads where only small deformation can be accepted, relatively larger deformations are often acceptable in unpaved roads. Accordingly, the reinforcement function of a geosynthetics can potentially provide significant benefits in unpaved roads.

Within the realm of geosynthetic materials, geotextiles provide good separation, drainage and filtering characteristics, in addition to reinforcement capability. By providing higher tensile strength at low strains, woven geotextiles (with higher tensile modulus) are generally considered better reinforcement materials than nonwoven geotextiles (with low tensile modulus). For geotextile-reinforced unpaved structures, there are currently two design methods, which were developed by Giroud and Noiray (1981) and Milligan et al. (1989a and 1989b). In the Giroud and Noiray (1981) method, the static performance of reinforced and unreinforced base courses was compared to estimate a thickness reduction due to reinforcement inclusion, with consideration for membrane effect and improvement in bearing capacity of subgrade. The required thickness of unreinforced base layer as a function of repeated loads is calculated using empirical formulas. The method proposed by Milligan et al. (1989a and 1989b) was based on the static equilibrium of a wedge under plane strain condition, with assumption that the reinforcement can completely carry interface shear stress between base layer and subgrade. An empirical formula is used to calculate an equivalent monotonic load as a function of the cyclic load amplitude and the number of cycles.

Another type of geosynthetic material used in reinforcement application is geogrid, which offers improved interface shear resistance due to interlocking as compared to geotextile. A currently available semi-empirical design method using

geogrids was developed by Giroud et al. (1984), based on some theoretical considerations and data from limited field trials of unreinforced sections. This method followed the same logic used for the geotextile-reinforced unpaved road design method (Giroud and Noiray, 1981). The difference between the two methods lies in improved stress distribution was determined for the geogrid-reinforced structure using finite element analysis with linear elastic assumption.

## 1.2 Problem statement

This research is focused on developing improved model for analysis and design of geogrid-reinforced unpaved structures under cyclic loads. Unpaved structures are used for either temporary or permanent transportation purposes, such as haul roads, access roads and parking lots.

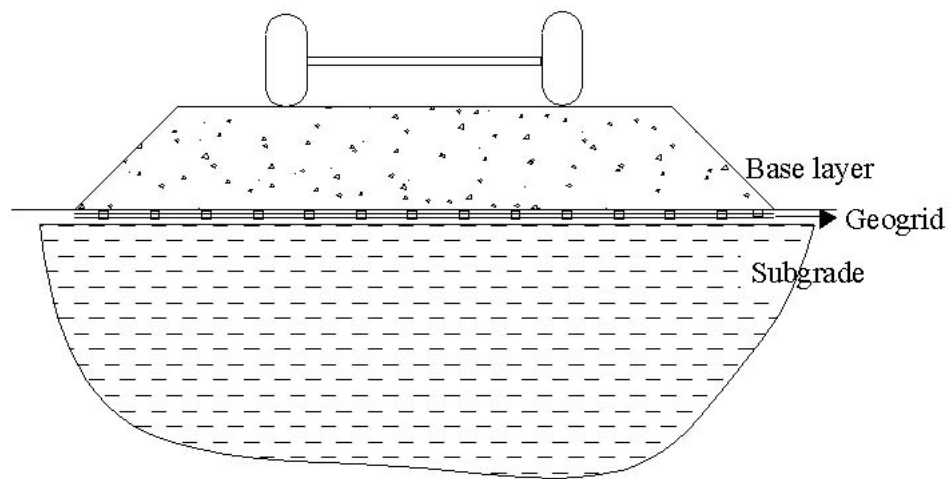


Figure 1. Typical section of reinforced unpaved road

Figure 1 shows a typical section of reinforced unpaved road, which consists of a aggregate base layer, a subgrade layer, and a reinforcement layer usually placed between the base course and subgrade. The base course and geogrid transmit the traffic load to the top of the subgrade, which will deform under the transmitted stress. Under repeated load,

the behavior of the base-geogrid-subgrade system is complicated. The overall behavior depends on the properties of geosynthetics, soil characteristics, and the interaction between the soil and the reinforcement.

Some researches (Milligan and Love, 1984; Fannin, 1987; Fannin and Sigurdsson, 1996) have been conducted on the behavior of geogrid-reinforced unpaved structures by means of model tests under monotonic loading, model tests under cyclic loading, a field test program of unpaved road. Although these studies have provided data that aid in describing the mechanisms of geosynthetic reinforcement, more experimental information is needed to fully understand the behaviors of the composite system is not available. Additionally, past efforts to provide design solutions have been largely based on empirical relationships and considerations. The existing design method (Giroud et. al., 1984) used for unpaved structure was based on static plane-strain analysis and empirical equation from unreinforced unpaved roads (Hammit, 1970; Giroud and Noiray, 1981).

### **1.3 Scope and objectives**

The main objective of the research is two fold. First to understand the mode of geosynthetic reinforcement to the stability of unpaved roads and how this contribution is manifested as a function of the deformation level. The second objective is to develop an improved design method that encompasses the discerned contribution of reinforcement with allowance for degradation of the aggregate base course and cyclic loading.

The research scope includes experimental and theoretical studies. Cyclic plate loading tests on geogrid-reinforced unpaved structure are conducted. Based on the test data, numerical and theoretical analyses have been performed to study and model the contribution of the reinforcement to unpaved section performance. Using the developed model, a parametric study is performed to identify key factors related to the design of reinforced unpaved roads. These factors are quantified and an improved design method for reinforced unpaved structure is proposed.

### **1.3.1 Experimental study**

The objective of experimental study is to measure the load deformation response and stress distribution of test sections during cyclic plate load testing, with different reinforcement grades and types, and two kinds of base course layer thickness. A total of fourteen cyclic load tests are performed on reinforced and unreinforced soil sections composed of aggregate base course (ABC) layer overlying soft subgrade layer. The geosynthetic reinforcement is installed at the interface between ABC layer and subgrade layer. The ABC is obtained from a local quarry; the subgrade soil was a mixture of 85% Lillington sand and 15% Kaolinite, with the CBR value of 3. The tests were performed in a 1.5 m × 1.5 m × 1.35 m (length × width × depth) steel box. The thickness of the subgrade layer is maintained at approximately 0.75 to 0.90 m. The thickness of ABC layer is 152 mm or 254 mm. Geosynthetic reinforcement is achieved using Tensar BX 1100 geogrid, BX 1200 geogrid, BX 4100 geogrid, BX 4200 geogrid, an experimental geogrid (Max30), a drainage geonet (DC6200) with and without BX1100 reinforcement. Vertical stress distribution on the top of subgrade and surface deformation are measured during the cyclic tests.

### **1.3.2 Analysis and modeling of reinforced unpaved structure**

The analytical study includes characterization of permanent deformation and degradation under cyclic load, analysis of stress distribution and soil geogrid interaction and modeling of geosynthetic reinforcement mechanisms for unpaved road design.

#### **i) Degradation and plastic deformation analysis**

The base course degrades during the cyclic loading because of contamination due to subgrade pumping and breakdown of aggregate particles, with some thickness decrease due to lateral spread. The degradation is represented as a decrease in load spread ability (stress attenuation) of base course under cyclic load. Based on the stress data from cyclic loading tests, the degradation of base course with number of cycles is evaluated in terms of stress distribution angle and elastic modulus ratio.

Under cyclic loading, the plastic deformation of unpaved structure accumulates. If the accumulated surface deformation is greater than acceptable deformation, it is called rutting failure. The plastic strain of both subgrade and base layers leads to plastic surface deformation of the unpaved structure. The plastic deformation of an unpaved structure is studied based on surface deformation data from cyclic loading tests. A method is proposed to predict the plastic deformation of unpaved structures under cyclic load, with consideration for base layer thickness and geogrid torsional stiffness.

## **ii) Finite element analysis**

Static finite element method (FEM) is used to analyze stress and strain distribution of unpaved sections using elasto-plastic soil properties and a friction model for the soil-reinforcement interaction. The modeled unpaved sections are analyzed under axi-symmetric conditions, with different reinforcement stiffness, interface properties, and thickness of the aggregate base layer.

The analysis is conducted considering base course and subgrade layer to be stress dependent and with isotropic elasto-plastic models (extended Drucker-Prager model) used to simulate constitutive relationship. Geosynthetic reinforcement is simulated using membrane elements, which can transfer in-plane normal tensile stress only. Interfaces of base course and subgrade, and interfaces of geosynthetic and soils are simulated by interface friction model. Stresses, strains and deformations of the modeled sections and the shear-resistance interaction at the interface are numerically evaluated and presented. Different modulus ratios of aggregate base course and subgrade are used during the static FEM analysis, to approximately simulate the degradation of modeled test section under cyclic load.

## **iii) Reinforcement mechanism analysis and modeling**

It is hypothesized that geosynthetic reinforcement at the interface of subgrade and base course can improve the engineering behavior of the unpaved structure. The modeled sections under axi-symmetric condition are studied for this purpose, with considerations of geosynthetic/base aggregate interaction and geosynthetic/subgrade interaction. Improvement due to geosynthetic reinforcement, in terms of stress and strain distribution,

stress transfer, and deformation, is discussed. The increase of subgrade bearing capacity, geosynthetic tension membrane effect and the decrease of base layer degradation under cyclic load due to reinforcement are also investigated.

### **1.3.3 Design method development**

Based on results from the cyclic load plate tests and analysis of geogrid-soil layer performance, a design method is proposed. The method is proposed based on axisymmetric condition, with consideration of the aggregate-geogrid interaction, the degradation of unpaved roads, and mobilization of subgrade bearing capacity. The proposed design method has been compared to the field test data (Fannin and Sigurdsson, 1996).

## **Chapter 2 LITERATURE REVIEW**

### **2.1 Mechanisms of soil reinforcement**

Geotextiles and geogrids are the two main geosynthetic products usually used for soil reinforcement. While geotextiles can be used for separation, drainage and filtration, or as reinforcement element, geogrids are mainly used for reinforcement applications. Stiff geogrids with aperture sizes properly configured for the intended backfill material size offer high tensile moduli and lateral confinement effects (due to interlocking). Previous studies (Giroud and Noiray, 1981; Giroud et. al, 1984; Perkins et. al., 1997) involving geosynthetic reinforcement of roadways have identified three reinforcement mechanisms: lateral confinement, increased bearing capacity, and tension membrane effect. These three mechanisms were originally based on observation and analysis under static load. They were also observed by some other studies under cyclic loading condition (Fannin, 1987; Haas et. al., 1988; Webster, 1992).

#### **2.1.1 Lateral confinement**

Lateral confinement (Figure 2.1(a)) is induced by frictional interface and interlocking between the aggregate base course and the geosynthetic. Repeated wheel loads induce shear stress at the bottom of base layer and create a spreading effect of the base layer over subgrade. Such spreading may be reduced if the geosynthetic is properly positioned at the location of maximum lateral strain within the subject layer. The interface shear resistance between base course aggregate and the geosynthetic transfers shear stresses from the base layer to the geosynthetic reinforcement. Such action can limit the extensional tensile and shear strains in the base course layer. As lateral movement of base course aggregate leads to vertical strain (and rutting of unpaved road), lateral confinement can effectively limit the plastic deformation.

By interlocking the aggregate, geogrids provide confining effect on the base layer and therefore increase the modulus of base layer. Geogrids can also reduce lateral sliding or displacement of aggregate, which results in less vertical deformation of the roadway

surface. Geotextiles provide little benefit if any with regard to lateral displacement because of relatively poor frictional characteristics between the aggregate and geotextiles (Webster, 1992).

### **2.1.2 Increase of the bearing capacity**

The function of increasing the bearing capacity (Figure 2.1(b)) is attributed to the “forced” initiation of the potential failure surface along an alternate plane, with modified configuration, providing a higher total resistance. The geosynthetic reinforcement can decrease the shear stresses transferred to the subgrade and provide vertical confinement on the subgrade outside of the loaded area where heave happens, thus decrease the shear strain near the top of subgrade and limit subgrade rutting and upheaval. The bearing failure model of subgrade may change from punching failure without reinforcement to general failure with ideal reinforcement. Biquet and Lee (1975) initially established this finding.

### **2.1.3 Tension membrane effect**

The tension membrane effect (Figure 2.1(c)) develops as a result of vertical deformation creating a concave shape in the tensioned geosynthetic layer. The vertical component of the tension membrane force can reduce the vertical stress acting on the subgrade. Some displacement is needed to mobilize the tension membrane effect. Generally, a higher deformation is required for the mobilization of tensile membrane resistance as the stiffness of the geosynthetic decreases. In order for this type of reinforcement mode to be significant, there is a consensus that the subgrade CBR should be less than 3 (Barksdale et al., 1989).

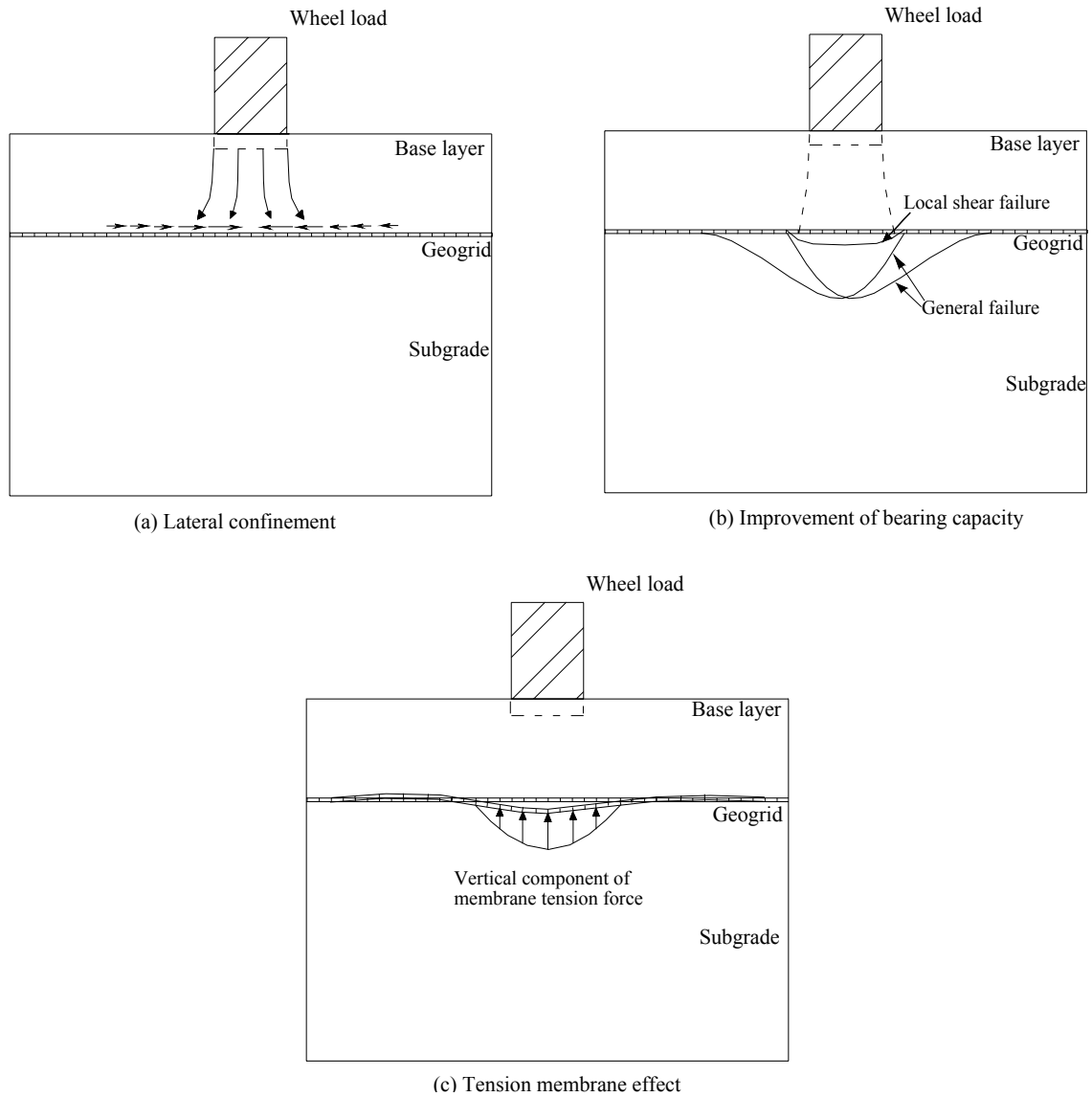


Figure 2. Three mechanisms of soil reinforcement

## 2.2 Analysis for layered system

For an unpaved structure, transient traffic load is directly applied on the top of the aggregate base layer. The subgrade soil and aggregate layers both exhibit non-linear stress-strain relationships, which are influenced by a range of variables including soil properties and loading conditions. On the other hand, the low frequency cyclic loading condition due to traffic is different from earthquake, or machine vibration problems. It is

difficult to analyze the cyclic stresses and strains in the aggregate and subgrade. There is a lack of well-documented field observations of unpaved structures' performance. Therefore, simplifications are often made in order to simulate loading condition, and stress distribution, and compute deformation. In analysis and design, a single wheel loading is usually represented by uniformly distributed pressure over a circular area, and both base and subgrade layers are assumed to be elastic materials.

### 2.2.1 Two-layer system elastic theory

For flexible circular foundation under uniform load, the deflections of a two-layer soil system have been investigated by several researchers (Burmister, 1943; 1956; Ueshita and Meyerhof, 1967; Huang, 1969).

For the axi-symmetrical problem (Figure 3), the basic equations to determine stress distribution satisfy equilibrium and compatibility relationships. For a surfaced load of  $-mI_0(mr)$ , the vertical displacement of the surface is given as follow (Milovic, 1992):

$$w(r) = I_0(mr) \frac{2(1-\mu_1)}{E_1} \cdot \left[ \frac{1 + 4Kmhe^{-2mh} - KLe^{-4mh}}{1 - (L + K + 4Km^2h^2)e^{-2mh} + KLe^{-4mh}} \right] \quad (1)$$

Where,

$$n = \frac{E_2(1+\mu_1)}{E_1(1+\mu_2)}$$

$$K = \frac{1-n}{1+n(3-4\mu_1)}$$

$$L = \frac{(3-4\mu_2) - n(3-4\mu_1)}{(3-4\mu_2) + n}$$

$I_0$  = Bessel function of the first kind and order of zero;  $m$  = dimensionless parameter;  $r$  = horizontal distance from centerline;  $h$  = thickness of the first layer;  $E_1, E_2$  = elastic modulus of first layer and second layer;  $\mu_1, \mu_2$  = Poisson's ratio of first layer and second layer.

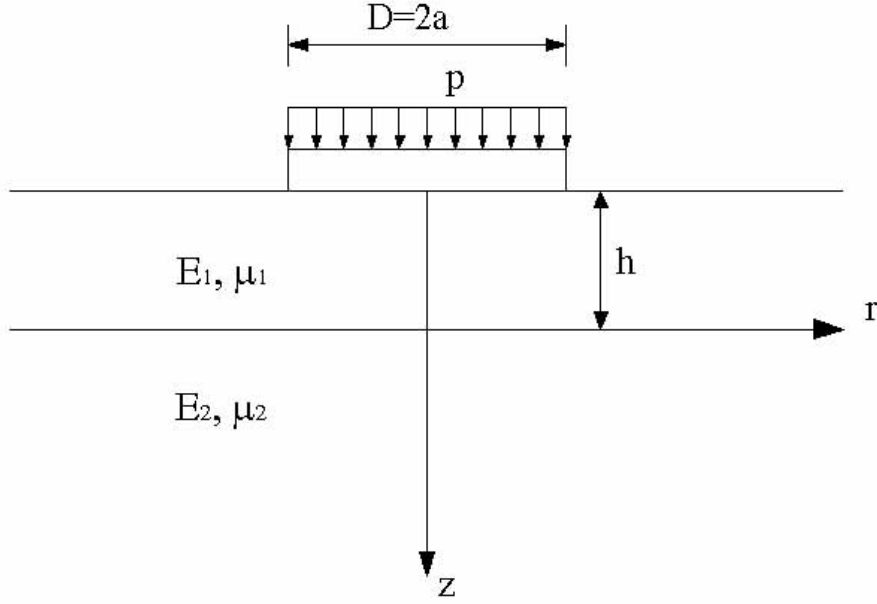


Figure 3. Geometry of the two-layer problem

For the stresses and deformation at the interface between two layers, Burmister (1943) obtained the following equations:

$$\sigma_z = mI_0(mr) \left\{ \frac{[1+mh-L/2-0.5K(1+2mh)]e^{-mh} + [KL(1-mh)-L/2-0.5K(1-2mh)]e^{-3mh}}{1-(L+K+4Km^2h^2)e^{-2mh} + KLe^{-4mh}} \right\} \quad (2)$$

$$\tau_{rz} = -mI_1(mr) \left\{ \frac{[mh+L/2-0.5K(1+2mh)]e^{-mh} + [KLmh-L/2+0.5K(1-2mh)]e^{-3mh}}{1-(L+K+4Km^2h^2)e^{-2mh} + KLe^{-4mh}} \right\} \quad (3)$$

$$w = I_0(mr) \frac{1+\mu_1}{E_1} \left\{ \frac{[2-2\mu_1+mh+L/2+0.5K(3-4\mu_1)(1+mh)]e^{-mh} - [KL(2-2\mu_1-mh)+L/2+0.5K(3-4\mu_1)(1-2mh)]e^{-3mh}}{1-(L+K+4Km^2h^2)e^{-2mh} + KLe^{-4mh}} \right\} \quad (4)$$

If the elastic properties ( $E$  and  $\mu$ ) are equal in the two layers, the coefficients  $K$  and  $L$  are equal to zero and the above equations reduce to Boussinesq's equations. The main assumptions in layered elastic theory are that the two-layer system is linear elastic,

and there is no relative displacement at the interface between two layers (perfectly rough interface).

### 2.2.2 Interface of the two-layer system

Based on elastic analysis, Fox (1948) provided a solution to the vertical stress  $\sigma_z$  on the top of second layer for perfectly rough interface and perfectly smooth interface. Figure 4 provides the vertical stress on the axis for the case with  $a/h=1$ . Here  $a$  = radius of the circular footing,  $h$  = thickness of the first layer,  $d$  = depth,  $p_z$  = the vertical pressure on the circular footing,  $p_0$  = the pressure on the circular footing. As shown in Figure 4, the first layer transfer less vertical stress to the second layer if the interface is rough. The vertical stress ratio of rough interface / smooth interface is 0.646-0.722, 0.292-0.305 and 0.081-0.082 for  $E_1/E_2 = 1, 10, 100$ . As the elastic modulus increases, the advantage of rough over smooth interface reduces with almost no advantage when  $E_1/E_2 = 100$ .

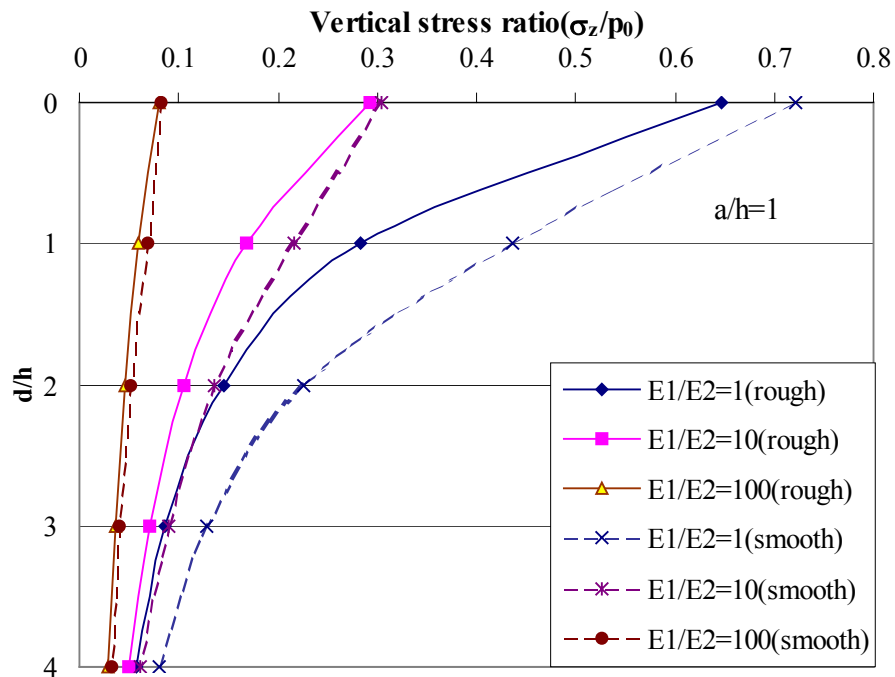


Figure 4. The vertical stress distribution on the second layer of two-layer system

(Fox, 1948, data from Poulos, 1973)

However, relatively high shear stress is usually seen at the interface of base and subgrade layer for unpaved structure. If the stress is higher than the shear resistance at the interface, there is tendency of base layer to spread laterally. Lateral spreading will result in increase of the vertical deformation and decrease of the modulus of the base layer. Stress redistribution will take place and more vertical load will be transmitted to the subgrade layer. In this case, geosynthetic reinforcement placed at the interface of base course can resist the shear stress and improve the stress distribution on the subgrade and thereby reducing the plastic deformation.

### **2.2.3 Nonlinear properties of unbound materials**

Linear elastic analysis becomes inappropriate for unpaved or thinly paved structures, whose responses are dominated by the nonlinear properties of granular materials and subgrade soils. Based on linear elastic analysis, there are usually high tensile stresses computed at the bottom of the base layer. The unbound materials have negligible tensile strength, which comes from soil suction and interlocking. If there is a negative incremental horizontal stress (or tensile stress) at the bottom of base layer, failure will occur in a zone when horizontal compressive stress is too low to compensate. Selig (1987) explained that local failure with each loading would lower the stiffness of aggregate at the bottom of the base, thus decreasing or eliminating the tensile stress induced.

Under tensile stresses generated by traffic load, the unbound material will spread laterally and stress will be redistributed. In performing finite element analysis assuming the elastic layers, the “unrealistic” high tensile stress problems may be numerically solved by replacing the tensile stresses in the elements with negative normal mean stresses which sets tensile stresses to zero. Using equilibrium, the analysis is iterated until the maximum tensile stress becomes lower than a given limiting value. Some pavement analysis programs (Kenlayer, Illi-Pave and Mich-Pave) have incorporated nonlinear elastic models or plastic models for resilient properties of the granular materials. Such

characterization provides a more reasonable simulation of the stress distribution within the system.

## 2.3 Soil behaviors under repeated load

### 2.3.1 Resilient soil behavior

Resilient soil properties under repeated load have been reported in previous research. Resilient modulus was introduced by Seed et al. (1962) and defined as dynamic deviator stress divided by recoverable strain under a transient dynamic pulse load. Used for material characterization of unbounded pavement material layers (subgrade, subbase and base), the resilient modulus has become widely utilized in pavement analysis.

Early researchers provided linear relationships between California bearing ratio (CBR) and resilient modulus, where the resilient modulus was not stress-depend. Heukelom and Foster (1960)'s empirical equation was expressed as:

$$E_r = 10 \text{ CBR(MPa)} \quad (5)$$

Where  $E_r$  = resilient modulus;

However, the results from lab testing (Hicks and Monismith, 1971) and back-calculation of in-situ deflection tests (Brown and Pell, 1967), clearly showed that the resilient responses of both subgrade and base material were highly non-linear. The resilient modulus was related to mean normal stress and deviator stress. The most well known and widely used model is the  $k$ - $\theta$  model (Brown and Pell, 1967; Hicks and Monismith, 1972). This model was the first to describe the results of repeated-load triaxial tests with constant confining pressure. The model was expressed as:

$$E_r = k_1 p_a \left( \frac{3p}{p_a} \right)^{k_2} \quad (6)$$

$P$  = mean normal (principal) stress, defined by:

$$p = \frac{\sigma_1 + 2 \cdot \sigma_3}{3} = \frac{\theta}{3}$$

$p_a$  = reference stress equals 100 kPa;

$k_1, k_2$  = material parameters depending on the state and quality of the material;

$\sigma_1$  = principal vertical stress;

$\sigma_3$  = principal horizontal (cell) stress;

$\theta$  = bulk stress =  $\sigma_1 + 2 \sigma_3 = 3p$ ;

In this model, the Poisson's ratio is constant and the value generally adopted is  $\nu = 0.3$ .

More recently, Uzan et al. (1992) modified the initial  $k$ - $\theta$  model, by assuming that the resilient modulus depends on both the mean normal stress ( $p$ ) and the deviator stress ( $q$ ), as follows:

$$E_r = k_1 p_a \left( \frac{3p}{p_a} \right)^{k_2} \left( \frac{q}{p_a} \right)^{k_3} \quad (7)$$

Deviator stress  $q$  was defined by:

$$q = \sigma_1 - \sigma_3$$

$K_1, K_2, K_3$  are material parameters depending on the state and quality of the unbound granular material.

### 2.3.2 Permanent deformation

Both subgrade and aggregate base course are essentially elasto-plastic materials. If plastic deformation accumulated beyond a limit, it is called rutting failure. Plastic deformation of base course and subgrade is an important consideration for the analysis of unpaved road and flexible pavement. Compared with resilient behavior, less successful research has been devoted to permanent deformation. Some empirical models of subgrade and base course have been proposed based on cyclic triaxial test results.

O'Reilly et al. (1989) demonstrated that silty clay subgrade responded in a viscous manner and it was possible to apply transient stresses above the static yield surface without significant plastic strains developing immediately. However, under cyclic loading, such strains may accumulate, their magnitude depending on the cyclic deviator

amplitude. Brown et al. (1987) reported this type of behavior for overconsolidated silty clay with OCR values of 6, 12, and 18. The specimens were tested in undrained condition with pore pressure and deformation measurement. The results indicated a possible threshold stress level, above which plastic strains accumulated and below which the strain and pore pressures were negligible. A similar pattern can be found from the data obtained by Loach (1987) from repeated load triaxial tests on compacted specimens of three clays with degree of saturation in excess of 85 %. These results suggested a simple design criterion for subgrade to prevent significant permanent deformation, and the ratio of deviator stress to mean normal effective stress need to be kept below a critical value. A more extensive testing by Cheung (1994) on compacted clays produced the relationship of plastic strain vs. repeated deviator stress, shown in Figure 5. The repeated loading tests involved 1000 cycles at a frequency of 2 Hz on compacted, unconfined London Clay. The results demonstrated a sharp change in slope at deviator stress of approximately 25 kPa. Chueng (1994) proposed the following relationship between the accumulated plastic strain ( $\epsilon_p$ ) and cycle number (N) based on testing up to 1000 cycles.

$$\epsilon_p = A \left( \frac{q_r}{s} \right)^b (\log N + B) \quad (8)$$

Where A, b and B are coefficient for the particular soil; s is the shear strength of the soil;  $q_r$  is the repeated deviator stress.

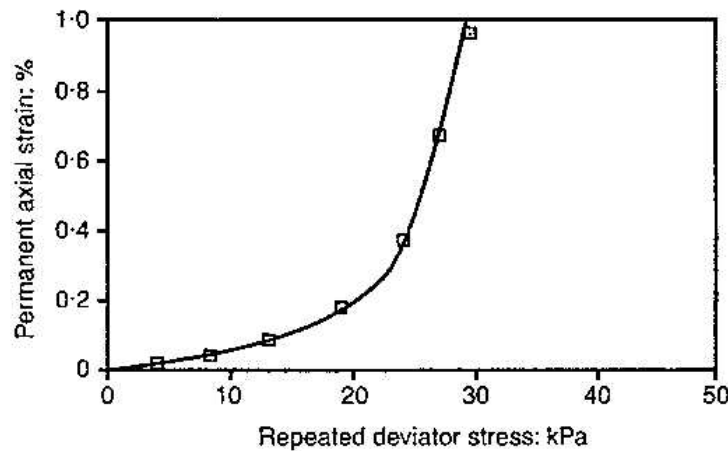


Figure 5. Plastic strain after 1000 cycles against repeated deviator stress for compacted silty clay (after Cheung, 1994)

Raymond and Komos (1978) studied permanent settlement of footing under cyclic loading, by conducting laboratory model tests of strip footings with widths of 75 mm. and 228 mm. resting on Ottawa sand, with various magnitudes of cyclic load ( $\sigma_d/q_u=13.5 - 90\%$ ).  $\sigma_d$  is the average pressure on the footing and  $q_u$  is the ultimate static bearing capacity. The load settlement relationships obtained from the tests for 228 mm footing are shown in Figure 6.

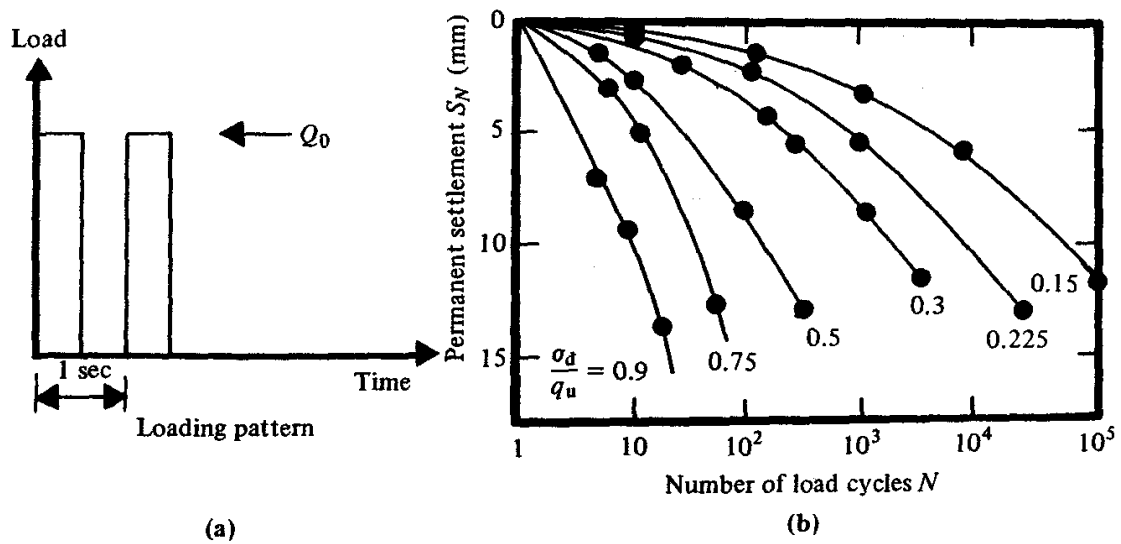


Figure 6. Plastic deformation due to repeated loading in plane strain tests  
(Raymond and Komos (1978), after Das (1983))

An empirical relationship (Raymond and Komos, 1978) of the permanent settlement of the footing ( $S_N$ ) and the number of cycles of load ( $N$ ) was given as:

$$S_N/\log N = a + bS_N \quad (9)$$

Where,  $a$  and  $b$  are two constants related to the width of footing and the magnitudes of cyclic load.

With regard to granular materials, previous experimental results revealed that the permanent deformation of unbound granular materials is affected by several factors

including stress level, number of load applications, stress history, and granular material properties (moisture content, density, grading and aggregate type). Several empirical models described the effects of the number of load repetitions and applied stresses on the plastic strain. Barksdale (1972) proposed the variation of permanent strains with the number of cycles as follow:

$$\epsilon_p = a + b \log(N) \quad (10)$$

Where, a and b are regression parameters.

Hornych et al. (1993) proposed a model for plastic strain after first 100 cycles ( $\epsilon_{1,p}^*$ ):

$$\epsilon_{1,p}^* = A \left( 1 - \left( \frac{N}{100} \right)^{-B} \right) \quad (11)$$

Here A and B are two positive parameters. A value is related with the stress level.

### 2.3.3 Degradation of subgrade and base course

#### Subgrade degradation

Undrained shear strength of subgrade is an engineering property, which governs the behavior of the soft subgrade. The progressive deterioration of the subgrade soil can be expressed by the decrease of its undrained shear strength as the number of the load cycles increases. Coefficient  $\lambda$  proposed by Giroud et al. (1984) represents the progressive deterioration or fatigue of the subgrade soil under cyclic loading due to traffic, with the empirical equation:

$$\lambda = C_{uN}/C_u = 1 / \left[ \frac{1 + (\log N)^{3/2} C_u}{1000} \right] \quad (12)$$

Where,  $C_{uN} = \lambda C_u$  = undrained shear strength of the subgrade at the passage of N (kN/m<sup>2</sup>);  $C_u$  = undrained shear strength of the subgrade before or at the passage of 1 (kN/m<sup>2</sup>);

## **Degradation of aggregate base course**

During the cyclic loading test, aggregate material generally experiences initial compaction, which can result in a little improvement of mechanical properties, followed by progressive deterioration or degradation that may decrease the effective thickness and the mechanical properties of the aggregate. The degradation of the aggregate base course gradually increases stresses on the subgrade soil. For unpaved structure, progressive deterioration of the base layer occurs through the following mechanisms (Giroud et al., 1984):

- 1) Lateral displacement of the base layer material resulting from tensile and shear strains related to bending and low confining stresses at the bottom of the base layer;
- 2) Contamination of the base layer by fine particles moving upward from subgrade, especially when the subgrade is very soft ( $BCR < 3$ );
- 3) Sinking of the base course layer aggregate into subgrade soil;
- 4) Breakdown of base layer aggregate due to repeated loading;

## **2.4 Geogrid reinforcement under cyclic load**

### **2.4.1 Geogrid constitutive relationship**

Geosynthetic materials are known to exhibit visco-elasto-plastic behavior that is direction-dependent. Rigo and Perfetti (1982) proposed a rheological model for geosynthetics under static and cyclic loading. The model consists of springs, dashpot and ratchet. The springs represent elastic, recoverable strains, the dashpot accounts for the time-dependent viscous component of the displacement, and the ratchet represents the unrecoverable plastic strain. Perkins (2000) provided a constitutive model of geosynthetics as direction-dependent elastic, plastic, and time-dependent creep materials, shown in Figure 7.

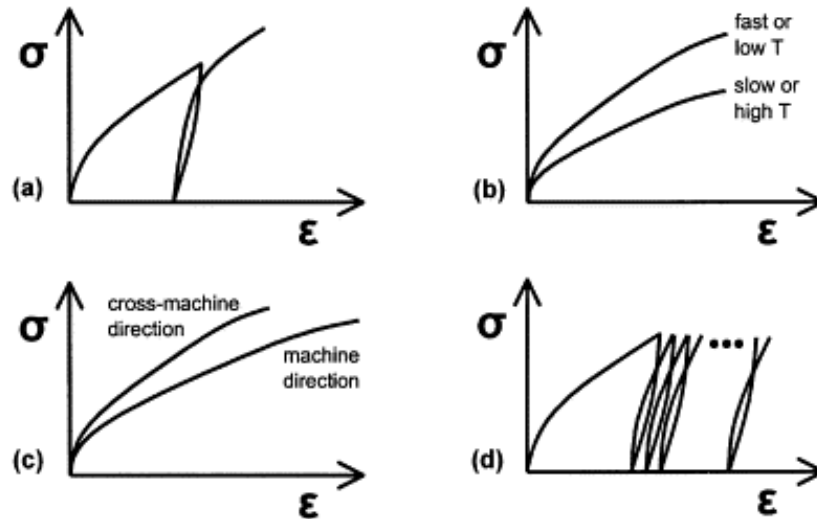


Figure 7. Stress-strain behavior of geosynthetics (a) elastic-plastic (b) thermovisco  
(c) anisotropic (d) ratcheting (after Perkins, 2000)

Nicola and Filippo (1997) tested two types of geogrids in HDPE (High Density Polyethylene) and PET (Polyester) under cyclic loading. The unload-reload tensile modulus was mainly a function of the applied load and secondarily a function of cycle frequency. It increased with frequency and decreased with tensile load. The modulus increased during the first 10 cycles. Afterward it remained mainly constant when tensile load  $T \leq 40\%T_{\max}$  (maximum tensile strength), or decreased if  $T > 40\%T_{\max}$ .

#### 2.4.2 Aggregate - geogrid interaction

The shear-resistance interaction of geosynthetics and soils is usually evaluated by pullout tests. For sheet or strip reinforcement, the soil reinforcement interaction is controlled by friction between the soil and the reinforcement. As schematically illustrated in Figure 8 by Wrigley (1984), the soil reinforcement interaction is controlled by friction between the soil and the reinforcement, the friction between soil and soil, and the bearing resistance of the soil on the transverse member of grid.

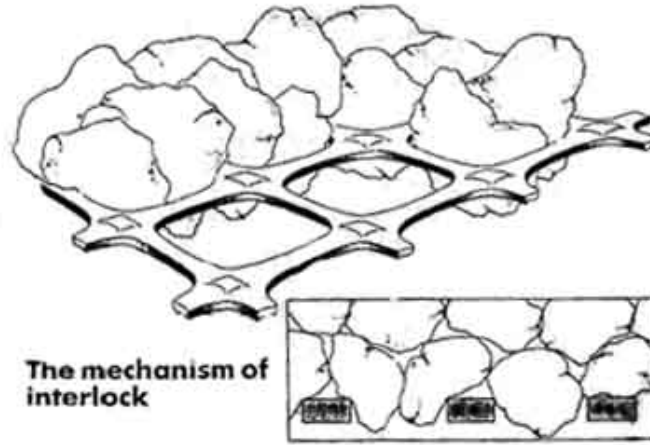


Figure 8. The mechanism of interlock (Wrigley, 1989)

Shear resistance between the reinforcement and soil has two components: the shear resistance between the soil and the reinforcement-plane surface area, and the soil-to-soil shear resistance at the grid opening (Jewell et al., 1984). The shear resistance was expressed by Jewell et al. (1984) as:

$$P_s = \sigma_n A [\alpha_{ds} \tan \delta + (1 - \alpha_{ds}) \tan \phi_{ds}] \quad (13)$$

Where,  $\sigma_n$  is normal stress,  $\phi_{ds}$  is the friction angle of soil in direct shear,  $\delta$  is the skin-friction angle between the reinforcement shear surface,  $\alpha_{ds}$  is the ratio between the reinforcement shear area and the total shear area, is the normal stress at the shear plane, and A is the total shear area.

The passive bearing resistance is evaluated by bearing capacity theory (Matsui et al., 1996):

$$P_{sb} = \frac{F}{W} = Nd\sigma_n N_q \quad (14)$$

Where,  $F_b$  is total bearing resistance, W, N, d are width, numbers, diameter of transverse members respectively,  $\sigma_n$  is normal stress acting on the transverse members. The bearing

resistance can be determined by using either general failure model (Perterson and Anderson, 1980) or punching shear failure mode (Jewell et al., 1984).

The overall pullout resistance is established with respect to an interaction factor  $F^* \alpha$  (Christopher et al., 1990; Chang et al., 1995) or an apparent coefficient of friction  $\mu^*$  (Ingold, 1982), defined by the following equation:

$$F^* \alpha = \mu^* = \frac{P}{2LW\sigma'_v} = \frac{\tau_{av}}{\sigma'_v} \quad (15)$$

Where, P is the pullout force, L is the embedment length, W is the specimen width,  $\tau_{av}$  is the mean shear stress acting on the specimen, and  $\sigma'_v$  is the effective vertical stress.

As geosynthetics made of polymeric material are relatively extensible, the pullout resistance is mobilized through progressive strain of geosynthetics. The interaction factor  $F^* \alpha$  for static loading tests is governed by the magnitude of relative displacement at the geosynthetic- soil interface. Christopher et al. (1990) suggested that the interaction factor for dynamic loading be taken as 80% of that for static loading. Raju and Fannin (1998) presented the results of pullout tests on HDPE and PET geogrids under monotonic and cyclic loading. PET geogrids showed higher pullout resistance than HDPE geogrids. On the other hand, HDPE geogrid yielded a pullout resistance in cyclic tests greater than or equal to the monotonic response. In contrast, PET geogrid yielded a cyclic resistance less than or equal to the static response.

Koerner (1997) provided direct shear test data, which showed that biaxial geogrid and sand interface shear resistance angle is close to the shear resistance angle of the test soil (efficiency= 97%-107%). The tests were performed in 450mm × 450mm shear box, with the test soil being sand with shear resistance angle of 43-46 degree.

## 2.5 Unpaved structure design methods

In the unpaved road design, a major concern is to prevent rutting failure and subgrade bearing capacity failure under traffic load. The performance of unpaved road on

soft subgrade can be improved by increasing base layer thickness and using geosynthetic reinforcement. For unreinforced unpaved roads, the current design methods (Hammit, 1970; Giroud and Noiray, 1981) are based on empirical design equations from filed tests. For reinforced unpaved roads, there are mainly two design methods based on two different mechanisms: small displacement mechanism and large displacement mechanism. All the methods are based on the analysis under plain-strain condition.

### 2.5.1 Unreinforced unpaved road design methods

An extensive testing program on unreinforced unpaved roads has been performed by Corps of Engineer (Hammit, 1970). A formula was proposed for determining the thickness of aggregate for unpaved structure as to produce a rut depth less than 3 in (or 75mm). The formula converted to the SI-system is as follows:

$$h_{os} = (0.0236 \log N + 0.0161) \sqrt{\frac{P}{\text{CBR}}} - 17.8 A \quad (16)$$

Where,  $h_{os}$  = design thickness of the base layer (m);  $N$  = number of passages;  $P$  = single wheel load (kN);  $A$  = tire contact area ( $\text{m}^2$ );  $\text{CBR}$  = California Bearing Ratio of subgrade.

Giroud and Noiray (1981) proposed the following formula to predict the required thickness to the cases with rut depth ( $r$ ) other than 0.075 m:

$$h_{os} = \frac{[0.190 \log N - 0.445(r - 0.075)]}{\text{CBR}^{0.63}} \quad (17)$$

Where,  $h_{os}$  and  $r$  are in unit of meter,  $N$  = the number of passages of standard axle load 80 kN. The formula is not recommended for  $N$  larger than 10000 or  $N$  less than 20. The failure mechanism addressed here is actually rutting. For  $N$  less than 20, Giroud and Noiray proposed to use a quasi-static analysis instead.

These two design equations are not based on theory, and include no consideration for base course properties. As shown in Figure 9, these equations do not correlate well with field test results reported by Fannin and Sigurdsson (1996). The filed test data from Fannin and Sigurdsson (1996) corresponding to the rutting depth of 0.075m (3 inches)

and 0.10 m (4 inches) are plotted in Figure 9, along with the predicted results from the two design equations (Hammit, 1970 and Giroud and Noiray, 1981). Hammit (1970) method and Giroud and Noiray (1981) method produced similar results from which the design base layer thickness was less than values from field test results.

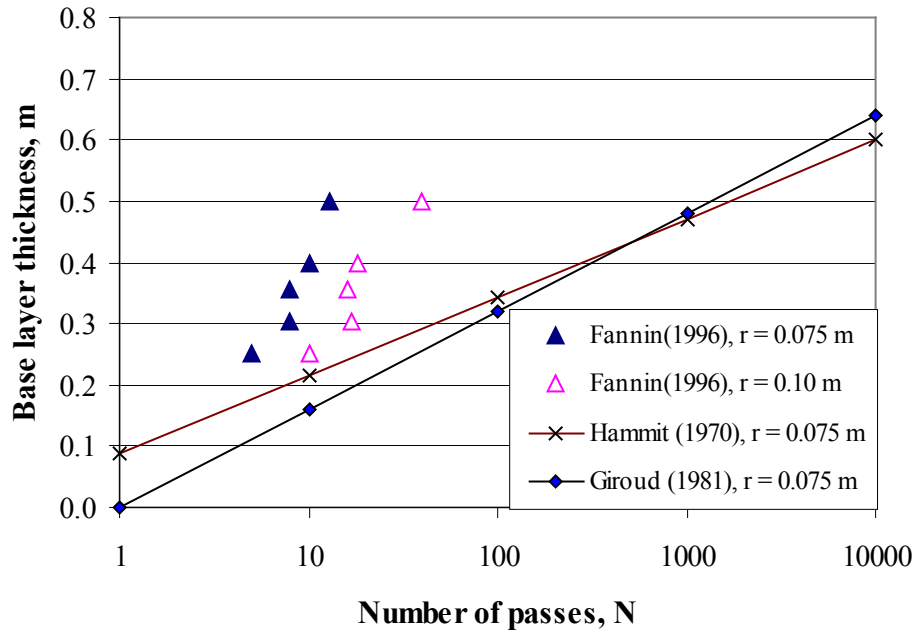


Figure 9. Unreinforced base course thickness vs. number of passes

Another way to consider the traffic is by using an equivalent load for N passes of a real axle load, or an allowable design load for N applications of the load. Based on the observation that rutting due to 100 passes of a 100 kN axle was equivalent to the rut depth calculated for a single 210 kN load, Sellmeijer and Kenter (1982) proposed the following equation to calculate the equivalent static load ( $P_e$ ) for N passes of axle load P.

$$P_e = PN^{0.16} \quad (18)$$

De Groot et al. (1986) proposed allowable design load ( $P_N$ ) for N application as a function of static failure load ( $P_s$ ):

$$P_N = \frac{P_s}{N^{0.16}} \quad (19)$$

However, this simplified method using the equivalent load was not verified by the field tests, which limited its application in unpaved road design.

## 2.5.2 Large displacement method of reinforced unpaved structure

The Large displacement mechanism assumes that large deformations are required to occur before reinforcement contribution is realized. Most of these large displacement mechanisms are associated with the vertical support of deformed membrane. Giroud and Noiray (1981) proposed a design method for reinforced unpaved roads base on such mechanism. This design model, which was based on the analysis of a membrane effect, dealt with the interaction that occurs between two wheel loads on the supporting layers and made the implicit assumption that the clay subgrade behaves in a rigid-perfectly plastic manner. The design method is summarized as follows:

### Simplified stress distribution

A simple load-spread mechanism was used in the method of Giroud and Noiray (1981). As shown in Figure 10, the load applied at the surface was assumed to be uniformly distributed over an area at base of the base layer with a load-spread angle ( $\alpha_0$  for unreinforced case and  $\alpha$  for reinforced case).

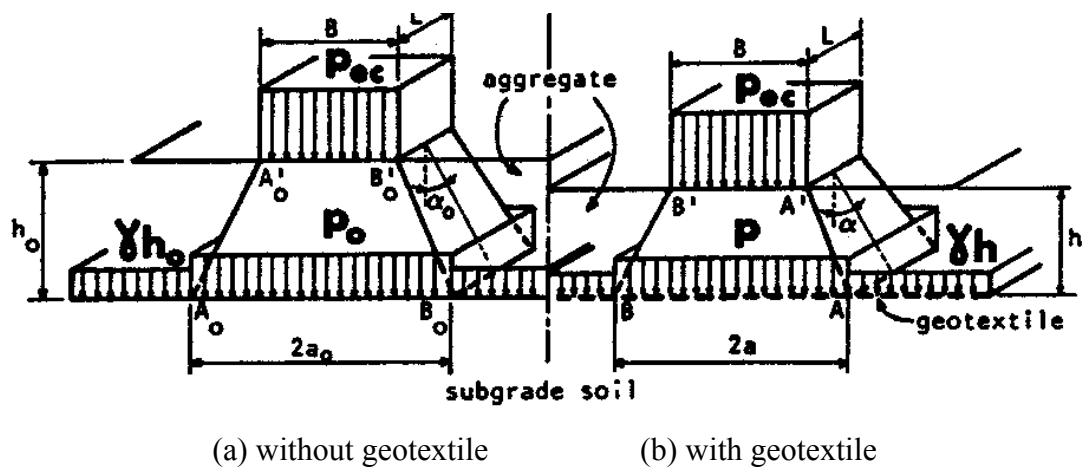


Figure 10. Simplified stress distribution Giroud and Noiray (1981)

### Bearing capacity of subgrade

Without reinforcement, the bearing capacity of subgrade was given by bearing capacity against punching failure:

$$q_{ult} = \pi C_u + \gamma h \quad (20)$$

Where,  $\gamma$  = unit weight of aggregate base course.

With reinforcement, the bearing capacity of subgrade was given by bearing capacity against general failure:

$$q_{ult} = (\pi + 2) C_u + \gamma h \quad (21)$$

### Vertical support from membrane

The reinforcement was assumed to be linearly elastic sheet of material placed at the bottom of base layer. The deformed shape of reinforcement was approximated by three parabolas, as shown in Figure 11. The points of zero vertical displacement (A and B) correspond to the edges of loaded area at the bottom of the base layer. The displacement of the wheel on the surface of the base layer was assumed to be equal to the displacement of the reinforcement beneath the wheel centerline. The mean reinforcement strain is obtained from the assumption that the reinforcement was fixed at points A and B.

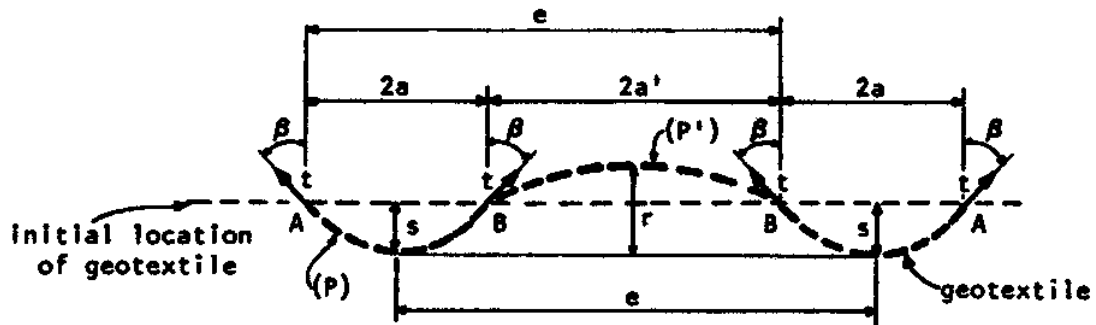


Figure 11. Membrane analysis for Giroud and Noiray (1981)

Here,

$$2a = B + 2h \tan \alpha$$

$$2a' = e - b - 2h \tan \alpha$$

For  $a < a'$ ,

$$s = \frac{ra'}{a + a'} \quad (22)$$

Elongation of geotextile

$$\varepsilon = \frac{b + b'}{a + a'} - 1 \quad (23)$$

For  $a > a'$ ,

$$s = \frac{2ra^2}{2a^2 + 3aa' - a'^2} \quad (24)$$

Elongation of geotextile

$$\varepsilon = \frac{b}{a} - 1 \quad (25)$$

Where  $b, b'$  = half length of parabolas AB and BB.

The additional resistance mobilized due to  $\varepsilon$  is  $E_f \varepsilon$  and the corresponding membrane support ( $p_m$ ) is expressed as:

$$p_m = \frac{E_f \varepsilon}{a \sqrt{1 + \left(\frac{a}{2s}\right)^2}} \quad (26)$$

Where,  $E_f$  = the tensile stiffness of geosynthetic.

The contribution of the reinforcement force to the strength of the system was assessed by considering the equilibrium of the portion of the reinforcement beneath the wheel. The assumption of the reinforcement fixity lead to model that may predict an excessively stiff response (Burd, 1986) and large rut depth for the case with stiff reinforcement.

### 2.5.3 Small displacement method of reinforced unpaved structure

Milligan et al. (1989a and 1989b) proposed a method based on the stress analysis at the shear interface of the base and subgrade. It was assumed that the shear stresses are resisted by the reinforcement and only pure vertical forces were transmitted to the subgrade, allowing the full bearing capacity of subgrade to be mobilized. As shown in

Figure 12, the vertical stress within the fill was estimated using a load spread angle ( $\alpha$ ). The vertical stress at a depth  $z$  below the surface within the region of ABED was given by:

$$\sigma'_v = \gamma z + \frac{pa}{a + z \tan \alpha}$$

Outside of the ABED region:

$$\sigma'_v = \gamma z$$

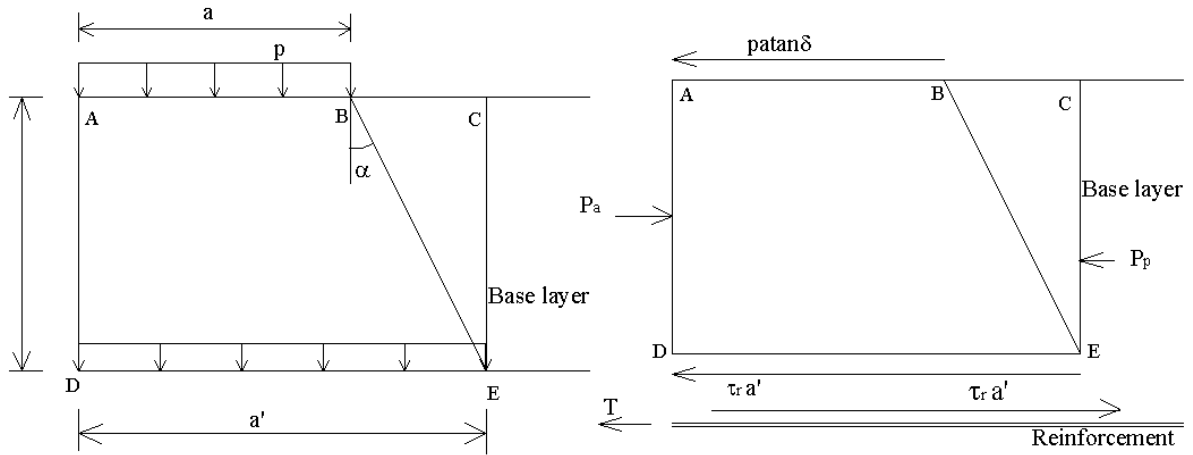


Figure 12. Load spread and equilibrium analysis for the reinforced strip footing

Assuming the base material tends to move outwards from underneath of footing, the minimum value of the horizontal stress on the surface AD was expressed as:

$$P_a = K_a \int_0^h \sigma'_v dz = 0.5 K_a \gamma h^2 + \frac{K_a pa}{\tan \alpha} \ln\left(\frac{a'}{a}\right) \quad (27)$$

Where,  $a' = a + h \tan \alpha$ ,  $K_a$  = active earth pressure coefficient.

Assuming passive pressures were developed outside of the footing, the maximum value of the horizontal stress on the surface CE was expressed as:

$$P_p = 0.5 K_p \gamma h^2 \quad (28)$$

Where,  $P_p$  = passive earth pressure coefficient.

The friction force on the base of footing was  $pa \tan \delta$ ,  $\delta$  is the friction angle between footing and base course.

The minimum tensile force of reinforcement required for equilibrium was given as:

$$\tau_r a' = 0.5(K_a - K_p) \gamma h^2 + \frac{K_a pa}{\tan \alpha} \ln\left(\frac{a'}{a}\right) - pa \tan \delta$$

The relationship between the required shear stress factor ( $\alpha_r = \tau_r/S_u$ ) acting on the subgrade and the bearing capacity factor ( $N_{cr} = pa/S_u a'$ ) for the subgrade was expressed by:

$$\alpha_r = 0.5(K_a - K_p) \frac{\gamma h^2}{S_u a'} + N_{cr} \left[ \frac{K_a}{\tan \alpha} \ln\left(\frac{a'}{a}\right) - \tan \delta \right] \quad (29)$$

Based on the bearing capacity, for unreinforced case ( $\alpha_r = 1$ ), the plastic solution yielded bearing capacity factor  $N_{cr} = (\pi/2+1)$  for the subgrade. For fully reinforced case ( $\alpha_r = 0$ ), the plastic solution yielded bearing capacity factor  $N_{cr} = (\pi+2)$  for the subgrade. The required reinforcement force may be calculated by:

$$T = \tau_r a' = \alpha S_u a' = \alpha S_u (a + h \tan \alpha) \quad (30)$$

Where  $\alpha$  is the  $\alpha_r$  value for  $N_{cr} = 5.14$  (fully reinforced case). It is also necessary to check the bearing capacity of the base course.

### 2.5.3 Geogrid-reinforced unpaved structure design method

Giroud et al. (1984) proposed a design method of geogrid-reinforced unpaved structure based on the Giroud and Noiray (1981) design method. The design method is summarized below.

## Material properties and assumptions

The aggregate material of base layer in this method was assumed to have good quality with CBR value larger than 80. Subgrade soil was assumed to be saturated low permeability soil (silt and clay), and the undrained strength was approximated using the relationship of  $C_u \text{ (kN/m}^2\text{)} = 30 \text{ CBR}$ . Two types of geogrids, in terms of reinforcement grade, were included: BX1100 geogrid (SS1) with average tensile stiffness of 300 kN/m and BX1200 geogrid (SS2) with average tensile stiffness of 500 kN/m.

Interface friction between geogrid and base layer was assumed to approximate the friction resistance of base aggregate. Thus, geogrids have adequate friction characteristics to prevent failure by sliding along the interface with the base layer. The vertical support from membrane effect of geogrid was neglected.

## Unreinforced unpaved structure

### a) Required thickness of base course

Empirical method of Giroud and Noiray (1981) (equation (31)) was used to predict the required thickness of the base layer as function of the CBR or undrained strength  $C_u$  of the subgrade, and the number of passages, as shown in Figure 13. Here the load is assumed to be of standard axle load 80 kN and the rut depth is 0.075m.

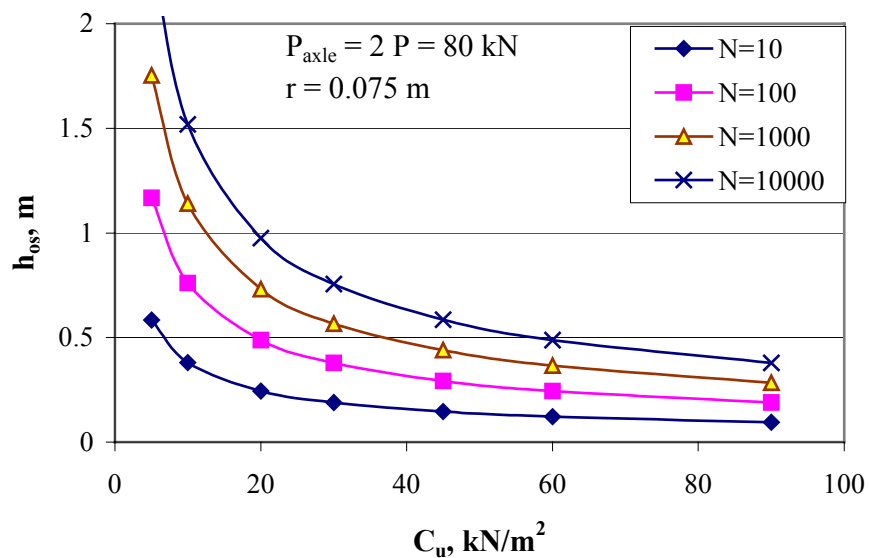


Figure 13. Unreinforced base layer thickness vs. subgrade shear strength

Based on work by Webster and Alford (1978) and Giroud and Noiray (1981), the following formula was used to predict the required thickness ( $h_{os}$ ) for the design rutting depth ( $r$ ):

$$h_{os} = \frac{[125 \log N - 294(r - 0.075)]}{C_u^{0.63}} \quad (31)$$

Here,  $h_{os}$  and  $r$  are in unit of meter,  $N$  = the number of passages of standard axle load 80 kN and  $C_u$  is in  $N/m^2$ .

The progressive deterioration of the subgrade soil can be expressed by the decrease of its undrained shear strength with the number of the passage (Equation 12).

#### b) Load spread of the base layer

Giroud et al. (1984) proposed a method based on the assumption that base layer provide pyramidal distribution of the wheel loads and vertical stress on the subgrade equals to the elastic limit. The vertical stress on the subgrade was expressed as follows:

$$P_{os} = \frac{0.5 P_s}{(B + 2 h_{os} \tan \alpha_0)(L + 2 h_{os} \tan \alpha_0)} + \gamma h_{os} \quad (32)$$

Where,  $\alpha_0$  = the load distribution angle for unreinforced unpaved structure;  $P_s$  = standard axle load (80 kN);  $L \times B$  = Contact area of a tire ( $m^2$ ). In the case of on-highway trucks,

$$L = B / \sqrt{2}$$

$$B = \sqrt{P/p_c}$$

Where,  $p_c$  = tire inflation pressure ( $kN/m^2$ ), 620  $kN/m^2$  for American-British standard.

Progressive deterioration of the base layer was expressed by the decrease of the load distribution angle. The deformation of the surface of the subgrade and the rut depth become large if the vertical stress on the subgrade exceeds the elastic limit ( $p_e$ ).

$$p_e = \pi C_{uN} + \gamma h_{os} \quad (33)$$

For  $p_{os} = p_e$  and  $r = 0.075$  m, the required stress distribution ability ( $\tan \alpha_0$ ) can be estimated as:

$$\tan \alpha_0 = \frac{\sqrt{(\sqrt{2}-1)^2 P_s / (2 p_c) + 2 P_s / (\lambda \pi C_u)} - (\sqrt{2}+1) \sqrt{P_s / (2 p_c)}}{6.5 \log N / C_u^{0.63}} \quad (34)$$

Where,  $\alpha_0$  = the load distribution angle;  $N$  = the number of passages;  $P_s$  = standard axle load (80 kN);  $p_c$  = tire inflation pressure;  $C_u$  = undrained shear strength of the subgrade;

### **Reinforced unpaved structure**

#### **a) Improved stress distribution**

The vertical stress transmitted to the upper face of the geogrid:

$$p' = \frac{0.5 P}{(B + 2 h \tan \alpha)(L + 2 h \tan \alpha)} + \gamma h \quad (35)$$

Where,  $\alpha$  = the load distribution angle for reinforced unpaved structure;

Elastic finite element method was used by Giroud et al. (1984) to evaluate the load spread ability due to geogrid reinforcement. Three cases of reinforced base layer were considered using different elastic modulus values of aggregate base course, while  $\tan \alpha_0 = 0.6$  was used for unreinforced case. Figure 14 shows the Load distribution improvement ratio ( $\tan \alpha / \tan \alpha_0$ ) as function of the thickness of the unreinforced base layer ( $h_0$ ). Curve 1 is for BX1100 (or SS1) with consideration of aggregate contamination (high number of vehicle passes); curve 2 is for BX1100 without consideration of aggregate contamination (low number of vehicle passes); curve 3 is for BX1200 (or SS2) without consideration of aggregate contamination (low number of vehicle passes). The aggregate contamination was simulated in the finite element analysis by decreasing elastic modulus of base layer.

The vertical stress below the geogrid was assumed as follows:

$$p = p' - p_m$$

where,  $p_m$  is the normal stress difference due to tension membrane effect.

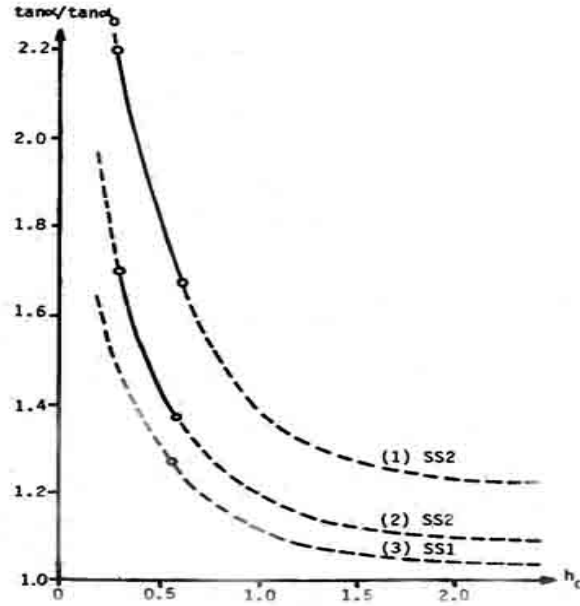


Figure 14. Load distribution improvement ratio ( $\tan\alpha / \tan\alpha_0$ ) as function of the thickness of the unreinforced base layer ( $h_0$ )

b) Thickness ratio:

A thickness ratio depicting decrease in thickness due to inclusion of reinforcement was presented as follows:

$$R = h/h_0 = \frac{\sqrt{(B-L)^2 + 4Y} - (B+L)}{4h_0 \tan\alpha} \quad (36)$$

$$Y = \frac{1}{\frac{1+2\pi}{(B+2h_0 \tan\alpha_0)(L+2h_0 \tan\alpha_0)} + \frac{2p_m}{P}} \quad (37)$$

Giroud et al. (1984) provided the simple chart based on  $\tan\alpha_0 = 0.6$  and  $p_m = 0$ , as shown in Figure 15.

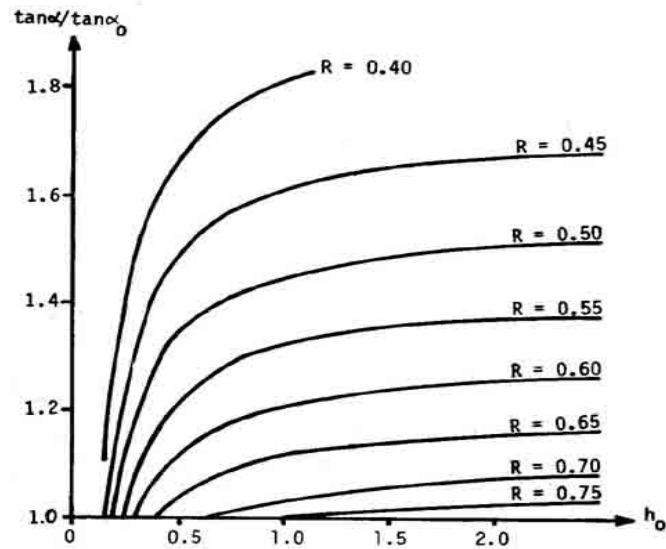


Figure 15. Thickness ratio ( $R$ ) versus load distribution improvement ratio ( $\tan \alpha / \tan \alpha_0$ )

Fannin and Sigurdsson (1996) provided field test data for BX1100 (SS1) geogrid-reinforced unpaved roads. The predicted base layer thickness from Giroud et al. (1984) method and test results Fannin and Sigurdsson (1996) are shown in Figure 16. Giroud (1984) method underpredicted the required thickness measured in the field based on number of load passes for the same rutting depth of 0.075m.

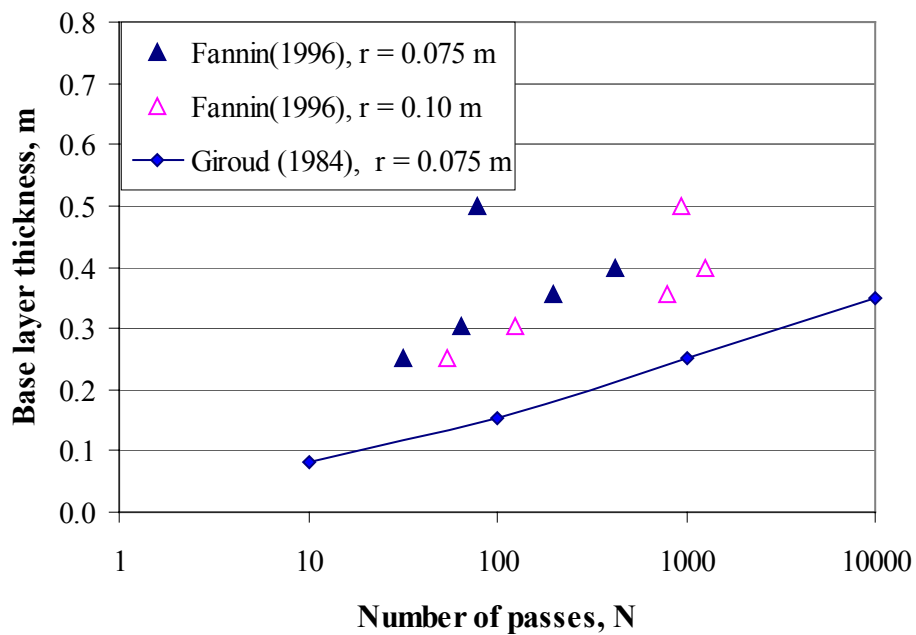


Figure 16. Reinforced base layer thickness vs. number of passes

#### **2.5.4 Gaps in the reinforced unpaved structure design method**

Based on review of literature, gaps in the design/analysis of reinforced unpaved structure design method are identified as follows:

- i) Current methods are mainly based on empirical equations based on unreinforced field tests as well as limited laboratory static tests. There is a lack of performance database to discern the behavior of reinforced unpaved structure against cyclic load.
- ii) Current analyses are based on static equilibrium and bearing capacity analysis under plain strain condition. For unpaved structure, the stress condition is close to axisymmetric, while the permanent deformation that develops along the load track is similar to plain strain condition.
- iii) CBR values of subgrade and base course are used in the design, with the major assumptions that base course remains as good quality with  $\text{CBR} \geq 80$  and subgrade has undrained shear strength  $C_u \text{ (kN/m}^2\text{)} = 30 \text{ CBR}$ . The analysis is therefore focused on these specific conditions, which may not be suitable in other situations, such as poor base course properties.
- iv) There is no consideration to the dependency of the mobilized subgrade bearing strength on the basis of deformation level.
- v) Load distribution improvement ratio used in literature design charts is based on specific and limiting assumptions. There is no explicit method to describe the load distribution angle based on the properties of subgrade, base course and reinforcement, and the changes in the stress distribution angle as deterioration of properties under traffic load.
- vi) Current design method only considered the degradation of subgrade with empirical relation of undrained shear strength with number of cycles. The degradation of base course and affect of reinforcement are not included.

## Chapter 3 CYCLIC LOAD PLATE TESTS

### 3.1 Cyclic load plate testing program

A total of 14 cyclic load plate tests were performed on two-layer systems of ABC and subgrade, with variation of reinforcement types and thickness of ABC layer, as shown in Table 1. The 152-mm ABC tests included one unreinforced case, one BX1100 geogrid-reinforced case, and two BX1200 geogrid-reinforced cases (one repeated). The 254-mm ABC tests included two unreinforced cases, two BX1100 geogrid reinforced case (one repeated), two BX1200 geogrid-reinforced cases (one repeated), one experimental geogrid (Max 200) reinforced case, one geonet reinforcement case, and one BX1100 geogrid plus geonet reinforced case. In general, repeated tests were performed to ascertain the accuracy of the measured data with the inherent variability of the prepared test sample.

Table 1. Summary of the testing program

| Thickness<br>of ABC | Reinforcement |        |        |        |       |        |                    |         |
|---------------------|---------------|--------|--------|--------|-------|--------|--------------------|---------|
|                     | BX1100        | BX1200 | BX4100 | BX4200 | Max30 | Geonet | BX1100<br>+ Geonet | No Rfrc |
| 152-mm              | 1             | 2      | -      | -      | -     | -      | -                  | 1       |
| 254-mm              | 2             | 2      | 1      | 1      | 1     | 1      | 1                  | 1       |

Monitored data included surface deformation with number of cycles as well as vertical pressure distribution at the interface of ABC layer and subgrade. Before the commencement of cyclic loading, static load-deformation response was measured under a load of 10 kN. The surface contours of base course layer, and subgrade layer, were

surveyed manually after soil preparation and after the completion of the cyclic loading test.

The dimensions of the test box were  $1.5 \text{ m} \times 1.5 \text{ m} \times 1.35 \text{ m}$  as shown in Figure 17. This selected size was based on minimizing interference from the box boundaries on the test results given the 0.305-m plate diameter. Previous plate load tests (Gabr et. al., 1998) with three pressure cells placed on the walls of the box, with depth, indicated that almost no stress transfer at the walls under applied surface pressure of 700 kPa. The thickness of the subgrade layer varied from approximately 0.75 – 0.90 m. The cyclic load was applied to the test plate using a computer-controlled servo hydraulic actuator, with amplitude of 40 kN and frequency of 0.67 Hz.

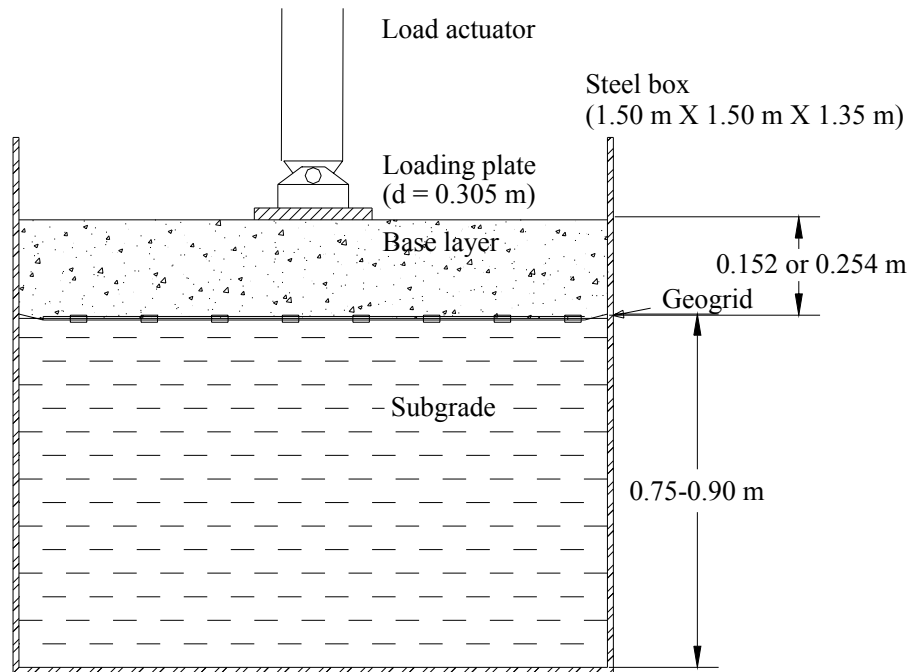


Figure 17. Schematic diagram of the test box and loading configuration

### 3.1.1 Testing materials

#### Aggregate Base Course (ABC)

The Aggregate base course (ABC) used in the testing program was obtained from a local quarry. This ABC material is typically used for flexible road bases. Grain size

analysis (ASTM D 422) was performed on ABC specimens in accordance with ASTM (1997). The grain size distribution curve is shown in Figure 18 and indicates that 100% of the particles passes the 30-mm sieve with  $C_U$  of 15 and  $C_C$  of 6. The ABC is classified as GW according to the Unified soil Classification system (USCS).

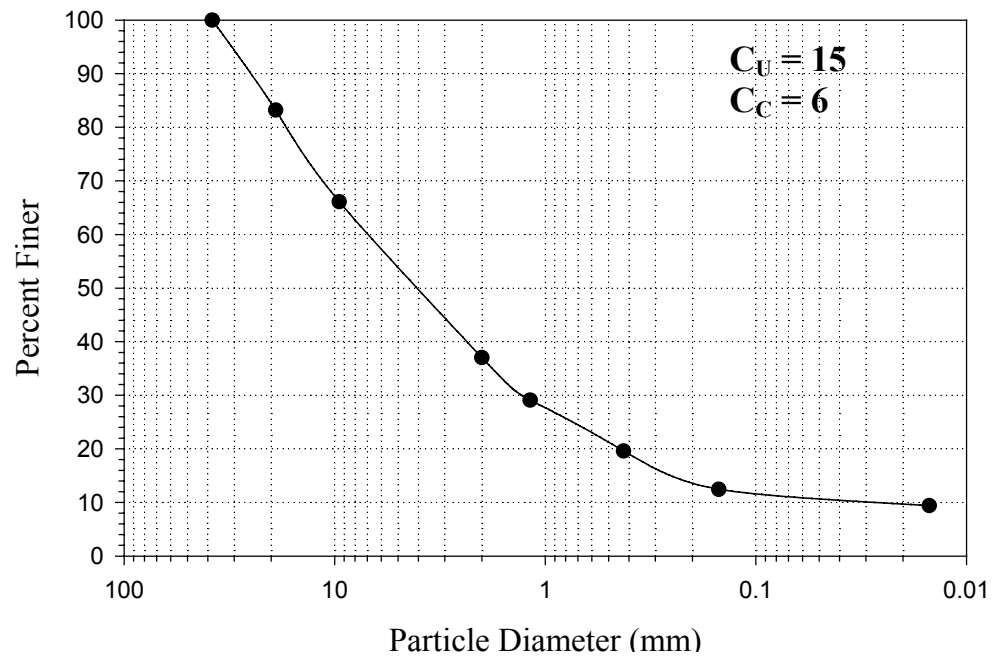


Figure 18. Grain Size Distribution of ABC stone

### Subgrade Soil

The subgrade soil was composed of as a mixture of 85% Lillington Sand and 15% Kaolinite. The Kaolinite was added in order to obtain low CBR values. Proctor analysis and CBR tests were performed on subgrade specimens. As shown in Figure 19, Standard Proctor compaction tests yielded a maximum dry density of 113.5 pcf (17.82 kN/m<sup>3</sup>) at optimum moisture content of 13.5%. Figure 20 shows the variation of CBR with compaction moisture content. Since the CBR value at 0.2 inch (5.08mm) penetration are greater than CBR value at 0.1 inch (2.54mm) penetration, the CBR value at 0.2 inch (5.08mm) penetration was used to represent the subgrade. Based on the CBR-moisture content curve determined in the lab, the material was typically compacted at moisture content of 14.5 – 15.3% with a corresponding laboratory-measured CBR value of 3 approximately.

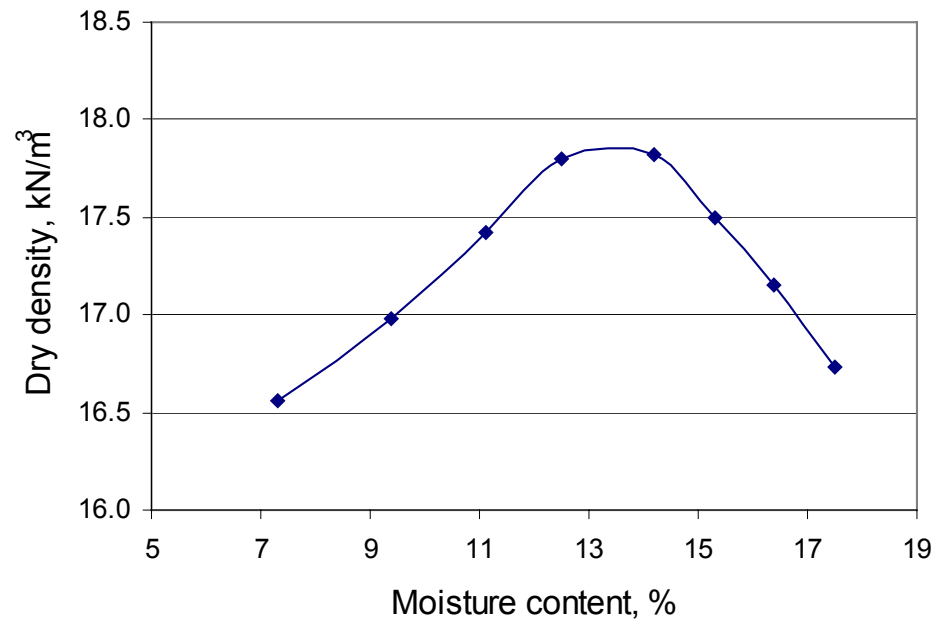


Figure 19. Proctor analysis of subgrade soil

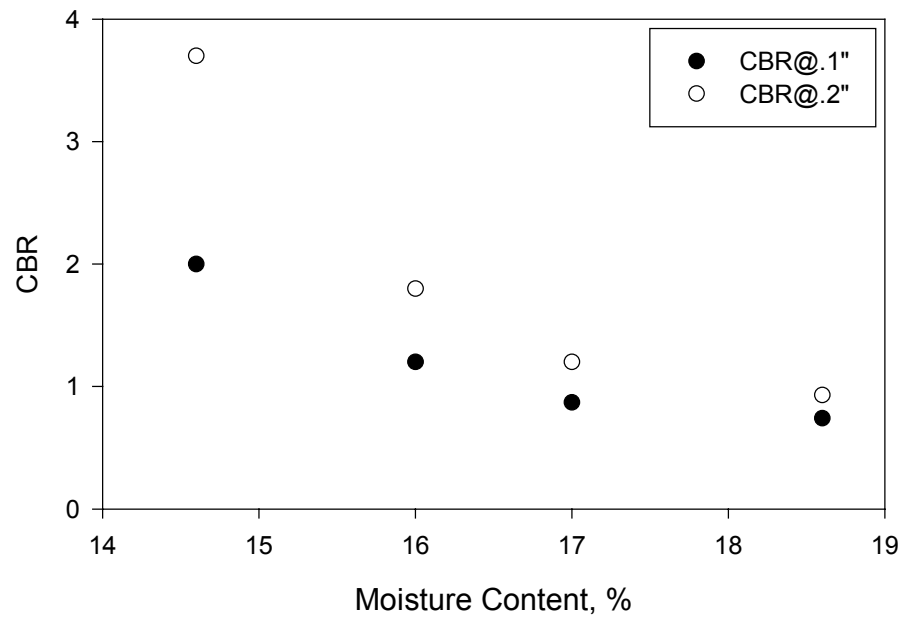


Figure 20. CBR versus compaction moisture content for subgrade

## **Geosynthetic Reinforcement Materials**

Six types of biaxial, polypropylene (PP) geogrids were utilized in the testing program: Tensar BX1100, BX1200, BX4100, BX4200, and experimental geogrid (Max 30). Tensar's biaxial geogrids have relatively large stiffness in both of the longitudinal and transverse directions with torsional rigidity. Table 2 presents a summary of the nominal dimensions and tensile strength of the reinforcement material. For all the reinforcement materials, one sheet of geogrid was used. A geonet composite material (DC6200) was also used in the testing program. It consists of a sheet of geonet with nonwoven geotextile on both sides. The dimension of reinforcement used in the testing program was  $4.9\text{ft} \times 4.9\text{ft}$  ( $1.49\text{m} \times 1.49\text{m}$ ).

### **3.1.2 Cyclic load plate testing process**

#### **a) Sample Preparation**

The sample for each test was prepared by placing the subgrade soil in 0.25-m layers with proper volume of water. Once water was mixed with soil and the desired thickness was achieved, a jackhammer with an  $0.203\text{-m} \times 0.203\text{-m}$  tamping plate was used for vibratory compaction. The jackhammer delivered 40.7 m-N blows at the rate of 850 blow/minute. The compaction commenced in one corner and proceeded to the other corner while staying on each plate footprint for ten seconds. This process was repeated until the entire subgrade layer was uniformly compacted. After the completion of subgrade preparation, pressure cells and the geosynthetic reinforcement materials were installed. The base course layer was consequently prepared by placing 0.075 m layers of aggregate, and compacting it inside the box after mixing with desired moisture volume. Compaction of this layer was performed in a manner similar to compaction of the subgrade soil.

Table 2. Properties of geogrids and geonet (Properties from manufacturer's data)

| Geosynthetic Type                                | BX1100  | BX1200  | BX4100  | BX4200  | Geonet DC6200 <sup>1</sup> | Max30 <sup>2</sup> |
|--------------------------------------------------|---------|---------|---------|---------|----------------------------|--------------------|
| Mass/Unit Area (kg / m <sup>2</sup> )            | 0.204   | 0.313   | 0.168   | 0.257   | 1.666                      | 0.236              |
| <u>Aperture Size (mm)</u> MD × TD                | 25 × 33 | 25 × 33 | 33 × 33 | 33 × 33 | N/A                        | 44 × 43            |
| <u>Tensile Strength (kN/m)</u> @2 % strain<br>MD | 4.1     | 6.0     | 3.6     | 5.5     | N/A                        | 11.7               |
|                                                  | 6.6     | 9.8     | 5.1     | 7.4     | N/A                        | 10.2               |
| <u>Tensile Strength (kN/m)</u> @5 % strain<br>MD | 8.5     | 11.8    | 7.3     | 10.5    | N/A                        | 24.0               |
|                                                  | 13.4    | 19.8    | 9.5     | 14.6    | N/A                        | 20.9               |
| <u>Ultimate Strength (kN/m)</u><br>MD            | 12.4    | 19.2    | 12.8    | 19.7    | 16                         | 36.4               |
|                                                  | 19.0    | 28.8    | 13.5    | 22.5    |                            | 39.0               |
| <u>Initial Modulus (kN/m)</u><br>MD              | 221     | 481     | 221     | 282     | N/A                        | N/A                |
|                                                  | 360     | 653     | 284     | 424     | N/A                        | N/A                |
| Flexural Stiffness (mg-kG)                       | 250000  | 750000  | 250000  | 750000  | N/A                        | N/A                |
| Torsional Stiffness (kg-cm/deg)                  | 3.2     | 6.5     | 2.8     | 4.8     | N/A                        | 6.9                |

Note 1: Geonet DC6200 is drainage composite not intended for reinforcement.

Note 2: Values given for Max30 are measured values, not Minimum Roll Values.

Note 3: Tensile strength of drainage geonet only. Grab tensile strength of the geotextile is 160 pounds.

The nuclear density/moisture gage was used to measure the moisture content and unit weight distribution according to ASTM (1997) D 2922-96 for density and D3017-95 for moisture content. The nuclear gage was orientated in the long direction with its sides parallel to the box's sides. The nuclear moisture/density tests were performed for duration of one minute in the direct transmission mode. After these tests were completed, the gage was rotated 180 degrees and the tests were repeated. For each layer, five tests were performed at the four corners and center of the subgrade. In general, the average moisture content and dry unit weight were 5.1 % and 20.1 kN/m<sup>3</sup> for the ABC , and 14.9 % and 17.6 kN/m<sup>3</sup> for the subgrade, respectively.

Table 3. Configuration and soil properties of each test

| Test Number | ABC thickness mm | Geosynthetic Reinforcement | Moisture content % |          | Dry density kN/m <sup>3</sup> |          |
|-------------|------------------|----------------------------|--------------------|----------|-------------------------------|----------|
|             |                  |                            | ABC                | Subgrade | ABC                           | Subgrade |
| 6-1         | 150              | None                       | 4.7                | 14.1     | 19.6                          | 17.5     |
| 6-2         | 163              | BX1100                     | 5.1                | 15.0     | 19.3                          | 17.4     |
| 6-3         | 157              | BX1200                     | 4.9                | 15.2     | 20.0                          | 17.6     |
| 6-4         | 160              | BX1200<br>(repeated)       | 5.2                | 15.3     | 20.4                          | 17.6     |
| 10-1        | 259              | None                       | 5.4                | 14.2     | 20.1                          | 18.0     |
| 10-2        | 274              | BX1100                     | 5.3                | 14.9     | 20.4                          | 17.9     |
| 10-3        | 262              | BX1100<br>(repeated)       | 5.5                | 15.2     | 20.5                          | 17.4     |
| 10-4        | 269              | BX1200                     | 5.1                | 14.7     | 20.1                          | 17.8     |
| 10-5        | 259              | BX1200<br>(repeated)       | 5.0                | 15.1     | 20.2                          | 17.5     |
| 10-6        | 257              | BX4100                     | 5.1                | 15.2     | 20.2                          | 17.6     |
| 10-7        | 262              | BX4200                     | 5.0                | 14.6     | 20.1                          | 17.4     |
| 10-8        | 259              | Max30                      | 5.0                | 14.8     | 20.1                          | 17.5     |
| 10-9        | 259              | Geonet                     | 5.1                | 15.2     | 20.1                          | 17.3     |
| 10-10       | 254              | BX1100 plus<br>Geonet      | 5.2                | 15.2     | 20.5                          | 17.4     |

## b) Load Control

A servo hydraulic MTS system was used for applying the cyclic test load. The system consists of a loading frame, a hydraulic actuator, and a servo-control unit connected to both a data acquisition system and a hydraulic control valve. Before the cyclic loading test, a static load test was performed to a maximum load of 10 kN (loading pressure of 20 psi or 137 kPa), which is recommended by AASHTO for evaluating subgrade reaction. The amplitude of the cyclical load was 40 kN (loading pressure of 80psi or 548 kPa) with a frequency of 0.67 Hz. The computer program MTS TESTSTAR was set up to control and acquire the load data as well as the deformation data. Figure 21 shows the input load pulse and corresponding actual load cell measurement of the applied load. This pulse has a 0.3 second period of linear load increase from 0.5 kN to 40 kN, followed by a 0.2 second period where the load is held as 40 kN, followed by a linear load decrease to 0.5 kN over 0.3 second period, then followed by 0.5 second period of 0.5 kN before the next loading cycle.

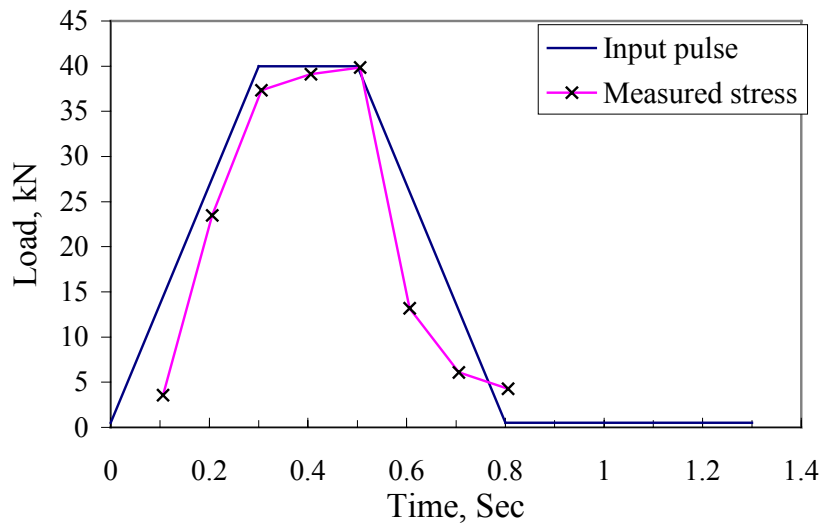


Figure 21. The input load pulse and corresponding load cell measurement

## c) Test data

Up to 8000 loading cycles have been applied through a circular steel plate (with diameter of 0.305-m). The vertical stresses at the interface (on the top of subgrade) were monitored at cycles number N=1, 10, 100, 500, 1000, 1500, 2000, 2500, 3500, 4000,

5000, 6000, 7000, and 8000. The vertical deformation of the plate's center was monitored using the transducer inside the actuator piston and from two LVDT dial gages mounted on a reference beam at the edge of the plate. Each gage has a resolution of 0.00254-mm (0.0001-inch). All the settlement data were recorded using the computer's data acquisition system. The deformation measured from the transducer of the actuator is used in graphs presented later. The readings from the periphery dial gage are used to check if there is obvious tilting during the test.

Four total pressure cells with the diameter of 50.8-mm were used to measure the vertical stress at the interface of the base course and subgrade layers. As shown in Figure 22, the pressure cells were placed at the interface of base course and subgrade with a varying distance from the center of the loading plate ( $r$ ), at  $r = 0$ , 152-mm, 305-mm and 457-mm, respectively, where  $r = 0$  designates the center of the plate. Since the pressure cells were not made to read dynamic load, the stress readings were manually performed by pausing the cyclical loading after the desired number of cycles was reached, then applying the load of 40 kN (equivalent to 548 kPa) and recording the pressure data.

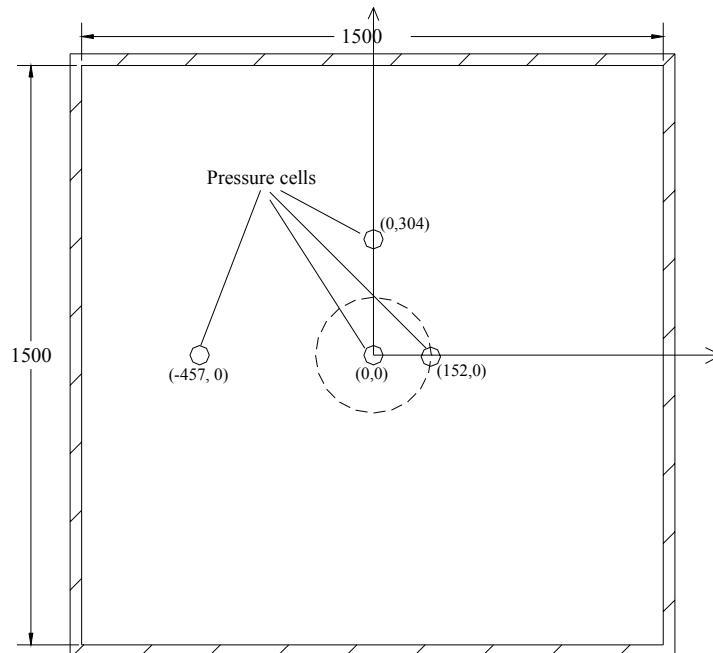


Figure 22. Location of pressure cells

The surface contours of base course and subgrade were measured after soil preparation and after loading test. The measurements of static load-deformation response under 10 kN, and surface contours of ABC layer and subgrade layer were only conducted for some of the tests.

### **3.1.3 Subgrade under cyclic load**

In a conjunction with cyclic load plate tests on the unpaved structures, three cyclic load tests were performed on the subgrade with load of 10 kN, 15 kN and 18 kN. The permanent vertical deformation was recorded in order to study the plastic deformation of subgrade under repeated loads in an attempt to discern the plastic deformation from subgrade and base layer in the unpaved structures.

## **3.2 Testing results**

### **3.2.1 Surface deformation**

The surface deformation curves for the 152-mm ABC tests (Without reinforcement, BX1100 and BX1200), 254-mm ABC tests (Without reinforcement, BX1100, BX1200), 254-mm ABC tests (Without reinforcement, BX4100, BX4200 and Max30), and 254-mm ABC tests (Without reinforcement, geonet and BX1100 plus geonet) are shown in Figures 23, 24, 25 and 26 respectively.

The overall results show that surface deformations accumulated with the number of load repetitions. The surface deformation increased quickly at the onset of the loading cycles. The increase rate of surface deformation decreased until it reached a steady increase-rate after a certain level of load repetitions. Compared with the unreinforced section, the reinforced sections exhibited a slower increase in the rate of surface deformation. Load-deformation response with geosynthetic reinforcements in general shows reduction in the surface deformation of the test samples, as compared to unreinforced samples.

For the 152-mmABC tests, the geogrid reinforcement effectively decreased the surface deformation with such an effect being more pronounced for the higher modulus geogrid BX1200, as shown in Figure 23. At the end of 8000 loading cycles, the total deformation decreased from 71.6 mm for unreinforced case, to 60.7 mm for BX1100 reinforced case, and to 48.3 mm for BX1200 reinforced case. The repeatability of two tests for the BX1200 reinforcement is also shown in Figure 23.

For 254-mm ABC tests, the geogrid reinforcement also decreased the surface deformation with BX1100 and BX1200 yielding the similar deformation after 8000 cycles. As data in Figure 24 are compared with data in Figure 23, it is apparent that the deformation rate-of-increase as well as the total magnitude of deformation are less for the 254-mm ABC tests. After 8000 loading cycles, the total deformation was 52.6 mm for unreinforced test, 44.2 mm (average) for two BX1100 tests and 41.4 mm (average) for two BX1200 tests. In general, it seems that the benefit from the geogrid reinforcements (BX1100 and BX1200) decreased as the thickness of base course increased from 152-mm to 254-mm. The deformation with the BX1200 reinforcement was 67% of the deformation measured for the unreinforced 152-mm test while this value was 78% for the 254-mm tests.

For 254-mm ABC tests, the other five reinforced cases (BX4100, BX4200, Max30, geonet and BX1100 plus geonet) also showed improvement in the deformation of the test sections. As shown in Figure 25, the BX4200 geogrid (with total deformation of 40.6 mm) provided better improvement than BX4100 geogrid (with total deformation of 48.5 mm), perhaps due to higher tensile modulus. The experimental geogrid (Max30) similarly provided improvement in deformation with number of cycles, with a total deformation of 41.4 mm, a magnitude similar to that obtained using BX4200.

As shown in Figure 26, the use of geonet composite material indicated some improvement in surface deformation, with total deformation of 48.5 mm as compared to 52.6 mm for the unreinforced case. The added reinforcement benefit, in addition to the primary function of separation and drainage, may be secondarily considered of substance although the impact of tensile strain on drainage efficiency should be investigated. As a BX1100 layer was added beneath the geonet composite a total deformation of 44.5 mm was measured. This magnitude is similar to value measured for the BX1100 by itself.

These results confirm the strain compatibility condition where the higher modulus material carries most of the applied load. Both, the geonet test and BX1100 plus geonet test showed a larger increase in the rate of surface deformation, as compared with other geogrid-reinforced tests, during the “steady state” deformation period. This behavior maybe explained by the compression of the geonet material, slippage of geonet in relation to geogrid, or geonet material fatigue under the cyclic load. Comparison of all the 254-mm ABC tests suggests that several factors affect reinforcement contribution to deformation level including modulus and aperture properties of geosynthetics, and geometry and stiffness properties of base course and subgrade soil. In this testing program, geogrids BX1200, and BX4200 provided the best contribution to reducing the surface deformation ratcheting effect and magnitude.

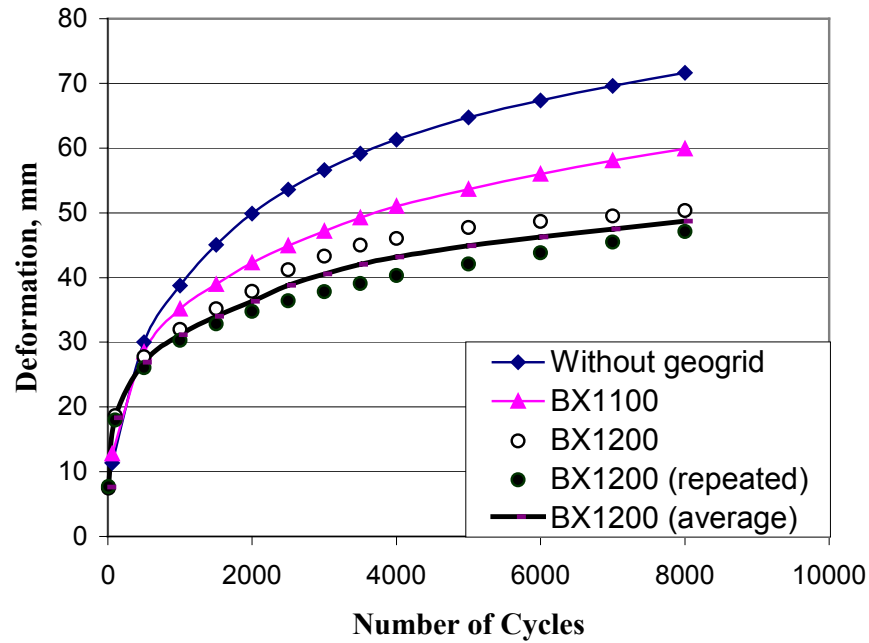


Figure 23. Surface deformation development of 152-mm ABC tests  
(Without reinforcement, BX1100, and BX1200)

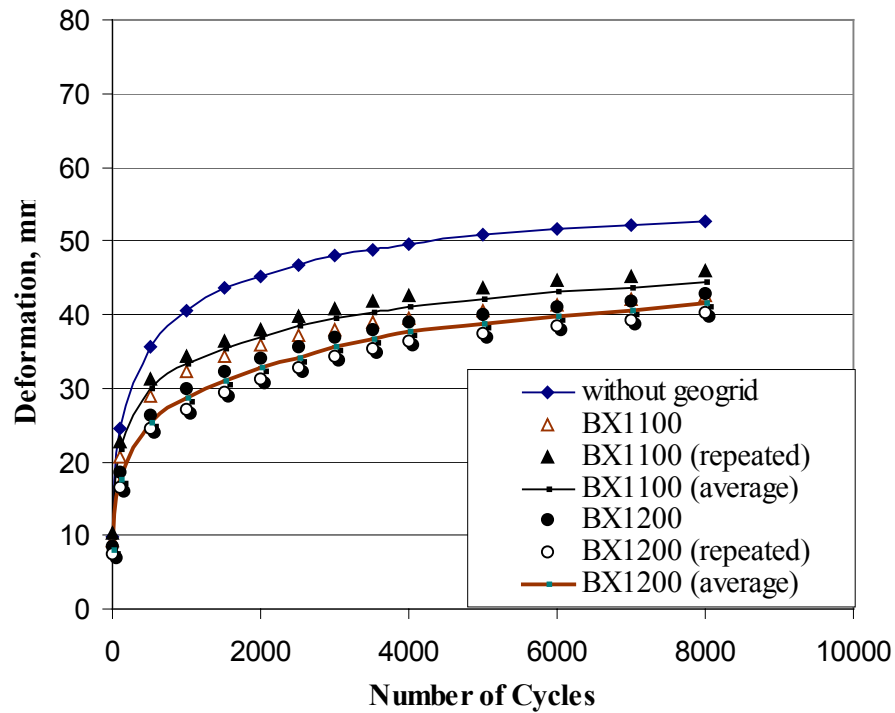


Figure 24. Surface deformation development of 254-mm ABC tests  
(Without reinforcement, BX1100, and BX1200)

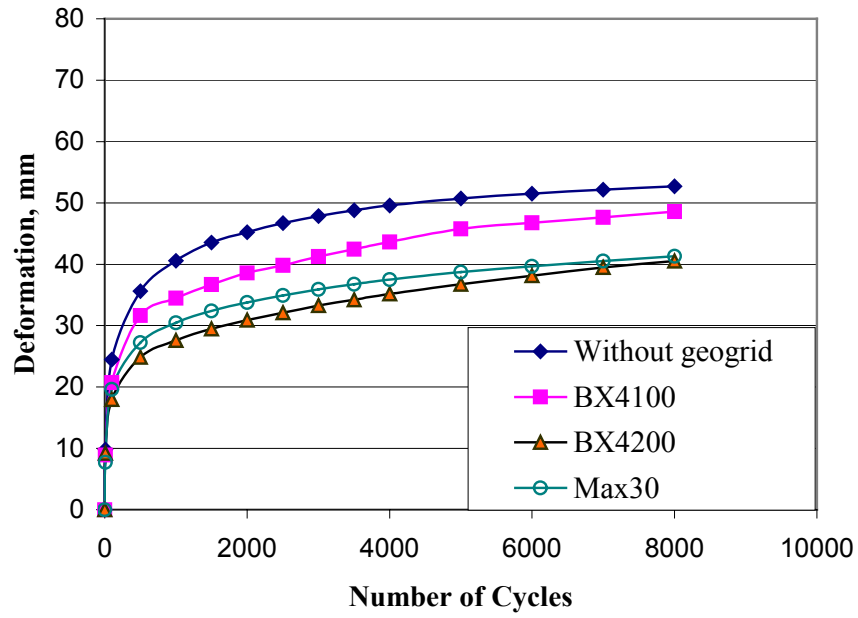


Figure 25. Surface deformation development of 254-mm ABC tests  
(Without reinforcement, BX4100, BX4200, and Max30)

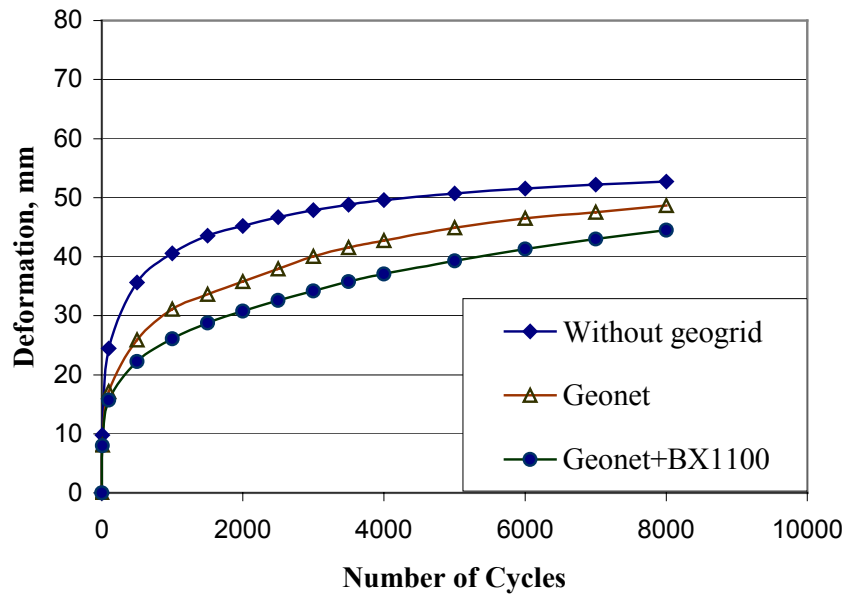


Figure 26. Surface deformation development of 254-mm ABC tests  
(Without reinforcement, geonet, and BX1100 plus geonet)

### 3.2.2 Stress magnitude on the subgrade

For each test, the vertical stresses were measured at a distance from the center,  $r = 0$ , 152-mm, 305-mm and 457-mm. There was almost no change in stress data measured by the cell located at  $r = 457$ -mm during the tests, which implies an influence diameter at the interface that is less than 1.5 times the plate diameter. The change in stresses at  $r = 305$ -mm was discernable during the tests. The stress at this location increased by approximately 30 kPa for the 152-mmABC tests and 50 -100 kPa for 254-mmABC tests (applied surface stress equal to 548 kPa). The change in stress at the points with  $r = 0$  and  $r = 152$ -mm was prominent. The stress development during the tests indicated that geosynthetic reinforcement decreased the vertical stresses on the top of subgrade and led to a more uniform stress redistribution will be shown later.

The vertical stresses at the center ( $r = 0$ ) for 152-mmABC tests, 254-mmABC tests (without reinforcement, BX1100, BX1200), 254-mmABC tests (without reinforcement, BX4100, BX4200, and Max30), and 254-mmABC tests (without reinforcement, geonet (DC6200) and BX1100 plus geonet) are shown in Figures 27, 28, 29 and 30 respectively. Compared with the results from the unreinforced tests, the reinforcement at the interface of base course and subgrade changed the stress distribution. The overall results mainly showed two types of stress development. For unreinforced tests and BX1100 tests, 152-mm ABC BX1200 test, 254-mmABC tests with BX4100, geonet (DC6200), and BX1100 plus geonet reinforcement, the vertical stress at the center experienced a quick increase in magnitude at the beginning of load application, then the increase rate was reduced and the stress became almost constant. For the 152-mm ABC test with BX1200 (repeated), 254-mm ABC tests with BX1200, BX4200 reinforcement, and BX1100 plus geonet (DC6200) reinforcement, the vertical stress at the center also experienced a quick increase, reached a maximum value, then the stress reduced slowly until it became stable.

In the case of 152-mmABC tests, the geogrid reinforcements effectively decreased the vertical stresses at the center. A higher modulus results in lower stresses as the stress decreased from 360kPa, for the unreinforced case, to 330kPa using BX1100 geogrid, to 220 kPa (average) using BX1200 geogrid as shown in Figure 27. For 254-mmABC tests, the vertical stress measured at the center was lower than that of 152-

mmABC tests. The steady state stresses were approximately 200kPa, 180kPa (average) and 125 kPa (average) for no reinforcement, BX1100 reinforcement and BX1200 reinforcement, respectively, and as shown in Figure 28. The steady state stresses were 160 kPa using BX4100 reinforcement, 140 kPa using BX4200, 145 kPa using Max20, 125 kPa using Geonet (DC6200), and 105 kPa using BX1100 plus geonet test, respectively (Figures 29 and 30). For geogrids with same material and aperture size, higher modulus geogrids (BX1200 and BX4200) provided a better load spreading effect than the corresponding low modulus geogrids (BX1100 and BX4100). Geonet test and BX1100 plus geonet test showed a bigger reduction of center stresses perhaps due to additional tensile membrane effect provided by the geonet (with two layers of geotextile).

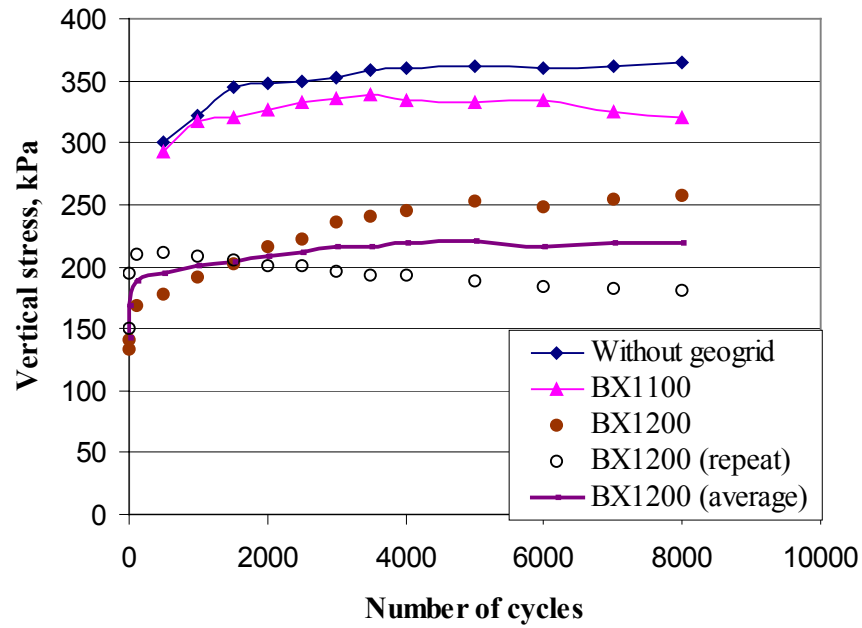


Figure 27. Vertical stresses at the center for 152-mm ABC tests  
(Without reinforcement, BX1100 and BX1200)

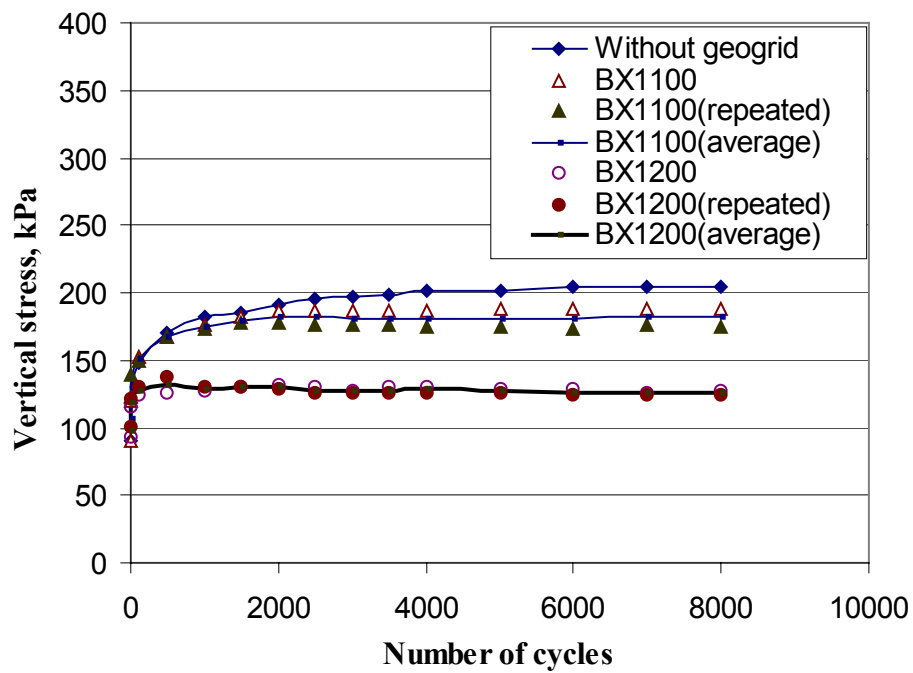


Figure 28. Vertical stresses at the center for 254-mm ABC tests  
(Without reinforcement, BX1100 and BX1200)

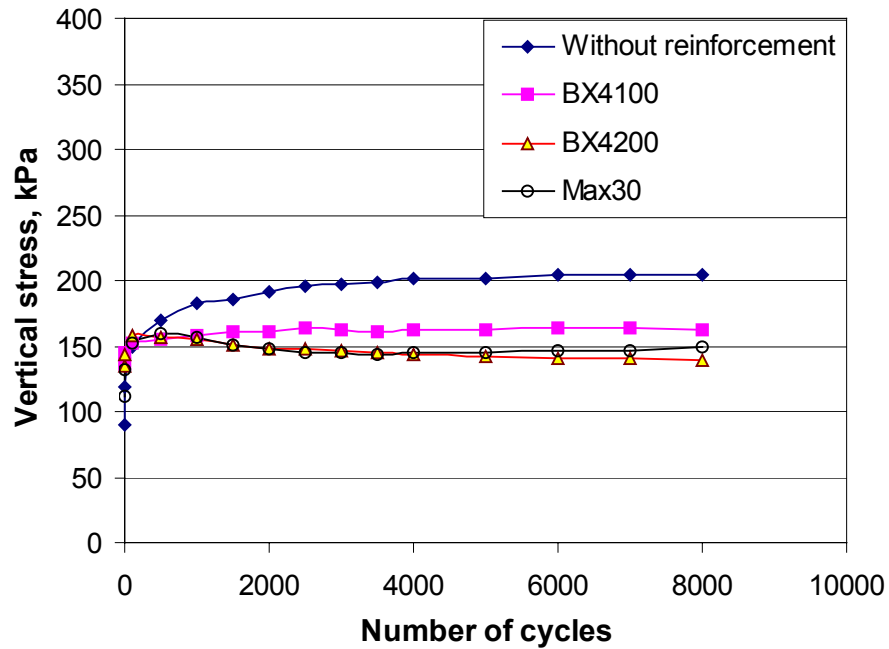


Figure 29. Vertical stresses at the center for 254-mm ABC tests  
(Without reinforcement, BX4100, BX4200, Max30)

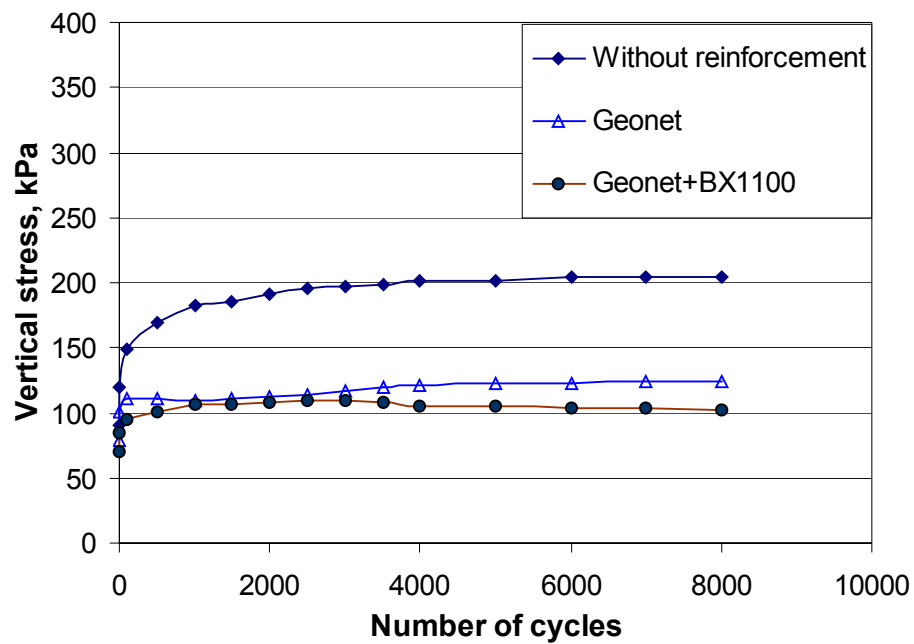


Figure 30. Vertical stresses at the center for 254-mm ABC tests  
(Without reinforcement, geonet and BX1100 plus geonet)

### **3.2.3 Vertical Stress distribution on the subgrade**

The data from different pressure cell positions showed the stress distribution on the top of subgrade changing with the number of cycles. The vertical stress distribution measured at the number of 8000 cycles for different configurations is presented in Figures 31, 32, 33 and 34. Figure 31 shows the data for no-reinforcement, BX1100, and BX1200 (repeated) for the 152-mm ABC configuration, and Figure 32 shows the data for similar testing conditions but with 254-mm ABC layer. Figure 33 shows data from tests with BX4100, BX4200, and Max30 using 254-mm ABC and Figure 34 shows data for the geonet and BX1100 plus geonet cases. It is apparent from the test results that the surface stresses were transferred to a wider area on the subgrade as geosynthetic reinforcement was included. For the 152-mm ABC tests, the improvement in stress attenuation was shown as a decrease in stress magnitude at center but with an increase of stresses at the positions of 152-mm and 305-mm away from the centerline. For most of the 254-mm ABC tests, the improvement is mainly shown as a decrease in stresses at the center and 152-mm away from the center but with stress increase 305-mm away from the center of the test plate. With 254-mm ABC, only the results from test using Max30 showed a decrease of stress at center and increase of stresses 152-mm and 305-mm away from the centerline.

The stress distribution on the subgrade is affected by the reinforcement material properties and the thickness/properties of the base course. For the same thickness of base course, higher modulus geogrids (BX1200 and BX4200) provided a more uniform stress distribution than corresponding lower modulus geogrids (BX1100 and BX4100). A more uniform stress distribution with a reduced magnitude can lead to less magnitude of total and differential settlement on the subgrade. The BX1100 plus geonet test showed the most advantageous stress distribution as compared to results from the other tests.

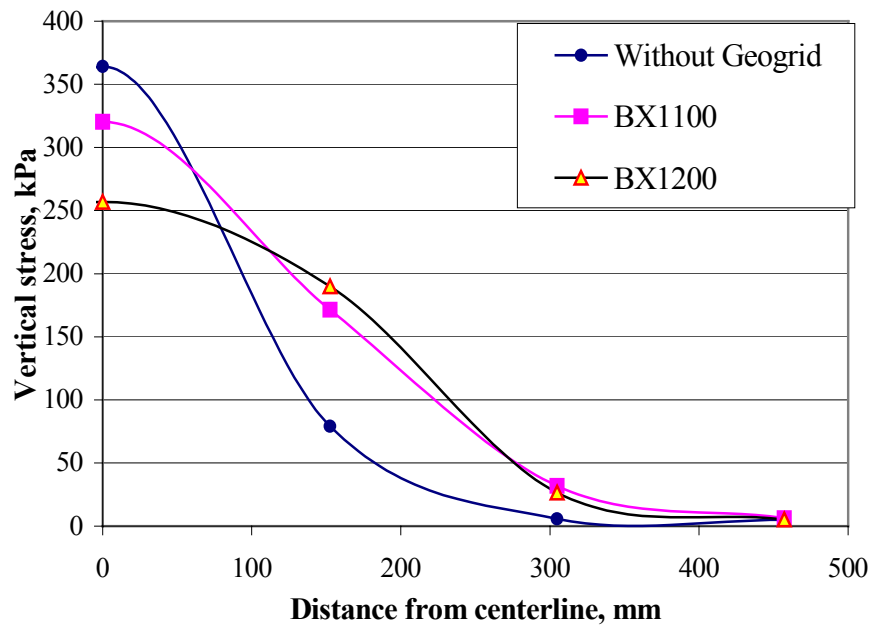


Figure 31. Vertical stress distribution at N=8000 (152-mm ABC tests)  
(Without reinforcement, BX1100 and BX1200)

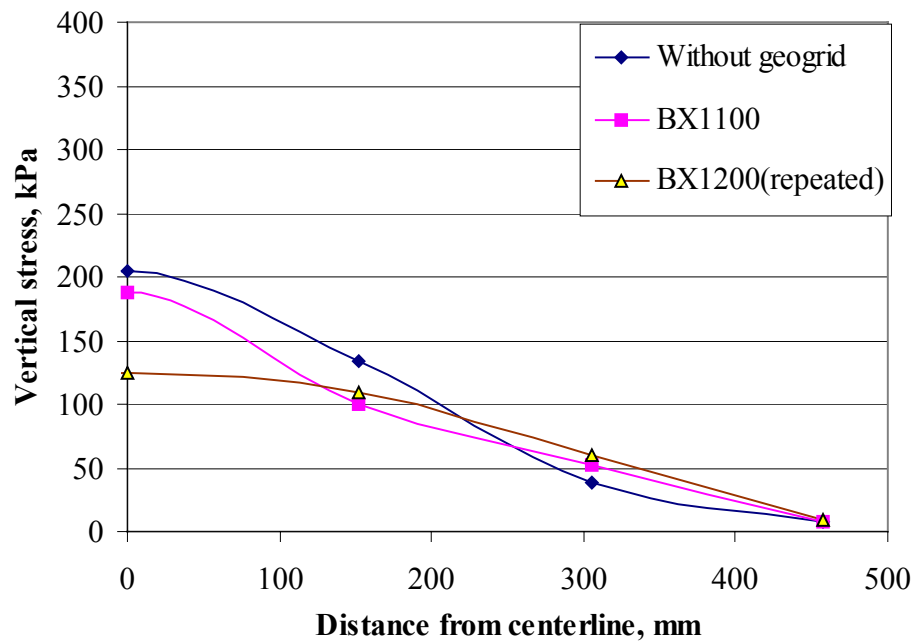


Figure 32. Vertical stress distribution at N=8000 (254-mm ABC tests)  
(Without reinforcement, BX1100 and BX1200)

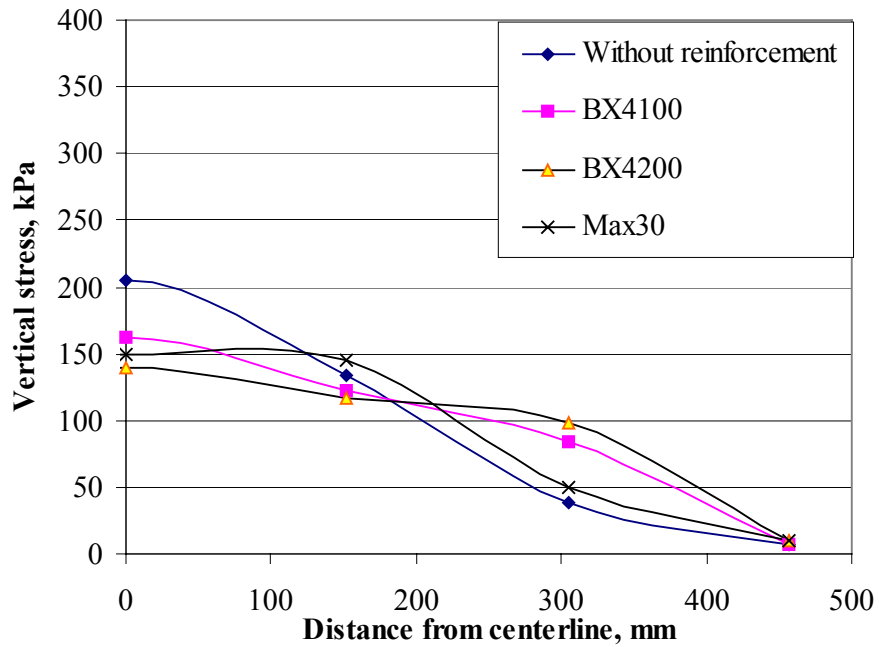


Figure 33. Vertical stress distribution at N=8000 (254-mm ABC tests)  
(Without reinforcement BX4100, BX4200, and Max30)

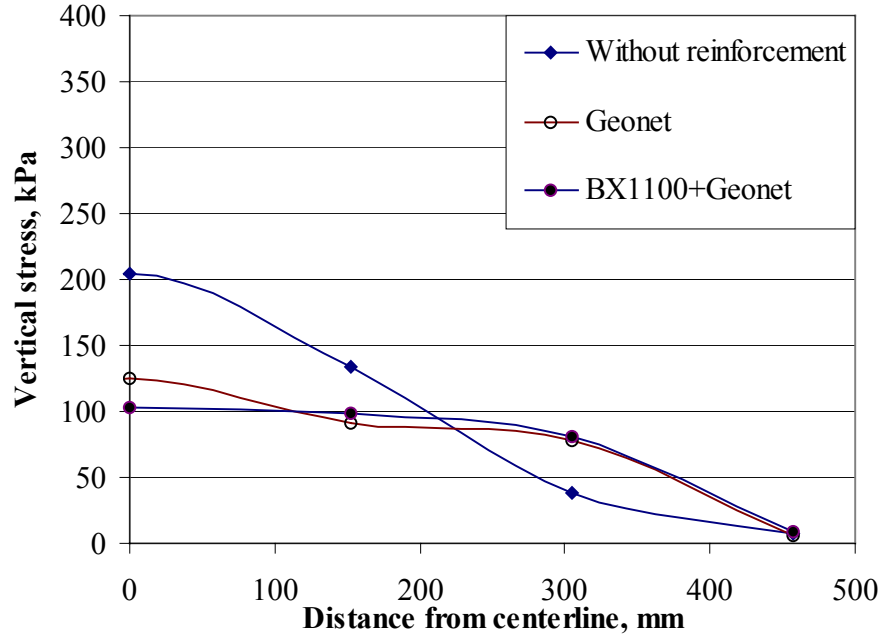


Figure 34. Vertical stress distribution at N=8000 (254-mm ABC tests)  
(Without reinforcement, geonet and BX1100 plus geonet)

### 3.2.4 Surface contours of base course and subgrade

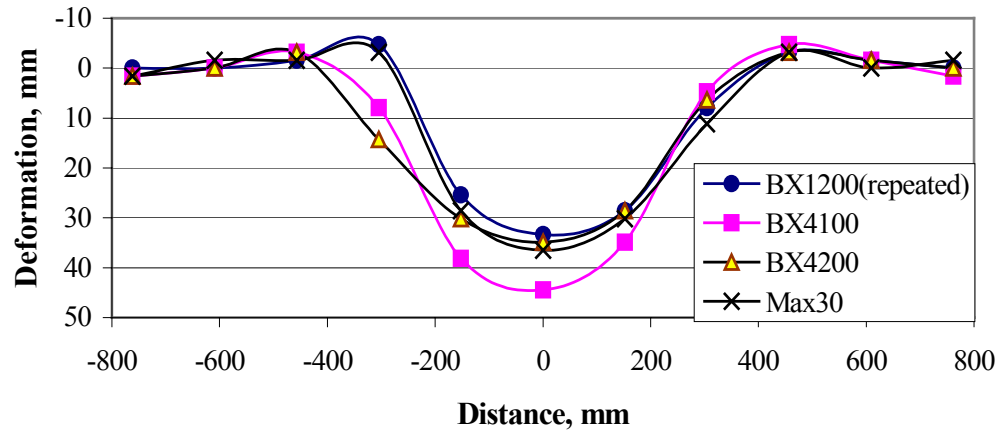
Surface contours of the top of ABC and subgrade layers were manually surveyed before and after testing. The survey was performed using a scale and a reference beam. The difference between surface survey reading before and after testing, is close to the surface deformation measured during the cyclic loading test. The deformation contour of ABC layer was almost uniform with a deformation radius of 152-mm, as shown in Figure 35. The deformation on the subgrade layer was variable but consistently indicating maximum deformation below the center of loading plate. Table 4 shows the maximum contour deformation on base layer and subgrade layers for some of 254-mm ABC tests. The maximum surveyed deformation on base course layer was larger than deformation measured in cyclic loading tests. It was not possible to survey permanent deformation resulted from setting the loading plate, and applying the static loading of 10 kN.

Table 4. Maximum contour deformation on base layer and subgrade (254-mm ABC tests)

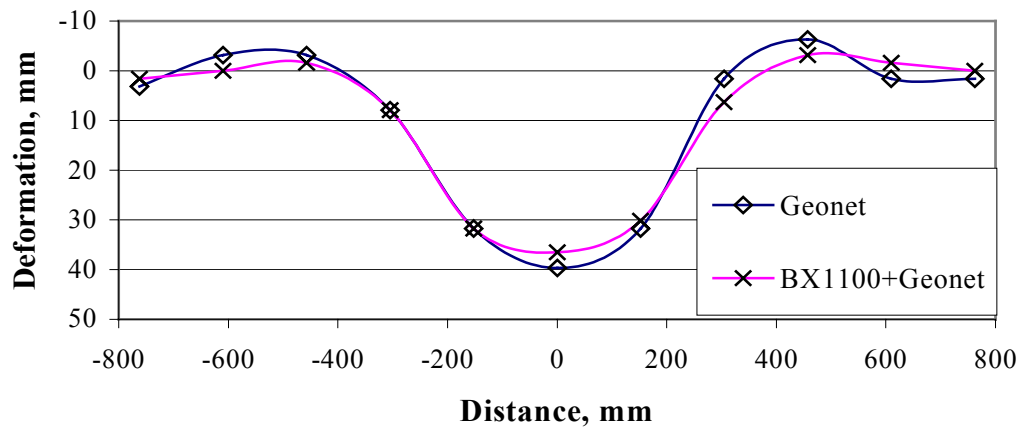
| Test                          | BX1200<br>(repeated) | BX4100             | BX4200 | Max30 |
|-------------------------------|----------------------|--------------------|--------|-------|
| Deformation on base layer, mm | 49.3                 | 58.7               | 50.8   | 49.3  |
| Deformation on Subgrade, mm   | 33.3                 | 44.5               | 35.1   | 36.6  |
|                               |                      |                    |        |       |
| Test                          | Geonet               | Geonet +<br>BX1100 |        |       |
| Deformation on base layer, mm | 60.5                 | 52.3               |        |       |
| Deformation on Subgrade, mm   | 39.6                 | 36.6               |        |       |

The maximum surveyed deformation of subgrade was 66 % to 76 % of the maximum surveyed top deformation, which indicated compression of the base layer through perhaps lateral spreading of the base course to take place during the test. Figure 35 Shows subgrade surface contours of eight 254-mm ABC tests. The subgrade settled down in the area within radius of 30 – 40 mm, beyond which there was upward deformation (heave) of approximately 3 – 5 mm. The accuracy of these measurements is

rather low since at the times it was difficult to make measurements when ABC particles intruded into subgrade or subgrade were pumped up into the ABC layer. It was found that only the geonet (with two geotextiles) totally separated base course aggregate and subgrade, as no soil mixing and contamination were observed in the two test sections with geonet and with geonet plus BX1100.



(a)



(b)

Figure 35. Surface contours of subgrade for 254-mm ABC tests

### 3.2.5 Static loading response

A static loading test was performed before the cyclic loading test. The maximum static load applied was 10 kN, which imposed average pressure of 137 kPa on the loading plate. Table 5 lists the total deformation, residual deformation and back-calculated modulus of each test.

For rigid plate on homogeneous elastic half-space, the deflection of the plate is

$$w_0 = \frac{\pi(1-\mu^2)qa}{2E} \quad (38)$$

Where,  $w_0$  = the deflection of loading plate;

$\mu$  = Poisson ratio of half-space;

$E$  = Elastic modulus of half-space;

$q$  = load on the loading plate;

$a$  = radius of the loading plate

The vertical deformation here is the deflection of loading plate. Assuming unpaved structure as homogeneous elastic half-space with Poisson's ratio of 0.4, the equivalent secant modulus and elastic modulus can be back-calculated by using total deformation and rebound deformation. The second modulus corresponds to the deformation that includes plastic deflection component while the elastic modulus is the modulus of the slope of the rebounding load-deformation curve.

$$E = \frac{\pi(1-\mu^2)qa}{2w_0} \quad (39)$$

The 152-mmABC BX1200 test showed a higher deformation with a lower modulus as compared to the 254-mmABC tests. For the 254-mmABC tests, most reinforced sections were slightly stiffer than the unreinforced section with an apparent contribution by the geosynthetic reinforcement. The sections with stiffer reinforcement showed a relatively higher modulus. The slight difference in modulus values shown between reinforced and unreinforced sections maybe due to rather low mobilization of geosynthetic reinforcement with small deformation under the applied load conditions.

Table 5. Static loading test data (Maximum load = 10 kN)

|                         |                       |                    |                  |
|-------------------------|-----------------------|--------------------|------------------|
| <b>152-mm ABC tests</b> | Without reinforcement | BX1100             | BX1200 (average) |
| Total deformation, mm   | NM                    | NM                 | 7.67             |
| Residue deformation, mm | NM                    | NM                 | 3.48             |
| Secant modulus, kPa     | NM                    | NM                 | 3546             |
| Elastic modulus, kPa    | NM                    | NM                 | 6490             |
| <b>254-mm ABC tests</b> | Without reinforcement | BX1100 (average)   | BX1200 (average) |
| Total deformation, mm   | 4.22                  | 4.09               | 3.66             |
| Residue deformation, mm | 1.57                  | 1.05               | 1.49             |
| Secant modulus, kPa     | 6451                  | 6651               | 7437             |
| Elastic modulus, kPa    | 10297                 | 8961               | 12525            |
| <b>254-mm ABC tests</b> | BX4100                | BX4200             | Max30            |
| Total deformation, mm   | 4.04                  | 3.61               | 3.68             |
| Residue deformation, mm | 1.78                  | 1.52               | 1.35             |
| Secant modulus, kPa     | 6735                  | 7541               | 7385             |
| Elastic modulus, kPa    | 12032                 | 13059              | 11640            |
| <b>254-mm ABC tests</b> | Geonet                | Geonet plus BX1100 |                  |
| Total deformation, mm   | 3.91                  | 3.63               |                  |
| Residue deformation, mm | 1.70                  | 1.27               |                  |
| Secant modulus, kPa     | 6954                  | 7489               |                  |
| Elastic modulus, kPa    | 12309                 | 11515              |                  |

Note: NM = No measurements were taken

### 3.2.6 Cyclic plate load tests on subgrade

Three cyclic load tests were performed on the subgrade to discern the performance of permanent deformation of subgrade layer itself. The amplitude of cyclic loads applied were 10 kN, 15 kN and 18 kN, which imposed average pressure of 137 kPa, 201 kPa and 247 kPa on the loading plate. Three values were chosen to represent magnitude of stress transferred to the subgrade during testing of the full sections. The permanent deformation accumulation under repeated load is shown in the Figure 36. The permanent deformation of subgrade is related to the stress amplitude of the vertical load applied on the loading plate.

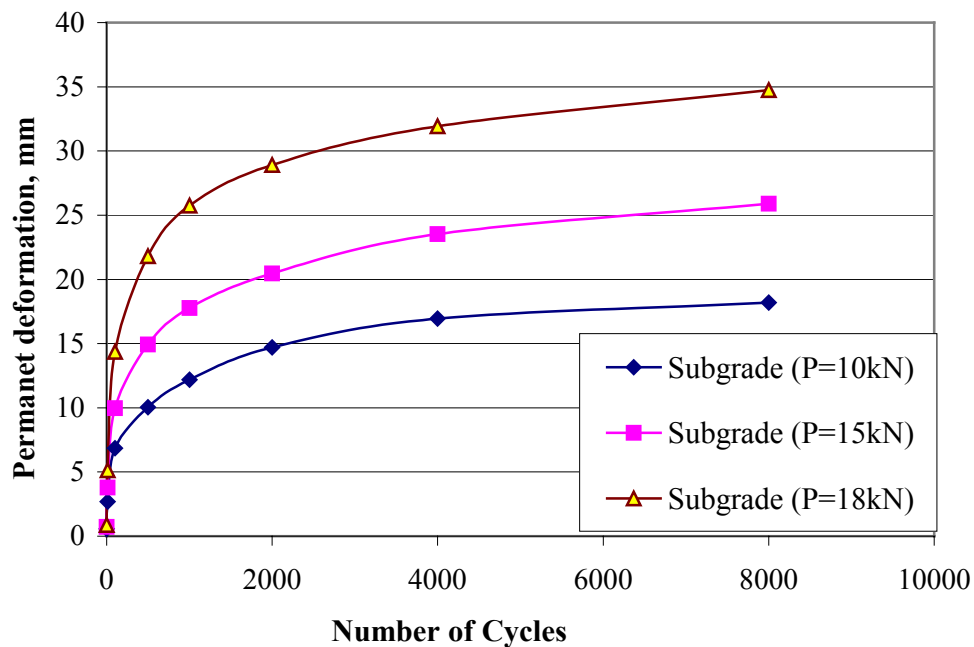


Figure 36. Plastic deformation development

### 3.3 Summary and discussions

Fourteen large-scale cyclic loading tests were performed in the laboratory on sections representing unpaved roads. The test sections were composed of aggregate base course (ABC) with variable thickness, subgrade soil and geosynthetic reinforcement. The subgrade soil was clayey sand with a laboratory-measured CBR of 3%. Both 152-mm

ABC tests and 254-mm ABC tests provided data on the surface deformation accumulation and stress change under cyclic loading. In general, results indicated that reinforcement improved the deformation and led to decreasing the surface deformation, improving stress distribution transferred to the subgrade, and decreasing of degradation of the ABC modulus. In this testing program, geogrids BX1200, BX4200 and Max30 provided the best reinforcement effect in reducing surface deformation. Compared with geogrid-reinforced sections, the section with geonet reinforcement and the section with BX1100 plus geonet reinforcement showed a larger increase rate of surface deformation during steady state period, which maybe due to fatigue of geonet (with two layers of geotextiles) under cyclic loading.

The improvement in stresses due to geosynthetic inclusion at the interface of ABC and foundation soil was shown in two aspects: decreasing the maximum stress (under the center of the test plate) and producing a more uniform stress distribution on the subgrade soil. As presented in previous literature, the improvement is related to two mechanisms: lateral confinement of base course and tensile membrane effect of geosynthetics. Higher modulus geogrids (BX1200 and BX4200) provided a better stress attenuation effect compared to lower modulus geogrids (BX1100 and BX4100). Geonet and BX1100 plus geonet test results showed a larger reduction of center stresses specially at high number of cycles perhaps due to additional tensile membrane effect from the geonet (with two geotextiles).

Surveyed surface contours of base course and foundation layers showed deformation on the subgrade to be smaller than the surface deformation on the surface of base course layer. This indicated that the total surface deformation has two components: base course compression due to lateral spreading and subgrade soil deformation. The static loading tests show that reinforced sections have a slightly higher modulus than unreinforced section. At small applied stresses, reinforcement contribution in terms of membrane action is not be fully mobilized, and the increase in modulus values may be attributed to confinement effect. The deformation modulus decreased as surface deformation accumulated with the increase of the number of cycles. These data are indicative of the degradation in ABC modulus and can be correlated to resilient modulus

of the materials. With higher modulus of geogrid reinforcement, a less degradation of equivalent modulus was obtained.

It is of interest to note that the decrease in surface deformation and the improvement of stress condition on the subgrade were not always compatible. It seems that improvement in stress condition on the subgrade soil was mainly related to the modulus or stiffness of geosynthetics. The geonet test and BX1100 plus geonet test showed better improvement in stress condition on the subgrade than geogrid alone. However, the surface deformation of the test sections includes a component due to lateral spreading of base course. As the ABC degrades under cyclical load, the contribution to deformation from lateral spreading of the ABC layer increases.

## **Chapter 4 DEGRADATION AND PLASTIC DEFORMATION**

While the unbounded material of unpaved road does not necessarily fatigue under cyclic load like asphalt, it incrementally degrades under repeated load. Damage is seen in the accumulation of plastic deformation over many load cycles, or rutting.

Previous analysis and design methods of geosynthetic-reinforced unpaved structure focused on the performance of subgrade, in terms of the reinforcement impact on subgrade bearing capacity and vertical stress attenuation due to tension membrane effect. The base layer was assumed to be of good quality without loss in attenuating function of the vertical stress transferred to subgrade, under the repeated traffic load. The basic equations of analysis were based on static limit analysis, and consideration of cyclic load was incorporated using empirical relation between required thickness of base layer and number of load cycles (Hammit, 1970). Giroud and Noiray (1981) and Giroud et al. (1984) used such relation for the reinforced unpaved structure. The benefit of geosynthetic reinforcement was considered as an equivalent increase in base layer thickness. There are two shortcomings of this method. The performance of aggregate under cyclic load was not clearly explained and the degradation of such layer was not explicitly considered.

Work presented in this chapter aims at analyzing the performance of reinforced aggregate base course placed over soft soil under cyclic load. The results of the experimental study are analyzed to explain the performances with degradation of the ABC layer and associated permanent deformation. Based on the test results, a correlation between ABC-subgrade elastic modulus ratio and number of applied load cycles, a correlation between the stress distribution angle and number of load cycles, and a correlation between the plastic deformation and number of load cycles, are developed to evaluate the degradation of base layer and permanent surface deformation, with the geogrid torsional stiffness used as index for reinforcement performance.

## 4.1 Degradation of unpaved structure

Undrained shear strength of subgrade is a mechanical property, which governs the behavior of the soft clay subgrade. The progressive deterioration of the subgrade soil can be expressed by the decrease of its undrained shear strength as the number of the traffic cycles increase. Grioud et al. (1984) proposed a coefficient  $\lambda$  to represent the progressive deterioration of the subgrade undrained strength under cyclic loading due to traffic, as was shown in Equation 12. It was found that the deterioration was significant for subgrade soils with high strength. For the unpaved structures on the soft subgrade, the degradation was mainly from the deterioration of base layer (Grioud et. al., 1984).

As a structure layer directly exposed to cyclic load, the base course aggregate experiences progressive deterioration or degradation of its mechanical properties, as well as its effective thickness. The degradation of the aggregate base course leads to an increase in stresses transferred to subgrade. There are many factors affect the degradation of the base layer, such as lateral spreading of base layer, the contamination by the fine particles from subgrade, and breakdown of base course aggregate. As these mechanisms are complicated, there is not a method currently available to explicitly evaluate the degradation of base course aggregate.

During the plate cyclic load tests, the degradation of unpaved structure was manifested as deterioration of load spreading ability of the base layer, or increase in vertical stress transferred to the subgrade. Based on the vertical stresses measured on the subgrade during the testing program, the degradation of aggregate base layer is characterized through the change in elastic modulus ratio of ABC layer to modulus of subgrade, and back-calculated stress distribution angle.

### 4.1.1 Back-calculation analysis

Based on elastic layer analysis, vertical stresses transferred to the top of subgrade are related to the thickness of base layer and the ratio of elastic modulus of the base course layer to that of the subgrade layer. With the measured stress on the subgrade, an

elastic layer analysis method is used to back-calculate the ratio of elastic modulus of the base course to that of the subgrade.

The back calculation is performed using Odemark's method (Ullidtz, 1987), which is an approximate method describing stress distribution within an elastic layered system. The principle of this method is to transform a system consisting of several layers with different moduli into an equivalent system where all layers have the same modulus, and on which Boussinesq's method may be used.

For a two-layer system, the stresses, strains and compression of the first layer above an interface are calculated by treating the system as homogenous elastic half-space with modulus  $E_1$ . When calculating the stresses, strains, and deflections at the interface or below the interface, the top layer is transformed to an equivalent layer with modulus  $E_2$ . To keep the same stiffness as the original layer, the equivalent thickness  $h_e$  is:

$$h_e = h \left[ \frac{E_1(1-\mu_2^2)}{E_2(1-\mu_1^2)} \right]^{\frac{1}{3}} \quad (40)$$

Here,  $h$  = thickness of base layer,  $E_1$  = elastic modulus of base layer,  $E_2$  = elastic modulus of subgrade,  $\mu_1$  = Poisson's ratio of base layer,  $\mu_2$  = Poisson's ratio of subgrade.

Based on the Odemark's method, the interface vertical stress ( $\sigma_c$ ) underneath the center of the loaded area (with radius =  $a$ ) can be expressed as:

$$\sigma_c = p \left[ 1 - \frac{h_e^3}{(a^2 + h_e^2)^{1.5}} \right] \quad (41)$$

The vertical deflection on the subgrade ( $w_c$ ) and the compression of the base layer ( $\Delta w_1$ ) can be expressed as:

$$w_c = \frac{(1+\mu_2)pa}{E_2} \left\{ \frac{a}{(a^2 + h_e^2)^{0.5}} + \frac{1-2\mu_2}{a} \left[ (a^2 + h_e^2)^{0.5} - h_e \right] \right\} \quad (42)$$

$$\Delta w_1 = \frac{(1 + \mu_1)pa}{E_1} \left\{ 1 - \frac{a}{(a^2 + h^2)^{0.5}} + \frac{1 - 2\mu_1}{a} [a + h - (a^2 + h^2)^{0.5}] \right\} \quad (43)$$

Ullidtz (1987) discussed the limitations of the use of equivalent thickness method. One is that the modulus should be decreasing with depth, preferably with the modulus ratio of the upper layer to the lower layer larger than 2. Another is that the equivalent thickness should preferably be larger than the radius of the loaded area. Using Odemark's method and Boussinesq's solution, the vertical stress at the interface with different elastic modulus ratio and depth of first layer is shown in Figure 37. The Poisson's ratios selected are 0.35 and 0.42 for base layer and subgrade, respectively, which are the averages of the typical Poisson's ratios of unbound granular material (0.2 - 0.5) and subgrade (0.3 - 0.5) (Yoder, 1975). Boussinesq's method was directly used for the case with  $E_1/E_2 = 1$ . For  $E_1/E_2 = 1-10$ , and  $a/h = 0.15 - 1.5$ , vertical stress distribution based on Odemark's method is close to the results from other elastic layer methods (e.g. Huang, 1969). For example, given  $E_1/E_2 = 10$  and  $a/h = 1.2$ , the predicted  $\sigma_c/p$  is 0.36 from Odemark's method and 0.37 from Huang (1969); given  $E_1/E_2 = 2.5$  and  $a/h = 0.4$ , the predicted  $\sigma_c/p$  is 0.13 from Odemark's method and 0.15 from Huang (1969).

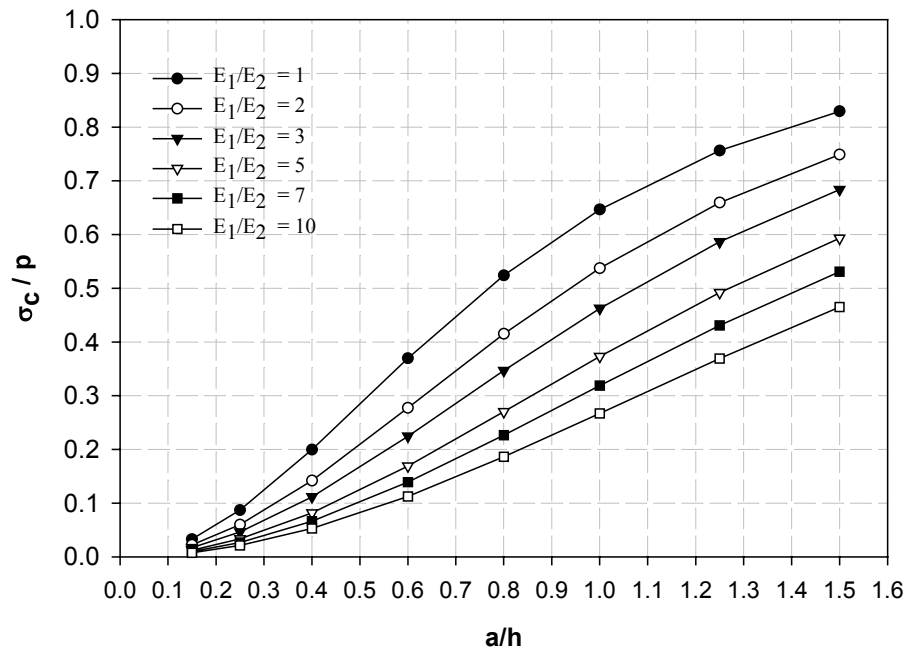


Figure 37. Vertical interface stress for two-layer system based on Odemark's method

#### 4.1.2 Degradation of modulus ratio

The Back-calculated elastic ratio of 152-mm ABC tests and 254-mm ABC tests (without reinforcement, with BX1100, and BX1200) are shown in Figure 38 and 39 respectively. The regression data of the degradation curves are also presented. Other 254-mm ABC test results are presented in Table 6, for the case of 8000 cycles. The degradation coefficient ( $\lambda_1$ ) of the elastic modulus ratio can be expressed as:

$$\lambda_1 = \frac{(E_1/E_2)_N}{(E_1/E_2)_1} = \frac{1}{1 + k_1 \log N} \quad (44)$$

Where,  $(E_1/E_2)_N$  = elastic modulus ratio at the cycle number of N,  $(E_1/E_2)_1$  = elastic modulus ratio at first load,  $k_1$  = constant, representing the degradation rate of elastic modulus ratio.

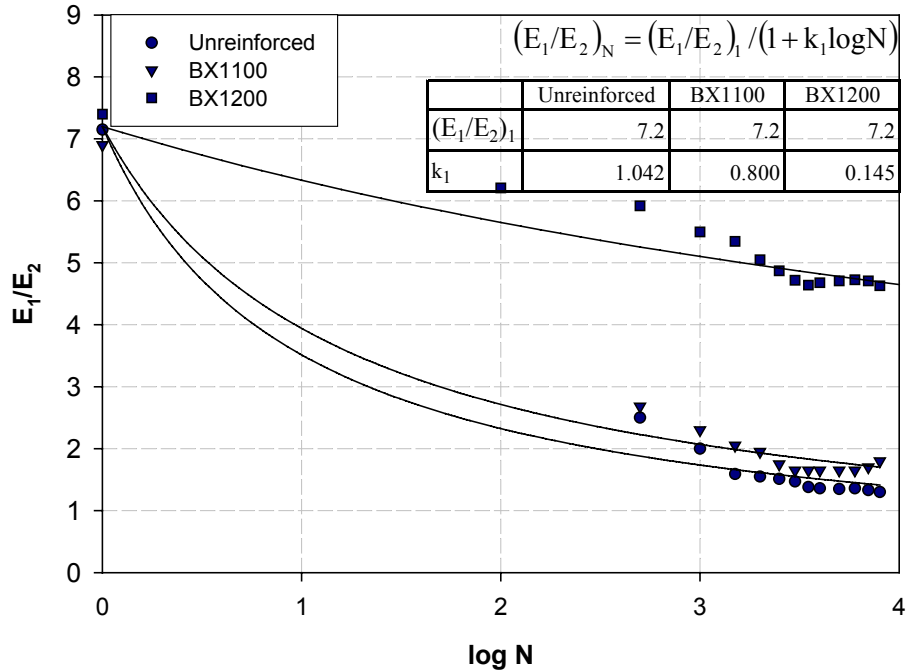


Figure 38. Elastic modulus ratio of 152-mm ABC tests  
(Without reinforcement, BX1100 and BX1200)

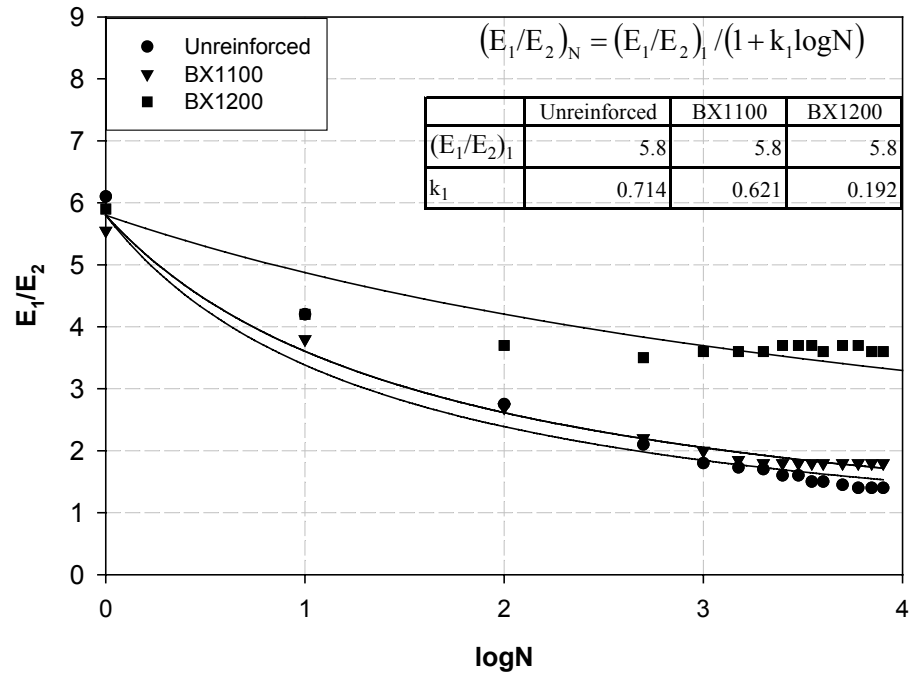


Figure 39. Elastic modulus ratio of 254-mm ABC tests  
(Without reinforcement, BX1100 and BX1200)

Table 6. Back calculated modulus ratio ( $E_1/E_2$ ) at the end of 8000 load cycles

| 152-mm ABC tests |              |        |        | 254-mm ABC tests |               |        |        |
|------------------|--------------|--------|--------|------------------|---------------|--------|--------|
| case             | Unreinforced | BX1100 | BX1200 | case             | Unreinforced  | BX1100 | BX1200 |
| N=8000           | 1.3          | 1.7    | 4.6    | N=8000           | 1.4           | 1.8    | 3.7    |
|                  |              |        |        |                  |               |        |        |
| 254-mm ABC tests |              |        |        |                  |               |        |        |
| case             | BX4100       | BX4200 | Max30  | Geonet           | Geonet+BX1100 |        |        |
| N=8000           | 2.3          | 3.1    | 2.8    | 3.8              | 5.4           |        |        |

The initial elastic modulus ratio is almost the same for unreinforced, BX1100, and BX1200 sections. This value is approximately 7 for the 152-mm ABC tests, and 6 for the 254-mm ABC tests. The geogrid reinforcements do not show significant impact on the

initial elastic modulus ratio. This indicates that the benefits of geogrid reinforcements are mainly related to the number of load repetitions or the deformation level.

The elastic ratio decreased with number of cycles. For the same subgrade with  $\text{CBR} = 3$  and the same base course aggregate, the degradation was related to the thickness of ABC, and geosynthetic reinforcements. After 8000 load cycles, the elastic modulus ratio of the 152-mm ABC tests decreased to 1.3, 1.7, and 4.6 for unreinforced, BX1100, and BX1200 sections respectively. The elastic modulus ratio of the 254-mm ABC tests decreased to 1.4, 1.8, and 3.7 for unreinforced, BX1100, and BX1200 sections. The degradation coefficient as a function of load cycles can be approximately presented by Equation 44. The degradation rates of 152-mm test was higher than the rate corresponding to the 254-mm tests. Compared with unreinforced sections, the degradation of reinforced test sections with increasing number of cycles was slower. The geogrid performance seems to be related to stiffness ( $J_g$ ). With the same aperture size, geogrid BX1200 ( $J_g = 480 - 650 \text{ kN/m}$ ) showed better performance than geogrid BX1100 ( $J_g = 220 - 360 \text{ kN/m}$ ).

For the 254-mm ABC tests, other geosynthetic reinforcements were used in the testing program. Back-calculated modulus ratios of BX4100, BX4200, Max30, Geonet, Geonet plus BX1100 are 2.3, 3.1, 2.8, 3.8, and 5.4 after 8000 load cycles. The geogrids with higher tensile stiffness (e.g. BX4200) showed better response in term of slower degradation. It is interesting to note that Geonet which has more separation function than reinforcement effect also decreased degradation of ABC. The section with Geonet plus BX1100 shows the best performance against degradation.

#### **4.1.3 Degradation of stress distribution angle**

The previous research (Love et al., 1987) showed that geosynthetic inclusion at the interface of base course and subgrade can improve the stress distribution at low deformation and provide larger stress distribution angle than unreinforced case. The vertical stress distribution at the interface of two-layer system is affected by several factors: properties of base course and subgrade, the thickness of base course and the interface properties. In a practical design, the stress distribution angle is introduced to

represent load distribution through base layer, and the benefit of geogrid reinforcement can be viewed as an increased stress distribution angle.

In this research, the maximum vertical stress on the subgrade (underneath the center of load area) was conservatively to determine the stress distribution angle. This method is simple and conservative. For the circular loading plate with radius of  $a$ , the angle of stress distribution is calculated as:

$$\tan\alpha = \frac{a}{h} \left( \sqrt{\frac{p}{\sigma_c}} - 1 \right) \quad (45)$$

Where,  $\alpha$  = stress distribution angle,  $a$  = radius of plate,  $h$  = thickness of base layer,  $p$  = pressure applied on the plate,  $\sigma_c$  = maximum vertical stress on the subgrade.

Using the vertical stress predicted by Odemark's method (equation 38 and 39), the stress angle of two-layer system is can be expressed as a function of elastic modulus ratio ( $E_1/E_2$ ) and the radius to thickness ratio ( $a/h$ ):

$$\tan\alpha = \frac{a}{h} \left( \sqrt[1.5]{\frac{\left( \left( \frac{a}{h} \right)^2 + \left( \frac{E_1(1-\mu_2^2)}{E_2(1-\mu_1^2)} \right)^{\frac{2}{3}} \right)^{1.5}}{\left( \left( \frac{a}{h} \right)^2 + \left( \frac{E_1(1-\mu_2^2)}{E_2(1-\mu_1^2)} \right)^{\frac{2}{3}} \right)^{1.5} - \frac{E_1(1-\mu_2^2)}{E_2(1-\mu_1^2)}}}} - 1 \right) \quad (46)$$

The stress distribution angle with varied modulus ratio ( $E_1/E_2$ ) and the radius to thickness ratio ( $a/h$ ) is shown in Figure 40, with Poisson's ratio of base layer  $\mu_1 = 0.42$ , Poisson's ratio of subgrade  $\mu_2 = 0.35$ .

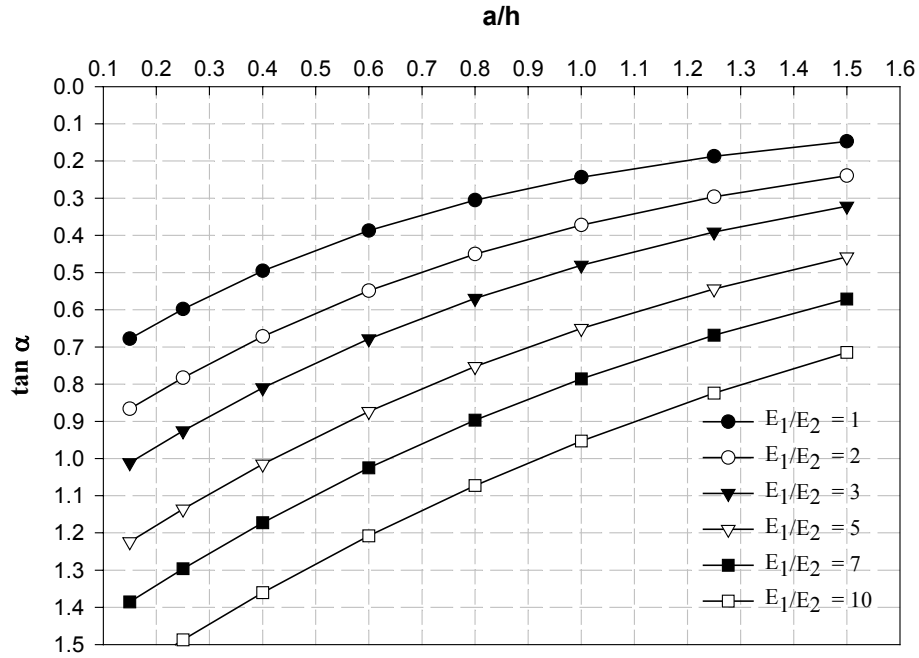


Figure 40. Stress distribution angle for two-layer system based on Odemark's method ( $\mu_1 = 0.42$  and  $\mu_2 = 0.35$ )

Back-calculated stress distribution results of 152-mm ABC tests and 254-mm ABC tests (without reinforcement, with BX1100 and BX1200) are shown in Figure 41 and 42 respectively.

A degradation coefficient can also be presented as stress distribution angle decrease with number of cycles. The degradation coefficient ( $\lambda_2$ ) of the stress distribution angle can be expressed as:

$$\lambda_2 = \frac{\tan \alpha_N}{\tan \alpha_1} = \frac{1}{1 + k_2 \log N} \quad (47)$$

Where,  $\alpha_N$  = stress distribution angle at the cycle number of  $N$ ,  $\alpha_1$  = stress distribution angle at first load,  $k_2$  = constant. The  $k_2$  value is indicative of the degradation in  $\tan \alpha$ .

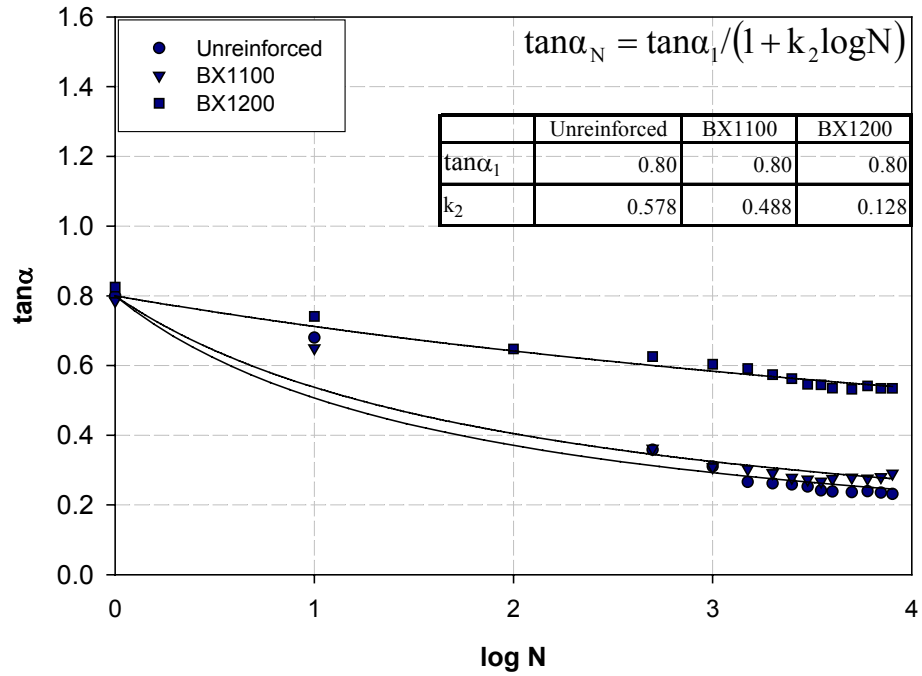


Figure 41. Stress distribution angle of 152-mm ABC tests  
(Without reinforcement, BX1100 and BX1200)

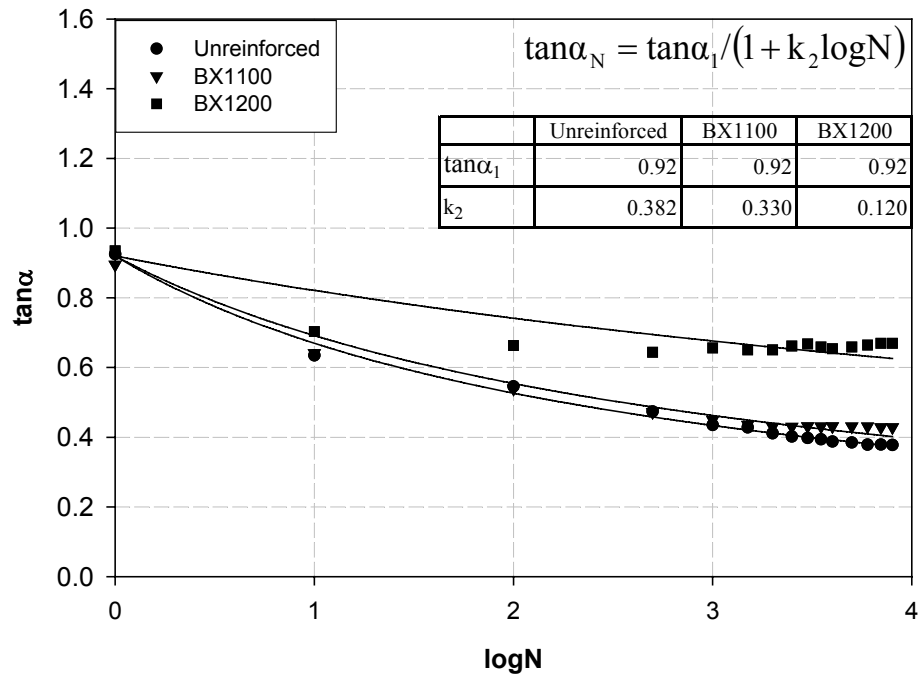


Figure 42. Stress distribution angle of 254-mm ABC tests  
(Without reinforcement, BX1100 and BX1200)

For all the 152-mm ABC tests and 254-mm ABC tests, the initial  $\tan\alpha$  value is approximately similar (around 0.8 for 152-mm ABC tests, 0.9 for 254-mm ABC tests) for unreinforced, BX1100, and BX1200 sections. After 8000 load cycles, the  $\tan\alpha$  value of the 152-mm ABC tests decreased to 0.24, 0.30, and 0.56 for unreinforced, BX1100, and BX1200 sections respectively. The  $\tan\alpha$  value of the 254-mm ABC tests decreased to 0.38, 0.43, and 0.65 for unreinforced, BX1100, and BX1200 sections respectively. As the degradation coefficient under cyclic load can be represented by Equation 47, it is clear that the degradation rates were higher for thinner base layer (152-mm tests). The high modulus geogrid BX1200 yielded better performance in reducing the degradation of stress distribution angle.

## **4.2 Plastic deformation of unpaved structure**

The plastic deformation can be viewed within the context of two mechanisms. If the base layer consists of weak granular materials, local shear stress in the vicinity of the wheel load may result in heave adjacent to the wheel track, as granular material undergoes large plastic shear strains. This type of rutting is thus largely due to inadequate shear strength of granular material. When aggregate quality is good, the section as a whole may still rut if the subgrade deforms underneath the granular layer. It was ideally assumed that there is no thinning or lateral spread of base layer. This assumption is used in many unsurfaced haul road design methods (e.g. Giroud and Noiray, 1981).

In general, rutting will be a combination of the above two mechanisms, lateral spreading of the granular layer and depression of the subgrade. Under cyclic loading, plastic deformation or rutting of railroad ballast, roadway foundations, and embankment was reduced with inclusion of geosynthetic reinforcement (Milligan et al., 1986). Previous research has noted that there is an optimum position to place a geosynthetic reinforcement to prevent rutting. Bathurst et al. (1986) reported that it was at a depth of about a third of the width of the loaded area (for a dual tire pair this is around 0.3m). The best position must be the depth at which the tensile strain (and hence reinforcement contribution) will be a maximum.

#### 4.2.1 Empirical correlation of plastic deformation

In pavement analysis, the study on cyclic plastic deformation of base course and subgrade is somewhat limited to the laboratory cyclic triaxial testing and modeling. The permanent deformation is affected loading conditions, such as stress level, principal stress orientation, number of load application, and soil properties, such as moisture content, stress history, density, fine content and grading (Lekarp, et al., 2000). It is difficult to predict unpaved road performance under the larger deformation and degradation. Several empirical formulas have been introduced in the literature for the prediction of plastic deformations of reinforced soils. Based on test results on geogrid-reinforced railroad ballast on compressible subgrade, Bathurst et al. (1986) proposed the permanent deformation on the surface be expressed as:

$$s = a + b(\log N) + c(\log N)^2 \quad (48)$$

Where,  $s$  = permanent deformation,  $N$  = number of cycles, and  $a$ ,  $b$ , and  $c$  = empirical coefficients obtained through regression analysis.

In this study, the relationship of the permanent surface deformation and the number of cycles number similarly. The permanent deformation it is rewritten as:

$$s_N = s_1 + b(\log N) + c(\log N)^2 \quad (49)$$

Where,  $s_N$  = total permanent surface deformation after  $N$  load cycles;  $s_1$  = permanent deformation under the first load cycle (3 mm used in this research according to the average plastic deformation at first load),  $b$  and  $c$  = empirical coefficients obtained through regression analysis. Both  $s_N$  and  $s_1$  are in unit of mm here.

Figure 43 and Figure 44 present the regression results of 152-mm ABC tests and 254-mm ABC tests. It seems that the empirical coefficient “ $c$ ” relates to the effect of base layer thickness, and “ $b$ ” related to the effect of geogrid improvement. The plastic deformation rate  $s/\log N$  equals to  $b + c \log N$ . At the end of 8000 cycles, the deformation rate was mainly related to  $c$  value (or related to base layer thickness). In this study,  $c$  value can be expressed as a function of  $a/h$  as follows:

$$c = c_{a/h=1} \left( \frac{a}{h} \right)^{0.67} = 2.23 \left( \frac{a}{h} \right)^{0.67} \quad (50)$$

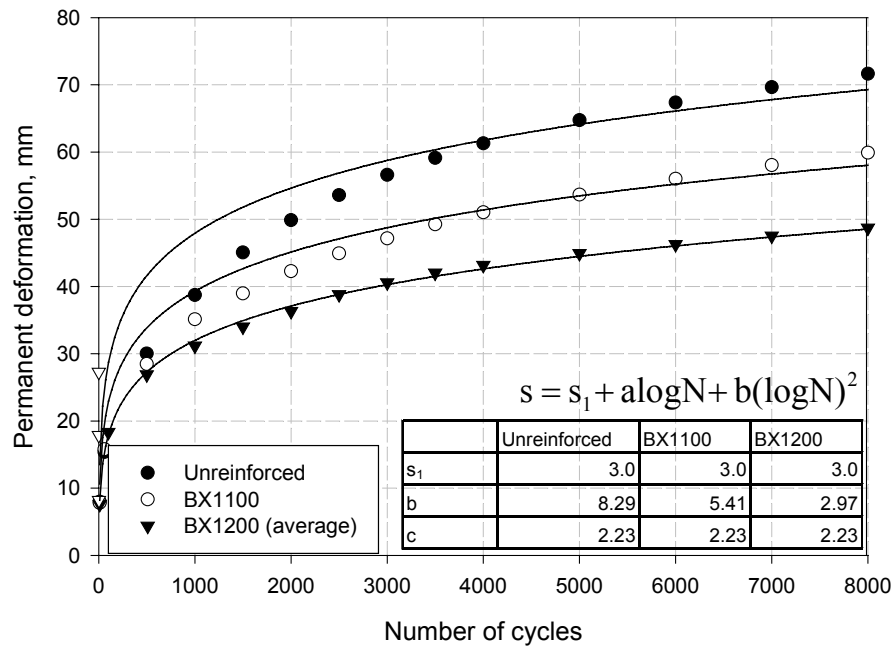


Figure 43. Permanent deformation for 152-mm ABC tests

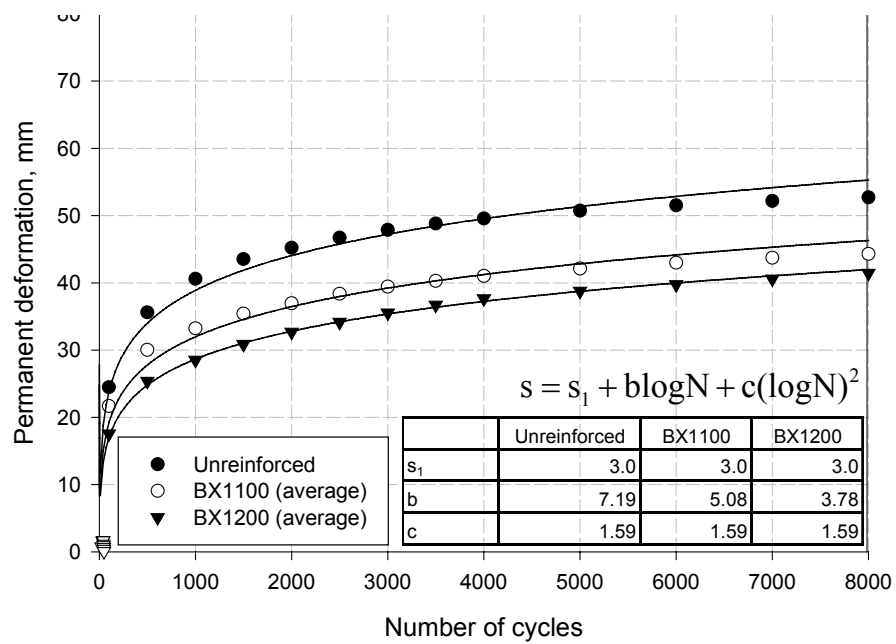


Figure 44. Permanent deformation for 254-mm ABC tests

#### 4.2.2 Plastic deformation component: subgrade and base layer

The plastic deformation of the subgrade only has been studied, with the surface load of 138 kPa, 206 kPa, and 248 kPa ( $P = 10, 15, 18$  kN on the loading plate). The testing information was used to back calculate the subgrade plastic deformation of the test sections.

For the rigid plate tests on the section with subgrade layer only, the deflection is:

$$w = \frac{\pi(1-\mu^2)pa}{2E}$$

For test sections with both subgrade layer and base layer, vertical stress transferred to the subgrade layer can be simplified as flexible load plate, with diameter of  $a' = a + h \tan \alpha$ . The deflection is:

$$w' = \frac{2(1-\mu^2)\sigma_c a'}{E} = \frac{2(1-\mu^2)\sigma_c (a + h \tan \alpha)}{E}$$

The back calculated subgrade plastic deformation  $s_s$  can be presented as:

$$s_s = \frac{w'}{w} s_t = \frac{4}{\pi} s_t \quad (51)$$

Where,  $S_t$  = total permanent deformation of the two-layer test section,  $S_{bc}$  and  $S_s$  = permanent deformation of base layer and subgrade.

The plastic deformation of the base layer (base layer compression) can be evaluated by subtracting the plastic deformation of subgrade from the total plastic deformation.

$$S_{bc} = S_t - S_s \quad (52)$$

Following the above procedures, the plastic deformation on the subgrade and base course compression can be calculated. Table 7 lists the computed plastic deformation results of 254-mm ABC tests at the end of 8000 cycles. The back calculation was not performed for 152-mm ABC as the magnitude of stresses are beyond the stresses used in the subgrade cyclic load tests. It is obvious that the magnitude of the vertical plastic

deformation of subgrade ( $S_s$ ) was dependent on the amount of the vertical stress transferred to the subgrade ( $\sigma_c$ ). Reinforcement with better performance on reducing degradation of load spread ability ( $\tan\alpha_N$ ) can reduce more subgrade deformation. Both reinforcement and separation functions of geosynthetic have impact on reducing subgrade deformation under many cycles. Compared with unreinforced section, the base course deformation can be limited by geogrid reinforcements (due to interlock). However, test sections with geonet show more base course plastic information than the unreinforced section and other reinforced sections with geogrid only, due to relatively low lateral confinement (without interlock) on the base course aggregate. The back calculated deformations were relatively close to the contour survey results listed in Table 4( with 60-70% of total deformation from subgrade settlement).

Table 7. Back calculated permanent deformation at the end of 8000 load cycles

| Reinforcement    | Unreinforced | BX1100 | BX1200        | BX4100 | BX4200 |
|------------------|--------------|--------|---------------|--------|--------|
| $S_t$ , mm       | 52.7         | 44.3   | 41.4          | 48.6   | 40.5   |
| $\sigma_c$ , kPa | 203          | 182    | 128           | 163    | 140    |
| $\tan \alpha_N$  | 0.38         | 0.43   | 0.65          | 0.49   | 0.56   |
| $S_s$ , mm       | 33.2         | 31.6   | 27.7          | 30.3   | 28.7   |
| $S_{bc}$ , mm    | 19.5         | 12.7   | 13.8          | 18.3   | 11.9   |
| $S_s / S_t$      | 0.63         | 0.71   | 0.67          | 0.62   | 0.71   |
| $w_c / w$        | 0.52         | 0.57   | 0.70          | 0.61   | 0.67   |
|                  |              |        |               |        |        |
| Reinforcement    | Max30        | Geonet | Geonet+BX1100 |        |        |
| $S_t$ , mm       | 41.3         | 48.6   | 44.5          |        |        |
| $\sigma_c$ , kPa | 149          | 124    | 103           |        |        |
| $\tan \alpha_N$  | 0.54         | 0.65   | 0.79          |        |        |
| $S_s$ , mm       | 29.3         | 27.5   | 25.8          |        |        |
| $S_{bc}$ , mm    | 12.0         | 21.1   | 18.7          |        |        |
| $S_t$ , mm       | 41.3         | 48.6   | 44.5          |        |        |
| $S_s / S_t$      | 0.71         | 0.57   | 0.58          |        |        |
| $w_c / w$        | 0.65         | 0.71   | 0.76          |        |        |

Table 7 also shows that the plastic deformation ratio ( $S_s/ S_t$ ) can be approximated by using a deformation ratio ( $w_c /w$ ) from elastic layer analysis. The ratio of subgrade deformation ( $w_c$ ) and total deformation ( $w = w_c + \Delta w_1$ ) can be determined by using Equation 42 and 43. The estimated deformation ratio for different  $a/h$  values ( $a/h = 0.4, 0.6, 1.0, 1.5$ , and  $3.0$ ) and  $E_1/E_2$  values ( $E_1/E_2 = 1, 2, 3, 5, 10$ ) is represented in Figure 45. And the deformation ratio can be expressed by:

$$w_c/w = 1 - e^{-1.06(a/h)\sqrt{E_1/E_2}} \quad (53)$$

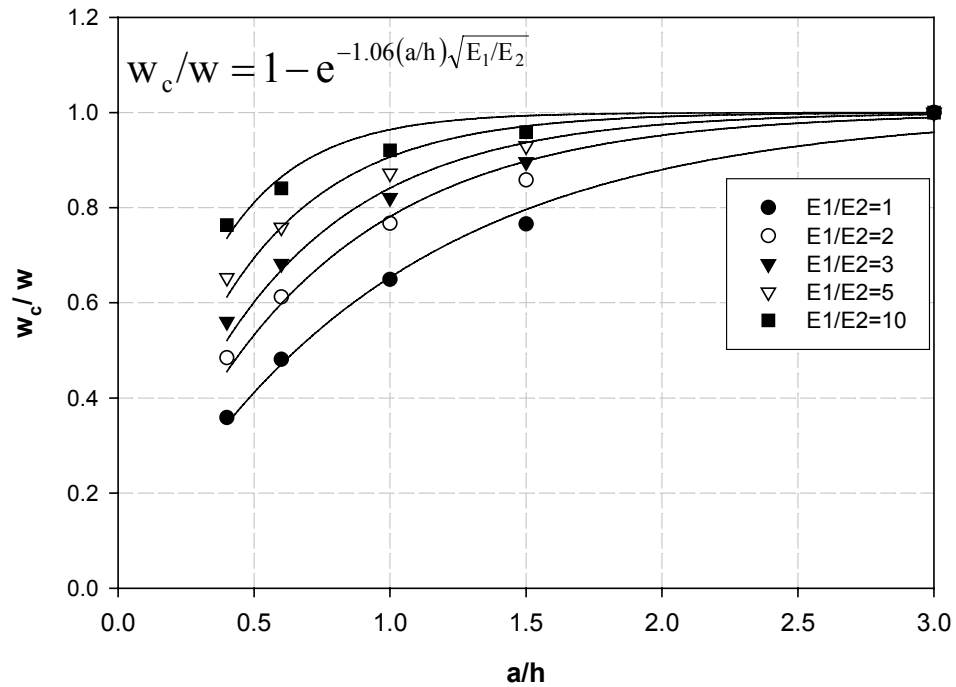


Figure 45. Estimated deformation ratio of two layer system

## **4.3 Modeling performance under cyclic load**

### **4.3.1 Key properties of geogrid reinforcement**

Giroud and Noiray (1981) used geotextile tensile modulus to evaluate the performance of geotextile-reinforced unpaved road, where tension membrane effect was the main contribution of geotextile reinforcement. For geogrid-reinforced unpaved road, the mechanism of geogrid reinforcement is more complicated, and the performance largely depends on the interaction of geogrid and aggregate (interlock effect). However, some attempts at relating the geometry and strength properties of geogrids to the performance of geogrid-reinforced aggregate under cyclic load were not successful.

Webster (1992) summarized several geogrid properties: aperture size, shape and aperture stability; rib thickness, stiffness and shape; junction strength; and geogrid tensile modulus. These factors affect the performance of reinforced base courses for flexible pavements as follows:

- i) Geogrid aperture must be large enough to permit aggregate strike-through, but small enough to provide effective interlock; an aperture size between 0.75 inches (19 mm) and 1.5 inches (38 mm) was suggested as a good target range for road base aggregate materials; geogrids with higher aperture stability perform better.
- ii) Square or rectangular ribs provide better interaction with soil and aggregate than rounded or curved ribs; geogrids with thicker ribs perform better.
- iii) A minimum junction strength is needed for geogrid to effectively interlock aggregate.
- iv) Geogrids with higher tensile modulus provide better tension membrane effect and better performance in mobilizing the interaction of geogrid and aggregate.

Webster (1992)'s study showed that the geometry and strength properties of various geogrids could not be related to the traffic improvement factor from geogrid-reinforced flexible pavement field test results. However, geogrid torsional stiffness (secant aperture stability modulus) showed good correlation with the traffic improvement factor. The torsional stiffness is a measure of the geogrid in-plane stiffness (in unit of cm-kg/degree) at a torque of 20 cm-kg. Webster (1992) and Kinney and Yuan (1995) indicated that this property effectively captures the complex interaction properties such as initial tensile modulus, stiffness, confinement, and stability.

#### 4.3.2 Correlation with torsional stiffness

In this research, the aperture dimensions of geogrids are between 0.9 inches (23 mm) and 1.8 inches (45 mm) in machine and cross-machine directions. Geogrids have rectangular ribs with rib-thickness greater than or equal to 0.03 inches (0.76 mm). Properties of geogrids are shown in Table 2. The performance of test section can be correlated to the geogrid torsional stiffness, based on the results from the sections without reinforcement, with BX1100, and BX1200. The remaining test results will be used for verification of the correlation.

The torsional stiffnesses of geogrids BX1100 and BX1200 are 3.2 and 6.5 kg-cm/deg. Zero torsional stiffness is used for the cases without geogrid reinforcement. Through regression analysis, the degradation parameters  $k_1$ ,  $k_2$  and the deformation constant  $b$  can be empirically correlated to the torsional stiffness of geogrids ( $J_t$ ) as follows:

Degradation parameter of elastic modulus ratio:

$$k_1 = (a/h)^{0.72} (1.04 - 0.019J_t^{2.0}) \quad (54)$$

Degradation parameter of stress distribution angle:

$$k_2 = (a/h)^{0.85} (0.58 - 0.0074J_t^{2.1}) \quad (55)$$

Empirical coefficient used in the plastic deformation equation:

$$b = 8.3(a/h)^{0.28} - 0.82(a/h)^{0.68} J_t \quad (56)$$

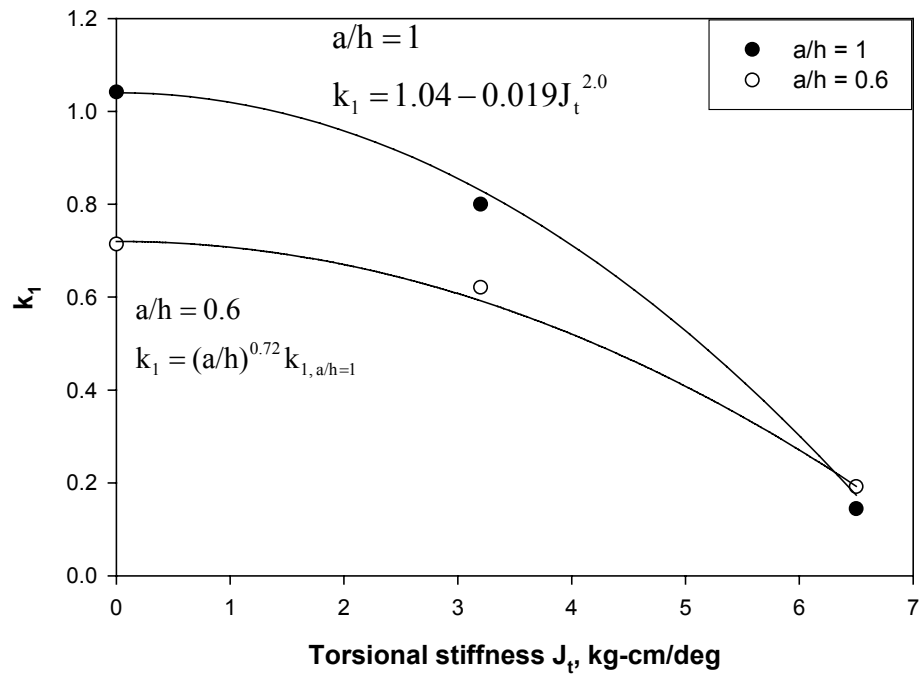


Figure 46. Influence of geogrid torsional stiffness on  $k_1$

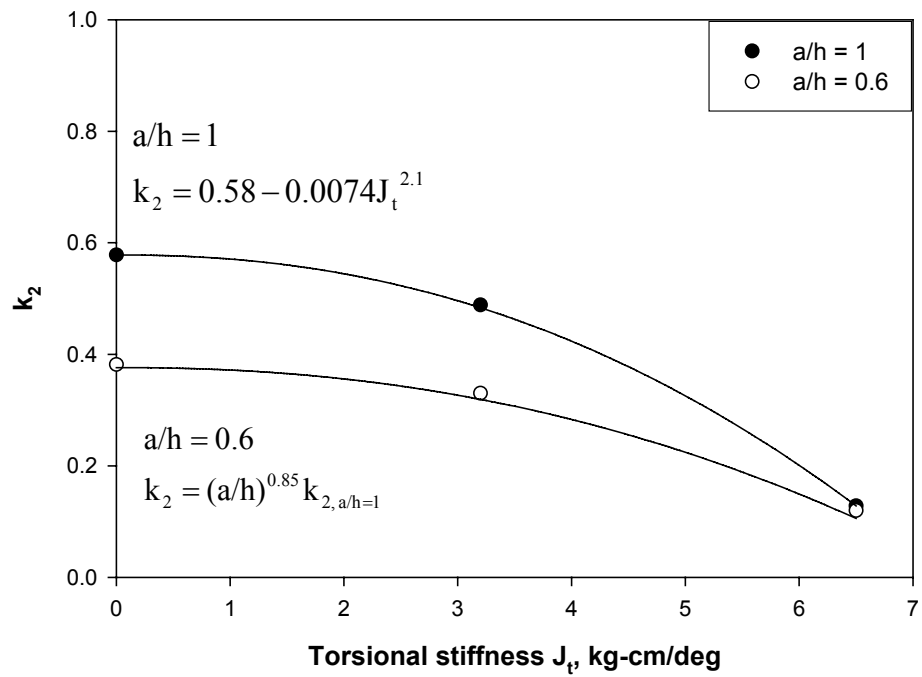


Figure 47. Influence of geogrid torsional stiffness on  $k_2$

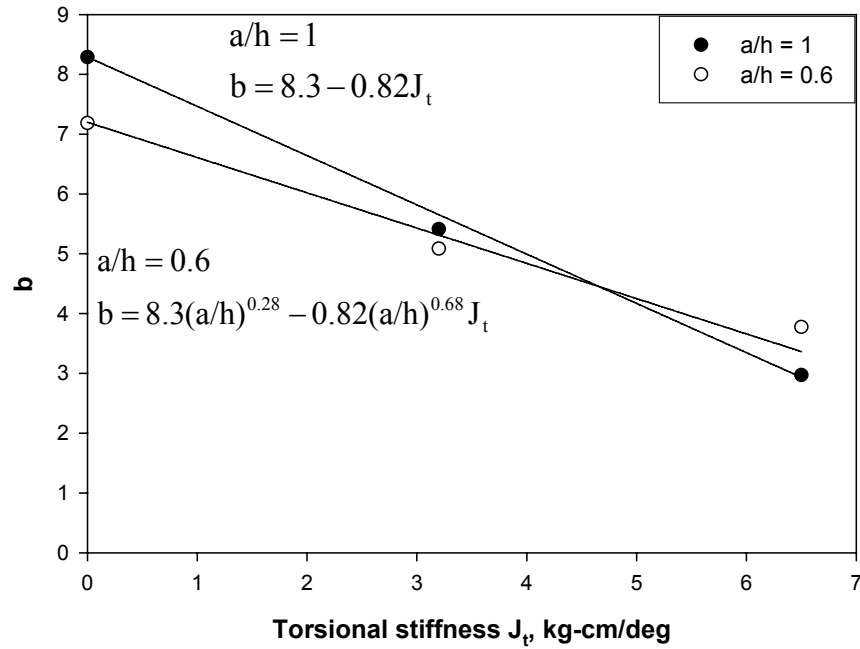


Figure 48. Influence of geogrid torsional stiffness on  $b$  value

The influence of geogrid torsional stiffness on  $k_1$ ,  $k_2$  and  $b$  are shown in Figure 46, 47 and 48 respectively. Other geogrids BX4100, BX4200, MS220 and Max30 with different torsional stiffness ( $J_t$ ) values were also used as reinforcements in 254-mm ABC tests. The test results (vertical stress on the subgrade and plastic surface deformation) and computed results using the above empirical model are shown in Table 8, with the same initial  $E_1/E_2 = 5.8$ . For the vertical stress on the subgrade, it seems the consideration of the degradation of base course provided good prediction of the vertical stress on the top of subgrade with cycle number from 10 to 1000, then vertical stress on the subgrade was slightly overestimated at the end of 8000 load cycles, except for the case with Max30 geogrid reinforcement. BX4100 and BX4200 have aperture size of 33 mm  $\times$  30 mm, which is very close to BX1100 and BX1200 (25 mm  $\times$  33 mm); while Max (44 mm  $\times$  43 mm) has a larger aperture size than BX1100 and BX1200. As shown in Table 8, the computed plastic surface deformation based on the empirical equation matched well with the measured results, even though the mode parameters were obtained only from tests without reinforcement, with BX1100, and with BX1200.

Table 8. Comparison of measured results and computed results

| Geogrid | Jt<br>kg-cm/deg. | N    | Stress on the Subgrade, kPa |          |            | Surface deformation, mm |          |            |
|---------|------------------|------|-----------------------------|----------|------------|-------------------------|----------|------------|
|         |                  |      | Measured                    | Computed | ratio(C/M) | Measured                | Computed | ratio(C/M) |
| BX4100  | 2.8              | 10   | 125                         | 117      | 0.94       | 8.9                     | 10.2     | 1.14       |
|         |                  | 100  | 148                         | 147      | 1.00       | 20.7                    | 20.5     | 0.99       |
|         |                  | 1000 | 158                         | 174      | 1.10       | 34.5                    | 34.0     | 0.99       |
|         |                  | 8000 | 163                         | 196      | 1.20       | 48.6                    | 48.9     | 1.01       |
| BX4200  | 4.8              | 10   | 129                         | 109      | 0.85       | 9.2                     | 9.0      | 0.98       |
|         |                  | 100  | 149                         | 132      | 0.89       | 18.0                    | 18.2     | 1.01       |
|         |                  | 1000 | 155                         | 154      | 0.99       | 27.6                    | 30.5     | 1.10       |
|         |                  | 8000 | 141                         | 172      | 1.22       | 40.5                    | 44.3     | 1.09       |
| Max30   | 6.9              | 10   | 120                         | 95       | 0.79       | 7.7                     | 7.8      | 1.01       |
|         |                  | 100  | 144                         | 104      | 0.73       | 19.6                    | 15.7     | 0.80       |
|         |                  | 1000 | 156                         | 114      | 0.73       | 30.5                    | 26.8     | 0.88       |
|         |                  | 8000 | 148                         | 122      | 0.83       | 41.3                    | 39.6     | 0.96       |

Note: C/M = computed value/measured value

#### 4.3.3 Generalization of model parameters

From above analysis, the degradation and surface deformation of the test sections can be largely correlated to the interaction of geogrid and aggregate or geogrid-aggregate interlock. The geogrid torsional stiffness seems so far the best index to evaluate the compatibility of the reinforcement and backfill material for better performance of reinforced unpaved road. However, for a field situation with different subgrade and aggregate properties, further research is needed to establish the relationship of the reinforced aggregate base and geogrid torsional stiffness.

The empirical relationships of  $k_2$ ,  $a$  and  $b$  with torsional stiffness of geogrids and  $a/h$  were based on the results of the laboratory cyclic loading tests with subgrade CBR = 3. For the field situations with different subgrade CBR values, both empirical equations for calculating the degradation parameter ( $k_1$ ,  $k_2$ ) and the plastic deformation related parameters need to be modified according to subgrade CBR values.

The following equation has been used to predict the cumulative subgrade plastic strain ( $\epsilon_p$ ) of subgrade under cyclic load (Ullidtz 1987; Li and Selig, 1996):

$$\epsilon_p (\%) = a \left( \frac{\sigma_d}{\sigma_{sd}} \right)^m N^b \quad (57)$$

Where,  $\sigma_d$  = applied deviator stress on the subgrade;  $\sigma_{sd}$  = undrained shear strength of subgrade; a, b and m are parameters dependent on soil type, with  $m = 1.7 - 2.4$  recommended by Li and Selig (1996) for subgrade soil from silt to fat clay.

According to Equation 57, the plastic deformation on the subgrade is proportional to  $(1/\sigma_{sd})^m$ . An average m value of 2.0 can be used in the prediction of subgrade deformation. Assuming the total surface deformation to be proportional to the deformation on the subgrade, and assuming the undrained shear strength is linearly correlated to subgrade CBR value, the total plastic deformation for subgrade with CBR value other than 3 can be approximated by using the following equation:

$$S_N(\text{mm}) = \left( \frac{3}{\text{CBR}_{sb}} \right)^m S_{N, \text{CBR}_{sb}=3} = \left( \frac{3}{\text{CBR}_{sb}} \right)^2 [s_1 + b \log N + c(\log N)^2] \quad (58)$$

$$s_1 = 3\text{mm}$$

$$b = 8.3(a/h)^{0.28} - 0.82(a/h)^{0.68} J_t$$

$$c = c_{a/h=1} (a/h)^{0.67} = 2.23(a/h)^{0.67}$$

For the degradation parameter  $k_2$ , there are no data available to modify for subgrade soils with different CBR values. Using modification coefficients similar to the modification coefficient of plastic deformation  $(3/\text{CBR}_{sb})^m$ , the degradation related parameters ( $k_1$  and  $k_2$ ) can be approximately evaluated as follows:

$$k_1 = \left( \frac{3}{\text{CBR}_{sb}} \right)^{n_1} k_{1, \text{CBR}_{sb}=3} \quad (59)$$

$$k_{1, \text{CBR}_{sb}=3} = (a/h)^{0.72} (1.04 - 0.019 J_t^{2.0})$$

$$k_2 = \left( \frac{3}{\text{CBR}_{sb}} \right)^{n_2} k_{2, \text{CBR}_{sb}=3} \quad (60)$$

$$k_{2, \text{CBR}_{sb}=3} = (a/h)^{0.85} (0.58 - 0.0074 J_t^{2.1})$$

Here  $n_1$  and  $n_2$  are positive parameters, representing the effects of subgrade CBR values on the degradation parameters  $k_1$  and  $k_2$ . Further testing data on subgrade different CBR values are needed to calibrate  $n_1$  and  $n_2$ . Later, back-figured values for  $n_1$  and  $n_2$  will be presented in field verification analysis.

#### **4. 4 Summary**

Unpaved structures suffer degradation and plastic deformation accumulation under cyclic load. Elastic layer method has been used to back-analyze the performance of unpaved test sections. The degradation was expressed as decay of elastic modulus ratio ( $E_1/E_2$ ) and stress attenuation ability of the base layer with number of load cycles. The improvement of structure performance due to geogrids was investigated, and the interaction between base course aggregate and geogrid (interlock effect) is believed to be a primary indicator of geogrid performance. With torsional stiffness used as index for reinforcement performance, an empirical method based on the test results of the sections without reinforcement, with geogrid BX1100, and BX1200 has been developed to evaluate the degradation of base layer and the permanent surface deformation. The method was verified by other tests with different geogrids (BX4100, BX4200, MS220 and Max30). As this method was based on the testing results from the subgrade with CBR of 3, proposed modifications were recommended for other situations with different subgrade CBR values.

## **Chapter 5 FEM ANALYSIS AND MODELING**

### **5.1 Introduction**

The behavior of reinforced earth structures depends on the properties of geosynthetics and soil layers as well as the interface interaction between the soil layers and the reinforcement. For geosynthetic-reinforced unpaved roads, previous laboratory and field studies have provided data such as lateral confinement of base course, bearing capacity improvement of subgrade, and membrane effect of geosynthetics that aided in describing the mechanisms of reinforcement. It was observed that the use of geogrids, with aperture sizes properly configured for the intended backfill material sizes, offers an improved lateral confinement effect (due to interlocking). However, additional information is required to further understand the complex behavior of such composite system especially with respect to the mode of reinforcement contribution as a function of deformation level.

Previous analysis and design methods of geosynthetic-reinforced unpaved structures incorporate one or more reinforcement mechanisms in design, and rely on simplified stress distribution concepts within the layers of the unpaved structure. Such models were presented by Giroud and Noiray (1981) and Miligan et al. (1989). These models do not typically consider degradation of base course layer with the repeated traffic loading.

Unpaved structures usually experience relatively large deformation under traffic load, while base course and subgrade showing significant plastic behaviors. Both the stiffness of geosynthetics and the interaction between geosynthetics and soil layers play a role in the response behavior of reinforced unpaved road. In traditional numerical analysis methods, linear elastic material behavior and completely rough interface are usually assumed to model the soils and the interface, respectively.

Work conducted in this part of the research aims at numerically analyzing the performance of reinforced aggregate base course placed over soft subgrade. An axis-symmetric Finite Element Method (FEM) analysis using the computer program Abaqus is

conducted to discern the stress and strain magnitude of geogrid-reinforced layered system under varying parameters including reinforcement stiffness, interface properties, and thickness of the aggregate base course (ABC) layer. The numerical results are compared with the experimental data from geogrid-reinforced and unreinforced sections composed of aggregate base course overlying soft subgrade. The stresses, strains, and deflections of the modeled sections and the effects of reinforcement stiffness on deformation and vertical stress distribution are numerically evaluated and presented.

## **5.2 Material and interface modeling**

### **5.2.1 Elasto-plastic model for base and subgrade materials**

Nonlinear constitutive models are needed to simulate behaviors of granular materials under large deformation. These models are essentially stress-dependent plasticity models. In the geotechnical engineering field, granular materials exhibit significantly different yield behavior in tension and compression. For unpaved structure with aggregate over subgrade, traditional linear elastic method or nonlinear elastic methods can not simulate yielding due to tensile stress generated at the bottom of the aggregate layer. Wathugala et al. (1996) performed a numerical simulation of geosynthetic-reinforced flexible pavements. They concluded that elastic analyses predict unrealistic tensile stresses in the base layer and therefore may show little improvement in pavement performance due to reinforcement. In contrast, elasto-plastic analyses predict compressive only stresses in the base layer and improvement in pavement behaviors due to geosynthetic reinforcement becomes apparent.

An extended Drucker-Prager model (Drucker and Prager, 1952) with hyperbolic yield criterion is used in this research. The model is available in Abaqus/Standard. The hyperbolic yield criterion is a continuous combination of maximum tensile stress condition of Rankine (tensile cut-off) and the linear Drucker-Prager condition at high confining stress, as shown in Figure 49. The yielding criterion is expressed as (Hibbitt, Karlsson & Sorensen, Inc., 2001):

$$F = \sqrt{(d'|_0 - p_{t|0} \tan \beta)^2 + q^2} - p \tan \beta - d' = 0 \quad (61)$$

Where,  $p$  and  $q$  are the two stress invariants,  $p_{t|0}$  is the initial hydrostatic tension strength of the material,  $d'|_0$  is the initial value of  $d'$  (a hardening parameter related to the initial yielding stress), and  $\beta$  is the slope of the yield surface in the  $p$ - $q$  stress plane.

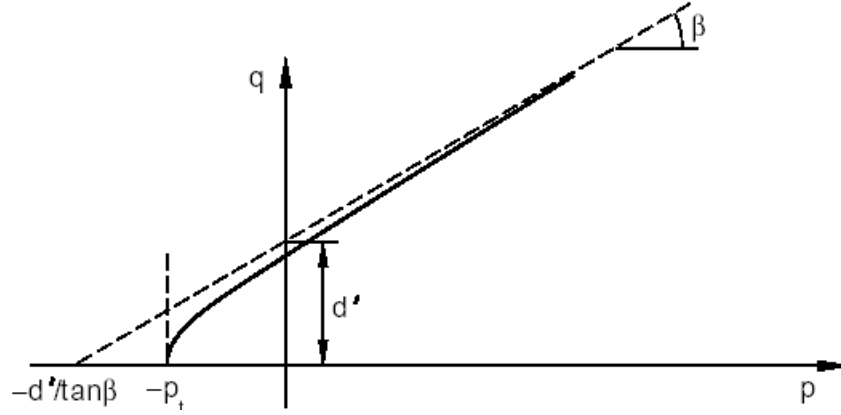


Figure 49. Hyperbolic yield criteria of extended Drucker-Prager models  
(Hibbitt, Karlsson & Sorensen, Inc., 2001)

In a triaxial compression test,  $p = 1/3(\sigma_1 + 2\sigma_3)$ ,  $q = \sigma_1 - \sigma_3$ . The extended Drucker-Prager model parameters ( $\beta$  and initial compression yielding stress  $\sigma_c^0$ ) can be derived from Mohr-Coulomb model parameters ( $\phi$  and  $c$ ). For the material with low friction angle (less than  $22^\circ$ ), parameters can be determined as follows (Hibbitt, Karlsson & Sorensen, Inc., 2001):

$$\beta = \tan^{-1} \left( \frac{6 \sin \phi}{3 - \sin \phi} \right) \quad (62)$$

$$\sigma_c^0 = 2c \frac{\cos \phi}{1 - \sin \phi} \quad (63)$$

For the material with high friction angle, the above equations may provide a poor match with the Mohr-Coulomb parameters, and  $\beta$  can be approximated as equal to  $\phi$  value.

The hyperbolic model provides a nonlinear relationship between deviatoric and mean stress at low confining pressures, which may provide a better match for the triaxial experimental data. Isotropic hardening and unassociated flow rules are also used. Flow potential ( $G$ ) is chosen in these models as a hyperbolic function as follows (Hibbitt, Karlsson & Sorensen, Inc., 2001):

$$G = \sqrt{(\varepsilon \sigma_{|_0} \tan \psi)^2 + q^2} - p \tan \psi \quad (64)$$

Where,  $\psi$  is the dilation angle measured in the  $p$ - $q$  plane at high confining pressure;  $\sigma_{|_0}$  is the initial equivalent yield stress; and  $\varepsilon$  is a parameter, referred to as the eccentricity, that defines the rate at which the function approaches the asymptote (the flow potential approaches to a straight line as the eccentricity approaches to zero). The function asymptotically approaches the linear Drucker-Prager flow potential at high confining stress and intersects the hydrostatic pressure axis at  $90^\circ$ .

### 5.2.2 Soil-geosynthetic interface

The soil-reinforcement interface friction (or shear resistance interaction) properties are one of the basic factors influencing the deformation and strength of reinforced soil matrix. Results from the direct shear box test do not represent the behavior of geosynthetic reinforcements subjected to tensile load, and therefore a commonly adopted method is the pullout test. Pullout resistance is expressed by using an apparent interface friction coefficient  $\mu^*$  (same as the interaction factor defined in Equation 15) or interface friction angle  $\delta$ , which describes the interface shear resistance mobilized between geosynthetic and the backfill soil.

$$\mu^* = \tan \delta = \frac{\tau_{av}}{\sigma'_v} \quad (65)$$

Where,  $\tau_{av}$  is average shear stress acting on the specimen, and  $\sigma'_v$  is the effective vertical stress.

As polymeric geosynthetics materials are relatively extensible, the pullout resistance is mobilized through progressive shear strain between soil and geosynthetic. Lopes and Lopes (1999) found that the relative dimensions of soil particles, geogrid apertures, and the thickness of bearing members or transverse ribs influenced the soil-geogrid interface shear strength and mobilization of interface shear stress. Lopes and Lopes (1999) also indicated that an increase in soil-geosynthetic shear resistance was observed when the soil contained a significant percentage of particle sizes slightly greater than the thickness of the geogrid bearing member but less than the geogrid aperture size. Perkins and Cuelho (1999) studied base course aggregate and geosynthetic interface strength based on pullout tests. The test geosynthetics included a biaxial geogrid (BX1100) and a woven geotextile. Under confining pressures of 5 to 35 kPa, peak friction angles for the geogrid were 54 to 58 degrees in the machine and cross-machine directions, respectively. The peak friction angles for the geotextile from pullout tests ranged from 37 to 53 degrees, with larger values in the machine directions or under larger confining pressure.

A Coulomb friction model is used in this research to simulate the shear resistance interaction between geogrid and base course aggregate. The friction model usually contains two material properties, a friction coefficient ( $\mu^*$ ), and an elastic slip ( $E_{slip}$ ) value. The elastic slip is the limit of elastic shear displacement before the critical interface shear stress is reached. The elastic slip represents the elastic shear stiffness of the interface, as was presented by Perkins (2001) shown in Figure 50. Shearing resistance ( $\tau$ ) is a function of the amount of relative shear displacement ( $\Delta$ ) between the aggregate layer and the geosynthetic. The initial part of the  $\tau$  vs.  $\Delta$  curve is elastic, with the slope of the curve (or elastic shear stiffness) dictated by specification of  $E_{slip}$ . Ultimate shearing resistance is reached according the relationship between  $\tau$  and  $\sigma$  (normal stress), which is specified by the interface friction coefficient ( $\mu$  or  $\mu^*$ ). From Figure 50, the shear stiffness of the interface increases as normal stress on the interface increases.

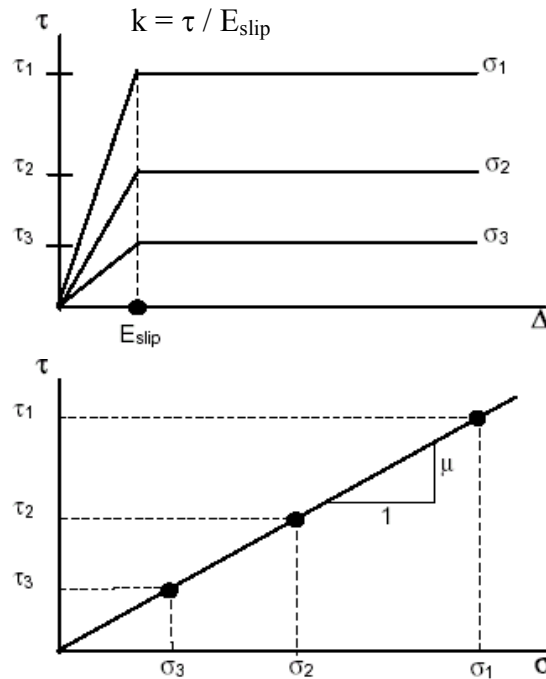


Figure 50. Geosynthetic/aggregate interaction model (Perkins, 2001)

Perkins (2001) numerically studied geosynthetic/aggregate interaction by simulating pullout tests, and cyclic plate tests on flexible pavement. Data showed that the input elastic slip values affected the performance of geosynthetic/aggregate interaction, especially under small strain when the ultimate interface shear strength was reached. The elastic slip value of 0.001 m provided best prediction matching with the testing results of Perkins and Cuelho (1999). Perkins's study also showed that the input elastic slip value affected the performance of modeled reinforced flexible pavements, when the relative shear displacement was less than the elastic slip and the interface shear strength was not reached.

### 5.3 FEM modeling of unpaved structure

A static finite element analysis is performed to evaluate stresses and strains of the laboratory plate load tests. As only surface deformation on the base layer, and vertical stress on the subgrade were measured during the laboratory load testing, the numerical analysis provides insight information about the intricate stress and strain of geogrid-

reinforced sections. The results will be used to assist in understanding of the interaction of aggregate-reinforcement-subgrade, and in evaluating the stress distribution within the modeled section layers.

### 5.3.1 FEM mesh and load conditions

A typical FEM mesh used in the analysis is shown in Figure 51. The axis-symmetric mesh has a radius of 0.75m and total depth of 0.90 m. It includes 84 elements for the base layer, 14 element for the geogrid reinforcement at the interface, and 140 elements for the subgrade. A load of 40 kN (pressure = 550 kPa), simulating a single wheel load, is applied to a circular area with a radius of 0.152 m, to simulate magnitude of load during the testing program. Solid elements are selected for the base course and the subgrade. The reinforcements between the base course and the subgrade is simulated by 14 membrane element with thickness of 0.003 m. The membrane elements have a tensile modulus transmitting tensile force only.

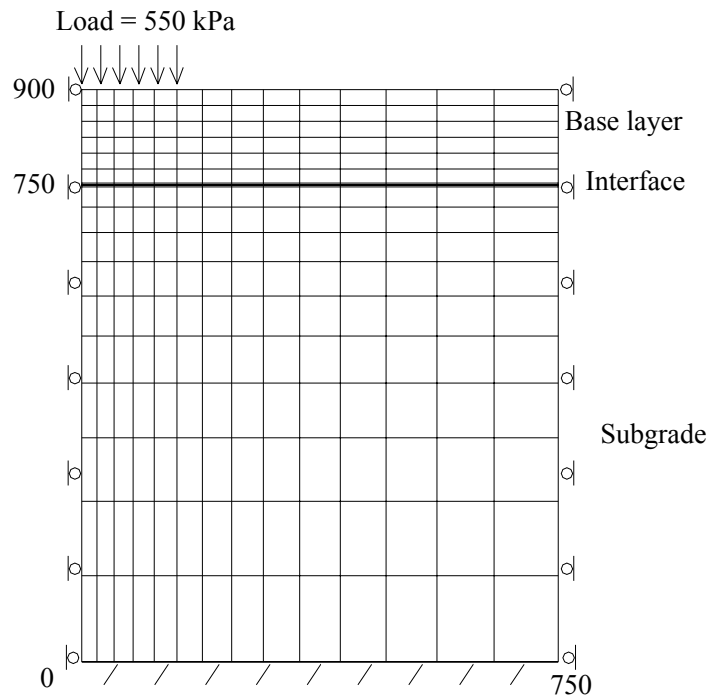


Figure 51. Axi-symmetric mesh for numerical analysis

### 5.3.2 Representation of material properties

Three types of materials are involved in modeling the composite system. Solid elements are selected for the base course aggregate and the subgrade. The reinforcements between the base layer and the subgrade are simulated by membrane elements with thickness of 0.003 m. The membrane elements have load carrying capacity in tension but no resistance to bending. Isotropic elasto-plastic constitutive models are used to simulate the base course, subgrade and geogrid. The extended Drucker-Prager model with hyperbolic yield criteria is used to model the base course aggregate and the subgrade. Geogrid is simulated as a normal elasto-plastic material. The model parameters are list in Table 9. Three cases of ABC thickness and four cases of geogrid moduli are used in FEM analysis.

Table 9. Parameters of materials in the FEM analysis

| Materials | Element<br>(type)  | Model and parameters <sup>1</sup>                                      | Yielding stress<br>(kPa) | Thickness<br>(M)     | E<br>(MPa)                        | $\nu$ |
|-----------|--------------------|------------------------------------------------------------------------|--------------------------|----------------------|-----------------------------------|-------|
| ABC       | Solid<br>(CAX4R)   | Drucker-Prager<br>$\beta=40^\circ$ , $p_{t 0}=20$ kPa, $\psi=10^\circ$ | 150                      | 0.15<br>0.20<br>0.25 | $E_1^2 = 50$                      | 0.35  |
| Subgrade  | Solid<br>(CAX4R)   | Drucker-Prager<br>$\beta=10^\circ$ , $p_{t 0}=10$ kPa, $\psi=0^\circ$  | 43.6                     | 0.75                 | $E_2^2 = 10$                      | 0.42  |
| Geogrid   | Membrane<br>(MAX1) | Elasto-plastic                                                         | 3000                     | 0.003                | $E_g^3 = 50$<br>100<br>200<br>400 | 0.35  |

Note:

1. Corresponding Mohr-coulomb parameters:  $\phi = 40^\circ$ ,  $c = 35$  kPa for ABC,  $\phi = 5^\circ$ ,  $c = 20$  kPa for subgrade.
2. Elastic moduli of subgrade and base layer are selected on the basis on the static plate load results of the testing program.

3.  $E_g = J/t$  or tensile stiffness/Thickness, tensile modulus  $E_0$  of 50, 100, 200 and 400 MPa represent tensile stiffness of 150, 300, 600, 1200 kN/m respectively. Testing geogrids BX1100 and BX1200 have tensile stiffness of 221-360 kN/m and 481-653 kN/m.

### 5.3.3 Interface properties

There are two interfaces for the geogrid reinforcement. One is between ABC and geogrid, and another is between geogrid and subgrade. In the case of unreinforced sections, only one interface (between ABC and subgrade) was used. Because the geogrid doesn't totally separate the ABC from subgrade, the interface between geogrid and subgrade is assumed to have similar interface properties as the interface of ABC and subgrade. In normal direction, the interface contact is assumed to be "hard contact" and no separation is allowed; in tangential direction, Coulomb friction model is used to simulate the shear resistance interaction.

Three friction coefficients ( $\mu^*$ ) of 0.5, 1.0 and 1.5, corresponding to interface friction angles of approximately  $27^\circ$ ,  $45^\circ$ , and  $56^\circ$  and covering the range of angles reported by Perkins and Cuelho (1999) were used for the interface between the geogrid and the ABC layer. An elastic slip of 0.001 m was used to represent the magnitude of elastic shear displacement allowed along the interface of geogrid and aggregate. Lagrange multiplier method and a friction coefficient of 0.25 were used for the interface of the ABC and the subgrade (unreinforced case), as well as the interface of the geogrid and the subgrade (reinforced case). With Lagrange multiplier method, there was no relative slip allowed until the critical shear stress was reached.

While normally the interface between subgrade and base layer has been modeled using rough interface, with the assumption of they are well bonded. A friction model was used in this study to make the convergence of the solution more rapid. On the other hand, the interface friction coefficient 0.25 was selected to make sure the interface shear strength was a little bit higher than the shear strength of subgrade ( $\phi = 5^\circ$ ,  $c = 20$  kPa). It only had negligible affect on the accuracy of the solution as the shear resistance near the interface still controlled by the shear strength of subgrade.

## 5.4 FEM analysis of unpaved structure

A “base” case, for the sake of the analysis, is defined as a section having ABC thickness of 0.15 m, ABC modulus of 50 MPa, subgrade modulus of 10 MPa, geogrid modulus of 100 MPa (stiffness of 300 kN/m), and friction coefficient of 1.0. The stress distribution underneath the center of the loaded area, as well as interface shear resistance interaction effect, are evaluated for the base case and corresponding unreinforced case. Other key performance data, such as surface deformation, vertical stress on the subgrade, tensile stress of geogrids, and vertical strain underneath the center of the loaded area, are also evaluated with varied ABC thickness, geogrid tensile stiffness and interface friction coefficients between ABC and geogrid.

Before the FEM analysis, the affect of element number on the FEM analysis results was checked by comparing the FEM analysis results from the two cases with different element number. As shown in Table 10, a “check” case had four times ABC elements, four times subgrade elements and two time geogrid membrane elements, as many as the “base” course. FEM analysis predicted almost the same maximum surface deformation and maximum subgrade stress for both cases.

Table 10. Element size effect on the FEM analysis results

| Case         | Element Number |          |         | Maximum surface deformation (m) | Maximum subgrade stress (kPa) |
|--------------|----------------|----------|---------|---------------------------------|-------------------------------|
|              | ABC            | Subgrade | Geogrid |                                 |                               |
| “base” case  | 84             | 140      | 14      | 0.011232                        | 220.428                       |
| “check” case | 336            | 560      | 28      | 0.011248                        | 220.470                       |
| Base /check  |                |          |         | 0.99857                         | 0.99981                       |

### 5.4.1 Stress distribution underneath the center of loading area

The stress distribution underneath the center of the loaded area was evaluated using elasto-plastic analysis (Abaqus) for the “base” case and the corresponding unreinforced case. Homogenous elastic analysis (Boussinesq’s solution) and elastic layer

analysis using Kenlayer program (Huang, 1993) were also performed, with the same elastic modulus and Poisson's ratio used in the Abaqus analysis.

Vertical stress distribution underneath the center of the loaded area is shown in Figure 52. Because the shear resistance interfaces were included in Abaqus analysis, the vertical stress is not smoothly continuous from ABC layer to subgrade layer. Given the model geometry of ABC thickness and assuming no degradation of ABC, the results indicate that the geogrid reinforcements slightly decrease the vertical stress transferred to subgrade, while changing the stress distribution in the ABC layer. As geogrids provide tensile resistance and limit tensile yielding at the bottom of ABC layers, reinforced ABC layers become stiffer and higher vertical stresses develop in the ABC layers. Compared with Abaqus results, Boussinesq's solution appears to overestimate the vertical stress in ABC layer as well as the vertical stress transferred to subgrade layer. Elastic layer prediction (Kenlayer), appears to underestimate the vertical stress transferred to subgrade layer. While the elasto-plastic Abaqus results are expected to be between Boussinesq's solution and Kenlayer results, data in Figure 52 shows the vertical stress in subgrade layer from Abaqus is higher than the results from Boussinesq's solution and Kenlayer, for the depth larger than 0.3. The reason is that Abaqus FEM analysis deals with limited depth while Boussinesq's solution and Kenlayer both simulate half space problem.

Horizontal stress distribution underneath the center of the loaded area is shown in Figure 53. Due to the shear-resistance interfaces, the horizontal stress of Abaqus results is also not continuous from the ABC layer to the subgrade layer. The geogrid reinforcement yields a small decrease of the horizontal stress transferred to subgrade, and a considerable increase of horizontal stress near the bottom of ABC layer. It indicates that shear-resistance interaction between geogrid and ABC provides lateral confinement at the bottom of base layer and improves the performance of unpaved structures. Compared with Abaqus results, Boussinesq's solution predicts higher horizontal stress in the base layer and lower horizontal stress in the subgrade layer. The elastic layer method predicts high tensile stress in the base layer, which is not realistic for the base course materials. The elasto-plastic analysis using Abaqus, which predicts only compressive stress in the base, layer and subgrade, appears to predict more realistic horizontal stress distribution.

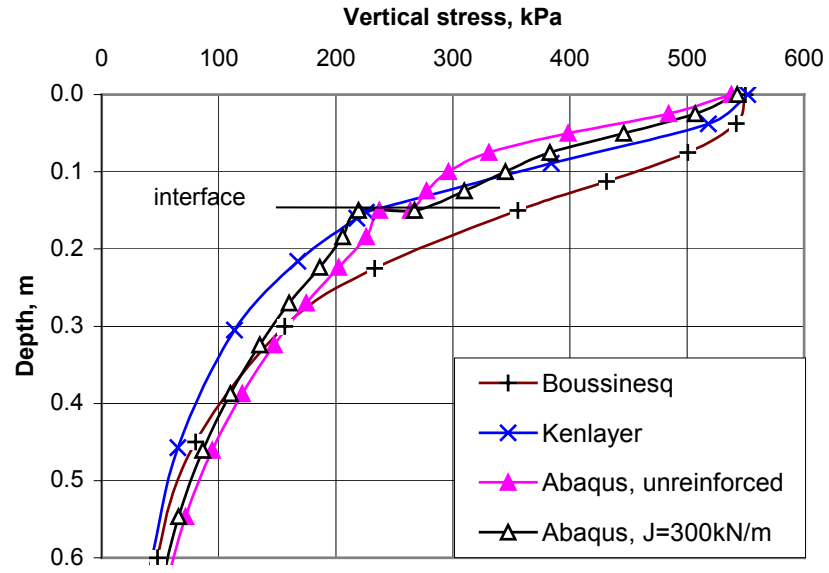


Figure 52. Vertical stress distribution underneath the center of the loaded area

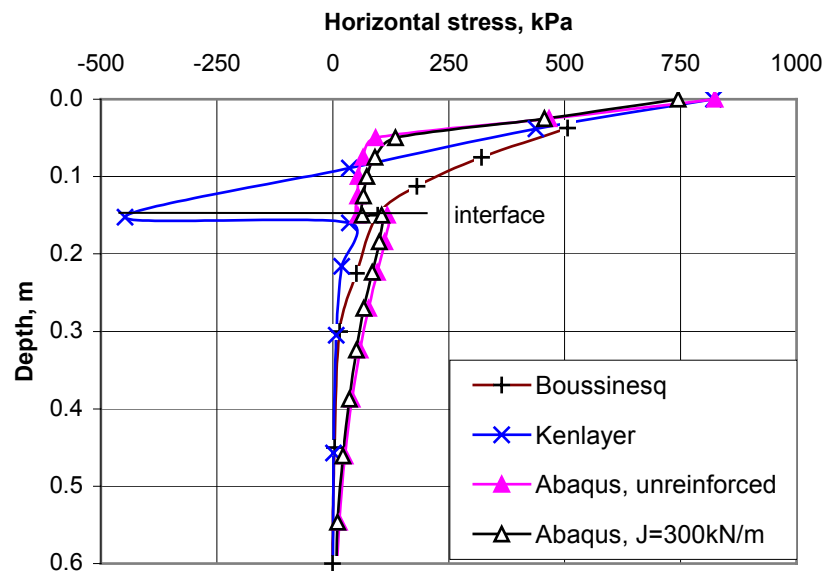


Figure 53. Horizontal stress distribution underneath the center of the loaded area

### 5.4.2 Shear-resistance interaction at the interface

The shear-resistance interaction at the interface was studied for the “base” case and the corresponding unreinforced case.

Figure 54 shows the mean stress (defined as the average of the three principal stresses), along the bottom of the base layer. The results show that lateral confinement due to the geogrid reinforcement results in approximately 10% increase in mean stress at the bottom of the base aggregate, in the area with distance from centerline less than 0.16m. As the modulus of the base course aggregate is stress-related, the increase of the mean stress implies the improvement of the base aggregate mechanic properties.

Figure 55 shows the interface shear stress at the bottom of base layer versus lateral distance from the centerline. It indicates that the interface shear stress in the “base” reinforced case is higher than that in the corresponding unreinforced case, both with the maximum interface shear stress at the position 0.15m away from centerline. Figure 56 shows the relative displacement between the base layer and the geogrid. It indicates that the interface shear strength is reached within the zone 0.13 to 0.22 from centerline, where the value of  $E_{slip} = 0.001$  m is exceeded.

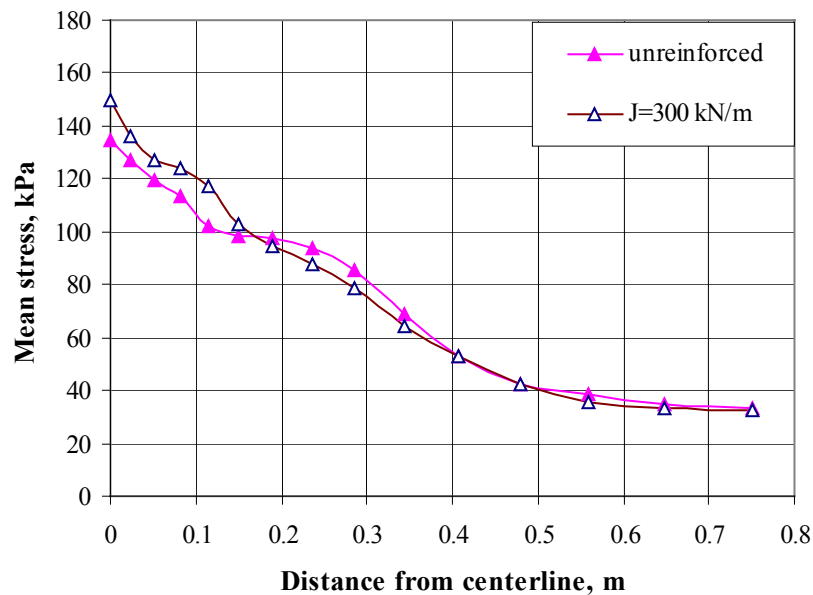


Figure 54. Mean stress at the bottom of the base layer

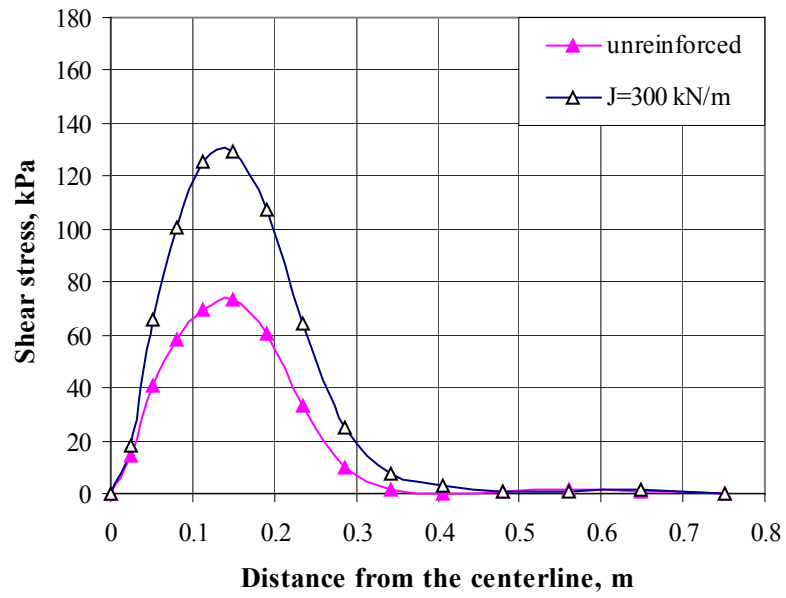


Figure 55. Interface shear stress at the bottom of the base layer

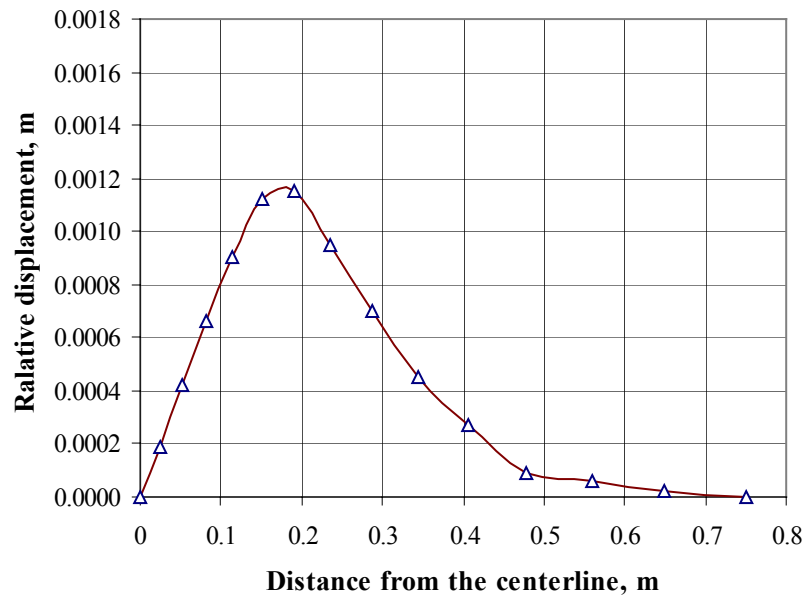


Figure 56. Relative displacement between the base aggregate and the geogrid

### 5.4.3 Surface deformation on the base layer

Surface deformation on the base layers is shown in Figure 57 and Figure 58, with consideration of ABC thickness, geogrid tensile modulus, and interface property of geogrid- ABC. For reinforced case (geogrid with tensile modulus of 100 MPa, or tensile stiffness 300 kN/m) versus unreinforced case, Figure 57 shows that maximum surface deformation of base layer (at the center of load area) decreased as the ABC thickness increased from 0.15 m to 0.25 m. A reduction of the maximum deformation of approximately 20% is obtained for 0.15 m-ABC, compared to the unreinforced case, with such reduction decreasing to 13% of 0.25 m-ABC thickness.

Figure 58 shows the effect of the tensile modulus and the interface properties on surface deformation. As geogrid stiffness increased from 150 kN/m to 1200 kN/m, the reduction of maximum surface deformation increased from approximately 15% to 30%. The rate of decrease in surface deformation decreased as the geogrid modulus was increased. Besides the ABC thickness and the geogrid tensile modulus, the surface deformation was affected by the interface friction coefficient. For example, in case of geogrid stiffness of 300 kN/m, and as the friction coefficient was increased from 0.5 to 1.5, the maximum surface deformation was reduced by 17% to 20%, as shown in Figure 58. However, on the basis desired deformation criterion, the difference in  $\mu^*$  may lead to significantly different results in terms of the needed reinforcement stiffness to maintain such deformation criterion. For example, in order to limit maximum deformation to 0.011 m as compared to 0.014m of unreinforced case, the J value required for geosynthetic reinforcement is 300 kN/m for  $\mu^* = 1.5$ , 400 kN/m for  $\mu^* = 1$ , and 950 kN/m for  $\mu^* = 0.5$ .

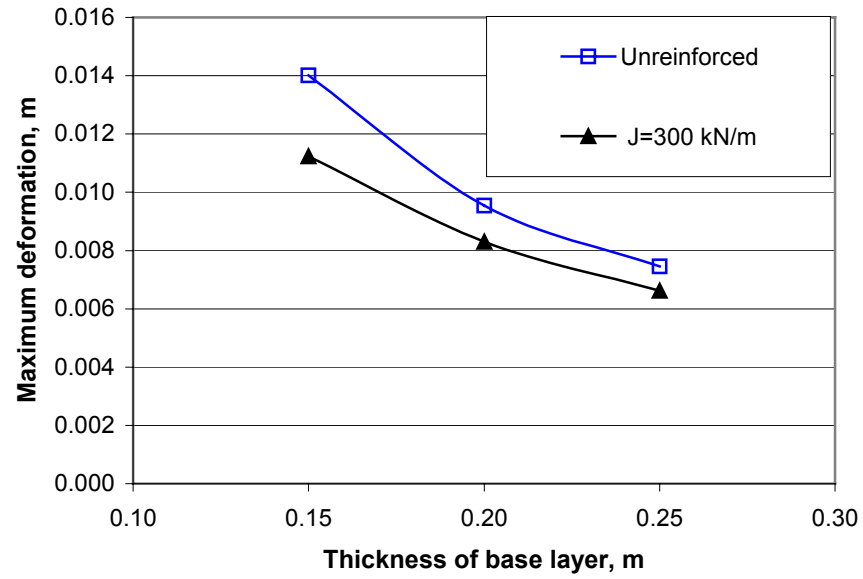


Figure 57. Influence of ABC thickness on surface deformation

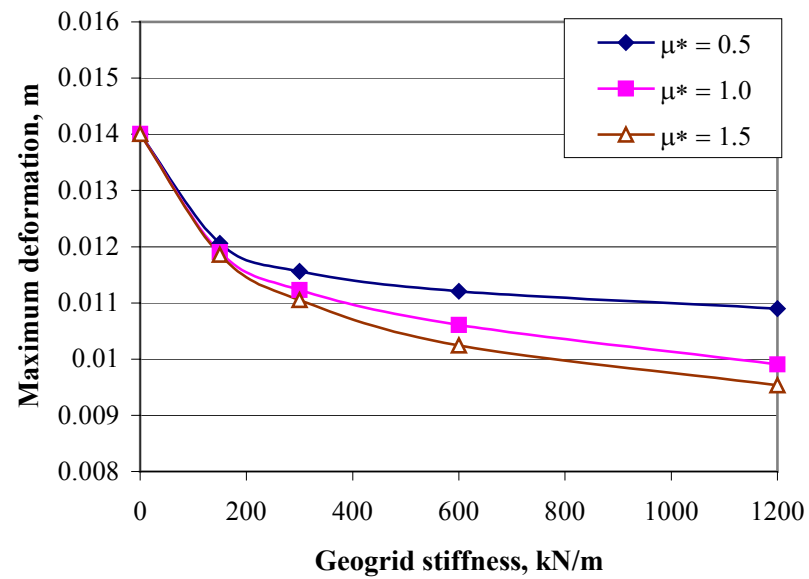


Figure 58. Influence of geogrid modulus and interface property on surface deformation

#### 5.4.4 Vertical stress on the subgrade

Vertical stress on the subgrade layer under the center of the loaded area is shown as a function of ABC thickness in Figure 59. The result shows that the vertical stress on the subgrade is mainly related to the thickness of base layer. The maximum vertical stresses for unreinforced /reinforced cases ( $J = 300 \text{ kN/m}$ ) are 237 / 220 kPa for 0.15 m ABC cases, 182 / 171 kPa for 0.20 m ABC cases, 146 / 139 kPa for 0.25 m ABC cases. The reduction of vertical stress is 7.2 %, 6.0 % and 4.8 % for the ABC thickness of 0.15 m, 0.20 m and 0.25 m, receptively. The improvement of vertical stress on the subgrade due to geogrid is relatively small under static loading condition, which coincides with the test results at the beginning of cyclic load tests. As shown in Figure 60, the increase in geogrid stiffness and interface friction coefficient leads to slight decrease in vertical stress on the subgrade. The geogrid with higher stiffness ( $J = 600 \text{ kN/m}$ ) provided a slightly improvement as the geogrid with lower stiffness ( $J = 300 \text{ kN/m}$ ). The higher friction coefficient provides better reduction in vertical stress. Higher stiffness of geogrid or better geogrid-ABC interaction results in less deformation. Analysis in this case indicated that, under static condition with low deformation, the benefit of higher modulus geogrid or better interface property is not significant.

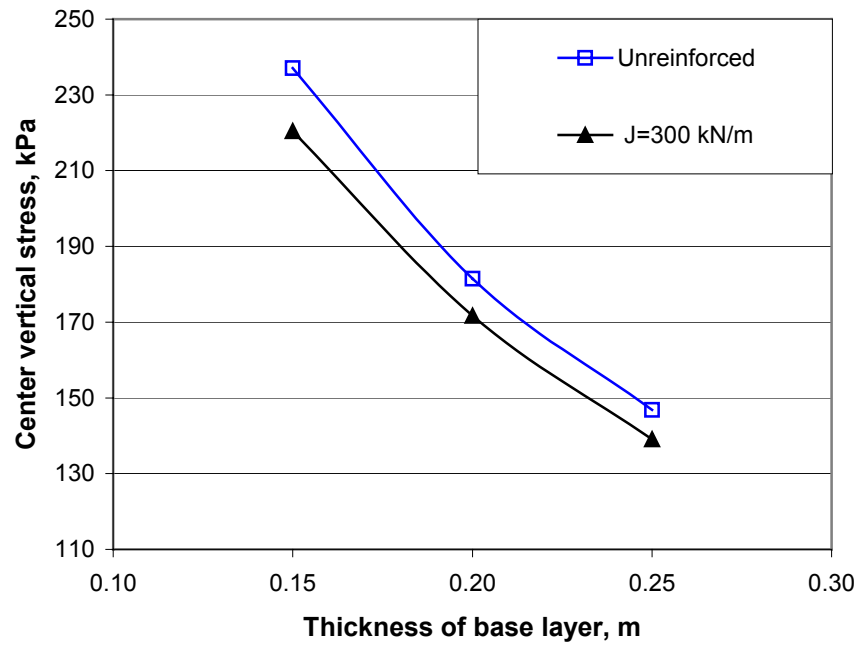


Figure 59. Influence of ABC thickness on vertical stress on the subgrade

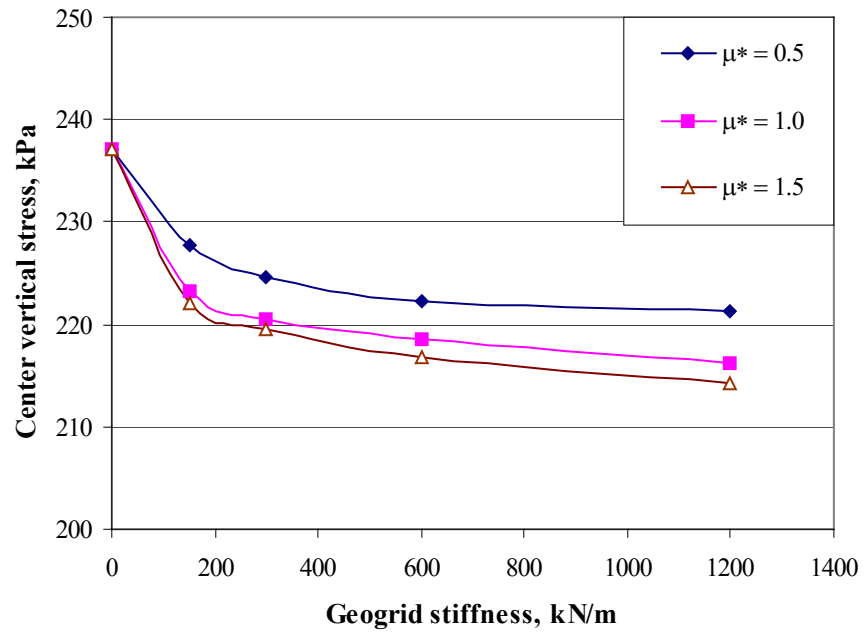


Figure 60. Influence of geogrid modulus and interface property on vertical stress on the subgrade

#### **5.4.5 Tensile stress of geogrids**

Tensile stress of geogrids is shown in Figure 61 with different of ABC thickness. The results from FEM analysis indicate that the tension stress is not uniform along the geogrid length. There is a zone of outward tension with the radius of 0.21 – 0.29 m, and a zone of small inward tension outside this radius. The zone of small inward tension is actually a zone of small compression, which indicates the lack of a tensioned-membrane effect at such area but a realization of a lateral confinement effect. Experimental results from other researchers (Haas et al., 1988; Miura et al., 1990; Perkins, 1999) showed a similar distribution presented with tensile strain of geosynthetic. As ABC thickness increases, the maximum tensile stress mobilized decreases due to the decrease of deformation. Under static load, the maximum tensile force mobilized is 2.5 kN/m for 0.25 m ABC, 4.1 kN/m for 0.20 m ABC, and 5.6 kN/m for 0.15 m ABC. As the geogrid yielding force used in the analysis is 9 kN/m, the tensile strength of geogrid is not fully mobilized in each case. The maximum tensile stress of geogrids is shown in Figure 62 with different of geogrid stiffness and interface property of geogrid-base layer. As geogrid modulus is increased, a quick buildup of tensile stress is detected, while the corresponding deformation decreases. Interface property also affects tensile strength mobilization. More tensile stress can be generated with larger friction.

#### **5.4.6 Vertical strain underneath the center of loading area**

Vertical strain along the centerline is shown in Figure 63 with consideration of ABC thickness. The results show there is a large vertical strain at the bottom of base layer, which is related to outward lateral spreading of ABC, or yielding of ABC under low confining stress (due to low tensile strength). As geogrid reinforcement is included, the tensile strains at the bottom of ABC are reduced significantly, with the reduction of 33 % to 45 %. In addition, results also indicate that the vertical strain in the subgrade is also limited by the geogrid reinforcements. As ABC thickness increases, the corresponding vertical strain decreases. Generally, the geogrid reinforcement with higher modulus and better interface property shows better performance on limiting the vertical strain at the bottom of base layer, as shown in Figure 64.

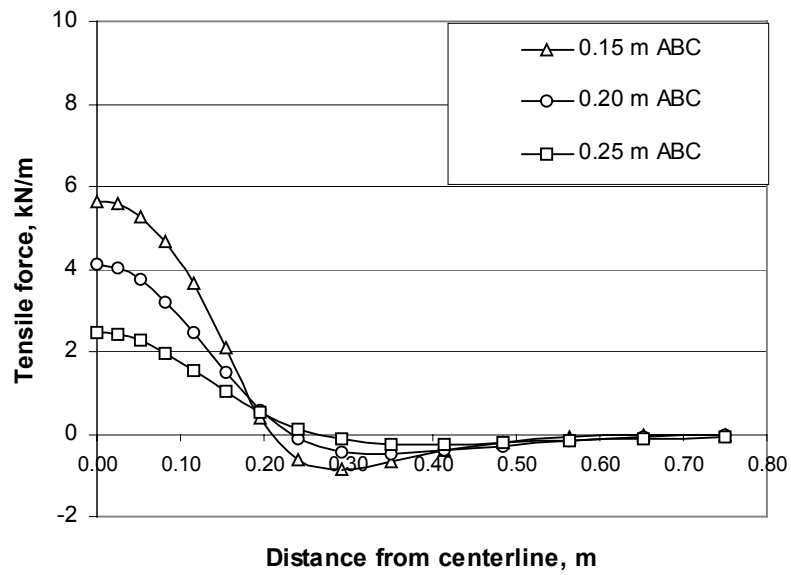


Figure 61. Influence of ABC thickness on mobilized tensile force of geogrids

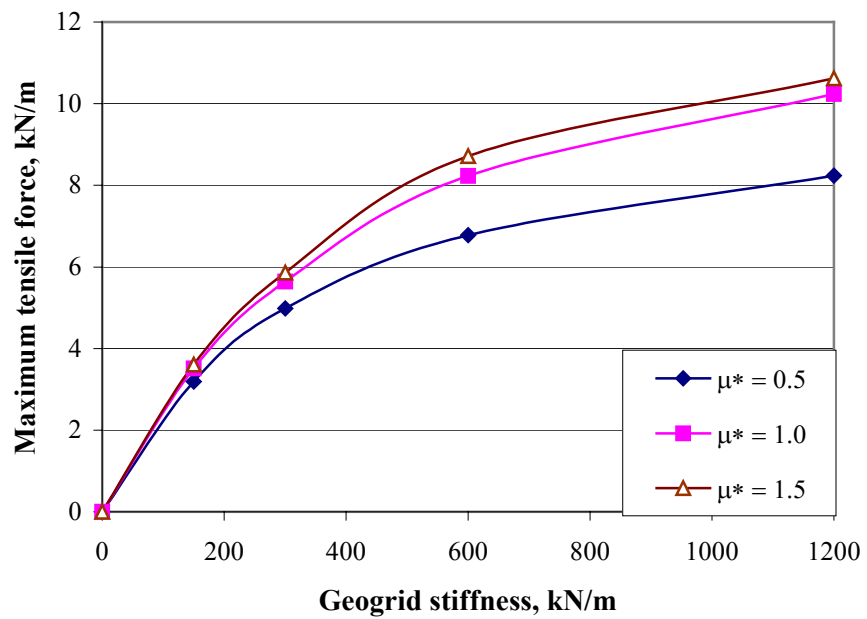


Figure 62. Influence of geogrid modulus and interface property on mobilized tensile force of geogrids

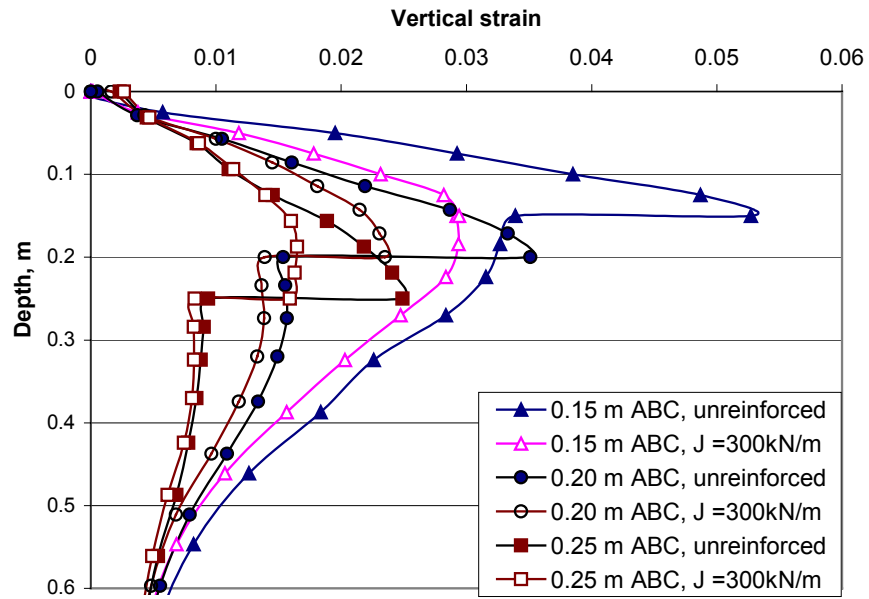


Figure 63. Influence of ABC thickness on vertical strain underneath the center of the loaded area

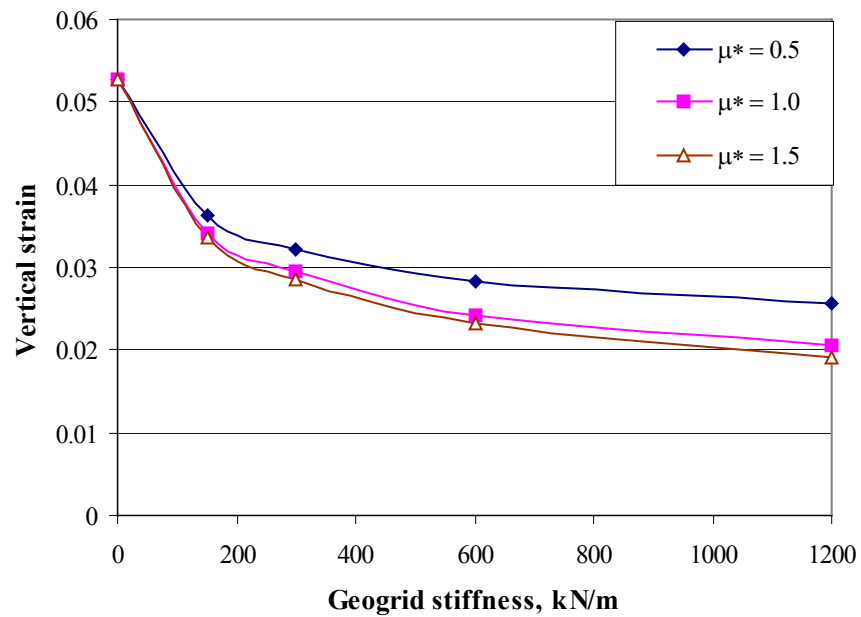


Figure 64. Influence of geogrid modulus and interface property on vertical strain at the bottom of base layer

## 5.5 Degradation study and comparison to testing results

FEM analysis provides information to understand the stress-deformation response of reinforced unpaved structure under static load. However, the behavior of an unpaved structure is complicated under cyclic load, as large plastic deformation and degradation (deterioration) usually happen in the process. The numerical analysis method used herein is not capable of simulating the plastic deformation accumulation and the degradation under cyclic load.

The cyclic load tests show that the degradation of unpaved structure is mainly represented as a decrease in stress attenuation ability of base layer. As the stress attenuation ability of base layer is related to base layer thickness and the elastic modulus ratio of base layer and subgrade, the degradation of ABC layer can be approximately simulated by using decreased elastic modulus ratio. In static FEM analysis, the subgrade modulus of 10 MPa (measured from plate load test) was kept as a constant and different ABC elastic modulus varied from 100 Mpa, 50 Mpa, 30 Mpa, 20 Mpa and 10 Mpa. One ABC thickness (0.25 m), one friction coefficient (1.0), and two types of geogrid tensile stiffness (300 kN/ m and 600 kN/ m) were used. The maximum surface deformation and the maximum vertical stress on the subgrade are shown in Figure 65 and Figure 66 respectively. In this case, the ABC elastic modulus or elastic modulus ratio decreases the surface deformation increases and more vertical stress is transferred to the top of subgrade. The benefits (reducing surface deformation and attenuating the vertical stress on subgrade) from geogrid reinforcement increase as the elastic modulus ratio decreases.

The permanent deformation and vertical stress on the subgrade were recorded in the cyclic plate load tests. From the 254-mm ABC tests, the vertical stresses transferred to the subgrade are approximately 120 kPa at the very beginning of cyclic load and the geogrid reinforcement show little improvement on the stress distribution. The static FEM analysis can predict the similar results with  $E_{ABC}/E_{subgrade}$  of 7. With the number of cycles increasing, the vertical stress transferred to subgrade can be predicted by updating the modulus of ABC material, which may degrade under cyclic load. For 254-mm ABC tests, the elastic modulus ratios of unpaved sections were back-figured to be 1.4, 1.8 and 3.7 after 8000 cycles, for tests without reinforcement, BX1100 geogrid, and with BX1200

geogrid, respectively. The modulus ratio values can be used to predict the maximum vertical stress on the subgrade and the maximum surface deformation on the subgrade, which are shown in the Figure 65 and Figure 66. Table 11 shows the comparison of static FEM results and the cyclic load test results at the end of 8000 cycles. As expected in this case, the FEM analysis prediction of maximum vertical stress matches well with cyclic load testing results. Since the modulus ratio was back-calculated from the experimental program, the stress distribution can be predicted from static FEM analysis. The static FEM analysis with changing elastic modulus ratio can not directly predict the large plastic deformation accumulation of unpaved road sections during under cyclic load. However, the permanent deformation is possible to be calculated by using the permanent strain relationship from cyclic triaxial tests and the stress distribution.

Table 11. Static FEM results and the cyclic load tests results (N = 8000 cycles)

| Geogrid reinforcement | Vertical stress, kPa<br>(Test results) | $E_{ABC}/E_{subgrade}$<br>(back-figured) | Vertical stress, kPa<br>(FEM prediction) |
|-----------------------|----------------------------------------|------------------------------------------|------------------------------------------|
| Without               | 200                                    | 1.4                                      | 198                                      |
| BX1100                | 180                                    | 1.8                                      | 175                                      |
| BX1200                | 130                                    | 3.7                                      | 147                                      |

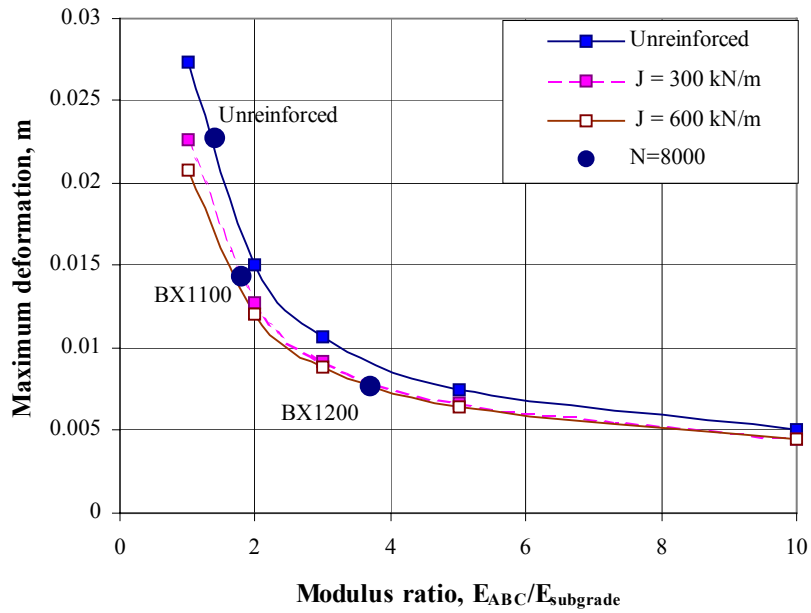


Figure 65. Influence of modulus ratio on surface deformation ( $h_{ABC} = 0.25$  m,  $E_{subgrade} = 10$  MPa,  $\mu^* = 1.0$ )

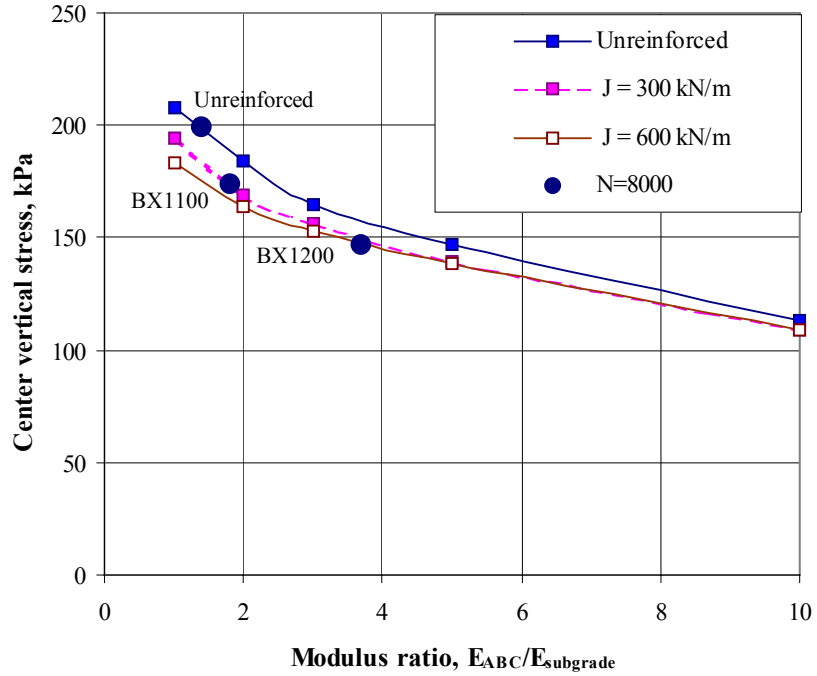


Figure 66. Influence of modulus ratio on vertical stress on the subgrade ( $h_{ABC} = 0.25$  m,  $E_{subgrade} = 10$  MPa,  $\mu^* = 1.0$ )

## 5.6 Summary

The performance of a geogrid-reinforced section tested in laboratory was numerically simulated using FEM program Abaqus. Compared with Bousinesq's solution and elastic layer analysis, the elasto-plastic model used in the numerical analysis provided a more "realistic" prediction of stress distribution.

The FEM study indicates the geogrid reinforcement placed between the base layer and the subgrade layer can provide lateral confinement at the bottom of the base layer. Geogrid reinforcement improves the interface shear resistance, which results in increased mean stress at the bottom of the base layer. The results of the numerical study also demonstrated that the inclusion of geogrids between base layer and subgrade can reduce the surface deformation on the unpaved structure, improve the stress distribution inside base layer and subgrade layer. The extent of improvement depends on the ABC thickness, geogrid tensile modulus, geogrid-ABC interface property, and the elastic modulus ratio of base layer and subgrade. Results also indicated that a significant vertical strain at the bottom of base layer was generated, and geogrids can provide tensile resistance to limit the lateral spread of ABC, and thereby decrease the vertical strain generated at the bottom of base layer.

As the ABC thickness decreases, or the elastic modulus ratio decrease, the benefit due to geogrid reinforcement becomes more apparent. In general, geogrid with higher tensile modulus and better interface property with base course aggregate shows better reinforcement effect. The degradation of unpaved structures under cyclic load was approximated by using FEM analysis with decreased elastic modulus ratio of base layer and subgrade. The predicted vertical stresses on the subgrade were close to cyclic load test results. Analyses indicated that geogrid with higher tensile modulus provided better attenuation of stresses than lower tensile modulus, due to better performance in decreasing the degradation of the ABC layer.

## **Chapter 6 DESIGN METHOD OF REINFORCED UNPAVED STRUCTURE**

### **6.1 Reinforced unpaved structure modeling**

Unpaved roads usually suffer degradation and plastic deformation under cyclic load. Large-enough thickness of the base layer is required to attenuate stress and prevent subgrade bearing capacity failure, and limit the plastic deformation magnitude. The geogrid reinforcement at the interface of subgrade and base layer can improve the performance of unpaved roads by providing shear resistance interaction on both subgrade and base layers. The interaction of the geogrid and the subgrade can be manifested as improvement of subgrade bearing capacity, and the interaction of the geogrid and the base layer can be manifested as improvement of the load spreading ability of the base layer and improvement of vertical stress distribution on the top of the subgrade layer.

#### **6.1.1 Geogrid-subgrade interaction**

##### **Bearing capacity analysis of subgrade**

A key aspect of unpaved structure design is to control the stress transferred to the subgrade as not to exceed the bearing capacity of subgrade. Based on the low bound plasticity theory for undrained loading on semi-infinite saturated clay (Bolton, 1979), the bearing capacity factor ( $N_c$ ) for plain strain problem can be expressed as (Miligan et al., 1989):

$$N_c = 1 + \frac{\pi}{2} + \cos^{-1} \alpha_a + \sqrt{1 - \alpha_a^2} \quad (66)$$

Here, shear stress factor  $\alpha_a = \tau_a / C_u$ ;

$\tau_a$  = shear stress acting on the top of subgrade;

$C_u$  = undrained shear strength of subgrade.

The bearing capacity factor is dependent on shear stress transferred from base course layer. If the shear above subgrade is zero ( $\alpha_a = 0$ ),  $N_c$  becomes  $(\pi+2)$ ; if the outward shear above subgrade is equal to shear strength of subgrade ( $\alpha_a = 1$ ), then  $N_c$  becomes  $(\pi/2+1)$ . For reinforced unpaved road, the bearing capacity factor of subgrade is also a function of the interface property between reinforced base layer and subgrade. For a case with smooth interface (zero interface shear strength,  $\alpha_a = 0$ ) between geosynthetic-base system and subgrade soil,  $N_c = (\pi+2)$ . For the case with rough interface (maximum interface shear strength,  $\alpha_a = -1$ ),  $N_c = (3\pi/2+1)$  with inward shear above subgrade equal to shear strength of subgrade.

For axi-symmetric problem, Giroud (2000) recommended  $N_c = 5.69$  (from Cox et al., 1961) for zero interface shear strength case, and  $N_c = 6.04$  (from Eason and Shield, 1960) for maximum interface shear strength case.

### **Bearing capacity factors in the unpaved road design**

Barenberg et al. (1975) evaluated the performance of model footing on an aggregate-soft clay system. With failure defined by excessive rutting over 50 mm, the bearing capacity coefficient  $N_c = 3.3$  without reinforcement, and  $N_c = 6.0$  with reinforcement. Steward et al. (1977) found the  $N_c$  values used for unpaved road design were related to traffic level and road performance (rutting), as shown in Table 12.

Table 12. Bearing capacity factors for unpaved roads from Steward et al. (1977)

| Traffic Level* | $N_c$ | Performance                        |
|----------------|-------|------------------------------------|
| High           | 2.8   | Very little rutting without fabric |
| Low            | 3.3   | Deep rutting Without fabric        |
| High           | 5.0   | Very little rutting With fabric    |
| Low            | 6.0   | Deep rutting With fabric           |

High traffic level: > 1000 passes of a 80 kN axle load;

Low traffic level: < 100 passes of the same load.

In Giroud and Noiray (1981) method, the bearing capacity factor  $N_c = \pi$  (elastic limit) was used for unreinforced unpaved road.  $N_c = (\pi+2)$  was used for geotextile-reinforced road for a general bearing capacity failure with a smooth interface, because the subgrade deformation under control by the geotextile reinforcement.

Geotextiles and geogrids placed between base layer and subgrade provide different interface friction properties. The geotextile separates the base layer from subgrade, and the interface shear strength is determined by the adhesion between geotextile and subgrade. The maximum interface shear strength is generally low and relatively large deformation is needed to mobilize the maximum shear strength. The geogrids do not completely separate the base layer aggregate from subgrade, but the relevant interface between the subgrade soil and interlocked aggregate is expected to provide relatively high interface shear strength. As geogrid-reinforced aggregate has relatively high shear modulus at the interface, interface shear strength can be mobilized under relatively low deformation. It may be reasonable therefore to choose different bearing capacity factor for geogrid-reinforced unpaved from geotextile-reinforced unpaved road.

There is no report of  $N_c$  value for unreinforced case under axi-symmetric condition. As  $N_c$  reported was ranged from  $\pi/2+1$  to 3.3 for the plain strain problem,  $N_c$  value under axi-symmetric condition can be approximately 1.3 times of the  $N_c$  value under plain strain condition, with the range of bearing capacity factor from 3.3 to 4.3 with average value of 3.8. In this research and as a part of the model to be presented later, bearing capacity factors are assumed according to axi-symmetric condition as follow:

$N_c = 3.8$ , for unreinforced road

$N_c = 5.69$ , for geotextile-reinforced unpaved road

$N_c = 6.04$ , for geogrid-reinforced unpaved road

### **Mobilization of subgrade bearing capacity**

A criterion of surface deformation or rutting has been widely used in the design method. Based on elastic analysis, the mobilized bearing capacity of subgrade soil is related to the ratio of deformation on the top of subgrade ( $w_s$ ) and the radius of the loaded area ( $a' = a + h \tan \alpha$ ).

For the case with subgrade only ( $h = 0$ ,  $a' = a$ ), critical surface deformation is assumed equal to the subgrade deformation ( $w_s = w_{cr}$ ) required to mobilize the bearing capacity (i.e. elastic limit). The bearing capacity of subgrade soil can be calculated using elastic analysis of flexible plate loading condition:

$$N_c C_u = p_{cr} = \frac{E_s w_{cr}}{2(1-\mu_s^2)a} \quad (67)$$

$$w_{cr} = w_s$$

Here,  $p_{cr}$  = critical subgrade bearing resistant;  $E_s$  = elastic modulus of subgrade;

For the case with base layer over subgrade condition, the mobilized subgrade bearing resistant ( $p_{cm}$ ) under the total deformation of  $w_{cr}$ :

$$N_{cm} C_u = p_{cm} = \frac{E_s w_s}{2(1-\mu_s^2)(a + h \tan \alpha)} = \left( \frac{a/h}{a/h + \tan \alpha} \right) \left( \frac{w_s}{w_{cr}} \right) N_c C_u \quad (68)$$

$$w_{cr} = w_s + \Delta w_1$$

Where,  $N_{cm}$  = the mobilized bearing capacity factor corresponding to critical surface deformation;  $w_s$ ,  $\Delta w_1$  (base layer compression) and  $\tan \alpha$ , can be determined using Equation 40, 41 and 44 respectively.

A modified bearing capacity ratio ( $m = N_{cm}/N_c$ ) can be introduced here to represent the relative mobilized bearing capacity of subgrade as a function of deformation level. Figure 67 shows the modified bearing capacity ratio with varied  $a/h$  values ( $a/h = 0.4, 0.6, 1.0, 1.5$ , and  $3.0$ ) and  $E_1/E_2$  values ( $E_1/E_2 = 1, 2, 3, 5, 10$ ). Using data from Equation 67 and 68, the regression curve in Figure 67 shows that the modified bearing capacity ratio can be approximately expressed as a function of  $a/h$ :

$$m = N_{cm}/N_c = 1 - e^{-0.78(a/h)} \quad (69)$$

The above analysis of the modified bearing capacity ratio is based on the condition that the surface deformation is the critical deformation required for mobilizing the bearing capacity of subgrade. For unpaved road design, the typical allowable rutting

is 2-4 inches (50 mm –100 mm). This allowable rutting may be higher than the critical deformation of subgrade as long as the stress transferred to subgrade is less than the bearing capacity of subgrade. In this model, it is assumed that the bearing capacity of subgrade can be fully mobilized when the rutting on subgrade is 2 inches (50 mm) or the critical rutting ( $r_{cr}$ ) is 2 inches (50 mm). A modified bearing capacity modification factor can be adjusted according to the rutting criterion ( $r$ ) used for the design:

$$m = \left[ 1 - e^{-0.78(a/h)} \right] \frac{r}{r_{cr}} \leq 1 \quad (70)$$

The modified bearing capacity can be expressed by:

$$q_{cm} = m N_c C_u \quad (71)$$

The bearing capacity of subgrade includes a part from surcharge of base layer, which is neglected as safety reserve.

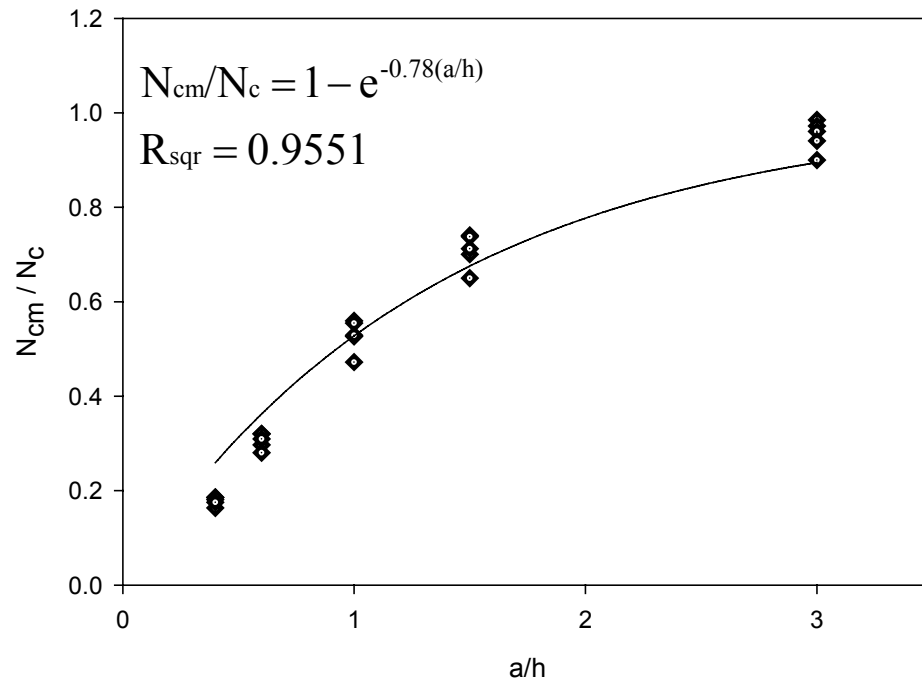


Figure 67. estimated modified bearing capacity ratio of unpaved road

### 6.1.2 Geogrid-base course aggregate interaction

Geogrid-base course aggregate interaction is seen as the increase of base layer stress attenuation ability, and the improvement of the vertical stress distribution on the subgrade. Previous research (Love et al., 1987) showed that the geosynthetic inclusion at the interface of base course and subgrade can improve the stress distribution.

Elasto-plastic FEM analysis (Chapter 5) has shown that the vertical stress distribution at the interface of two-layer system is affected by properties of base course and subgrade ( $E_1/E_2$ ), the thickness of base course ( $a$ ) and the interface property. For reinforced case, stress distribution is also related to the properties of reinforcement (tensile modulus) and the interaction of reinforcement and soils (interface friction coefficient), as well as deformation of unpaved structure. Some deformation is required to mobilize the geosynthetic reinforcement effect. For the unpaved road under static load with low deformation, FEM analysis shows slight improvement of stress distribution on the subgrade due to geosynthetic reinforcement. The vertical stress on the subgrade increases under cyclic load, due to the degradation of base layer modulus. The cyclic loading test results indicate that geosynthetic reinforcement is significant in decreasing the degradation of the aggregate base layer under cyclic load. The reinforced test sections show better vertical stress distribution on the subgrade, as compared to unreinforced test sections after some load cycles.

#### Stress distribution angle analysis

In the unpaved road design, the stress distribution angle ( $\alpha$ ) is usually used to simplify the analysis. In this research, the maximum vertical stress on the subgrade is used as the average stress to determine the stress distribution angle. The average vertical additional stress on the subgrade layer can be expressed as:

$$\Delta\sigma_v = \frac{pa^2}{(a + h\tan\alpha)^2} \quad (72)$$

Where,  $p$  is the pressure acting on the base layer;  $a$  is the radius of circular loading area;  $h$  is the thickness of base layer;  $\alpha$  is the stress distribution angle.

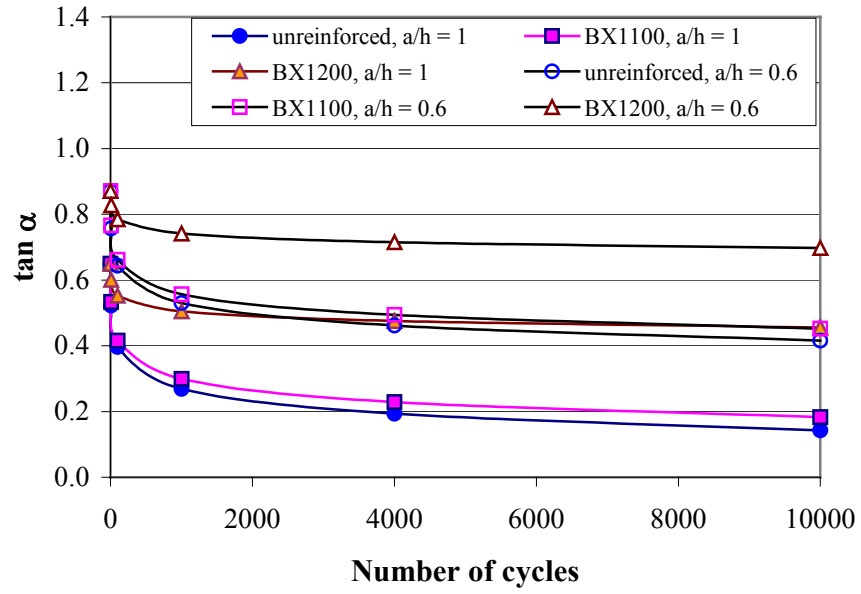
Typical values of  $\tan \alpha$  have been used in the static analysis of unpaved structure, such as 0.6 used in Grioud et al., (1984). Based on elastic layer analysis, under axis-symmetric condition, the stress distribution angle is related to the elastic modulus ratio ( $E_1/E_2$ ) and the ratio of base layer thickness to the radius of the loaded area ( $a/h$ ). The experimental results showed that the stress attenuation ability of base layer degrades under cyclic load. The degradation has been shown as a decrease of  $\tan \alpha$  value under cyclic load. Compared with unreinforced unpaved structure, the geogrid reinforcement can decrease the degradation of base layer and show larger stress distribution angle under cyclic load. It is reasonable to define the initial (or static) stress distribution angle based on elastic modulus ratio and the thickness of base layer. However, degradation needs to be incorporated in the design method, with consideration of reinforcement effect.

Base on the elastic layer analysis and the cyclic loading test results, the vertical stress distribution on the subgrade under cyclic load can be expressed by using:

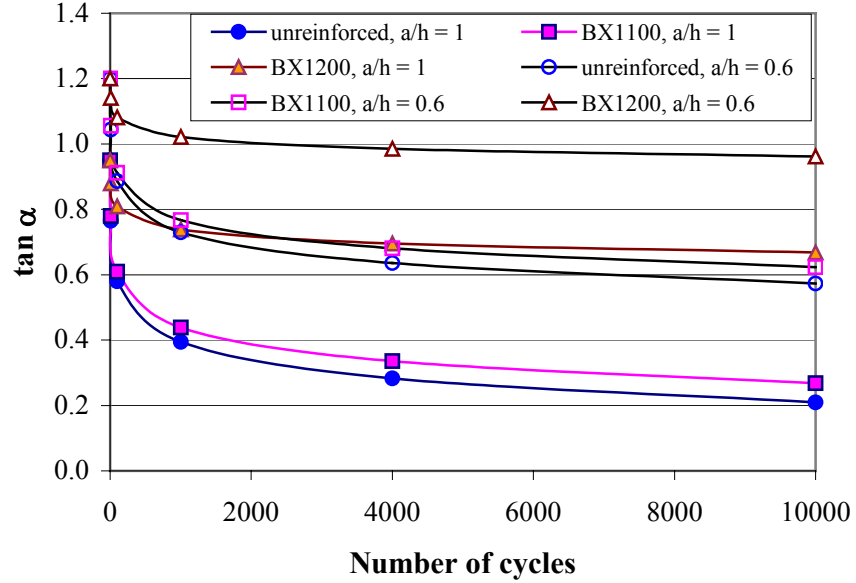
$$\tan \alpha_N = \tan \alpha_1 [1 + k_2 \log N] \quad (73)$$

Where,  $\tan \alpha_N$  is the  $\tan \alpha$  value at the N-th load cycle;  $\tan \alpha_1$  is the initial value of  $\tan \alpha$ , which can be determined by using Equation 46;  $k_2$  is a constant related to degradation, which can be determined by using Equation 60.

Figure 68 shows  $\tan \alpha$  value under cyclic load for a section with subgrade CBR =3. The initial elastic modulus ratio of base layer and subgrade is assumed to be  $E_1/E_2 = 5$  and 10, with two  $a/h$  values ( $a/h = 1$ , and 0.6) and reinforcement conditions (unreinforced, BX1100 geogrid and BX1200 geogrid).



(a)  $E_{ABC} / E_{subgrade} = 5$



(b)  $E_{ABC} / E_{subgrade} = 10$

Figure 68. Stress attenuation ability ( $\tan \alpha$ ) under cyclic load

### **Confinement of base course aggregate**

The response of reinforced section is related to the interface shear-resistance interaction between Geogrid and base course aggregate. The interfaces shear resistance provides additional confinement at the bottom of the base layer and induces tension in the geogrid. These two mechanisms also known as “confinement” of base course aggregate, and membrane effect of geogrid.

The geogrid reinforcement can provide additional confining effect at the bottom of base layer though interlocking with aggregates. FEM analysis showed that lateral confinement due to geogrid reinforcement increased the mean stress at the bottom of base layer. As the modulus of granular material depends on the confining pressure, such increase improves the stress-strain characteristics of base course aggregate, and the stress distribution on the subgrade.

### **Membrane effect of geogrid**

Previous membrane analysis method (Giroud and Noiray, 1981) in unpaved road focused on tensioned membrane support, which is the vertical component of normal stress of deformed geosynthetic membrane to resist the vertical load. Actually the shear stress along the interface between the base layer and the geosynthetic can also provide vertical component to resist the vertical load. The interface shear stress support usually contributes to the confinement of base course in addition of the lateral confinement.

Using the membrane equation in Espinoza’s study (1994) and modifying it from plane strain condition to axi-symmetric condition, total vertical support (tensioned membrane support and interface shear stress support):

$$q_m(r) = T_h(r) \frac{d^2 z(r)}{dr^2} \quad (74)$$

where,

$$T_h(r) = T(r) \cos \beta$$

$$\tan \beta = \frac{dz(r)}{dr}$$

$r$  = horizontal radial coordinate;  $z(r)$  = vertical deflection of geogrid;  $\beta$  = angle that deformed geogrid makes with horizontal plane;  $T(r)$  = geogrid tensile force, (equal to geogrid tensile stiffness  $J$  times tensile strain  $\epsilon_r$ );  $T_h(r)$  = horizontal component of the tensile force  $T(r)$ .

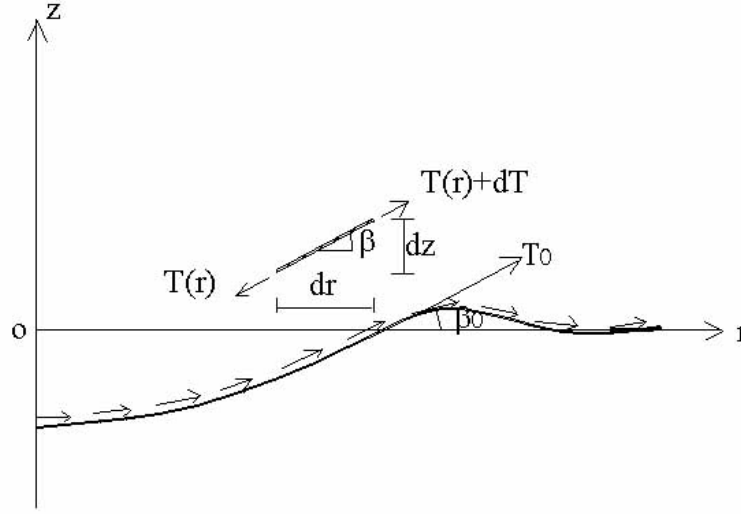


Figure 69. Deformed geogrid under axi-symmetric condition

Horizontal shear stress acting on the geogrid

$$\tau = -\frac{dT_h(r)}{dr} \quad (75)$$

Assuming the convex part of the deformed geogrid has radius of  $a'$  (shown in Figure 70), average vertical support can be expressed as:

$$\begin{aligned} \bar{q}_m &= \frac{2\pi \int_0^{a'} q_g(r) r dr}{\pi a'^2} = \frac{2 \int_0^{a'} T_h(r) r \frac{d^2 z(r)}{dr^2} dr}{a'^2} \\ &= \frac{2 T_h(r) r z'(r)}{a'^2} \Big|_0^{a'} - \frac{2 \int_0^{a'} \left[ T_h(r) + \frac{dT_h}{dr} r \right] \frac{dz(r)}{dr} dr}{a'^2} \\ &= \frac{2 T_h(a')}{a'} \tan \beta(a') + \frac{2 \int_{z(0)}^{z(a')} \tau(r) r dz}{a'^2} - \frac{2 \int_0^{a'} T_h(r) \tan \beta(r) dr}{a'^2} \end{aligned}$$

$$\bar{q}_m \approx \frac{T_0}{a'} \sin \beta_0 + \frac{2 \int_{z(0)}^{z(a')} \tau(r) r dz}{a'^2} \quad (76)$$

$$T_0 \leq \min\left(\frac{2L_a a' + L_a^2}{2a'} \gamma h \tan \varphi, T_{\max}\right) \quad (77)$$

$$\tau(r) \leq (\gamma h + \Delta \sigma_v) \tan \varphi \quad (78)$$

Here,  $T_0$  and  $\beta_0$  = the geogrid tensile force and inclination angle at the point  $r = a'$ , respectively;  $\varphi$  = interface friction angle;  $L_a$  = geogrid anchorage length;  $T_{\max}$  = geogrid maximum tension force;  $\gamma$  and  $h$  = unite weight and thickness of base aggregate, respectively;  $\Delta \sigma_v$  = additional vertical stress at the bottom of base layer due to surface stress (p).

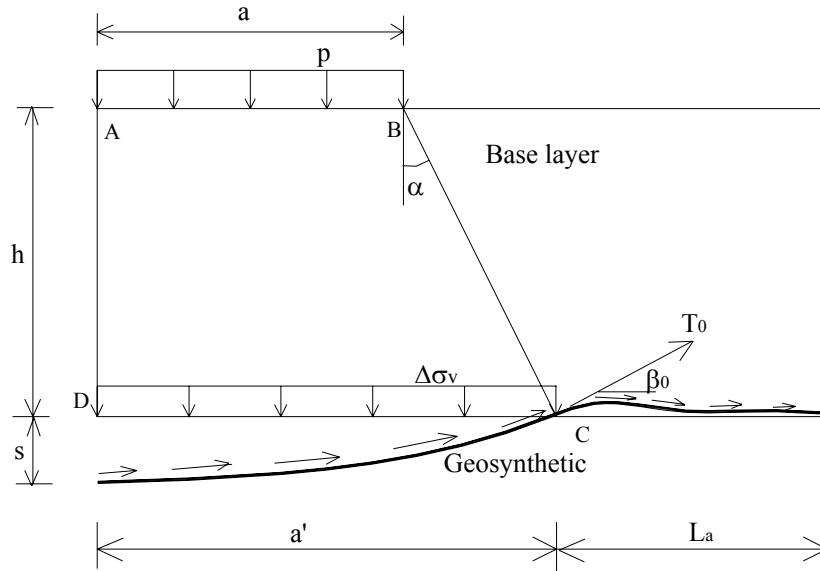


Figure 70. Membrane effect in the reinforced base course

Assuming shear stresses on the effective area ( $r \leq a'$ ) can be approximated using average shear stress:

$$\bar{\tau} = (\gamma h + \Delta \sigma_v) \tan \varphi_m = \left[ \gamma h + \left( \frac{a}{a'} \right)^2 p \right] \tan \varphi_m \quad (79)$$

The average vertical support can be simplified as:

$$\bar{q}_m \approx \frac{T_0}{a'} \sin \beta_0 + \frac{3s \left[ \gamma h + \left( \frac{a}{a'} \right)^2 p \right] \tan \varphi_m}{2a'} \quad (80)$$

Where,  $\varphi_m$  = mobilized interface friction angle;  $s$  = deflection on the subgrade

The first item in Equation 80 represents the tensioned membrane support from outside of the effective area ( $r \geq a'$ ), which depends on tensile force induced in the geosynthetic membrane as well as proper anchorage outside of the effective area. The second item represents the interface shear stress support within the effective area ( $r \leq a'$ ), which depends on the applied stress and the mobilized interface friction. As  $\sin \beta_0$  and deflection on the subgrade ( $s$ ) are both related to the deformation on the subgrade, a relatively large deformation is usually required to mobilize both tensioned membrane support and interface shear stress support components. This tendency was also apparent from the results from cyclic load test data and FEM analysis results.

### 6.1.3 Equilibrium equations for critical state analysis

Vertical force and horizontal force equilibrium conditions can be checked, as seen in Figure 71.

i) Vertical equilibrium condition:

$$\Delta \sigma_v = q_s + \bar{q}_m \quad (81)$$

Here,  $\Delta \sigma_v$  = the vertical stress acting on the geosynthetic;  $q_s$  = subgrade bearing capacity.

If the vertical stress due to traffic loading exceeds the subgrade bearing capacity plus the vertical support from membrane effect, the unpaved structure will undergo subgrade bearing failure.

ii) Horizontal equilibrium condition:

$$T = P_a - P_p \quad (82)$$

Assume that ABD is the wedge beneath the wheel load,  $P_a$  acting on BD is active force,  $P_p$  acting on BC is passive force, and the horizontal shear stress acting on CD is uniformly distributed.

$$P_a = 2\pi K_a \int_0^h \left( \gamma z + \frac{a^2 q_0}{(a + z \tan \alpha)^2} \right) \left( 1 - \frac{z}{h} \right) dz \quad (83)$$

$$P_p = 2\pi K_p \int_0^h \gamma z (a + z \tan \alpha) dz \quad (84)$$

$$T = 2\pi \int_0^{a'} \tau r dr = \pi (a + h \tan \alpha)^2 \left[ \gamma h + \left( \frac{a}{a + h \tan \alpha} \right)^2 p \tan \phi_m \right] \quad (85)$$

Here  $K_a$  and  $K_p$  are active earth pressure coefficient and passive earth pressure coefficient of the base layer.

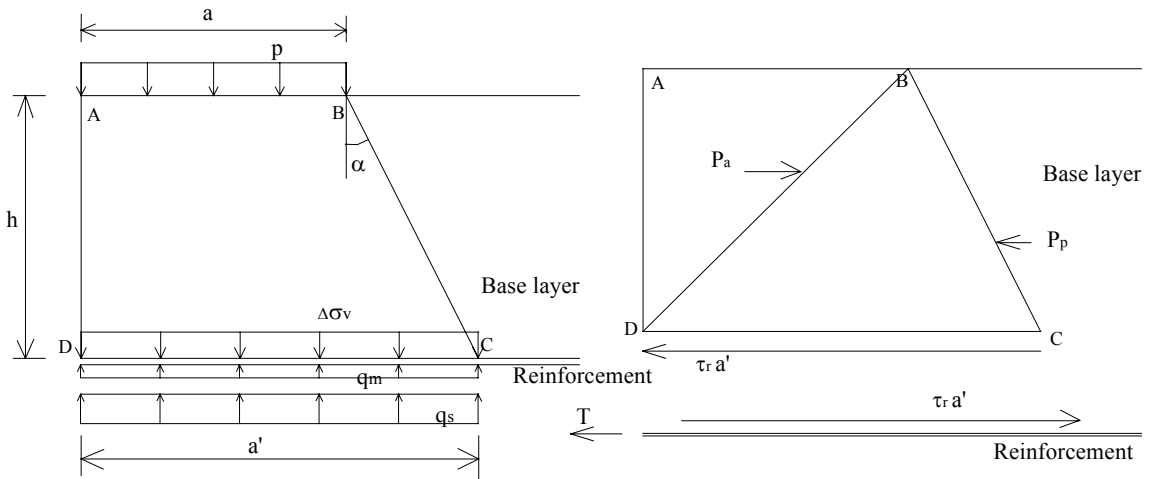


Figure 71. Vertical and horizontal equilibrium reinforced base course

If the lateral active force exceeds the resistance from passive force plus interface shear resistance, the unpaved structure will undergo base course lateral bearing failure, which usually happens when the friction angle of base layer and the friction coefficient of the interface are both low. With the mobilized interface shear stress determined from

horizontal equilibrium, the magnitude of vertical resistance due to membrane (Equation 80), at the critical condition (base course lateral bearing failure) can be estimated.

The mobilized interface shear resistance and membrane support are evaluated with the same conditions used in the testing program:  $q = 550$  kPa,  $a = 0.152$  m, initial  $E_1/E_2 = 7$ , and  $a/h = 0.6$ , and 1. The mobilized interface friction coefficient against base course lateral bearing failure increases as base aggregate friction angle or  $a/h$  decreases, as shown in Table 13:

Table 13. The mobilized interface friction against base course lateral bearing failure

| Friction angle of base aggregate, $\phi$ (degree) | Mobilized interface friction coefficient, $\tan \phi_m$ |           |
|---------------------------------------------------|---------------------------------------------------------|-----------|
|                                                   | $a/h = 0.6$                                             | $a/h = 1$ |
| 20                                                | 0.509                                                   | 0.525     |
| 30                                                | 0.283                                                   | 0.341     |
| 40                                                | 0.087                                                   | 0.197     |

Solution for the equilibrium equations can be obtained with simplified assumptions regarding the vertical stress distribution, subgrade bearing capacity, membrane support and shear stress distribution. The actual distribution of  $\Delta\sigma_v$ ,  $q_m$  and  $\tau$  depends on the geometric characteristics and material properties of the base course, subgrade and geosynthetic, and the interaction of soil and geosynthetic. Numerical schemes may be necessary for analysis of stress and strain of unpaved structure. As they may allow less restrictive assumptions and provide better simulation of soil properties, geosynthetic and their interaction.

## 6.2 Proposed design method

### 6.2.1 Proposed design method development

Assuming one loaded wheel contacting the unpaved road surface on a circular area, the vertical stress distributed on the subgrade layer need to be less than or equal to mobilized bearing capacity of subgrade in order to prevent the rutting failure and the bearing capacity failure of subgrade:

$$\sigma_c = \frac{P}{\pi(a + h \tan \alpha_N)^2} \leq m N_c C_u \quad (87)$$

Where,  $\sigma_c$  = the additional vertical stress on the subgrade;  $a$  = the radius of loaded area;  $P$  = the single wheel load;  $\alpha_N$  = stress distribution angle at the design load cycles  $N$ ;  $m$  = the modified bearing capacity ratio calculated by using Equation 70, which is related to  $a/h$  and the rutting criterion used in the design;  $N_c$  = the bearing capacity factor (3.8 for unreinforced road, 5.7 for geotextile-reinforced unpaved road, and 6.0 for geogrid-reinforced unpaved road);  $C_u$  = the undrained shear strength of subgrade;

The required thickness of base layer ( $h$ ) can be expressed as:

$$h = \frac{1}{\tan \alpha_N} \left( \sqrt{\frac{P}{\pi m N_c C_u}} - a \right) \quad (88)$$

Assuming that the wheel load is uniformly distributed on the contact area ( $P = p\pi a^2$ ), and the degradation of base layer can be determined as  $\tan \alpha_N = \tan \alpha_1 [1 + k_2 \log N]$ , the required base layer thickness and  $a/h$  can be determined as following:

$$h = \frac{a}{\tan \alpha_1 [1 + k_2 \log N]} \left( \sqrt{\frac{p}{m N_c C_u}} - 1 \right) \quad (89)$$

$$\frac{a}{h} = \frac{\tan \alpha_1 [1 + k_2 \log N]}{\sqrt{\frac{pa}{mN_c C_u}} - 1} \quad (90)$$

Where  $\tan \alpha_1$  is a function of initial  $E_1/E_2$  and  $a/h$ , and which can be determined by using Equation 46; degradation coefficient  $k_2$  can be determined by using empirical equation (Equation 60), which is related to  $a/h$  and the geogrid property (torsional stiffness  $J_t$ );

As  $h$  or  $a/h$  values are on both sides of the design equations (Equation 89 and 90), an iteration scheme is necessary in order to solve the equations. The required base layer thickness ( $h$ ) in this model is a function of the modulus ratio of base layer and subgrade ( $E_1/E_2$ ), the undrained shear strength of subgrade ( $C_u$ ), the torsional stiffness of geogrid reinforcement ( $J_t$ ), the bearing capacity factor ( $N_c$ ), the single wheel load ( $P$ ), the radius of loaded area ( $a$ ), and the number of design load cycles ( $N$ ).

### 6.2.2 Determination of design parameters

For the unpaved road design, some design parameters are usually not directly measured, such as modulus of base layer and subgrade, subgrade undrained shear strength. However, CBR values of subgrade and base course aggregate have been widely used in the roadway design. Some empirical relationships are available to evaluate undrained shear strength and modulus of subgrade as follows:

#### i) Subgrade undrained shear strength

The empirical relationship between subgrade undrained shear strength was proposed by Giroud and Noiray (1981):

$$C_u = 30 \text{ CBR}_{sb} \text{ (kPa)}$$

#### ii) Subgrade modulus

The most widely used relation of subgrade resilient modulus ( $E_2$ ) and CBR is the one developed by Shell (Heukelom and Klomp, 1962):

$$E_2 \text{ (MPa)} = 10 \text{ CBR}_{sb}$$

iii) Base course modulus

The empirical relationship of base course modulus and base course CBR value is represented in a design method proposed by Tensar Earth Technologies, Inc. (2001). Using the design chart (Figure 2.7) of the AASHTO Design Guide for Pavement Structure (1993), to obtain values of CBR versus base course modulus, the following equation is developed:

$$E_1 = 36 \text{ CBR}_{bs}^{0.3} \text{ (MPa)}$$

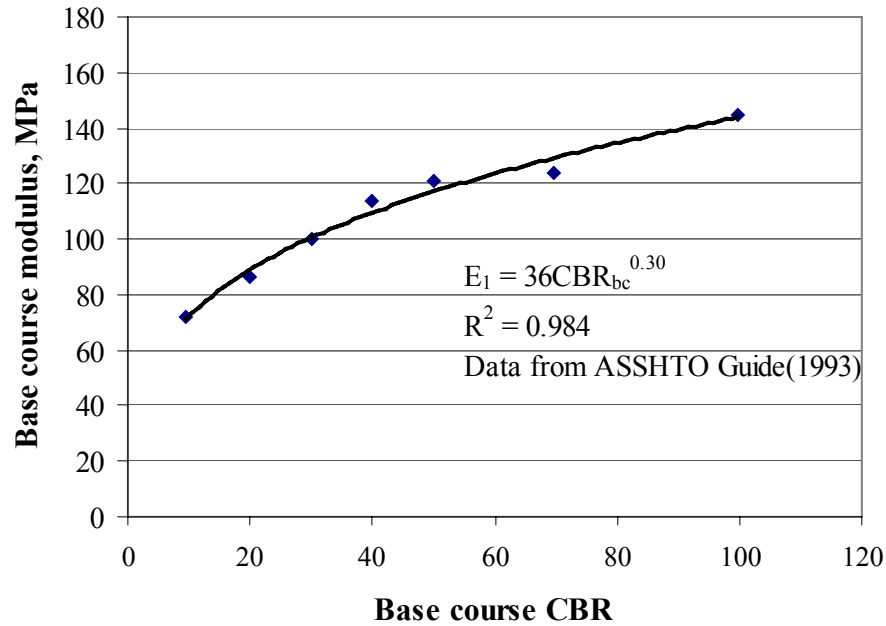


Figure 72. Correlation of base course modulus and CBR

iv) Elastic modulus ratio and CBR ratio of base course and subgrade

With the correlation equations of modulus and CBR value for subgrade and base course, the modulus ratio  $E_1/E_2$  can be expressed as:

$$\frac{E_1}{E_2} = \frac{36 \text{ CBR}_{bc}^{0.3}}{10 \text{ CBR}_{sb}} = \frac{3.6}{\text{CBR}_{sb}^{0.7}} \left( \frac{\text{CBR}_{bc}}{\text{CBR}_{sb}} \right)^{0.3} \quad (91)$$

The CBR value for aggregate base course placed on subgrade is related to the CBR value of subgrade. Aggregate base course placed on the good subgrade will have good support for good compaction, and the CBR value of base course can reach 80 to 100. If the base course is constructed on soft subgrade, the base course CBR value can be low as soft subgrade can not provide the support needed to obtain good compaction of the base course aggregate. Reported CBR values for base courses and subgrades in Hammit (1970) study are shown in Figure 73. The CBR ratio varied from 1.7 to 17 with an average of 5.2. For the cases without reported base course CBR values, it is recommended to use the average CBR ratio 5.2 to estimate the elastic modulus ratio  $E_1/E_2$ .

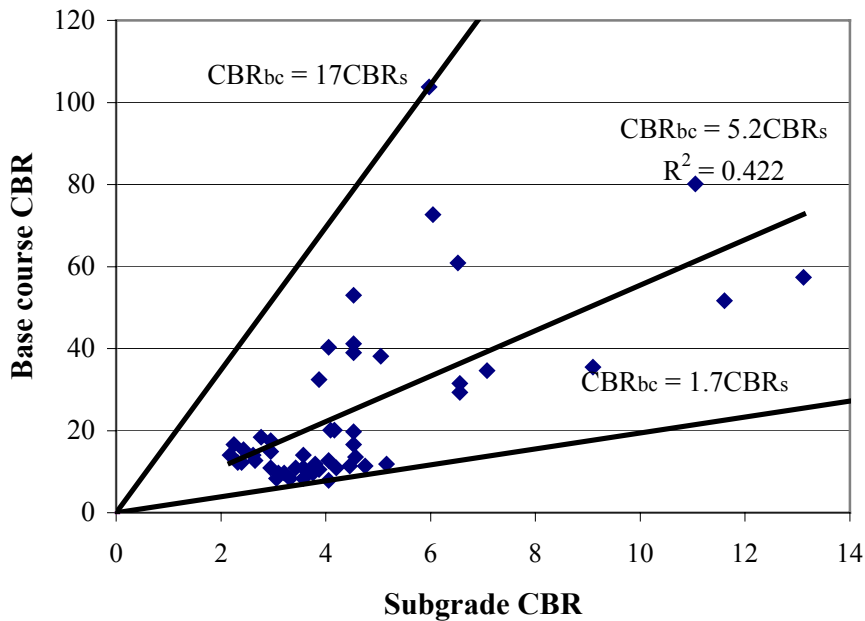


Figure 73. CBR values of base course and subgrade (data from Hammit, 1970)

### 6.3 Design method verification

A field truck testing program on unpaved roads has been reported by Fannin and Sigurdsson (1996). The test truck imposed a standard axle load of 80 kN. The subgrade was soft clayey silt with undrained shear strength approximately 40 kPa. One unreinforced test section and one test section with geogrid BX1100, were used. Rut path was recorded on each test sections of the roadway, where the initial base course thickness was 0.25, 0.3, 0.35, 0.4 and 0.5 m.

A subgrade CBR value of 1.3 was used based on undrained shear strength and CBR correlation. As there is no reported base course CBR value,  $CBR_{bc}$  of 6.8 was assumed using average field CBR ratio of 5.2. The modulus ratio  $E_1/E_2$  of 4.9 was selected using Equation 91.

The empirical relationship of  $k_2$  with torsional stiffness of geogrids and  $a/h$  is originally based on the results of the laboratory cyclic loading tests with a subgrade CBR of approximately 3. For the subgrade  $CBR_{sb} = 1.3$  in this case,  $k_2$  should be adjusted by using a modification coefficient  $(3/CBR_{sb})^n$ . Three different  $n$  values ( $n = 0, 0.5$  and  $1.0$ ) have been used for the modification. The predicted base course thickness after modification and the test results are shown in Figure 74 and 75 for the unreinforced cases and the reinforced cases. It seems that the predictions with  $n = 0.5$  matched the test results better than the predictions with  $n = 0$  and  $n = 1.0$ . And the prediction with  $n = 0.5$  will be used to compare with the test results and the predicted results from other design method.

The design methods of Hammit (1970), Giroud and Noiray (1981) and the one proposed in this study have been applied for the unreinforced case. Testing results and computed results from the three design methods are presented in Figure 76. Hammit (1970) method and Giroud and Noiray (1981) method produced similar results, which underestimated the thickness of the base layer. The proposed design method provided a better prediction, with the computed results reasonably matching with the measured data for rutting depths of 0.075m and 0.10 m.

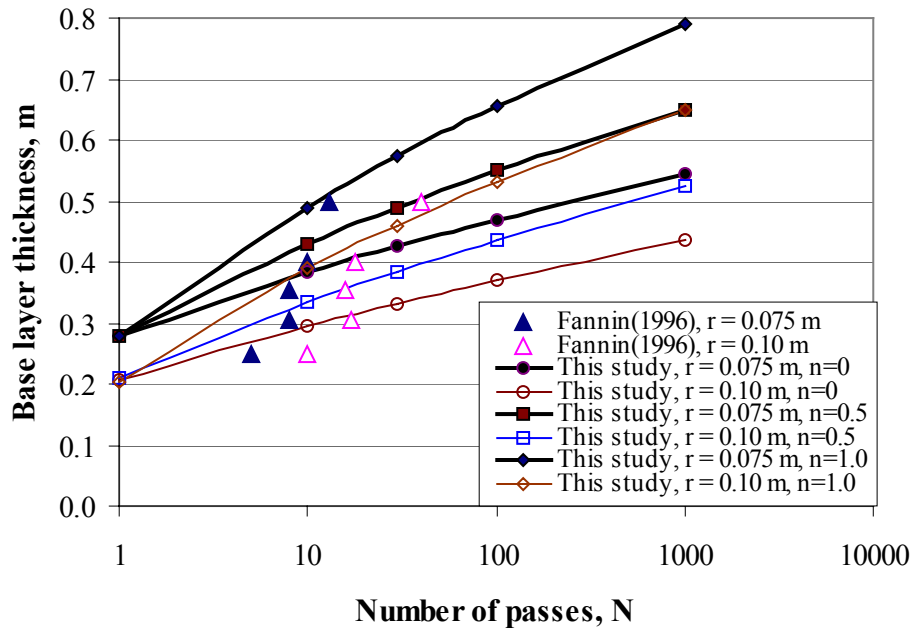


Figure 74. Modification of  $k_2$  for the unreinforced cases

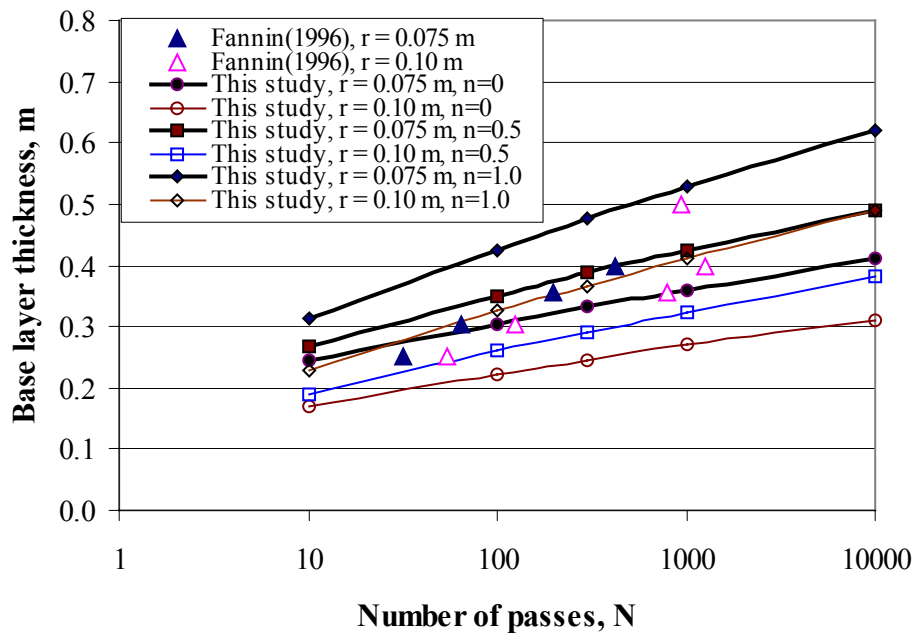


Figure 75. Modification of  $k_2$  for the reinforced cases

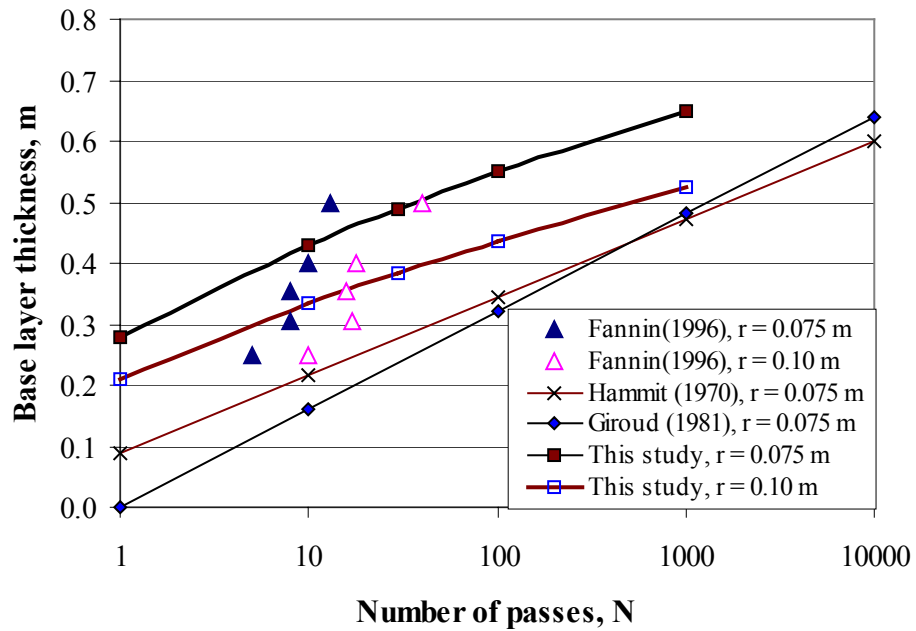


Figure 76. Base layer thickness vs. number of passes for the unreinforced cases

The design method from Giroud et al. (1984) and the proposed design method have been applied for the reinforced case with BX1100 geogrid. Testing results and computed results from the two design methods are presented in Figure 77. Similar to the reinforced cases, Giroud (1984) method underpredicted the base layer thickness. The proposed design method provided the better prediction as the predicted base course thickness matched reasonably well with the testing results for rutting depths of 0.075m and 0.10 m.

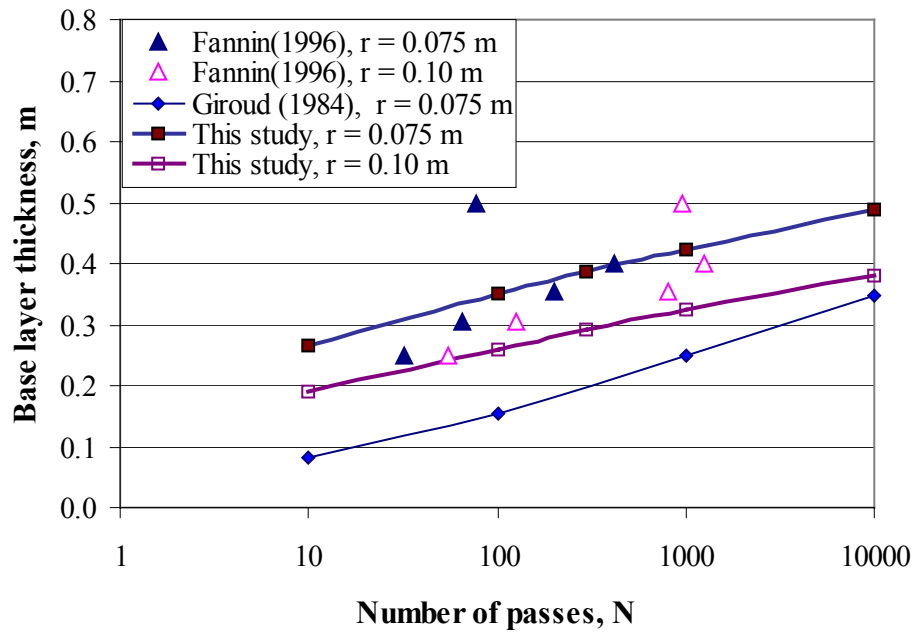


Figure 77. Base layer thickness vs. number of passes for the reinforced cases with BX1100 geogrid reinforcement

## 6.4 Summary

The performance of geogrid-reinforced unpaved road has been analyzed with consideration of geosynthetic/subgrade interaction and geosynthetic/base aggregate interaction. The geogrid reinforcement mechanisms are presented as the improvement of subgrade bearing capacity, the improvement of vertical stress distribution on the subgrade due to the lateral confinement of base course, the interface shear stress support and the tension membrane effect. Based on the study of the degradation of unpaved road from the laboratory test and the analysis of the subgrade bearing capacity and the mobilization of subgrade bearing capacity, a new design method was developed for both unreinforced and reinforced unpaved road. The new design method incorporates the base course property, the mobilization of subgrade bearing capacity with rutting depth, the degradation of the stress attenuation ability with cyclic load, and the effect of the geogrid inclusion. This design method has been verified by the field test study of Fannin and Sigurdsson (1996), with the predicted base course thickness matching well with the test results.

## **Chapter 7 SUMMARY CONCLUSIONS, AND CONTRIBUTIONS:**

### **7.1 Summary**

Past work in literature showed that geogrids could improve the behavior of the unpaved road under cyclic load. Induced reinforcement mechanisms due to geogrid inclusion included: lateral confinement, membrane effect and attenuation of stresses transmitted to subgrade. The contribution of the geogrid reinforcement can be specifically characterized by a wider stress attenuation angle, improvement in aggregate deterioration rate and modulus, tensioned membrane support, and an increase in subgrade bearing capacity. The current design method of reinforced unpaved structure is based on plain strain analysis with empirical relationships for effect of repeated traffic load. In general, there is a dearth of systematic laboratory and field test data to establish a model of reinforcement contribution to support of unpaved structures based on the soil properties, geogrid properties, and loading condition. The interaction of geogrid and aggregate base course, as well as degradation aspects under the cyclic loading, are not fully understood, and a comprehensive analysis model is lacking.

This research is undertaken to model the reinforced unpaved structure under cyclic load through laboratory cyclic plate load tests, numerical finite element analysis, and theoretical development to establish a design model. Fifteen laboratory large-scale cyclic load plate tests were conducted on unpaved structure sections with two base course thicknesses and several geosynthetic reinforcements. The subgrade soil was clayey sand with a laboratory-measured CBR of 3. The geosynthetic reinforcement was placed between subgrade and aggregate base course. The processes included measuring the stress deformation response of test section under cyclic load test, degradation and plastic deformation analysis of the test data, and study of interaction of aggregate-geogrid-subgrade system and the contribution of geogrid reinforcement mechanisms to stress distribution using finite element method. The performance of a geogrid-reinforced unpaved structure tested was numerically simulated by using the FEM program ABAQUS. The degradation of modeled sections was presented as the decrease of elastic

modulus ( $E_1/E_2$ ) or stress spread ability of the base layer ( $\tan \alpha$ ) with number of load cycles. A proposed design model to predict layer thickness and reinforcement grade based on given deformation criterion was developed and applied to field data by Fannin and Sigurdsson (1996).

## 7.2 Conclusions

Based on the results of this study, the following conclusions are drawn:

1. Degradation occurred during the cyclic tests, which was seen as increase of vertical stress magnitude on the subgrade.
2. Test results indicated that reinforcement improved the stress and deformation responses by improving stress distribution transferred to the subgrade, and decreasing of degradation of the base course stress attenuation ability and surface deformation accumulation.
3. The improvement in stress distribution due to geosynthetic inclusion at the interface of ABC and subgrade was manifested by a decrease in the maximum stress (at the center) and a more uniform stress distribution on the subgrade soil.
4. Geogrids reduced plastic surface deformation of tested sections, including vertical deformation on the subgrade and lateral spread of aggregate base course.
5. Higher modulus geogrids (BX1200 and BX4200) showed better stress attenuation effect and reduced plastic surface deformation, compared to lower modulus geogrids (BX1100 and BX4100).
6. The degradation of tested sections was also related to the thickness of the base layer and the interaction between base course aggregate and geogrid (interlock effect). The degradation of base layer and permanent surface deformation were correlated to geogrid torsional stiffness.
7. The FEM study indicated that geogrid reinforcement placed between the base layer and the subgrade layer can provide lateral confinement at the bottom of the base layer by improving interface shear resistance and increasing mean stress at the bottom of the base layer.

8. As the ABC thickness decreased, or the elastic modulus ratio decreased, the benefit due to geogrid reinforcement becomes more apparent. In general, geogrid with higher tensile modulus and better interface property with base course aggregate showed better reinforcement effect.

The effects of reinforcement were related to the ABC thickness, geogrid tensile modulus, geogrid-ABC interface property, and the elastic modulus ratio of base layer and subgrade. The geosynthetic/subgrade interaction and geosynthetic/base aggregate interaction have been theoretically analyzed by presenting the geogrid reinforcement mechanisms as improvement of subgrade bearing capacity, and improvement of vertical stress distribution on the subgrade due to the lateral confinement of base course, the interface shear resistance and the tension membrane effect. A new design method was developed for unpaved road based on the analysis of geogrid reinforcement mechanisms, degradation of base course, and the mobilization of subgrade bearing capacity. The estimation of the required base course thickness using this proposed design method matched well with the field test results from Fannin and Sigurdsson (1996).

### **7.3 Contributions**

Based on cyclic plate load tests, analytical and numerical analyses of the test data, a method of modeling the performance of unpaved structures has been developed. The contributions to the state of art are summarized as follows:

- Characterization of the stress deformation response of unreinforced and reinforced test sections under varying key parameters (thickness of ABC and geogrid grades) from experimental program;
- Development of analytical relationship to evaluate the degradation of base course and the plastic surface deformation of unpaved structures under cyclic load. The developed relationship accounts for thickness of base course, properties of base course and subgrade, and geogrid-aggregate interaction;
- Characterization of aggregate-geogrid-subgrade interaction by using test data and FEM analysis results. A analytical model explained the improvement of subgrade

bearing capacity, mobilization of subgrade bearing capacity, and the vertical stress on the subgrade with consideration of: i.) lateral confinement of base course, ii.) interface shear strength support, and iii.) tension membrane effect;

- Advancement of a proposed design method for unpaved road design incorporating base course degradation effect, subgrade bearing capacity, and vertical stress transferred to the subgrade. The analytical model is tested against field results performed by Fannin and Sigurdsson (1996) and is presented in a form useable by the engineering community involved in the design and construction of unpaved structures.

## **7.4 Recommendations**

Additional data are necessary to further verify the model for unpaved structures, including the empirical equations of degradation and plastic deformation related to plastic deformation torsional stiffness, and the modifications required for applying them in the situations with different subgrade soils. Additional research effort is also needed to investigate and model the permanent deformation of unpaved roads, and the effect of other geosynthetic functions (such as separation and filtration) on the performance of unpaved roads.

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