ABSTRACT


Existing soft subgrade soils are common in road construction projects. Long term serviceability of the roads depends on the performance of the quality of the subgrade soils. Subgrade soil with poor shear strength and stiffness properties can lead to excessive rutting and cracking of the asphalt layer. In place soil should be able to sustain heavy construction vehicles to avoid interrupting the construction. In addition to short term stability, soft subgrade can cause a significant amount of deformation under traffic load. The properties of subgrade soils can be evaluated by conventional in-situ tests such as Dynamic Cone Penetrometer (DCP), Light Weigh Deflectometer (LWD), Falling Weight Deflectometer, or by static and dynamic laboratory tests including monotonic Triaxial and Resilient modulus tests. Mechanical properties of the weak subgrade soils can be improved by replacing in place soil with higher strength and stiffer soils (select fill), and geosynthetics reinforcement.

This study investigates the performance of the geosynthetics-reinforced unsaturated subgrade soils and ABC, under cyclic loading, through field testing, laboratory measurements, and numerical analyses. Three test pads with comparable subgrade conditions were constructed in the Piedmont geologic area of North Carolina, using different subgrade stabilization measures, including select fill material, and geosynthetic reinforcement (Geogrid BX1200 and Geotextile HP570) together with a relatively thin layer of aggregate base course (ABC). Field loading was applied using 1,000 passes of a loaded construction truck. Several parameters were monitored during loading, including surface deformation, stress, deformation, and moisture and suction levels. Numerical analyses were performed using PLAXIS 3D software to study the deformation and stress distribution performance in the reinforced unsaturated subgrade soil under cyclic traffic loading within the context of advanced soil constitute model.

The experimental and FEM results indicated that measured vertical stress at the interface of the subgrade in the geosynthetic-reinforced sections increased and then declined with a number of cycles due to the mobilization of force in geogrid, as well as variation of modulus ratio of ABC to the subgrade. It was determined that the matric suction state of the ABC layer have a significant effect on the surface deformation of unreinforced unsaturated subgrade
under cyclic loading. FEM results indicated that the mobilized force in reinforcement layer, increased by a number of load cycles, as results of cumulative plastic deformation under cyclic loading. It was observed that an increase in ABC suction, escalates the mobilized force until a certain suction state, and then starts to decline. The numerical results showed that ABC layer behaves increasingly as a beam as the suction increases due to an increase in both the strength and stiffness properties of ABC material.

Numerical results indicated that regardless of ABC layer thickness, for a given circular loaded area, the surface deformation is minimized when the reinforcement is embedded at the depth equal to half of the radius of the loaded area, (D/r = 0.5). Analysis results indicated that required thickness of ABC is reduced when the reinforcement layer is implemented at the depth at which the maximum vertical strain occur right. It was observed from the numerical results that the higher tension force is mobilized in the reinforcement element when it is placed at the depth that the maximum vertical strain occurs, (D= 0.5r). On the other hand, the FEA results indicated that reinforcement layer can be ineffectual if it is placed at the interface between the ABC and the subgrade layer as is traditionally the case. This analysis approach however does not take “separation” benefits into consideration.

A novel approach for predicting resilient modulus of subgrade soils at various stress level based on light weight deflectometer (LWD), and dynamic cone penetrometer (DCP) data is introduced and validated by field measured data, and independent data from other studies reported in the literature. The proposed LWD and DCP models were shown to be capable of predicting the resilient modulus of low plasticity soils SM, ML, and SC (A-4a and A-4), with PI<5, and 40%<P200<55%; as a function of the stress state. Such values can be used along with the subgrade’s shear strength to discern the need for undercutting based on the criteria presented in Borden et al 2010.

Validity of existing stabilization criteria in the literature were examined for the Residual Piedmont subgrade soil, by utilizing field measured data. A new subgrade stabilization recommendation chart was developed for layered subgrade profile, based on the incorporation of in-situ DCP measurement and computed deformation response of subgrade soil from advanced 3D numerical analyses, under cyclic loading corresponding to the proof roller passes with taking into account the effect of the superposition of the multi-loaded areas. It was determined that the failure deformation occurs at DCPI value of 38 mm/blow for single layer
subgrade soil with the Mr/E50 of 6; which is consistent with NCDOT subgrade undercut criteria. A subgrade stabilization recommendation chart was developed based on the criterion of 25 cm plastic rut depth on top of the subgrade after two passes proof roller and incorporation of in-situ DCP measurement and numerical proof roll test results. Strong agreement were obtained with the 38 mm/blow NCDOT undercut criteria. The numerical results indicated that the subgrade with 30 cm stiff layer (DCPI < 20 mm/blow) on top does not require any subgrade soil stabilization, regardless of subgrade layers underneath, while subgrade needs to be stabilized if there is a very soft soil layer top 30 cm of subgrade (DCPI > 60 mm/blow), unless the sublayers underneath are composed of stiff soil (DCPI < 20 mm/blow).
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Experimental and Numerical Investigation of the Performance of Geosynthetics-Reinforced Unsaturated Subgrade Soils Under cyclic Loading

by
Seyyed Hamed Mousavi

A dissertation submitted to the Graduate Faculty of North Carolina State University in partial fulfillment of the requirements for the degree of Doctor of Philosophy

Civil Engineering

Raleigh, North Carolina

2016

APPROVED BY:

_______________________________
Dr. Mohammed Gabr
Co-Chair of Advisory Committee

_______________________________
Dr. Roy H. Borden
Co-Chair of Advisory Committee

_______________________________
Dr. Brina M. Montoya

_______________________________
Dr. Shamimur M. Rahman
DEDICATION

To my Mom and Dad, my very first teachers, for all sacrifices that they have made and guidance they gave me when I needed it most. I hope that this achievement will put a smile on their beautiful faces.

To my brothers and lovely only sister, Hamid Habib, and Bita, for supporting and encouraging me to believe in myself.

To my lovely angel, Atefeh, who was a charming companion during this journey.
BIOGRAPHY

Hamed Mousavi was born in a beautiful mountain city in eastern part of Iran, Hamadan, on May 1986. He was accepted to join to National Organization for Development of Exceptional Talents; where built the foundation of his engineering mindset, and he earned his high school diploma in mathematics and physic. Summer 2004 he was stood at rank 192nd among more than half a million students who took the university entrance exam, and enrolled in Civil Engineering at Sharif University of Technology. Five years later, after he earned his Bachelor degree, he pursued his education in Geotechnical Engineering at the Sharif University of Technology under the direction of Dr. Jafarzdeh by investigating mechanical properties of the plastic concert which used in the earth dam foundations. In August 2012 he moved to the United States and enrolled at North Carolina State University, in Raleigh to pursue his Ph.D. degree in Civil Engineering under the guidance of Prof. Gabr, and Borden
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CHAPTER 1: INTRODUCTION

Existing soft subgrade soils are common in road construction projects. Long term serviceability of the roads depends on the performance of the quality of the subgrade soils. Subgrade soil with poor shear strength and stiffness properties can lead to excessive rutting and cracking of the asphalt layer. In place soil should be able to sustain heavy construction vehicles to avoid interrupting the construction. In addition to short term stability, soft subgrade can cause a significant amount of deformation under traffic load. The properties of subgrade soils can be evaluated by conventional in-situ tests such as Dynamic Cone Penetrometer (DCP), Light Weigh Deflectometer (LWD), Falling Weight Deflectometer, or by static and dynamic laboratory tests including monotonic Triaxial and Resilient modulus tests. Mechanical properties of the weak subgrade soils can be improved by replacing in place soil with higher strength and stiffer soils (select fill), geosynthetics reinforcement and chemical stabilization.

1.1 Background

A country’s economic development is highly related to its transportation infrastructure. In the USA, trucks carry 60% of total shipments by weight and 70% by value (Palmeira and Antunes 2010). Therefore every year significant amount of resources is allocated for the maintenance and construction of pavements (FHWA 2006). If road construction over weak subgrade soil is unpreventable, the mechanical properties of subgrade soil need to be improved to increase the bearing capacity and avoid developing excessive plastic deformation under traffic load. In general, for addressing these situations, the undesirable subgrade soil is excavated (undercut) and replaced with other soil material, which has better mechanical properties that meet specifications or, alternatively, use chemical stabilization to reduce soil plasticity and improve strength and
workability, or geosynthetics reinforcement to reduce transferred stress to the weak subgrade (Koerner 2012).

Thus as a first step, it needs to specify whether or not any subgrade soil improvement is required. Hence the shear strength and stiffness properties of subgrade soils need to be determined precisely to prevent over cost the project. Over the years, several criteria have been proposed which evaluate the stability of subgrade soil based on the in-situ testing measurement, such as dynamic cone penetrometer, DCP, light weight deflectometer, LWD, and Proof roll test. NCDOT 38 mm/blow undercut criteria have been used for decades as a conventional approach to evaluate the stability of the subgrade soil. Borden et al. (2010) proposed undercut criteria based on Dynamic Cone Penetrometer (DCP) testing, and extensive numerical analyses. The proof roller test has also been carried out as a technique for subgrade quality assessment in road construction for decades. Hambleton and Drescher (2008) developed two theoretical criterions based on analytical and FEM methods, to relate surface plastic deformation of single homogenous subgrade layer to wheel geometry, wheel load, and soil strength parameters (friction angle and cohesion), while the soil stiffness plays a secondary role. White et al. (2009) performed a series of in-situ DCP test conjunction to the proof roll test. Although existing criteria fully take into account the shear strength properties of subgrade soil, however, further investigation needs to be performed to evaluate the performance of multi layered subgrade soils within the context of advanced constitutive soil model, under cyclic proof load with taking into account the effect of the superposition of the multi-loaded areas.

Resilient modulus has been used for decades as an important parameter in pavement structure design (AASHTO 1993, NCHRP 2004). The use of resilient modulus ($M_r$) has been substituted for the California Bearing Ratio (CBR) in pavement design in order to consider the deformation
behavior of base and subgrade layers under cyclic loading condition. The stiffness properties of soils depend on the soils physical properties such as density, the applied stress state, as well as matric suction state (Li 1994). The resilient modulus, \( M_r \), is determined directly from the laboratory resilient modulus tests. Although laboratory test provides most confident results but it requires well-trained operator and substantial time as well as advanced apparatus. An alternative to laboratory testing is empirical correlations developed from statistical analyses on mechanical and physical properties of soils and laboratory-measured resilient modulus. Carmichael et al. (1985), Elliott et al. (1988), Drumm et al. (1990), Farrar and Turner (1991), Hudson et al. (1994) proposed models to estimate the resilient modulus of subgrade soils from index properties. A shortcoming of these model is that they are not capable of capturing either stress dependency or undisturbed properties of soils. In order to taking into account the properties of undisturbed soils in the resilient modulus prediction, Hasan et al. (1994), MEPDG (2004), Herath et al. (2005) and Mohammad et al. (2008), proposed correlations to predict \( M_r \) from in-situ testing, dynamic cone penetrometer (DCP) measurement. White et al. (2007) and Mohammad et al. (2008), proposed correlations to estimate the resilient modulus of subgrade soil by LWD data. Although these empirical correlations consider in-situ properties of soils, the validity just at the one specific stress state, makes these models ineffectual for the other load combinations. Many comprehensive studies have been performed over the past two decades to model the stress dependency of the resilient modulus by predicting the coefficients of the universal constitutive models (e.g. NCHRP project 1-28A 2004) from the basic soils properties such as water content, plastic limit, liquid limit, \( P_4 \), \( P_{200} \) etc. Yau and Von Quintus (2002), Elias and Titi (2006), and Nazzal and Mohammad (2010), proposed different models to estimate NCHRP project 1-28A constitutive model coefficients \((k_1, k_2 \text{ and } k_3)\), Equation 1-1, from the physical properties of soils. Despite
consideration the stress dependency of the resilient modulus, these models do not consider in-situ properties of soils. Therefore new models are demanded to not only include properties of undisturbed in place soils but also can be capable of predicting the resilient modulus at any desired stress state.

\[
\mathrm{M}_r = k_1 \left( \frac{\theta}{P_a} \right)^{k_2} \left( \frac{\tau}{P_a} + 1 \right)^{k_3},
\]

Eq. (1-1)

Where:

\( \mathrm{M}_r \): resilient modulus

\( P_a \): atmospheric pressure

\( \sigma_1, \sigma_2, \sigma_3 \): principal stresses

\( \theta = \sigma_1 + 2\sigma_3 \): bulk stress

\( \tau_{oct} = \frac{\sqrt{2}}{3} (\sigma_1 - \sigma_3) \): octahedral shear stress

\( k_i \): regression constants

As previously mentioned, the resilient modulus of soil, as well as its shear strength properties, depend on the matric suction state (Li 1994). The Shear modulus at very small strain (G\(_{\text{max}}\)) increases with an increase in matric suction state (We et al. 1984, Qian et al. 1993). The effect of the matric suction is more pronounce on the maximum shear modulus of fine grained soils (Picornell and Nazarian 1998). Hence taking into consideration the matric suction level leads to the most accurate evaluation of the stiffness of subgrade soil. Thus, effects of subgrade matric suction level on the performance of un/reinforced subgrade and required thickness of base layer need more investigations to be addressed comprehensively.

Geosynthetics have been used in un/paved structure for decades as filtration, separation, and reinforcement, to improve mechanical properties of the subgrade soils, reduce the amount of the
cumulative rut depth, reduce required thickness of base layer, and increase the service life, by several mechanisms (Koerner 2012, Leng and Gabr 2005, Fannin and Sigurdsson 1996, Cowell et al. 2012, Brown et al. 2007, Mekkawy et al. 2011). Geogrid and geotextile are two types of geosynthetics that frequently used in pavement engineering.

Geotextiles form one of the two largest groups of the geosynthetics which can be utilized as filtration, drainage, separation, and reinforcement (Koerner 2012). Geotextile reinforcement provides better deformation performance compared to unreinforced subgrade, due to spreading the applied pressure on the larger area and reducing the transferred load to the subgrade as well as tensile membrane reaction, however the membrane effect is minor if the rut depth is small and it becomes significant only when a very large rut depth is reached (Giroud and Han 2004, Burd 1994, Espinoza 1993).

Another common category of the geosynthetics materials, is geogrids which increase bearing capacity of the un/paved structure and reduce permanent deformation by the same mechanisms provided by geotextile as well providing interface shear resistance due lateral confining ABC particles (Qian 2013, Han and Bhandari 2010, Giroud and Han 2004, Brown 2007). Confinement can reduce the vertical deformation of subgrade, increase the bearing capacity from elastic limit to ultimate state (i.e. plastic limit). As a result, the failure mode changes from local shear to general shear failure.

Over the last two decades, many studies were dedicated to investigate mechanism and performance of the geosynthetic-reinforced un/paved sections through experimental and numerical methods. It has been justified that inclusion of geosynthetic reinforcement reduces cumulative vertical deformation and applied stress on the top of the subgrade significantly, compare to the unreinforced section (Fannin and Sigurdsson 1999, Tingle and Webster 2003, Hufenus 2006, Tingle
and Jersey 2009, Cowell 2012). The contribution of geosynthetics-reinforcement in improving deformation behavior of the subgrade is intensified with a thinner thickness of aggregate base course (Fannin and Sigurdsson 1996, Hufenus 2006).

Along full scale testing studies, researchers have been performed extensive numerical analyses on the performance of geosynthetics-reinforced subgrade. The FEM analyses were performed under static load or a limited number of load cycles. The shear strength and stiffness properties of base and subgrade materials were input into to FEM analyses within the context of the soil constitutive models such as Isotropic linear elastic, Mohr-Coulomb, Drunker-Prager and Cam-Clay (Miura et al. 1990, Dondi 1994, Wathugala et al. 1996, Leng and Gabr 2005, Nazzal et al. 2010). The reported numerical results confirm full scale testing observations.

Despite extensive contributions of the previous studies on understanding the mechanisms of geosynthetics reinforced subgrades and their deformation behavior, the effects of the matric suction state on deformation behavior under cyclic loading, has not fully addressed yet. In addition, the numerical analyses of reinforced subgrades have been performed so far by utilizing conventional constitutive model (i.e. Mohr-Coulomb) under static load or a limited number of load cycles, which not able to capture stiffness reduction of soil with an increase of strain level and provide realistic results.
1.2 Objectives

This study investigates the performance of the geosynthetics-reinforced unsaturated subgrade soils and ABC, under cyclic loading, through field testing, laboratory measurements, and numerical analyses. The objectives of this study can be advanced as follow:

1. Assess performance of the reinforced unpaved sections, under cyclic loading within the context of the advanced soil constitutive model that is capable of simulating stress and strain dependency of the soil stiffness, by 3D FEM analyses, validated by full scale testing measurements.

2. Investigate effects of the ABC layer matric suction on the deformation performance of the un/reinforced subgrade soil, under cyclic loading.


4. Develop a new model based on the dynamic cone penetrometer test data that is capable of estimating the resilient modulus of soils at various stress state by estimating MEPDG constitutive model coefficients (k1, k2, and k3).

5. Develop a new approach in the resilient modulus prediction from in situ testing: light weight deflectometer, which not only it eliminates uncertainties associated with selecting a Poisson’s ratio and shape factor, but also capable of predicting the Mr at any desired load combination by predicting MEPDG constitutive model coefficients (k1, k2, and k3).

6. Develop a subgrade stabilization criteria by fully taking into account the shear strength and stiffness properties of layered subgrade soil by using in-situ measurement, and based on the estimation of deformation performance under proof roll test.
1.3 Layout

The results and finding of this study are presented within 7 chapters as follows:

Chapter 2. Details of the full scale testing, including site description, subgrade, aggregate base course, and reinforced sections’ properties and instrumentations. Numerical analyses results on effect of ABC suction of the deformation performance of un/reinforced unpaved road/

Chapter 3. Numerical investigation results on factors affecting the performance of a geogrid-reinforced granular base material under cyclic loading are presented.

Chapter 4. Laboratory and in-situ testing program results including tested material properties, resilient modulus test results, as well as subgrade resilient modulus prediction model from LWD measurements are introduced in this chapter. The proposed models are validated by data reported herein and in the literature.

Chapter 5. Laboratory resilient modulus and in-situ testing DCP measurement are presented. An empirical correlation for estimation of resilient modulus from in-situ DCP data is introduced in this chapter.

Chapter 6. A subgrade stabilization recommendation chart which was developed based on the criterion of 25 cm plastic rut depth on top of the subgrade after two passes proof roller and incorporation of in-situ DCP measurement and numerical proof roll test results, is presented.

Chapter 7. A summary of the research results and findings and conclusions of the study. Recommendations for future works are stated in this chapter.
References


CHAPTER 2: NUMERICAL AND EXPERIMENTAL INVESTIGATION OF GEOSYNTHETICS REINFORCED UNSATURATED UNPAVED ROAD

S. Hamed Mousavi, Corresponding Author
North Carolina State University
Graduate Research Assistant, Department of Civil, Construction, and Environmental Engineering, North Carolina State University, Raleigh, NC 27695-7908;
Tel: 919-995-8792; Email: smousav3@ncsu.edu

Mohammed A. Gabr
North Carolina State University
Professor, Department of Civil, Construction, and Environmental Engineering, North Carolina State University, Raleigh, NC 27695-7908;
Tel: 919-515-7904 FAX: 919-515-7908; Email: gabr@eos.ncsu.edu

Roy H. Borden
North Carolina State University
Professor, Department of Civil, Construction, and Environmental Engineering, North Carolina State University, Raleigh, NC 27695-7908;
Tel: 919-515-7630 FAX: 919-515-7908; Email: borden@ncsu.edu
ABSTRACT

The work described in this paper focuses on the deformation and stress distribution performance of geosynthetics reinforced unsaturated residual subgrade soil. Three test pads with comparable subgrade conditions were constructed in the Piedmont geologic area of North Carolina, using different subgrade stabilization measures, including select fill material, and geosynthetic reinforcement (Geogrid BX1200 and Geotextile HP570) together with a relatively thin layer of aggregate base course (ABC). Field loading was applied using 1,000 passes of a loaded construction truck. Several parameters were monitored during loading, including surface deformation, stress, deformation, and moisture and suction levels. Numerical analyses were performed using PLAXIS 3D software to study the deformation and stress distribution performance in the reinforced unsaturated subgrade soil under cyclic traffic loading within the context of advanced soil constitutive model. The experimental and FEM results indicated that measured vertical stress at the interface of the subgrade in the geosynthetic-reinforced sections increased and then declined with a number of cycles due to the mobilization of force in geogrid, as well as variation of modulus ratio of ABC to the subgrade. It was determined that the matric suction state of the ABC layer have a significant effect on the surface deformation of unreinforced unsaturated subgrade under cyclic loading. FEM results indicated that the mobilized force in reinforcement layer, increased by a number of load cycles, as results of cumulative plastic deformation under cyclic loading. It was observed that an increase in ABC suction, escalates the mobilized force until a certain suction state, and then starts to decline. The numerical results showed that ABC layer behaves increasingly as a beam as the suction increases due to an increase in both the strength and stiffness properties of ABC material.

Keywords: geosynthetics, FEM, full scale, unsaturated, unpaved, reinforcement
2.1 Introduction

With increased construction activities associated with the development road network, it is common to encounter soft subgrade soils. The long term serviceability of a given road not only depends on the performance of the asphalt layer but is also tied to the quality of the subgrade layer. In situ subgrade soil layers should be able to support heavy construction vehicles to enable a given project to be completed on schedule. In addition to the short term stability, soft subgrade layers need to sustain the lifetime traffic loading without experiencing excessive deformation. Excessive deformation can lead to accelerated degradation of the asphalt layer and the need for emergency repairs and maintenance. One example where problem soils are often found is within the Triassic Basin area of the Piedmont Physiographic region of North Carolina. In general, these situations are addressed by excavating (undercut) the undesirable soil, and replace it with materials that meet high quality specifications or, alternatively, use geosynthetics reinforcement to reduce transferred stress into the subgrade, and laterally confine aggregate base course particles, and improve bearing capacity of the soft subgrade (Binquet and Lee, 1975; Perkins, 1999; Abu-Farsakh and Nazzal, 2009).

Over the years many studies have been performed to investigate the deformation behavior and stress distribution of the un/reinforced unpaved road from physical testing that includes small-scale and large-scale laboratory tests and full-scale field tests. Despite the high quality control that could be obtained in laboratory testing, boundary effects can affect the results (Cote 2009). In addition, the load conditions produced by full-scale moving tires cannot be perfectly replicated in most laboratory settings. Therefore, full-scale testing is carried out to simulate the actual load conditions and to mitigate or eliminate the boundary effects that can affect results.
The performance of different types of geosynthetic-reinforced subgrades has been evaluated and compared to that of unreinforced sections through several large and full-scale testing programs over the past decades (Fannin and Sigurdsson 1996, Hufenus et al. 2005, Leng and Gabr 2006, Tingle and Jerzy 2009, Chen et al. 2009, White et al. 2011, Al-Qadi et al. 2012, Cowell et al. 2012, Thakur et al. 2012, Saghebfar 2014, Tang et al. 2015, Sun et al. 2015). Field-scale and laboratory test results indicate that the inclusion of geosynthetic reinforcement reduces the magnitude of both the permanent deformation and applied stress that is transferred onto the reinforced subgrade (Fannin and Sigurdsson 1996, Tingle and Webster 2003, Hufenus et al. 2005, Tingle and Jersey 2009, Cowell et al. 2012). Early on, researchers also observed that the use of geosynthetics could potentially reduce the required base layer thickness (Fannin and Sigurdsson 1996). Geosynthetic inclusions have been shown to be more beneficial when the ABC layers are relatively thin; moreover, with an increase in the thickness of the ABC, the contribution of the geosynthetic inclusion to the reduction of rut depth development decreases (Fannin and Sigurdsson 1996, Hufenus et al. 2006). Tingle and Jersey (2009) also observed that base layer material becomes stiffer after trafficking due to the densification of the base materials and the mobilization of the geosynthetic reinforcement. Cowell et al. (2012) reported that even a small difference in base layer thickness could cause a significant effect on the measured stress at the subgrade surface.

Giroud and Han (2004) developed Equation 2-1, that presents the variation of stress distribution angle, $\alpha$, with respect to the number of load cycle (N), geogrid stiffness properties (k), modulus ratio of base layer to the subgrade soil ($R_E$), and stress distribution under static loading ($\alpha_0$). It was reported that for a constant modulus ratio value, the stress distribution angle decrease with a number of cycles, and applied stress on the top of subgrade increase.

$$\frac{1}{\tan \alpha} = \frac{1 + k \log(N)}{\tan \alpha_0 [1 + 0.204(R_E - 1)]}$$

Eq. (2-1)
Leng and Gabr (2006), proposed Equation 2-2, from the analytical analyses and experimental results, to estimate stress distribution angle at the given load cycles, $\alpha_N$, with taking into stress the distribution angle under static load, ($\alpha_1$), number of load cycle, $N$, geogrid properties, $k_2$.

$$\frac{\tan \alpha_N}{\tan \alpha_1} = \frac{1}{1 + k_2 \log N}$$  \hspace{1cm} \text{Eq. (2-2)}

Although existing models consider stress distribution angle variation with a number of traffic, modulus ratio of ABC layer to the subgrade material is a major factor which controls magnitude the transferred pressure into the subgrade and stress distribution angle under traffic load, hence its variation corresponding to the strain levels, needs to be addressed in detailed.

Along the experimental studies, extensive numerical studies have been performed over the past few decades to evaluate the performance of reinforced unpaved and paved roads using finite element/difference analyses (Miura et al. 1990, Doni, 1994, Whathugala et al. 1996, Perkin et al. 2001, Leng and Gabr 2005, Kwon et al. 2005, Abu-Farsakh et al. 2009, Nazzal et al. 2010, Perkin et al. 2011). Various constitutive soil models were used in these studies to model the deformation behavior of ABCs and subgrade soils. For example, Dondi (1994) obtained numerical analysis results that were in good agreement with field observations and indicated a significant increase in the bearing capacity of the reinforced subgrade compared to the unreinforced section used in that study. Dondi (1994) and Whathugala et al. (1996) showed through numerical analysis that geosynthetic inclusions produced up to 20 percent reduction in vertical deformation compared to an unreinforced subgrade. Leng and Gabr (2005) showed that the contribution of geosynthetic reinforcement to reducing rut depth increases as the modulus ratio of the ABC to the subgrade decreases. Nazzal et al. (2010) found that the benefits of reinforcement are more pronounced in reducing permanent deformation than a recoverable strain. Based on analysis of these earlier studies, this study found that the deformation behavior and stress distribution within reinforced
unsaturated base/subgrade materials under cyclic loading have not yet been fully addressed. Furthermore, no explicit consideration of the base/subgrade matric suction seems to have been taken into account in evaluating the deformation behavior of pavement systems.

2.2 Background

As previously stated, the contribution of the matric suction of ABC layer into the deformation performance of un/reinforced unpaved section was investigated in this study. Matric suction of subgrade and base materials improve their shear strength and stiffness properties. Influence of matric suction on shear strength of soils can be presented within the contexts of three proven frameworks in the literature (Nuth and Laloui, 2007): Bishop’s (1960) single effective stress, independent state variable approach, Fredlund and Morgenstern (1977), Critical state concept proposed by Alonso et al. (1990), and developed constitutive model by (Sheng, 2011 and Toll, 1990). PLAXIS 3D EA software, used in this study, has the capability to incorporate the soil stiffness’s nonlinear dependency on the strain level, as well as simulate the stiffness hardening of soils that is due to an increase in confining pressure, within the advance soil constitutive model: Hardening Soil small-strain. Influence of the matric suction on the performance of unsaturated soil can be taken into account in PLAXIS through advanced analysis mode, by implementing Bishop’s single effective stress theory. Based on the Bishop’s approach, effective stress can be computed in unsaturated materials, by application of the matric suction into the extended from of the Terzaghi’s principle of effective stress for saturated soils and introducing a new parameter, $\chi$, as expressed in Equation 2-3:

$$\sigma' = (\sigma - u_a) + \chi (u_a - u_w)$$  \hspace{1cm} \text{Eq. (2-3)}

Where:
(\sigma - u_a): net normal stress

(u_a - u_w): matric suction

\( \chi \) is a parameter that represents the contribution of the matric suction state into the effective stress, which depends on soil grain size distribution, and the degree of saturation. Many studies have performed over the last years to investigate the value of \( \chi \), a parameter based on the degree of saturation. Lu et al. (2010) proposed that \( \chi \) can be substituted by the effective degree of saturation \( S_e \) from Van Genuchten’s SWCC model (1980), as presented in Equation 2-4:

\[
\chi = S_e = \frac{\theta - \theta_r}{\theta_s - \theta_r} = \frac{1}{[1 + (a \psi)^n]^m}
\]

Eq. (2-4)

The soil water characteristic curve (SWCC) correlates the soil matric suction state with the volumetric/gravimetric moisture content or degree of saturation. Several mathematical models have been developed over the last several decades to develop the SWCC from a limited number of points. For example, Fredlund and Xing (1994) proposed Equation 2-5:

\[
\theta(\psi, a, n, m) = C(\psi) \frac{\theta_s}{\{\ln[1 + (\psi/\psi_r)^n]\}^m}
\]

Eq. (2-5)

where:

\[
C(\psi) = 1 - \frac{\ln[1 + (\psi/\psi_r)]}{\ln[1 + (10^6/\psi_r)]}
\]

Eq. (2-6)

Van Genuchten (1980) proposed Equation 2-7 for developing the SWCC. PLAXIS software uses Van Genuchten’s model for unsaturated soil analysis.

\[
\theta = \theta_r + \frac{\theta_s - \theta_r}{[1 + (a \psi)^n]^m}
\]

Eq. (2-7)
Where:

\[ m = 1 - \frac{1}{n} \quad \text{Eq. (2-8)} \]

Ba et al. (2013) reported results regarding the SWCCs of different types of ABC. They developed and constructed the SWCCs based on laboratory suction measurements obtained using Fredlund and Xing’s model and Van Genuchten’s model. The results presented by Ba et al. (2013) are used here in the numerical analyses to estimate the SWCCs of the aggregate base materials utilized in the present study.

Wang (2014) performed a comprehensive study of the SWCCs of Piedmont residual soils near Greensboro, North Carolina. Wang conducted a series of pressure plate tests and tensiometer suction measurements to develop the SWCCs of natural A-7-5 soils. Because the full-scale testing took place in the same Piedmont geologic region that is the focus of this study, the SWCCs for the A-7-5 soil developed by Wang (2014) are used for the numerical analyses in this study. The developed SWCCs were validated using laboratory tensiometer matric suction measurements of field-obtained Shelby tube samples.

Developing SWCCs through laboratory testing (pressure plate tests, etc.) is time-consuming and requires advanced equipment; thus, empirical correlations have been proposed in the literature to estimate the Fredlund and Xing or Van Genuchten model coefficients \((a, n, m)\) based on soil grain size distribution data. In order to develop an SWCC for the A-4 soil with a plasticity index (PI) < 4, this study employed the model proposed by Zapata et al. (2000) that predicts the SWCC from the index properties of soils using PI = 0. Equations 2-9 through 2-12 predict the Fredlund and Xing model coefficients for soils with PI = 0:

\[ a = 0.8627(D_{60})^{-0.751} \quad \text{Eq. (2-9)} \]

\[ n = 7.5 \quad \text{Eq. (2-10)} \]
\[ m = 0.1772 \ln(D_{60}) + 0.7734 \]  
\[ \psi_r = \frac{1}{a} \left( \frac{D_{50}}{9.7e-3} \right) \]

**2.3 Test Site**

The test site is part of an NCDOT road widening project (State Project R-2413C) located in Rockingham and Guilford Counties, North of Greensboro, North Carolina. Guilford County is located in the Piedmont region, which encompasses approximately the middle third of the State of North Carolina (Ogunro et al. 2008). The specific test area is located on ramp B that connects US 65 East to US 220 South at coordinates 36.268082 and -79.93053, as marked on Fig. 2-1.

![Figure 2-1. Test site location.](image)
2.4 Test Sections Configuration

Fig. 2-2 shows a plan view and cross section of the test area. The area that encompassed the three test sections was 45.7 m (200 ft) long by 4.9 m (16 ft) wide and was divided into three sections of 15.2 m (50 ft) long sections in order to compare the performance of the different types of subgrade soil stabilization. The test sections were constructed in the following order: I) Section 1 was designed to be undercut by 79 cm (31 in.) and backfilled with Class II select fill material based on the NCDOT specification. After analyzing the LiDAR scan data, the research team observed that 74 (29 in.) to 79 cm (31 in.) was excavated and backfilled with 76 (30 in.) to 84 cm (33 in.) of select fill. II) Section 2 was excavated 23 cm (9 in.) and reinforced with biaxial geogrid (BX 1200), GG. The section was backfilled with 19 (7.4 in.) to 29 cm (11.2 in.) of Class IV ABC. III) Section 3 was undercut by 23 (9 in.) and backfilled with 25 (9.8 in.) to 29 cm (11.6 in.) of Class IV ABC on the top of high strength polyester woven geotextile, GT.

![Test section configuration and boreholes and in-situ tests locations](image-url)

Figure 2-2. Test section configuration and boreholes and in-situ tests locations
2.5 Laboratory Testing

The laboratory tests included the determination of index properties, resilient module, and monotonic axial compression triaxial, were performed using samples retrieved from the test site using Shelby tubes. Locations of the holes which Shelby tubes were collected, are shown in Fig. 2-2.

2.5.1 Tested materials

The index property tests included natural water content, specific gravity, grain size distribution, Atterberg limits, and standard compaction were performed following ASTM D2216, D854, D6913, D4318, D1557, respectively, on resilient modulus or triaxial test specimens which retrieved from the Shelby tubes; select fill, and ABC material.

2.5.1.1 Subgrade soil

Fig. 2-3 shows the range of the grain size distributions for the all subgrade soil types encountered, as determined by the index properties. According to the AASHTO engineering soil classification system, i.e., the Unified Soil Classification System (USCS), the natural Piedmont residual soils are classified as A-4 (SM, ML, CL), soft low plasticity soils located at the top 80 cm (32 in.), and A-7-5 (MH), stiff high plasticity materials. Table 2-1 presents a summary of the index properties of the subgrade soil specimens.
Table 2-1. Summary of Sample Properties

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Depth 1 cm</th>
<th>γ_total 2 kN/m²</th>
<th>w % 3 natural water content</th>
<th>e 4 void ratio</th>
<th>Gs 5 specific gravity</th>
<th>LL</th>
<th>PL</th>
<th>PI</th>
<th>P200%</th>
<th>USCS</th>
<th>AASHTO</th>
</tr>
</thead>
<tbody>
<tr>
<td>H1-1</td>
<td>79</td>
<td>16.9</td>
<td>34.0</td>
<td>0.98</td>
<td>2.70</td>
<td>72</td>
<td>48</td>
<td>24</td>
<td>80</td>
<td>MH</td>
<td>A-7-5</td>
</tr>
<tr>
<td>H2-2</td>
<td>86</td>
<td>17.7</td>
<td>27.8</td>
<td>0.80</td>
<td>2.66</td>
<td>58</td>
<td>36</td>
<td>22</td>
<td>72</td>
<td></td>
<td></td>
</tr>
<tr>
<td>H2-1</td>
<td>18</td>
<td>18.7</td>
<td>14.4</td>
<td>0.51</td>
<td>2.64</td>
<td>15</td>
<td>12</td>
<td>4</td>
<td>41</td>
<td>SM</td>
<td>A-4</td>
</tr>
<tr>
<td>H3-1</td>
<td>8</td>
<td>18.0</td>
<td>16.8</td>
<td>0.61</td>
<td>2.65</td>
<td>20</td>
<td>17</td>
<td>3</td>
<td>49</td>
<td></td>
<td></td>
</tr>
<tr>
<td>H4-3</td>
<td>61</td>
<td>18.4</td>
<td>22</td>
<td>0.74</td>
<td>2.67</td>
<td>14</td>
<td>10</td>
<td>4</td>
<td>44</td>
<td></td>
<td></td>
</tr>
<tr>
<td>H6-1</td>
<td>8</td>
<td>18.3</td>
<td>17.0</td>
<td>0.58</td>
<td>2.65</td>
<td>13</td>
<td>10</td>
<td>3</td>
<td>42</td>
<td></td>
<td></td>
</tr>
<tr>
<td>H3-2</td>
<td>23</td>
<td>18.8</td>
<td>16.0</td>
<td>0.51</td>
<td>2.62</td>
<td>18</td>
<td>17</td>
<td>2</td>
<td>51</td>
<td>ML</td>
<td>A-4</td>
</tr>
<tr>
<td>H3-3</td>
<td>61</td>
<td>19.1</td>
<td>19</td>
<td>0.63</td>
<td>2.63</td>
<td>14</td>
<td>10</td>
<td>4</td>
<td>50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>H4-1</td>
<td>23</td>
<td>18.6</td>
<td>21.8</td>
<td>0.63</td>
<td>2.60</td>
<td>20</td>
<td>17</td>
<td>3</td>
<td>54</td>
<td></td>
<td></td>
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<td>H4-2</td>
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<td>15.4</td>
<td>0.47</td>
<td>2.62</td>
<td>19</td>
<td>16</td>
<td>3</td>
<td>55</td>
<td></td>
<td></td>
</tr>
<tr>
<td>H5-1</td>
<td>0</td>
<td>19.4</td>
<td>13.3</td>
<td>0.46</td>
<td>2.61</td>
<td>22</td>
<td>19</td>
<td>2</td>
<td>51</td>
<td></td>
<td></td>
</tr>
<tr>
<td>H6-2</td>
<td>23</td>
<td>18.8</td>
<td>21.2</td>
<td>0.59</td>
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<td>0.52</td>
<td>2.61</td>
<td>18</td>
<td>12</td>
<td>6</td>
<td>54</td>
<td></td>
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<tr>
<td>H5-2</td>
<td>30</td>
<td>19.7</td>
<td>16.5</td>
<td>0.44</td>
<td>2.64</td>
<td>25</td>
<td>16</td>
<td>8</td>
<td>42</td>
<td>CL</td>
<td>A-4</td>
</tr>
</tbody>
</table>

1 cm
2 kN/m², 3 natural water content, 4 void ratio, 5 specific gravity.
6 Hj: borehole number, j: sample number

Figure 2-3. Range of grain size distribution of the subgrade, select fill, and ABC materials
2.5.1.2 Aggregate Base Course

ABC Class IV with a specific gravity value of 2.7 was used in this project. Fig. 2-3 shows the grain size distribution of the ABC material. Table 2-2 presents the index properties of the ABC material, and the modified Proctor compaction results, performed according to ASTM D1577. According to the AASHTO engineering soil classification system (i.e., USCS), the ABC is classified as A-1 (GW), a well-graded gravel with silt and sand.

<table>
<thead>
<tr>
<th>Material</th>
<th>LL</th>
<th>PI</th>
<th>$G_s$</th>
<th>Maximum dry unit weight (pcf)</th>
<th>Optimum water content (%)</th>
<th>USCS</th>
<th>AASHTO</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABC</td>
<td>NA</td>
<td>Na</td>
<td>2.7</td>
<td>134</td>
<td>7.5</td>
<td>GW</td>
<td>A-1</td>
</tr>
<tr>
<td>Select fill</td>
<td>32</td>
<td>2</td>
<td>2.71</td>
<td>117</td>
<td>14.5</td>
<td>SM</td>
<td>A-4a</td>
</tr>
</tbody>
</table>

2.5.1.3 Select Material

Test Section 1 was stabilized by select fill material. Table 2-3 and Fig. 2-3 provide summaries of the index properties and grain size distribution of the select material, respectively. Based on the index properties, the select material is classified as A-4 and meet the NCDOT specifications for Class II select material. According to the USCS, the select material is classified as a silty sand (SM).

2.6 Instrumentation

A combination of 229 mm (9-in.) diameter Earth Pressure Cells, EPC, Geokon® Models 3500-2 and 3500-3 with pressure transducers with voltage 0-5 VDC and current 4-20 mA output, respectively (instruction manual 3500 EPCs, Geokone 2013), was used in this study. Four EPCs were embedded in the wheel paths (two in each wheel path) of each test section, 76 mm (3 in.) below the interface of the subgrade and stabilized depth, to monitor the applied stress at the top of
the subgrade soil induced by traffic loading, as shown in Fig. 2-4. Two moisture sensors and one suction sensor were installed in the middle of each test section to monitor and record the volumetric water content and matric suction of the subgrade layer. One 10HS Decagon® moisture sensor, shown in Fig. 2-4, was installed horizontally at a depth of 76 mm (3 in.) and one was installed vertically at a depth of 152 mm (6 in.) below the stabilized zone. One MPS-2 Decagon® suction sensor was installed 76 mm (3 in.) below the undercut/treated depth, as shown in Fig. 2-4, and 76 mm (3 in.) away from the horizontal moisture sensor.

![Figure 2-4. Sensors layout (Dimensions in cm)](image)

### 2.7 Performance Testing

A triaxle dump truck was loaded with stone and weighed on a certified truck scale in order to determine the load distribution on each axle. The total gross weight was determined as 244.5 kN (54,980 lb). By comparing the truck weight and load distribution on each axle to those used in a similar project (Cowell et al. 2012), the front and two rear axle loads were estimated as 73.4 and
171.2 kN (16,500 and 38,500 lb), respectively, which were assumed to be evenly distributed on the tires for each axle. The tire pressure was checked before and intermittently during and after testing and was set to ~ 586 kPa (85 psi). A total of 1000 consecutive passes were conducted on the test sections within approximately the same wheel path over a period of one month, as the project had to be paused due to the construction problem.

The equivalent single-axle load (ESAL) is used to convert the loading from various axle configurations and load magnitudes into an equivalent number of standard single-axle, 80 kN (18-kip) loads, for pavement design. The ESAL for each test section in this study was calculated following the AASHTO 1993 procedure, assuming a terminal serviceability index of 2.5. An ESAL value of 2.51 and 2.56 were calculated for section 1 and sections 2 and 3, respectively, based on the load distribution on each truck axle and the calculated structure number (SN) of the test sections. Therefore damage caused by 1000 truck passes is equivalent to the damage induced by 2510 and 2560 passes of an 80 kN single axle load.

2.8 Lidar Scan Survey

To monitor the cumulative permanent deformation induced by traffic loading, LiDAR scan surveys were performed before starting the tests and after 1, 5, 10, 50, 100, 200, 300, 500, 700, 800, 900, and 1000 truck passes. Surveys also were conducted before and after excavation to determine the exact depth of the stabilization measure. A MATLAB code was developed to call the Lidar scan data files and open them in MATLAB. Because the spacing resolution of the scanned points was 13 mm (0.5 in.), grids in both the vertical and horizontal directions were generated in MATLAB with 25.4 mm (1-in.) node spacing by using the GPS coordinates of the corners of the test sections. By generating these grids, measured elevations of the nearest scanned point to the intersection of the horizontal and vertical grid lines could be assigned to the intersection point. By using the code
and calling the scanned data, the elevation changes for the different time steps were calculated with respect to the initial elevation before the sections were subjected to traffic loading and plotted for the entire test site. These data were used to discern the location of the wheel paths. As shown in Fig. 2-5, the colder colors indicate settlement and wheel paths and the hotter colors indicate pumping (elevation rise) after 1,000 truck passes. The results of these analyses indicate an 117 cm (46 in.) wide wheel path, based on the area of cold colors shown in the figure.

Figure 2-5. Pumping and rutting over test section 1, after 1,000 passes

Because the overlapping test sections could not accurately represent their respective stabilization measure (Cowell et al. 2012), the portion of the wheel path within each test section that was outside the two EPCs was excluded from further deformation analysis. As shown in Fig. 2-5, the maximum
cumulative rut depths did not occur right on the centerline of each wheel path. Therefore, and to be able to capture the maximum elevation change at each cross-section, the maximum rut depth on each horizontal grid was found and used to develop the longitudinal surface deformation of each wheel path. In order to track the trend of permanent deformation development under the applied truck load, the deformation calculations for under the wheel paths in each test section were focused on two zones: north and south. The Northwest (called North) and Southeast (called South) zones are 183 cm (6 ft.) long and cover the area 91 cm (3 ft.) Northwest and 91 cm (3 ft.) Southeast of each pressure cell, as shown in Fig. 2-6. The average elevation change along the longitudinal direction in each of the zones was calculated after 1, 10, 50, 100, 200, 300, 500, 700, 800, 900, and 1000 truck passes for both the inner and outer wheel paths in each test section.

Figure 2-6. Considered area for surface deformation analysis (Dimensions in cm)
2.9 Performance Test Results

2.9.1 Permanent surface deformation

Fig. 2-7 shows the cumulative deformation of Section 1 that was stabilized by the select fill materials. The figure indicates that the rut depth reached a relatively constant value after 700 and 500 passes in the north and south sections, respectively. As shown in Figs. 2-8 through 2-9, for sections 2 and 3 with GG and GT reinforcements, respectively, the cumulative rut depth values are significantly lower than the permanent deformation in Section 1, and reached a relatively constant value after 500 traffic passes. The figures also show that the cumulative vertical deformation reduced slightly at pass No. 700 where the drivers were switched. By switching the drivers at this pass number, the wheel paths moved slightly (due to driver variability); thus, the lateral displacement induced by the newly established wheel path and the cumulative vertical deformation under the wheel paths located directly over the EPC were reduced.
Figure 2-7. Cumulative deformation in section 1, select fill

Figure 2-8. Cumulative deformation in section 2, geogrid reinforced
2.9.2 Stress distribution

As previously mentioned four EPCs were installed 7.6 mm (3 in.) below the interface of the subgrade and treated zone in each test section (two in each wheel path), to monitor and record applied pressure at the top of the subgrade. A MATLAB code was developed to find the peak value recorded by each pressure cell during each traffic pass, as shown in Fig. 2-10.
The primary analysis results indicate a range of measured pressure values, which could be caused by the direction of traffic and driver habits. To illustrate the effect of traffic direction, the recorded data were separated into two direction categories: South to North and North to South. For each pressure cell, the range of measured pressure narrowed significantly. As shown in Fig. 2-11 (a-c), the variation of the applied stress reduced from 13.8 to 6.9 kPa (2 to 1 psi), 69 to 34.5 kPa (10 to 5 psi), and 55 to 27.5 kPa (8 to 4 psi) for EPCs in section 1, 2 and 3, respectively, after categorizing the recorded applied stress according to travel direction.

Based on a tire contact area that is assumed rectangular in shape and has dimension of 20x 137 mm (8 in. x 54 in.), the tire location can be 13 to 38 mm (5 to 15 in.), offset from the EPC location, depends on the traffic location and driver skills. These scenarios may explain the difference between the recorded pressure levels for the two traffic directions, north to south and south to north.
Ahlvin and Ulery (1962) presented a detailed tabulation to calculate the vertical stress below a uniformly loaded flexible area at any desired point. The vertical stress at any point located at depth \( z \) and distance \( r \) from the center of a loaded area can be calculated by Equation 2-13:

\[
\Delta \sigma_z = q(A' + B')
\]

Eq. (2-13)

where \( A' \) and \( B' \) are functions of \( z/a \) and \( r/a \) (and \( a \) = radius of loaded area).

The stress distributions with depth under a loaded area for the stated scenarios were calculated for a given rear axle load and tire pressure of 586 kPa. As shown in Fig. 2-12, the applied pressure with depth changes when the location of the loaded area moves laterally only a few inches. This scenario is more pronounced for shallower depths. For Sections 2 and 3, where the EPCs were located 76 mm below the subgrade (~28 mm below the ground surface), the pressure has the potential to vary between 13.8 and 138 kPa (2 psi and 20 psi); and in Section 1, with the EPC located 89 mm (35 in.) below the ground surface, the measured pressure might vary by 13.8 to 34.5 kPa (2 psi to 5 psi), which is consistent with the measured values. It is worth noting that, although the Ahlvin and Ulery (1962) table is intended for calculating the stress distribution with depth in single-layer elastic subgrade under static loading, it still presents useful results to explain the effects of lateral wheel wander on the pressure that is measured near the interface of the subgrade and stabilized layer.
Figure 2-11. Measured pressure by traffic direction a) sec. 1: EPC11, b) sec. 2, EPC 21 and c) sec. 3: EPC32 (1 psi = 6.9 kPa)
2.9.3 Moisture and suction measurements

The moisture and suction of the subgrade layers were monitored and recorded using 10HS Decagon® moisture sensors and a MPS-2 Decagon® suction sensor, respectively. The volumetric moisture content changed by 2 percent, from 19 to 21%, during testing in all the test sections, while the matric suction slightly declined over the first week after installation and started to increase by 10 kPa during the duration of the full-scale testing, as shown in Fig. 2-13.
2.10 Numerical Analysis

An extensive set of finite element analyses was performed using PLAXIS 3D software to evaluate the effect of matric suction state of the ABC layer on the deformation response of the reinforced unpaved road section under traffic load. The subgrade soil and ABC materials properties were defined within the context of the Hardening Soil Small Strain (HS Small Strain) constitutive models.

2.10.1 Soil constitutive model

As previously mentioned the HS Small strain soil constitutive model was implemented in this study, in order to take into account the stress and strain dependency of the stiffness of soils. Duncan and Chang (1970) proposed Equation 2-14 to capture the strain dependency of the shear modulus:

\[-\varepsilon_i = \frac{1}{E_i} \frac{q}{1 - q / q_a} \quad \text{q,<q}_f\]

Eq. (2-14)

where \(q_a\) and \(E_i\) are defined as Equations 2-15 and 2-16:
\[ q_s = \frac{q_f}{R_f} \]  
\[ E_i = \frac{2E_{so}}{2 - R_f} \]

Eq. (2-15)

Eq. (2-16)

Benz (2007) defined the stress-strain relationship in the context of the HS Small Strain model, as expressed in Equation 2-17:

\[ \tau = G_s \gamma = \frac{G_0}{1 + 0.385 \left| \frac{\gamma}{\gamma_{0.7}} \right|} \]

Eq. (2-17)

The stress dependency of the shear modulus is given by Equation 2-18:

\[ G_0 = G_0^{ref} \left( \frac{c \cos \varphi - \sigma_3 \sin \varphi}{c \cos \varphi + p^{ref} \sin \varphi} \right)^m \]

Eq. (2-18)

Where the initial elastic modulus is defined by Equation 2-19:

\[ E_0 = 2G_0(1 + \nu_{ur}) \]

Eq. (2-19)

2.10.2 Soils model properties

As previously described, the constitutive model requires stiffness and shear strength parameters as inputs. The results from the laboratory tests of the undisturbed specimens, as well as reported data in the literature, Borden et al. (1996), Janoo et al. (2004), and Ayithi and Hiltunen (2013), were used to assign the appropriate parameters, \( E_{ur}^{ref} \), \( E_{50}^{ref} \), \( E_{ode}^{ref} \), \( m \), \( G_0^{ref} \), \( \gamma_{0.7} \), \( \varphi' \), \( c' \), to the subgrade and ABC materials. The ABC and subgrade soils properties are summarized in Table 2-3.
Table 2-3. ABC and subgrade soils properties used in numerical analyses.

<table>
<thead>
<tr>
<th>Material</th>
<th>H (mm)</th>
<th>$\gamma_{\text{moist}}$ (kN/m$^3$)</th>
<th>$E_{50}^{\text{ref}}$ (MPa)</th>
<th>$E_{\text{oed}}^{\text{ref}}$ (MPa)</th>
<th>$E_{ur}^{\text{ref}}$ (MPa)</th>
<th>$\nu_{ur}$</th>
<th>$G_0^{\text{ref}}$ (MPa)</th>
<th>$\gamma'_{0.7}$ (%)</th>
<th>$\phi'$</th>
<th>$c'/s_0$(kPa)</th>
<th>$R_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABC</td>
<td>200</td>
<td>23</td>
<td>200</td>
<td>180</td>
<td>400</td>
<td>0.64</td>
<td>0.2</td>
<td>800</td>
<td>0.05</td>
<td>43</td>
<td>0</td>
</tr>
<tr>
<td>A-4</td>
<td>610</td>
<td>20</td>
<td>75</td>
<td>75</td>
<td>150</td>
<td>0.5</td>
<td>0.2</td>
<td>360</td>
<td>0.001</td>
<td>25</td>
<td>13.8</td>
</tr>
<tr>
<td>A-7-5</td>
<td>3760</td>
<td>19</td>
<td>48</td>
<td>24</td>
<td>150</td>
<td>0.2</td>
<td>0.2</td>
<td>550</td>
<td>0.003</td>
<td>NA</td>
<td>193</td>
</tr>
</tbody>
</table>

2.10.3 SWCC and matric suction state

The SWCCs, shown in Fig. 2-14, for the ABC, A-4, and A-7-5 soils were developed from the data collected in this study as well as proposed correlations and data found in the literature by Zapata et al. (2000), Ba et al. (2013), Wang (2014). These curves also were verified by laboratory-measured suction values obtained using a tensiometer, from undisturbed Shelby tubes samples. Table 2-4 presents the SWCC parameters for the two subgrade soil types and the ABC layer, employed in numerical analyses. It is worth noting again that Zapata et al. (2000) and Wang (2014)’s empirical correlations were developed to estimate the Fredlund and Xing (1994) SWCC model parameters, however PLAXIS software uses Van Genuchten (1980)’s model for unsaturated soil analysis, therefore the Van Genuchten’s model parameters were back-calculated from the curve fitting for the developed SWCC from the proposed empirical correlations. From field measured suction, a matric suction of 40 kPa was considered for subgrade soils and ABC materials.
Figure 2-14. Developed SWCCs and tensiometer measured suction values.

<table>
<thead>
<tr>
<th></th>
<th>$\theta_s$</th>
<th>$\theta_r$</th>
<th>a</th>
<th>n</th>
<th>m</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-4 (SM)</td>
<td>0.36</td>
<td>0.09</td>
<td>0.13</td>
<td>2.6</td>
<td>0.61</td>
</tr>
<tr>
<td>A-7-5 (MH)</td>
<td>0.54</td>
<td>0.03</td>
<td>0.14</td>
<td>1.15</td>
<td>0.13</td>
</tr>
<tr>
<td>ABC</td>
<td>0.1923</td>
<td>0.00001</td>
<td>0.20</td>
<td>1.66</td>
<td>0.40</td>
</tr>
</tbody>
</table>

2.10.4 Geosynthetics and interface

The geosynthetic reinforcements were modeled for finite element analysis as linear elastic materials. The only parameter needed for geosynthetic materials is the elastic normal stiffness, $EA$, which is given by Equation 2-20:

$$EA = \frac{T_{\theta \varepsilon}}{\varepsilon}$$

Eq. (2-20)
where $T_{\varepsilon_{\%}}$ is the axial strength at a given magnitude of strain, $\varepsilon_{\%}$. The $EA$ values of 240 kN/m (16200 lb/ft), was calculated for the geogrid reinforcements used in the full-scale testing, at a strain level of 5 percent. The interface layers are defined for both sides of the geogrid. A reduction factor of 1 (meaning no reduction in the interface strength, or 100% efficiency of the geosynthetic material) was assigned to the interface of the geogrid and ABC and of the geogrid and subgrade soil.

2.10.5 Cyclic loading

The shape of the cyclic load was captured from the EPC recorded data, as shown in Fig 2-15. As previously described, the tire pressure of 590 kPa (85 psi) was measured in the field, and the rear axle load of 40 kN (9,000 lb) was estimated from the measured truck weight. Therefore, the radius of the loaded area was computed as 147 mm (5.8 inches). Absorbent boundaries were assigned to the model to absorb stress waves without rebounding into the soil body.

Figure 2-15. One applied load cycle.
2.10.6 Model geometry specifications and mesh sensitivity

As previously stated, the radius of the loaded area was calculated as 147 mm (5.8 in.). The model domain extended 3048 mm (120 in.) in both the X and Y directions and 4572 mm (180 in.) in the Z direction in order to minimize the boundary effects and rebounding of the reflected wave into the loaded zone of interest (Howard and Warren, 2009). The absorbent boundaries also were considered for the X_max, Y_max, and Z_min planes. A set of the general fixities was imposed to the boundaries of the geometry model, as summarized in Table 2-5:

<table>
<thead>
<tr>
<th>Plane</th>
<th>Deformation fixities</th>
<th>Deformation</th>
</tr>
</thead>
<tbody>
<tr>
<td>X_min</td>
<td>Normally fixed</td>
<td>U_x=0</td>
</tr>
<tr>
<td>X_max</td>
<td>Normally fixed</td>
<td>U_x=0</td>
</tr>
<tr>
<td>Y_min</td>
<td>Normally fixed</td>
<td>U_y=0</td>
</tr>
<tr>
<td>Y_max</td>
<td>Normally fixed</td>
<td>U_y=0</td>
</tr>
<tr>
<td>Z_min (bottom)</td>
<td>Fully fixed</td>
<td>U_x=U_y=U_z=0</td>
</tr>
<tr>
<td>Z_max (surface)</td>
<td>Free</td>
<td>free</td>
</tr>
</tbody>
</table>

In order to eliminate the effect of the size of the generated mesh on the results, the ABC and soft A-4 subgrade soil were divided into three volumes, as shown in Fig. 2-16. Different combinations of coarseness factors were assigned to the soil volumes until no change in surface deformation under static load was obtained. More detailed can be found in Mousavi et al. (2016)
2.10.7 Model validation

Fig 2-17 shows the surface deformation of the reinforced section under 1000 load cycles. As shown, the results obtained from PLAXIS 3D provide a good agreement with the measured field surface deformation. After 200 and 500 load cycles, the rate of the cumulative deformation decreases as well, following the trend of the measured data.
2.10.8 Stress distribution analysis

Fig. 2-18 presents the field-measured pressure levels obtained from EPC in section 2 and the average of the computed stress values of two stress points from PLAXIS 3D at a depth of 76 mm below the interface of the subgrade and ABC layer (depth of the installed pressure cells in the field) versus the number of load cycles. Both the measured and computed pressure levels follow a similar trend, and the pressure measured at the top of the subgrade is seen to increase gradually for the first 60 cycles and then decrease as the number of load cycles increases beyond 60. The same trend was reported by Qian et al. (2013), Tingle et al. (2009), and Thakur et al. (2012). This reduction in stress at the top of the subgrade can be explained by the mobilization of the tensile strength of the geogrid (as nearly 6.3 mm (0.25 in.) of vertical deformation occurred after 60 load cycles), which produces uplift that attenuates the applied stress that is transferred into the subgrade layer (Qian et al. 2013).
In addition to the geogrid mobilization, the ratio of the ABC modulus value to the subgrade modulus value also affects the magnitude of the stress that is transferred to the subgrade layer. An increase in the modulus ratio leads to a reduction in the transferred stress, as was observed by Leng and Gabr (2002). Fig. 2-19 shows the ratio of the secant modulus of the ABC to that of the subgrade layer, as computed from PLAXIS model results. The secant modulus ratio decreases during the initial cycles of loading; therefore, more stress is transferred to the subgrade. However, as the modulus ratio starts to increase after about 70 load cycles, a reduction in applied pressure on the top of the subgrade soil layer is computed. Such a reduction is evident as long as the ratio of the ABC modulus value to the subgrade modulus value continues to increase.

Figure 2-18. Applied pressure at the top of the subgrade
Fig. 2-20 shows the calculated stress distribution angle from field measured EPC 21, PLAXIS 3D results, Giroud and Han (2004), and Leng and Gabr (2006). The corresponded stress distribution angle to each load cycle was calculated using Equation 2-21, as demonstrated by Leng and Gabr (2006), from field measured stress by EPC C21, and PLAXIS 3D analyses:

$$\tan \alpha_N = \frac{a}{h} \left( \frac{p}{\sigma_{cN}} - 1 \right)$$  

Eq. (2-21)

Where “a” is a radius of loaded area of 147 mm (5.8 in.), “h”: thickness of ABC layer equal to 203 mm (8 in.), “p”: tire pressure of 586 kPa (85 psi), and $\sigma_{cN}$: applied pressure at the top of the subgrade at a load cycle of “N”.

As previously mentioned, although both Giroud and Han (2004), and Leng and Gabr (2006)’s approaches taking into account for the effect of the number of load cycles, and initial modulus
ratio, and geogrid stiffness property; these correlations do not consider variation of modulus ratio of ABC to the subgrade layer under traffic load. It is worth noting again; that applied pressure at the top of the subgrade varies with generated deformation and mobilized force in the reinforcement structure, as well as alteration of modulus ratio of ABC layer to the subgrade implied by the different induced strain levels.

Figure 2-20. Computed stress distribution angle

2.10.9 Evaluate subgrade stabilization alternatives

Fig. 2-21 shows the permanent deformation of the saturated subgrade soil without ABC layer, reinforcement under cyclic loading. The deformation of saturated subgrade reached to 33 mm (1.3 in.) after 100 load cycles. It was observed that deformation of unsaturated subgrade with a suction of 40 kPa, as measured in the field, after 100 load cycles is about 30.5 mm (1.2 in.), that indicates unsaturated subgrade soil in the field was not stable and required to be stabilized. As shown in
Fig. 2-21, 200 mm (8 in.) compacted saturated ABC on top of the soft subgrade reduces the permanent deformation dramatically to a value of 18.3 mm (0.72 in.). It was observed that the reinforcement inclusion, in this case, reduces the surface deformation only by 12.5% after 100 load cycles, and decrease the deformation from 18.3 to 16 mm (0.72 to 0.63 in.).

![Graph showing cumulative deformation](image)

Figure 2-21. Cumulative deformation of subgrade stabilization alternatives

### 2.10.10 Influence of matric suction

The effect of the matric suction state of the ABC layer on the deformation behavior of the un/reinforced sections was investigated. The surface deformation under 100 load cycles was computed using ABC matric suction values of 0, 10, 40, and 120 kPa, with the results plotted in Fig. 2-22. The figure shows that an increase in the ABC matric suction could significantly reduce the cumulative deformation, as the shear strength and stiffness properties are improved.
Figure 2-22. Cumulative deformation at different ABC suction level, for un/reinforced sections.

2.10.10.1 Vertical displacement

Fig. 2-23 shows the permanent deformation after 100 load cycles for reinforced and unreinforced subgrade at different ABC suction levels. It can be seen that the matric suction of ABC material significantly reduces the cumulative permanent deformation under cyclic loading. It was observed that matric suction of ABC layer, can reduce the surface deformation for both unreinforced and reinforced sections by 50 to 85% for the given matric suction range. It was also determined that the contribution of the matric suction into reducing the surface deformation increase by a number of cycles and reaches to the constant value, as the HS Small Strain constitutive model records and keeps a strain history using a strain tensor to calculate the tangent shear modulus and strain values for the current step. Therefore, the effect of the matric suction on reducing the deformation accumulates under cyclic loading and becomes more noticeable with an increase in the number of load cycles.
2.10.10.2 Pumping deformation

Fig. 2-24 shows the permanent surface deformation after 100 load cycles, with distance from the center of the loaded area. It can be seen that the matric suction of ABC layer, not only significantly reduces the permanent rutting deformation beneath the loaded area, but also dramatically decreases the generated mounting deformation under cyclic loading out of the loaded area, because of increase in shear strength and stiffness of ABC material.
2.10.10.3 Mobilized force in Geogrid

Influence of matric suction on the mobilized force in reinforcement structure after 100 load cycles was investigated. As reported by Mousavi (2016), PLAXIS 3D EA 2015 does not compute the mobilized force in geogrid element under vertical load correctly, since the vertical deformation component is left out the equation and mobilized force is calculated in two directions, XX and YY, in horizontal XY plane, from corresponded deformations. The mobilized force was hand-calculated from FEM deformation results as demonstrated by Mousavi (2016) and expressed in Equation 2-21:

\[ L2 = \sqrt{(L1+(U_{x2}-U_{x1}))^2 + (U_{z1}-U_{z2})^2} \]

Eq. (2-21)
Where $U_{x_i}$ and $U_{z_i}$ are horizontal, and vertical deformation of a geogrid node, respectively. And L1 and L2 are the initial and final length of geogrid between two nodes.

Equations 2-22, and 2-23 formulate in-plane strain of geogrid and mobilized tension force, respectively:

$$\varepsilon = \frac{L2 - L1}{L1}$$  \hspace{1cm} \text{Eq. (2-22)}

$$F = (EA) \varepsilon$$  \hspace{1cm} \text{Eq. (2-23)}

Fig. 2-25 shows the maximum hand-calculated mobilized force in the reinforcement structure after 40, 80, and 100 load cycles at different ABC suction level. The results indicate that the mobilized force increased by a number of load cycles, as results of cumulative plastic deformation under cyclic loading. It was observed that an increase in ABC suction, escalates the mobilized force until a certain suction level, in this study 40 kPa, and reduces after on.

![Figure 2-25. Maximum mobilized force in reinforcement](image)
Fig. 2-26(a-d)? shows the horizontal strain in ABC volume after 100 load cycles at ABC suction 0, 10, 40, and 120 kPa. It can be seen that an increase in suction level from 0 to 120 kPa, significantly reduces the horizontal deformation, $U_x$, by 83% from 4.0 to 0.73 mm (0.158 to 0.029 in.). As shown in figures, it can be seen that with an increase in ABC suction state, the location that the maximum horizontal deformation takes place; moves downward to the interface of ABC and subgrade layer, where the reinforcement layer is located. An increase in ABC suction escalates both the strength and stiffness of the ABC material, therefore ABC layer increasingly behaves as a beam. The more the layer acts like a beam, the strains tend toward compressive at the top and tensile toward the bottom. Hence, as shown in Fig. 2-27 the generated horizontal deformation after 100 load cycles, at the interface level, is greater for ABC layer with the suction level of 40 and 120, compared with cases with zero and 10 kPa suction. It can be concluded that, although an increase in ABC suction level reduces the vertical deformation at the location of reinforcement level, which is located at the bottom of ABC layer, but inflates horizontal deformation, which causes the mobilized force increases until certain suction level.
Figure 2-26. Horizontal deformation in ABC layer, after 100 load cycles: a) 0, b) 10 kPa, c) 40 kPa, and d) 120 kPa. (Dimensions in inches, 1 in. = 25.4 mm)
Conclusions

This paper presented an observation of instrumented full-scale test and 3D FEM analyses results for evaluating deformation and stress distribution performance of geogrid reinforced unsaturated unpaved road under cyclic load. Based on the obtained results, the following conclusions were drawn:

- The vertical stress values measured near the interface of the subgrade for the geosynthetic-reinforced sections (Sections 2 and 3) decreased with traffic, from 138 to 69 kPa for Section 2 (geogrid-reinforced), and from 110 kPa to 55 kPa for Section 3 (geotextile-reinforced), whereas the pressure recorded for Section 1 (select fill) was almost constant, 27.5 +/- 13.8 kPa during traffic. This decrease in measured stress for the geosynthetic-reinforced
sections is attributable to the mobilization of the reinforced layer tensile strength, an increase in the matric suction in the subgrade layer from 25 kPa to 35 kPa as a result of the hot summer weather during which the project was conducted, as well as variation of modulus ratio of ABC to subgrade implied by the different induced strain levels.

- Because each driver was to drive over the EPCs that were located on the driver’s side of the vehicle, the EPCs on the other (passenger) side was off between 13 to 38 mm from the truck tires (loaded area). It was observed that the lateral wander of the truck tires had a significant effect on the stress that was measured in the shallow depth of the subgrade layer (in Sections 2 and 3). Hence, the monitored pressure was categorized according to traffic direction, i.e., North to South and South to North. The variation in the recorded pressure reduced from 13.8 to 6.9 kPa, 69 to 34.5 kPa, and 55 to 27.5 kPa in Sections 1, 2, and 3, respectively.

- The geosynthetic-reinforced sections experienced up to 17 mm (0.67 in.) and 21 mm (0.83 in.) of surface deformation for the geogrid- and geotextile-reinforced sections, respectively. The select fill-stabilized section showed 37.5 (1.47 in.) of deformation after 1,000 truck passes.

- The rut depth values (defined as the vertical displacement from the original ground surface) increased with the number of truck passes until they reached a relatively constant value within 1,000 truck passes. The rut depth value increased up to 500 truck passes in both the geotextile and geogrid sections (Sections 2 and 3, respectively). In Section 1 (select fill), the rut depth reached a limiting value after 700 passes.

- FEM analyses results indicated that the matric suction state of the ABC layer was shown to have a notable effect on the surface deformation of un/reinforced unsaturated subgrade
under cyclic loading. A 120-kPa increase in matric suction in the ABC layer (from 0 kPa to 120 kPa) was associated with a 85 percent reduction in the surface deformation of the reinforced subgrade under cyclic loading.

- FEM results indicated that the mobilized force increased by a number of load cycles, as results of cumulative plastic deformation under cyclic loading, and an increase in ABC suction, increases the mobilized force until a certain suction level, in this study 40 kPa, and reduces after on.

- The numerical results showed that ABC layer behaves increasingly as a beam as the suction increases due to an increase in both the strength and stiffness. The more the layer acts like a beam, the location that the maximum horizontal deformation takes place; moves to the bottom of ABC, where the reinforcement layer is located.
References


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CHAPTER 3: OPTIMUM LOCATION OF THE REINFORCEMENT IN UNPAVED ROAD

S. Hamed Mousavi, Corresponding Author

North Carolina State University
Graduate Research Assistant, Department of Civil, Construction, and Environmental Engineering, North Carolina State University, Raleigh, NC 27695-7908;
Tel: 919-995-8792; Email: smousav3@ncsu.edu

Mohammed A. Gabr

North Carolina State University
Professor, Department of Civil, Construction, and Environmental Engineering, North Carolina State University, Raleigh, NC 27695-7908;
Tel: 919-515-7904 FAX: 919-515-7908; Email: gabr@eos.ncsu.edu

Roy H. Borden

North Carolina State University
Professor, Department of Civil, Construction, and Environmental Engineering, North Carolina State University, Raleigh, NC 27695-7908;
Tel: 919-515-7630 FAX: 919-515-7908; Email: borden@ncsu.edu
ABSTRACT

This study evaluated the optimum location of a reinforcement layer in order to maximize the efficiency of the reinforcement inclusion in the unpaved road section. The analyses are used to not only investigate the optimum location of the reinforcement layer within the ABC layer, but to also explain the reason for improvement in performance. A series of 3D FEM analyses were performed in Plaxis to investigate the strain and deformation response of the reinforced unpaved road with two different thickness aggregate base course, ABC, under stationary cyclic loading with three different radii of the loaded area. The embedded depth of reinforcement was varied within the ABC layer. Results indicated that regardless of ABC layer thickness, for a given circular loaded area, the surface deformation is minimized when the reinforcement is embedded at the depth equal to half of the radius of the loaded area, (D/r = 0.5). Analysis results indicated that required thickness of ABC is reduced when the reinforcement layer is implemented at the depth at which the maximum vertical strain occur right. It was observed from the numerical results that the higher tension force is mobilized in the reinforcement element when it is placed at the depth that the maximum vertical strain occurs, (D= 0.5r). On the other hand, the FEA results indicated that reinforcement layer can be ineffectual if it is placed at the interface between the ABC and the subgrade layer as is traditionally the case. This analysis approach however does not take “separation” benefits into consideration.

Keywords: Reinforcement, geosynthetics, unpaved, location, finite element, cyclic load.
3.1 Introduction

Over the years, many studies have been performed to investigate the deformation behavior of the reinforced unpaved road. These included small-scale and large-scale laboratory testing and full-scale field tests as well as numerical and analytical analyses, (e.g. Fannin and Sigurdsson 1996, Hufenus et al. 2005, Leng and Gabr 2006, Tingle and Jersy, 2009, Chen et al., 2009, White et al. 2011, Abu-Farsakh et al. 2012, Cowell et al. 2012, Thakur et al. 2012, Qian et al 2013, Saghebfar 2014, Tang et al. 2015, Sun et al., 2015). Several of these studies have been performed to evaluate the influence of factors such as thickness of the aggregate base course (ABC), shear strength and stiffness properties of subgrade, location of the reinforcement layer within the ABC layer, and mechanical and geometric properties of geogrid, on deformation response of a reinforced unpaved road. Geosynthetic inclusions have been shown to be more beneficial when the ABC layers are relatively thin; moreover, with an increase in the thickness of the ABC, the contribution of the geosynthetic inclusion to the reduction of rut depth development decreases (Fannin and Sigurdsson 1996, Hufenus et al. 2006, Al-Qadi 2012). The results presented by Cancelli and Montanelli (1999), and Perkins (1999) indicated that the efficiency of geosynthetics reinforcement inclusion is more pronounced with soft subgrade soil. The optimum location of reinforcement layer has also been investigated in many studies over the last decades (Perkins et al. 1999). Al-Qadi et al. (2012) recommended to place the reinforcement layer at the upper one-third of the ABC layer; however for the thinner ABC layer, the reinforcement should be located at the interface of the ABC and subgrade layer. Abu-Farsakh et al. (2012) reported the same results regarding the optimum location of a single layer of reinforcement; however, it was determined that double reinforcement layers inclusion (i.e., at the upper and lower third of the sample height) led to the largest improvement regardless of the geogrid type. Numerical analyses results reported by Saad et al. (2006), determined that the best location which provides the highest efficiency of reinforcement inclusion,
regardless of the subgrade quality, is at the lower one-third of the base thickness. Abu-Farsakh et al. (2012), and Qian et al. (2013) reported that the triangle geogrid provides better improvement compared with the biaxial geogrid with similar tensile moduli.

An extensive set of finite element analyses was performed in this study, using PLAXIS 3D software to not only investigate the optimum location of the reinforcement layer within the ABC layer but to also explain the reason for the improvement in performance. Two ABC layer thicknesses, under stationary cyclic loading with various radiiuses of the loaded area are used in the analyses. The subgrade soil and ABC materials properties were defined within the context of the Hardening Soil small-strain (HS small-strain) constitutive models, which were validated by the field observation in the report presented by Mousavi et al. (2016).

3.2 Soil Constitutive Model

The Hardening Soil small strain constitutive model was implemented in this study, because of its capability to incorporate the soil stiffness’s nonlinear dependency on the strain level, as well as simulate the stiffness hardening of soils that is due to an increase in confining pressure. In order to describe the variation in stiffness value with strain and confining pressure, the HS small strain model requires 7 stiffness input parameters:

- $m$: Stress dependent stiffness according to a power law.
- $E_{50}^{m}$: plastic straining due to primary deviatoric loading
- $E_{oed}^{m}$: Plastic straining due to primary compression
- $E_{ur}^{m}$: elastic unloading/reloading
- $V_{ur}$: unloading/reloading Poisson’s ratio
- $G_{0}$: initial shear modulus.

\[ \gamma_{0.7} \]: shear strain level at which the $G_s$ is reduced to 70 percent of the $G_0$ value
The relationship between axial deviatoric stress and the axial strain was formulated first using the hyperbolic relationship developed by Konder (1963), and later implemented in the constitutive model by Duncan and Chang (1970), as expressed in Equation 3-1:

$$-\varepsilon = \frac{q}{E_i} \left( 1 - \frac{q}{q_a} \right) \quad q < q_f$$  \hspace{1cm} \text{Eq. (3-1)}

where $q_a$ and $E_i$ are defined as Equation 3-2 and 3-3:

$$q_a = \frac{q_f}{R_f} \hspace{1cm} \text{Eq. (3-2)}$$

$$E_i = \frac{2E_{s0}}{2 - R_f} \hspace{1cm} \text{Eq. (3-3)}$$

The $E_{s0}$ is the stress dependent stiffness modulus which is given by Equation 3-4:

$$E_{s0} = E_{s0}^{\text{ref}} \left( \frac{c \cos \varphi - \sigma_3^s \sin \varphi}{c \cos \varphi + p_{\text{ref}} \sin \varphi} \right)^m$$  \hspace{1cm} \text{Eq. (3-4)}

The stress dependency of the stiffness modulus is expressed by the power $m$. The $m$ value is recommended to be between 0.5 for sand and silt and 1.0 for soft clayey soil, (Janbu 1963, Soos 1990). Benz (2007) defined the stress-strain relationship in the context of the HS small-strain model, as expressed in Equations 3-5:

$$\tau = G_s \gamma = \frac{G_0}{1 + 0.385 \left| \frac{\gamma}{\gamma_{0.7}} \right|}$$  \hspace{1cm} \text{Eq. (3-5)}

The stress dependency of the shear modulus is given by Equation 3-6:

$$G_0 = G_0^{\text{ref}} \left( \frac{c \cos \varphi - \sigma_3^s \sin \varphi}{c \cos \varphi + p_{\text{ref}} \sin \varphi} \right)^m$$  \hspace{1cm} \text{Eq. (3-6)}

3.3 Input Material Properties

3.3.1 ABC and subgrade

The input materials properties were selected based on the laboratory test results, and numerical model validation by full scale field testing results as was describe in Mousavi et al (2016). The
ABC and subgrade soils properties are summarized in Table 3-1. As it will be discussed later, the analyses were performed for two thickness of ABC, (203, and 305 mm).

Table 3-1. ABC and subgrade soils properties used in numerical analyses.

<table>
<thead>
<tr>
<th>Material</th>
<th>H (mm)</th>
<th>$\gamma_{\text{moist}}$ (kN/m$^3$)</th>
<th>$E_{50}^{\text{ref}}$ (MPa)</th>
<th>$E_{\text{oed}}^{\text{ref}}$ (MPa)</th>
<th>$E_{ur}^{\text{ref}}$ (MPa)</th>
<th>$m$</th>
<th>$\nu_{ur}$</th>
<th>$G_0^{\text{ref}}$ (ksi)</th>
<th>$\gamma_{0.7}$ (%)</th>
<th>$\phi^\prime$</th>
<th>$c'/s_0$ (kPa)</th>
<th>$R_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABC</td>
<td>-</td>
<td>23</td>
<td>200</td>
<td>180</td>
<td>400</td>
<td>0.64</td>
<td>0.2</td>
<td>800</td>
<td>0.05</td>
<td>43</td>
<td>0</td>
<td>0.9</td>
</tr>
<tr>
<td>A-4</td>
<td>610</td>
<td>20</td>
<td>75</td>
<td>75</td>
<td>150</td>
<td>0.5</td>
<td>0.2</td>
<td>360</td>
<td>0.001</td>
<td>25</td>
<td>13.8</td>
<td>0.9</td>
</tr>
<tr>
<td>A-7-5</td>
<td>3760</td>
<td>19</td>
<td>48</td>
<td>24</td>
<td>150</td>
<td>0.2</td>
<td>0.2</td>
<td>550</td>
<td>0.003</td>
<td>NA</td>
<td>193</td>
<td>0.9</td>
</tr>
</tbody>
</table>

3.3.2 Geosynthetics and Interface

The geosynthetic reinforcements were modeled as linear elastic materials. The only parameter needed for geosynthetic materials is the normal elastic stiffness, $E_A$, which is given by Equation 3-7:

$$EA = \frac{T_{@\varepsilon\%}}{\varepsilon}$$  \hspace{1cm} \text{Eq. (3-7)}

Where $T_{@\varepsilon\%}$ is the axial strength at a given magnitude of strain, $\varepsilon\%$. The $EA$ values of 240 kN/m (16200 lb/ft) at a strain level of 5 percent, was selected for the geogrid reinforcements, as used in the full-scale testing reported by Mousavi et al. (2016). The interface layers are defined for both sides of the geogrid. A reduction factor of 1 (meaning no reduction in the interface strength, or 100% efficiency of the geosynthetic material) was assigned to the interface of the geogrid and ABC and of the geogrid and subgrade soil.

3.4 Cyclic Loading

The shape of the cyclic load was captured from the EPC recorded data, shown in Fig. 3-1. The radius of the loaded area was calculated as 147 mm (5.8 inches) corresponding to a tire pressure of 590 kPa (85 psi) and an axle load of 80 kN (18,000 lb).
3.5 Model Geometry

The model domain extended 3048 mm (120 in.) in both the $X$ and $Y$ directions and 4572 mm (180 in.) in the $Z$ direction in order to eliminate the boundary effects and rebounding of the reflected wave into the loaded zone of interest (Howard and Warren, 2009). The absorbent boundaries also were considered for the $X_{\text{max}}$, $Y_{\text{max}}$, and $Z_{\text{min}}$ planes to absorb stress waves without rebounding into the soil body. A set of the general fixities was imposed to the boundaries of the geometry model, as summarized in Table 3-2:

Table 3-2. Geometry boundary conditions.

<table>
<thead>
<tr>
<th>Plane</th>
<th>Deformation fixities</th>
<th>Deformation</th>
</tr>
</thead>
<tbody>
<tr>
<td>$X_{\text{min}}$</td>
<td>Normally fixed</td>
<td>$U_x=0$</td>
</tr>
<tr>
<td>$X_{\text{max}}$</td>
<td>Normally fixed</td>
<td>$U_x=0$</td>
</tr>
<tr>
<td>$Y_{\text{min}}$</td>
<td>Normally fixed</td>
<td>$U_y=0$</td>
</tr>
<tr>
<td>$Y_{\text{max}}$</td>
<td>Normally fixed</td>
<td>$U_y=0$</td>
</tr>
<tr>
<td>$Z_{\text{min}}$ (bottom)</td>
<td>Fully fixed</td>
<td>$U_x=U_y=U_z=0$</td>
</tr>
<tr>
<td>$Z_{\text{max}}$ (surface)</td>
<td>Free</td>
<td>free</td>
</tr>
</tbody>
</table>
In order to eliminate the effect of the size of the generated mesh on the results, the ABC and soft A-4 subgrade soil were divided into three volumes, as shown in Fig. 3-2. Different combinations of coarseness factors were assigned to the soil volumes until no change in surface deformation under static load was obtained. It was determined that maximum computed deformation does not change by utilizing more than 16000 elements in the model geometry. More details on mesh sensitivity analyses can be found on Mousavi et al. (2016).

![Figure 3-2. Model geometry and soil volumes.](image)

### 3.6 Numerical Analyses Results

Optimum reinforcement location within the ABC layer was investigated for ABC with a thickness, \( H \), of 203, and 305 mm (8, and 12 in.). To do so, effect of any matric suction was ignored and the surface deformation was computed under 3 loaded area with a radius \( r \) of 76, 152, and 305 mm.
The reinforcement layer was located at various depths (D) from the surface, as shown in Fig. 3-3.

Figure 3-3. Location of the reinforcement layer.

3.6.1 Effect of reinforcement location

Fig. 3-4 shows the variation of computed surface deformation after 100 load cycles, under the loaded area with a radius of 76, 152, and 305 mm, and ABC thickness of 203 and 305 mm. The data are shown in terms of the ratio of D/r. In this case, for all three loaded areas, regardless of ABC layer thickness, the surface deformation is minimized, when the reinforcement is embedded at the depth equal to half of the radius of the loaded area, (D/r = 0.5).
Fig. 3-5 shows the surface deformation after 100 load cycles, in terms of the ratio of D/H. As previously discussed, it can be seen that the location, which provides the maximum efficiency of the reinforcement inclusion, depends on the radius of the loaded area, but is independent of the thickness of the ABC layer. It was observed that the placing the reinforcement layer in the depth of half of the radius of loaded area compared to at the interface between the ABC and the subgrade layers can significantly decrease the surface deformation. Open symbols
Figs. 3-6, shows the variation of vertical with depth, beneath the center of the loaded area, for the unreinforced section, after 100 loaded cycles for ABC layer with a thickness of 203 mm (8 in.). In this case, the maximum vertical strain occur at the depth about half of the radius of the loaded area \( (z = 0.5r) \); which is consistent with Schmertmann (1970)'s vertical influence factor diagram under circular loaded area. Accordingly, to maximize the efficiency and performance of the reinforcement structure, the reinforcement layer should be placed at the depth that maximum vertical strain occur.
3.6.2 Mobilized tension force

Variation of mobilized tension force in the reinforcement element with respect to its embedded depth was investigated under each loaded area. Plaxis 3D calculates mobilized tension force in the reinforcement layer in two local directions 1 and 2, based on the deformation in these directions as demonstrated in Fig. 3-7 and expressed in Equations 3-8 and 3-9:
\[ N_1 = EA_1 \varepsilon_1 \quad \text{Eq. (3-8)} \]

\[ N_2 = EA_2 \varepsilon_2 \quad \text{Eq. (3-9)} \]

In this case, \( N_1, U_1 \) and \( N_2, U_2 \) are in \( XX \) and \( YY \) direction respectively. As shown in Figs. 3-8 (a-b), it was observed that the software does not rotate the local axis (\( U_1 \) and \( U_2 \)) directions as the plane start to deform under the applied load, and \( U_1 \) and \( U_2 \) stay parallel and equal to \( U_x \) and \( U_y \) of the reinforcement node, respectively. Consequently, it seems that the computed mobilized force in geogrid is independent of the vertical deformation, and is only contingent on the horizontal deformation in \( XY \) plane. Therefore, manual computation of the force in the reinforcement is necessary.
Figure 3-8. (a) Ux, (b) U1, of geogrid after 100 load cycles; H, D, and r = 305 mm
In order to compute the actual mobilized tension force in the geogrid elements, and taking into account the component, $U_z$, the mobilized tension force was hand calculated by using Plaxis $U_x$ and $U_z$ output, for each case. Fig. 3-9(a) shows the diagram of the 2 nodes of one geogrid element, with their $U_z$, and $U_x$, deformation in ZZ and XX direction. Fig. 3-10(b) illustrates how the new element length, $L_2$, can be calculated based on the relative nodes deformation, as expressed in Equation 3-10:

$$L_2 = \sqrt{(L_1 + (U_{x2} - U_{x1}))^2 + (U_{z1} - U_{z2})^2}$$

Eq. (3-10)

Equations 3-11, and 3-12 formulate in-plane strain of geogrid and mobilized tension force, respectively:

$$\varepsilon = \frac{L_2 - L_1}{L_1}$$

Eq. (3-11)

$$F = (EA)\varepsilon$$

Eq. (3-12)

Fig. 3-10 shows the hand calculated maximum mobilized tension force in geogrid elements after 100 load cycles, corresponding to the different D/r values used in this study. It was observed that greater tension force is mobilized when reinforcement implemented at the depth where maximum
vertical strain occur. Therefore this can also explain the observation that permanent surface deformation minimized when the reinforcement layer is placed at the depth at which maximum vertical strain occur, (D=0.5r)

![Graph showing the relationship between D/r and f (lb/in) for different r values and H heights.](image)

Figure 3-10. Maximum mobilized tension force in geogrid after 100 load cycles.

### 3.6.3 Influence of reinforcement inclusion

Fig. 3-11 shows the effectiveness of the reinforcement inclusion in terms of the ratio of D/r, for two thickness of ABC, which calculated as presented in Equation 3-13. It was observed that the FEM results are consistent with the literature, which provided improvement with reinforcement is more pronounced with a thinner thickness of ABC. As shown in Fig. 3-11, reinforcement inclusion can be ineffective if it is not implemented at the appropriate location. It can be seen that the
efficiency of the geogrid can significantly increase from less than 5% to more than 70% if the reinforcement located at its optimum location.

\[
\text{Efficiency: } \eta = \frac{\delta_{\text{unreinforced}} - \delta_{\text{reinforced}}}{\delta_{\text{unreinforced}}} \\
\text{Eq. (3-13)}
\]

Figure 3-11. Influence of reinforcement inclusion

Conclusions

FEM analyses were performed in Plaxis 3D under cyclic loading to evaluate the optimum location of a reinforcement layer that can be implemented to maximize the efficiency of the reinforcement inclusion in an unpaved road section. The analyses are used to not only investigate the optimum location of the reinforcement layer within the ABC layer, but to also explain the reason for
improvement in performance. Based on the numerical analyses result following conclusion can be drawn:

- Regardless of ABC layer thickness, the surface deformation is minimized, when the reinforcement is embedded at the depth of about half of the radius of loaded area, (D/r = 0.5).

- The inclusion of reinforcement can lead to even smaller surface deformation if it is placed at the depth at which maximum vertical strain occur, compared with the case with greater ABC thickness and reinforcement is located at the interface of the base and subgrade, conventionally.

- Computed mobilized force in geogrid in Plaxis 3D, under vertical load, is independent of the vertical deformation, which is the main component that mobilizes force in reinforcement, and contingent only on the horizontal deformation (in XY plane) of the geogrid node.

- The Higher tension force is mobilized in reinforcement element when it is placed at the depth that maximum vertical strain take place, (D= 0.5r), thus reinforcement inclusion is more beneficial in the reduction of surface deformation.

- FEM results confirmed the literature, which reinforcement produce a higher reduction in surface deformation with thinner ABC layer. It can be seen that the efficiency of the geogrid can significantly increase if the reinforcement located at a depth equal to half of the radius of the loaded area.
References


CHAPTER 4: SUBGRADE RESILIENT MODULUS PREDICTION FROM LIGHT WEIGHT DEFLECTOMETER

S. Hamed Mousavi, Corresponding Author

North Carolina State University
Graduate Research Assistant, Department of Civil, Construction, and Environmental Engineering, North Carolina State University, Raleigh, NC 27695-7908; Tel: 919-995-8792; Email: smousav3@ncsu.edu

Mohammed A. Gabr

North Carolina State University
Professor, Department of Civil, Construction, and Environmental Engineering, North Carolina State University, Raleigh, NC 27695-7908; Tel: 919-515-7904 FAX: 919-515-7908; Email: gabr@eos.ncsu.edu

Roy H. Borden

North Carolina State University
Professor, Department of Civil, Construction, and Environmental Engineering, North Carolina State University, Raleigh, NC 27695-7908; Tel: 919-515-7630 FAX: 919-515-7908; Email: borden@ncsu.edu
ABSTRACT

Resilient modulus has been used for decades as an important parameter in pavement structure design. Resilient modulus, like other elasticity moduli, increases with increasing confining stress and softens with increasing deviatoric stress. Several constitutive models have been proposed in the literature to calculate resilient modulus as a function of stress state. The most recent model, recommended by MEPDG and used in this paper, calculates resilient modulus as a function of bulk stress, octahedral shear stress and three fitting coefficients, $k_1$, $k_2$, and $k_3$. Work in this paper presents a novel approach for predicting resilient modulus of subgrade soils at various stress level based on light weight deflectometer (LWD) data. The proposed model predicts the MEPDG resilient modulus model coefficients ($k_1$, $k_2$, and $k_3$) directly from the ratio of applied stress to surface deflection measured during LWD testing. The proposed model eliminates uncertainties associated with needed input parameters for $E_{LWD}$ calculation, such as the selection of an appropriate value of Poisson’s ratio for the soil layer and shape factor. The proposed model was validated with independent data from other studies reported in the literature.

*Keywords*: resilient modulus, light weight deflectometer, subgrade, MEPDG
4.1 Introduction

The use of resilient modulus ($M_r$) has been substituted for the California Bearing Ratio (CBR) in pavement design in order to consider the deformation behavior of base and subgrade layers under cyclic loading condition. The magnitude of $M_r$ depends on, soil types, its structure, physical properties such as density and water content, as well as the applied stress state (Li 1994; Rahim and George 2004, Liang et al. 2008). Studies have shown that same as other soil modulus, resilient modulus of subgrade soils, increase by an increase of confining pressure and reduces with an increase in deviatoric stress. The value of $M_r$ is defined as the ratio of the cyclic axial stress to the recoverable or resilient axial strain (NCHRP project 1-28A, 2004), as shown schematically in Fig. 4-1 and expressed in Equation 4-1:

$$M_r = \frac{\sigma_{cyclic}}{\varepsilon_r}$$

Eq. (4-1)

Where:

$\sigma_{cyclic}$: cyclic axial stress

$\varepsilon_r$: resilient axial strain
Figure 4-1. Definition of resilient modulus

The $M_r$ of a subgrade layer can be determined from laboratory testing following the AASHTO T-307 test protocol, which uses fifteen stress combinations: five deviatoric stress levels 13.8, 27.6, 41.4, 55.2 and 69 kPa (2, 4, 6, 8 and 10 psi) applied at three confining pressures 41.4, 57.6 and 13.8 kPa (6, 4 and 2 psi). Different forms of constitutive models can be found in the literature that allow for computing the $M_r$ value as a function of one, two or three stress parameters such as confining pressure, deviatoric stress, bulk stress and octahedral shear stress (e.g. Dunlap, 1963; Seed et al., 1967; Witczak and Uzan, 1988; Pezo, 1993; NCHRP project 1-28A, 2004). The recent universal constitutive model proposed in NCHRP 1-28A (MEPDG) is presented in Equation 4-2:

$$
M_r = k_1 P_a \left( \frac{\theta}{P_a} \right)^{k_2} \left( \frac{\tau_{oc}}{P_a} + 1 \right)^{k_3}
$$

Eq. (4-2)

Where:
\( M_r \): resilient modulus

\( P_a \): atmospheric pressure

\( \sigma_1, \sigma_2, \sigma_3 \): principal stresses

\( \theta = \sigma_1 + 2\sigma_3 \): bulk stress

\( \tau_{oct} = \frac{\sqrt{2}}{3} (\sigma_1 - \sigma_3) \): octahedral shear stress

\( k_i \): regression constants; \( i:1, 2, \) and \( 3 \)

Despite the good accuracy of the use of laboratory testing to determine \( M_r \) values, the requirement of using an expensive and sophisticated device to perform the \( M_r \) test is considered as a disadvantage. Several studies have been undertaken to overcome this issue by estimating \( M_r \) through the development of empirical correlations with the physical properties of soil (e.g. Carmichael et al., 1985; Elliott et al., 1988; Drumm et al., 1990; Farrar and Turner, 1991; Hudson et al., 1994). These empirical correlations eliminate the expense of resilient modulus laboratory testing, but they are generally not capable of capturing the stress dependency of the \( M_r \), or simulate various stress conditions encountered in the field. Many studies have been performed over the past two decades to model the stress dependency of the resilient modulus by predicting the coefficients of constitutive model, NCHRP Project 1-28A 2004, using basic soil properties, such as water content, the plasticity limit, liquid limit, P4, P200, etc. For example, Yau and Von Quintus (2002), Rahim and George (2004), and Nazzal and Mohammad (2010) each proposed different models to estimate the NCHRP Project 1-28A constitutive model coefficients (\( k_1, k_2, \) and \( k_3 \)) based on the physical properties of soils. Another approach is the use of expedient in-situ approaches such as light weight deflectometer, LWD, and dynamic cone penetrometer, DCP, testing to estimate \( M_r \). For example, White et al. (2007) and Mohammad at el. (2008) have proposed correlations to
estimate the $M_r$ of subgrade soil by LWD at one specific confining pressure and deviatoric stress (confining pressure = 41.4 and 13.8 kPa and deviatoric stress = 41.4 kPa, respectively). Since the $M_r$ depends on the confining pressure and applied deviatoric stress, these models become inapplicable for cases with different stress states.

Work in this paper presents a model to compute $M_r$, values of A-4 (ML, SM, and CL) and A-7-5 (MH) soils from LWD data at any stress state by estimating the MEPDG recommended constitutive model parameters ($k_1$, $k_2$ and $k_3$). The proposed model is based on the use of LWD measured data, the ratio of applied stress to the measured surface deflection, rather than on the use of LWD-estimated $M_r$, in order to minimize uncertainties involved with the selection of Poisson’s ratio and shape factor input parameters. The validity of the proposed model for A-1-b (SP) and A-6 (CL-ML) soils, is examined by using data presented in the literature.

4.2 Background

The LWD is a portable falling weight deflectometer for measuring in-situ modulus of soil (Fleming 2007). Compared to the falling weight deflectometer (FWD), the LWD is cheaper and more convenient to perform. The device used in this study was a Prima 100, as shown in Fig. 4-2, and consisted of 10 kg falling weight, which can induce 15-20 ms pulse load up to 450 kPa, with its 20-cm diameter plate (radius = 10 cm). A geophone is used to measure surface deflection, right at the center of the plate load. Surface deflection and applied load are monitored and recorded through Prima 100 software. Fig. 4-3 shows an example of applied load and surface deflection for one drop.
Figure 4-2. Prima 100 sketch (after Vennapusa, 2008)

The \textit{in-situ} modulus is calculated based on Boussineq’s static elastic half space theory by assuming a homogeneous isotropic soil layer (Fleming 2007). Therefore Poisson’s ratio and shape factor are assigned as input parameters to the software to calculate the modulus per Equation 4-3 (Fleming 2007).

$$ E_{LWD} = \frac{f \cdot (1 - \nu^2) \cdot \sigma \cdot r}{\delta} $$

Eq. (4-3)

Where:

- $E_{LWD}$: surface Modulus (MPa)
- $\sigma$: applied stress (kPa)
- $\delta$: surface Deflection (µm)
- $f$: shape factor
- $\nu$: Poisson’s ratio
- $r$: radius of plate (mm)
As previously mentioned, there are few empirical correlations to approximate the \( M_r \) of subgrade soil from LWD measurements. Although the main advantage of these models is that they can capture actual moisture and density conditions of the soil layer, they are mostly limited to one specific stress state. White et al. (2007) proposed the model presented in Equation 4-4 to predict \( M_r \) of subgrade soil from \( E_{LWD} \) with an assumed Poisson’s ratio of 0.35 and shape factor of \( \frac{\pi}{2} \) and 2 for cohesive and cohesionless soils, respectively, at a confining pressure = 41.4 kPa (6 psi) and deviatoric stress = 69 kPa (10 psi).

\[
M_r = \frac{(E_{LWD} + 45.3)}{1.24}
\]  

Eq. (4-4)

With \( M_r, E_{LWD} \) in MPa

Figure 4-3. Example of recorded applied stress and surface deflection during LWD testing
Mohammad et al (2008) presented the model in Equation 4-5 in order to estimate $M_r$ from LWD data by assuming a Poisson’s ratio of 0.4 and a shape factor of $\frac{\pi}{2}$ for cohesive soil and 2 for cohesionless soils, at a confining pressure 13.8 kPa (2 psi) and deviatoric stress 41.4 kPa (6 psi). These stress values were chosen to represent the stress state at the top of the subgrade layer under standard single axle loading of 80 kN (18 kips) and tire pressure of 689 kPa (100 psi) with a 50-mm asphalt wearing course, 100-mm asphalt binder course and 200-mm aggregate base course (Mohammad et al. 2008; Rahim and George 2004; Asphalt Institute 1989).

$$M_r = 27.75 \times E_{LWD}^{0.18} \quad \text{Eq. (4-5)}$$

With $M_r$, $E_{LWD}$ in MPa

### 4.3 Experimental Program

A series of laboratory and in-situ LWD tests were performed to evaluate subgrade soil modulus properties of four 4.88-m (16-ft) wide by 15.2-m (50-ft) long test sections located in the Piedmont area, North of Greensboro, North Carolina. In this case, LWD testing was conducted in the field to monitor the variation of subgrade modulus across the test sections. As shown in Fig. 4-4, the LWD tests were carried out at four locations within each test section; those locations were offset 1-m (3.3-ft) away from boreholes, from which Shelby tubes were obtained. In parallel, a laboratory testing program including resilient modulus testing and physical properties characterization was performed on undisturbed samples retrieved from within the LWD influence zone, (a depth = 1.5~2 diameter of the LWD loading plate, as was specified by Mooney and Miller 2007; Khosravifar et al. 2013; Senseney et al. 2016), as presented in Table 4-1, and indicated in Fig. 4-1.
Figure 4-4. Location of resilient modulus specimens and LWD tests (dimensions in cm).

4.4 Materials

Basic index property tests, including grain size distribution, Atterberg limits, and specific gravity were conducted on the specimens after the $M_r$ tests were completed. As shown in Table 4-1 and Fig. 4-5, the site soils were classified as A-7-5 (MH), A4 and A4-a (SM, ML and CL, respectively). The high plasticity specimens, A-7-5, were taken from the deeper depths, and correspond to natural soil, while the low plasticity soils (A4 and A4-a) were located at shallower depths and corresponded to compacted fill.
Table 4-1. Summary of Sample Properties

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Depth</th>
<th>$\gamma_{\text{total}}$</th>
<th>$w$</th>
<th>$e$</th>
<th>$G_s$</th>
<th>$LL$</th>
<th>$PL$</th>
<th>$PI$</th>
<th>$P_{200%}$</th>
<th>$\text{Clay}&lt;5\mu m$</th>
<th>USCS</th>
<th>AASHTO</th>
</tr>
</thead>
<tbody>
<tr>
<td>H1-11</td>
<td>79</td>
<td>16.9</td>
<td>34</td>
<td>0.98</td>
<td>2.70</td>
<td>72</td>
<td>48</td>
<td>24</td>
<td>80</td>
<td>65</td>
<td>MH</td>
<td>A-7-5</td>
</tr>
<tr>
<td>H2-2</td>
<td>86</td>
<td>17.7</td>
<td>27.8</td>
<td>0.80</td>
<td>2.66</td>
<td>58</td>
<td>36</td>
<td>22</td>
<td>72</td>
<td>46</td>
<td>MH</td>
<td>A-7-5</td>
</tr>
<tr>
<td>H2-1</td>
<td>18</td>
<td>18.7</td>
<td>14.4</td>
<td>0.51</td>
<td>2.64</td>
<td>15</td>
<td>12</td>
<td>4</td>
<td>41</td>
<td>16</td>
<td>SM</td>
<td>A-4a</td>
</tr>
<tr>
<td>H3-1</td>
<td>8</td>
<td>18.0</td>
<td>16.8</td>
<td>0.61</td>
<td>2.65</td>
<td>20</td>
<td>17</td>
<td>3</td>
<td>49</td>
<td>19</td>
<td>SM</td>
<td>A-4a</td>
</tr>
<tr>
<td>H6-1</td>
<td>8</td>
<td>18.3</td>
<td>17.0</td>
<td>0.58</td>
<td>2.65</td>
<td>13</td>
<td>10</td>
<td>3</td>
<td>42</td>
<td>14</td>
<td>SM</td>
<td>A-4a</td>
</tr>
<tr>
<td>H3-2</td>
<td>23</td>
<td>18.8</td>
<td>16.0</td>
<td>0.51</td>
<td>2.62</td>
<td>18</td>
<td>17</td>
<td>2</td>
<td>51</td>
<td>16</td>
<td>ML</td>
<td>A-4a</td>
</tr>
<tr>
<td>H4-1</td>
<td>23</td>
<td>18.6</td>
<td>21.8</td>
<td>0.63</td>
<td>2.60</td>
<td>20</td>
<td>17</td>
<td>3</td>
<td>54</td>
<td>20</td>
<td>ML</td>
<td>A-4a</td>
</tr>
<tr>
<td>H4-2</td>
<td>23</td>
<td>19.3</td>
<td>15.4</td>
<td>0.47</td>
<td>2.62</td>
<td>19</td>
<td>16</td>
<td>3</td>
<td>55</td>
<td>20</td>
<td>ML</td>
<td>A-4a</td>
</tr>
<tr>
<td>H5-1</td>
<td>0</td>
<td>19.4</td>
<td>13.3</td>
<td>0.46</td>
<td>2.61</td>
<td>22</td>
<td>19</td>
<td>2</td>
<td>51</td>
<td>20</td>
<td>ML</td>
<td>A-4a</td>
</tr>
<tr>
<td>H6-2</td>
<td>23</td>
<td>18.8</td>
<td>21.2</td>
<td>0.59</td>
<td>2.61</td>
<td>19</td>
<td>15</td>
<td>4</td>
<td>49</td>
<td>19</td>
<td>ML</td>
<td>A-4a</td>
</tr>
<tr>
<td>H8-1</td>
<td>8</td>
<td>18.8</td>
<td>16.6</td>
<td>0.52</td>
<td>2.61</td>
<td>19</td>
<td>17</td>
<td>2</td>
<td>54</td>
<td>14</td>
<td>ML</td>
<td>A-4a</td>
</tr>
<tr>
<td>H5-2</td>
<td>30</td>
<td>19.7</td>
<td>16.5</td>
<td>0.44</td>
<td>2.64</td>
<td>25</td>
<td>16</td>
<td>8</td>
<td>42</td>
<td>16</td>
<td>CL</td>
<td>A-4a</td>
</tr>
</tbody>
</table>

1 cm, 2 kN/m², 3 natural water content, 4 void ratio, 5 specific gravity, 6 Liquid limit, 7 Plastic limit, 8 Plasticity index, 9 pass sieve No. 200, 10 Clay < 5μm, 11 Hi-j, i: borhole number, j: sample number

Figure 4-5. Grain size distributions of resilient modulus samples
4.5 LWD Measurements

Subgrade modulus values were measured at the test site by LWD (with 20 cm plate) following the ASTM E2583-07. To do so, the plate located horizontally on the surface, and first three seating drops were considered to provide full contact between the plate load and the subgrade; which followed by another three drops to measure the surface modulus, \( E_{LWD} \). The \( E_{LWD} \) values were calculated using Equation 4-3 and assuming a Poisson’s ratio of 0.35. A shape factor of \( \frac{\pi}{2} \) was selected for the MH soil, and 2 for the ML, SM, and CL soil, (Mooney and Miller 2007, White et al. 2007). Two LWD test stations were located about 1 m apart on each side of the boreholes, from which samples for resilient modulus testing were collected, as shown in Fig. 4-4. At each station, 3 \( E_{LWD} \) were measured. The Standard deviation and coefficient of variation of the six LWD measurements corresponding to the resilient modulus laboratory specimen are presented in Table 4-2, which indicate COV is less than 5%. Hence for comparison sake, the average of six field measured \( E_{LWD} \) values was used to estimate the \( M_r \) from LWD. The calculated \( M_r \) values were compared to the laboratory-measured \( M_r \) values for specimens retrieved from the corresponding borehole within the LWD effective zone, as summarized in Table 4-2.

As previously mentioned, the assumption of Poisson’s ratio and shape factor can lead to various estimates of \( E_{LWD} \). In order to overcome the ambiguities with which values to use, the ratio of applied stress to surface deflection, \( \frac{\sigma}{\delta} \), from LWD direct measurements; which is representative of soil layer elasticity, is directly used herein in the proposed model development, instead of computing the \( E_{LWD} \) by Equation 4-3. The applied stress to surface deformation ratios from the LWD measurements are summarized in Table 4-2 and are shown associated with the Specimen Number upon which the subsequent \( M_r \) tests were performed.
4.6 Laboratory Testing: Resilient Modulus

Resilient modulus tests were performed on undisturbed soil specimens following AASHTO T-307. Resilient modulus at each load combination was computed as the ratio of the cyclic axial stress to average resilient axial strain for the last 5 of the 100 applied load cycles. Laboratory results from each specimen were imported into MATLAB in order to evaluate fitting coefficients for the MEPDG universal constitutive model, as presented in Equation 4-2. The calculated $k_1$, $k_2$, and $k_3$ values are provided in Table 4-2 along with the respective coefficient of determination ($R^2$).

Table 4-2. Summary of LWD Measurements and $M_r$ Model Parameters

<table>
<thead>
<tr>
<th>Soil Classification</th>
<th>Specimen No.</th>
<th>$E_{LWD}$ MPa</th>
<th>Std. MPa</th>
<th>$C_v$ %</th>
<th>$\frac{\sigma}{\delta}$</th>
<th>$k_1$</th>
<th>$k_2$</th>
<th>$k_3$</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>MH (A-7-5)</td>
<td>H1-1</td>
<td>154</td>
<td>8.92</td>
<td>3.8</td>
<td>0.87</td>
<td>1310</td>
<td>0.230</td>
<td>-1.04</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td>H2-2</td>
<td>111</td>
<td>4.32</td>
<td>1.4</td>
<td>0.63</td>
<td>1440</td>
<td>0.329</td>
<td>-2.44</td>
<td>0.99</td>
</tr>
<tr>
<td>SM (A-4a)</td>
<td>H6-1</td>
<td>32</td>
<td>3.03</td>
<td>3.8</td>
<td>0.18</td>
<td>567</td>
<td>0.808</td>
<td>-2.11</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td>H3-1</td>
<td>41</td>
<td>1.03</td>
<td>1.3</td>
<td>0.23</td>
<td>513</td>
<td>0.970</td>
<td>-3.08</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td>H2-1</td>
<td>64</td>
<td>3.07</td>
<td>2.7</td>
<td>0.36</td>
<td>634</td>
<td>0.882</td>
<td>-2.50</td>
<td>0.94</td>
</tr>
<tr>
<td>ML(A-4a)</td>
<td>H5-1</td>
<td>48</td>
<td>2.94</td>
<td>4.7</td>
<td>0.27</td>
<td>680</td>
<td>0.910</td>
<td>-2.82</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td>H4-1</td>
<td>21</td>
<td>2.07</td>
<td>4.2</td>
<td>0.12</td>
<td>666</td>
<td>0.879</td>
<td>-3.28</td>
<td>0.94</td>
</tr>
<tr>
<td></td>
<td>H8-1</td>
<td>35</td>
<td>3.05</td>
<td>4.0</td>
<td>0.20</td>
<td>555</td>
<td>0.925</td>
<td>-2.82</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td>H4-2</td>
<td>30</td>
<td>1.72</td>
<td>3.08</td>
<td>0.17</td>
<td>646</td>
<td>0.896</td>
<td>-2.44</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td>H6-2</td>
<td>20</td>
<td>0.51</td>
<td>2.1</td>
<td>0.11</td>
<td>717</td>
<td>0.673</td>
<td>-4.39</td>
<td>0.86</td>
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<tr>
<td></td>
<td>H3-2</td>
<td>56</td>
<td>2.40</td>
<td>3.6</td>
<td>0.32</td>
<td>593</td>
<td>0.874</td>
<td>-3.09</td>
<td>0.95</td>
</tr>
<tr>
<td>CL(A-4a)</td>
<td>H5-2</td>
<td>20</td>
<td>3.31</td>
<td>9.2</td>
<td>0.11</td>
<td>897</td>
<td>0.897</td>
<td>-4.26</td>
<td>0.92</td>
</tr>
</tbody>
</table>

$^a E_{LWD} (MPa)$

$^b \sigma (kPa)$

$^c \delta (\mu m)$
4.7 Effect of LWD Input Parameters

Although input parameters such as Poisson’s ratio and shape factor can be approximated for particular cases from values presented in literature, it is difficult to select appropriate values for these input parameters for site specific conditions. Poisson ratio values between 0.2 to 0.5 are recommended in the literature (Bishop 1977). Various shape factors are recommended for different scenarios. The shape factor can be varied between $\frac{\pi}{2}$ for a rigid loading plate, 1.33 and 2.67 for parabolic contact stress distribution in cohesive soils and granular soils, respectively, and 2 for a uniform contact stress distribution (Terzaghi and Peck 1967, Mooney and Miller 2007, Vennapusa and White 2007, Prima 100 software). The uncertainties associated with selecting these parameters can induce significant variation in the value of calculated modulus, as illustrated in Fig. 4-6. For the case shown, the computed $E_{LWD}$ for sample H1-1 can change from 110 up to 220 MPa by changing the shape factor from 1.33 to 2.67 at a given Poisson’s ratio of 0.2. In addition, it can be observed that the effect of Poisson’s ratio becomes more pronounced with increasing shape factor, and can produce up to a 30% change in the computed elastic modulus value.
4.8 Existing Models

As previously mentioned, White et al. (2007) Mohammad et al (2008) proposed empirical correlations to approximate the $M_r$ of subgrade soil from LWD measurements at one specific confining pressure and deviatoric stress.

The performance of these correlations in estimating the laboratory-measured $M_r$ values from the current study are both shown in Fig. 4-7. It can be seen that both correlations have generally overpredicted the measured values; however the Mohammad et al. correlation underestimates the $M_r$ of the more highly plastic soils.
4.9 Development of LWD Correlation

As previously noted, the existing models for subgrade $M_r$ determination from LWD data are limited to a specific stress level. A change in pavement structure layer thickness, axial load, and tire pressure can lead to changes in stresses within the layers. In order to overcome this restriction and eliminate uncertainties associated with selecting appropriate Poison’s ratio and shape factor values, the ratio of applied stress to surface deflection as measured during LWD testing was used. The coefficients of the MEPDG model are functionally related to the elastic modulus ($k_1$), stiffness hardening ($k_2$) and strain softening ($k_3$) behavior of the soil (Yau and Quintus, 2002). Accordingly, the proposed model was developed to correlate $k_1$, $k_2$, and $k_3$ to the ratio of $\frac{\sigma}{\delta}$ obtained from LWD measurements, with the advantage of having the ability to estimate $M_r$ at other stress levels once
these parameters are defined. The new model was developed from regression analyses on the laboratory and field measurement data from the cohesive (A-7-5) and cohesionless (A-4a) soils.

4.9.1 **MEPDG coefficients from LWD**

Multilinear regression analyses was performed on three quarters of the data set to develop a model to calculate resilient modulus indirectly at any stress level from LWD data through estimating the MEPDG formula coefficients. The proposed correlation is presented in Equation 4-6, with the definition of constants presented in Table 4-3. Figs. 4-8(a-c) shows the calculated coefficients, $k_1$, $k_2$, and $k_3$, from curve fitting of the laboratory results versus the ratio of the measured applied stress to the surface deformation in the field. The proposed model predicts the $k_1$, $k_2$ and $k_3$ coefficients with a coefficient of determination ($R^2$) of 0.71, 0.82, and 0.55.

$$k_i = C_1 + C_2 \left( \frac{\sigma}{\delta} \right)$$

Eq. (4-6)

$i:1,2,3$

<table>
<thead>
<tr>
<th></th>
<th>$C_1$</th>
<th>$C_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k_1$</td>
<td>480</td>
<td>1040</td>
</tr>
<tr>
<td>$k_2$</td>
<td>1.0</td>
<td>-0.9</td>
</tr>
<tr>
<td>$k_3$</td>
<td>-3.7</td>
<td>2.8</td>
</tr>
</tbody>
</table>

Table 4-3. Constant Coefficients of Developed Model
Following AASHTO T-307, laboratory resilient modulus test is performed at 15 different stress level, 5 deviatoric stress, 13.8, 27.6, 41.4, 55.2 and 69 kPa (2, 4, 6, 8 and 10 psi), at 3 confining pressure, 41.4, 57.6 and 13.8 kPa (6, 4 and 2 psi); which provides 15 resilient modulus values corresponding to each stress level. Hence for each specimen, 15 resilient moduli are measured at different stress level. Fig. 4-9 shows the laboratory measured vs. predicted Mr, for ¾ of the samples at 15 different stress levels. The analyses results, illustrated in Fig. 4-9, show that the proposed
model is able to compute the laboratory-measured $M_r$ with a coefficient of determination ($R^2$) of $= 0.83$.

![Figure 4-9. Laboratory-measured $M_r$ vs. that computed prediction](image)

4.9.2 Model validation

Fig. 4-10 shows laboratory-measured vs. model-computed resilient modulus values using the remaining quarter of the data set which was not used in the initial statistical correlations, at 15 different stress level. The best-fit line for the data plotted shows that the proposed model slightly underestimates resilient modulus by 7% with a coefficient of determination of 0.83. The performance of the proposed model was also evaluated by utilizing data available from two other studies by White et al. (2007) and Mohammad et al. (2008). Data from White et al. (2007) included LWD measurements as well as laboratory-measured $M_r$ data at a confining pressure $= 41.4$ kPa (6 psi) and deviatoric stress $= 69$ kPa (10 psi), for A-6 (CL), sandy lean clay; and A-1-b (SP) soil,
poorly graded sand with silt and gravel. Mohammad et al. (2007) presented LWD and Mr measurements at a confining pressure 13.8 kPa (2 psi) and deviatoric stress 41.4 kPa (6 psi), for A-4(CL-ML) and A-6 (CL-ML) soils. In order to be able to utilize this data from the literature, the ratio of $\frac{\sigma}{\delta}$ values were back calculated using Equation 4-3. The parameters utilized in the back-calculation were Poisson’s ratio of 0.35 and 0.4, for White et al. (2007) and Mohammad et al. (2008), respectively, and shape factors of $\frac{\pi}{2}$ for cohesive soils and 2 for cohesionless soils, as originally reported by the authors. As shown in Fig. 4-11, the proposed model underestimates laboratory-measured Mr by 11% with a coefficient of determination of 0.96.

Figure 4-10. Laboratory-measured Mr vs. that predicted for one quarter of data set.
Summary and Conclusions

A model for estimating $M_r$ on the basis of LWD data is presented in this paper. A performance evaluation of existing models from the literature to assess $M_r$ based on LWD data indicated a limitation related to the ability of these model to predict $M_r$ at only a single stress level. Using a linear regression analyses approach, applied to laboratory measured $M_r$ and field measured LWD data, a new model is proposed to compute $M_r$ at any desired stress state. The use of such a model exclude uncertainties involved with the assumption of parameters needed for current LWD modulus determination. In the proposed model, the ratio of applied stress to surface deformation measured by LWD is used directly to compute MEPDG universal constitutive model coefficients ($k_1$, $k_2$ and $k_3$). Based on the results obtained in this study, the following conclusion can be stated:

- The proposed model is capable of predicting $M_r$ at various stress combinations with a coefficient of determination of, $(R^2) 0.83$. Good agreement was obtained between the
LWD-predicted $M_r$ and laboratory-measured data presented in other studies, with an 11% average underprediction of $M_r$ values with $R^2 = 0.96$.

- Examination of the ability of two existing models to predict $M_r$ from in-situ LWD data, showed a general trend toward overprediction, except for the higher plasticity soils tested in this study. For this soil the Mohammad et al. correlation was seen to underestimate the measured $M_r$ values.

- Input parameters needed to calculate elastic modulus by the LWD, such as Poisson’s ratio and shape factor, can have a significant effect on the calculated $E_{LWD}$. At a given Poisson’s ratio, the $E_{LWD}$ can change by a factor of 2 depending on the shape factor selected. The effect of Poisson’s ratio was shown to increase with increasing shape factor.

- The proposed model was demonstrated to predict the resilient modulus of A-1-b (SP), A-4 (ML, SM, CL), A-6 (CL-ML), and A-7-5 (MH) soil types using the measured ratio of applied stress to surface deflection from a Prima 100 LWD device, employed in this study. Further work will need to be performed to evaluate the applicability of the proposed approach to other soil types as well.
References


CHAPTER 5: RESILIENT MODULUS PREDICATION OF SOFT LOW PLASTICITY PIEDMONT RESIDUAL SOIL FROM DYNAMIC CONE PENETROMETER

S. Hamed Mousavi, Corresponding Author
North Carolina State University
Graduate Research Assistant, Department of Civil, Construction, and Environmental Engineering, North Carolina State University, Raleigh, NC 27695-7908;
Tel: 919-995-8792; Email: smousav3@ncsu.edu

Mohammed A. Gabr
North Carolina State University
Professor, Department of Civil, Construction, and Environmental Engineering, North Carolina State University, Raleigh, NC 27695-7908;
Tel: 919-515-7904 FAX: 919-515-7908; Email: gabr@eos.ncsu.edu

Roy H. Borden
North Carolina State University
Professor, Department of Civil, Construction, and Environmental Engineering, North Carolina State University, Raleigh, NC 27695-7908;
Tel: 919-515-7630 FAX: 919-515-7908; Email: borden@ncsu.edu
ABSTRACT

Dynamic Cone Penetrometer (DCP) has been used for decades in order to estimate the shear strength and stiffness properties of the subgrade soils. There are several empirical correlations in the literature to predict the resilient modulus values at only a specific stress state from DCP data, corresponding to a predefined thicknesses of pavement layers (50 mm asphalt wearing course, a 100 mm asphalt binder course and a 200 mm aggregate base course). In this study, field-measured DCP data were utilized to estimate the resilient modulus of low plasticity subgrade Piedmont residual soil. Piedmont residual soils are in-place weathered soils from igneous and metamorphic rock, as opposed to transported or compacted soils. Hence the existing empirical correlation might not be applicable for these soils. Experimental program was conducted incorporating field DCP tests and laboratory resilient modulus testing on “undisturbed” soil specimens. The DCP tests were carried out at various locations in four test sections to evaluate subgrade stiffness variation laterally and with depth. Laboratory resilient modulus test results were analyzed in the context of the MEPDG-recommended universal constitutive model. A new approach for predicting the resilient modulus from DCP by estimating MEPDG constitutive model coefficients ($k_1$, $k_2$, and $k_3$) was developed through statistical analyses. The new model is capable of not only taking into account the insitu soil condition on the basis of field measurements, but also representing the resilient modulus at any stress state which addresses a limitation with existing empirical DCP models and its applicability for a specific case. Validation of the model is demonstrated by using data that was not used for model development, as well as data reported in the literature.

Keywords: dynamic cone penetrometer, resilient modulus, MEPDG, residual soils, subgrade
5.1 Introduction

The resilient modulus of subgrade soils is a fundamental parameter in the design of pavement structures, as recommended in the mechanical-empirical pavement design guide (MEPDG 2004). The resilient modulus is defined as the ratio of the applied cyclic axial stress to the recoverable axial strain (NCHRP project 1-28A 2004), as expressed in Equation 5-1:

\[ M_r = \frac{\sigma_{\text{cyclic}}}{\varepsilon_r} \quad \text{Eq. (5-1)} \]

Where:

- \( \sigma_d \): applied deviatoric stress
- \( \sigma_{\text{cyclic}} \): cyclic axial stress (0.9\( \sigma_d \))
- \( \varepsilon_r \): resilient axial strain

While the resilient modulus, \( M_r \), can be determined from laboratory testing, performing the test requires a well-trained operator and substantial time, as well as advanced apparatus. An alternative to laboratory testing is the use of empirical correlations developed on the basis of statistical analyses and utilizing physical and engineering properties of soils. Carmichael et al. (1985), Elliot et al. 1988, Drumm et al. (1990), Farrar and Turner (1991), and Hudson et al. (1994) all proposed models to estimate the resilient modulus of subgrade soils on the basis of material index properties. As an alternative, Hasan et al. (1994), MEPDG (2004), Herath et al. (2005), George et al. (2006), and Mohammad et al. (2008) have proposed correlations to predict \( M_r \) from \textit{insitu} dynamic cone penetrometer (DCP) data. The advantage of using DCP is that of testing the soil in its natural density and moisture content state. These correlations, however, provide the \( M_r \) at only one specific stress state; that is, at a confining pressure of 13.8 kPa (2 psi) and a deviatoric stress of 41.7 kPa (6 psi). These values represent the stress level at the top of the subgrade layer under
standard single axle loading of 80 kN (18 kips) and tire pressure of 689 kPa (100 psi) with a 50 mm asphalt wearing course, a 100 mm asphalt binder course and a 200 mm aggregate base course (Mohammad et al. 2008, Rahim 2004, Asphalt Institute 1989). Since the resilient modulus depends on the confining pressure and applied deviatoric stress, any change in the pavement structure, axle load and tire pressure, will lead to a change in the stress state at the surface of the subgrade. Accordingly, the predicted \( M_r \) by existing correlations may not be representative of the field stress conditions.

On the other hand, many studies have been performed over the past two decades to model the stress dependency of the resilient modulus by predicting the coefficients of a general constitutive model (e.g. Dunlap 1963, Seed et al. 1967, Witzack and Uzan 1988, Pezo 1993, NCHRP project 1-28A 2004) on the basis of index soil properties. These properties included water content, \( w\% \); plastic limit, PL; liquid limit, LL; \% passing the No. 4 sieve, \( P_4 \); \% passing the No. 200 sieve, \( P_{200} \); etc. Yau and Von Quintus (2002), Elias and Titi (2006), Nazzal and Mohammad (2010), and Titi and English (2011), have each proposed different models to estimate the MEPDG (2004) constitutive model coefficients \((k_1, k_2 \text{ and } k_3)\), expressed in Equation 5-2; however these model have been developed based on the compacted specimens and do not consider the properties of the undisturbed soil in its natural state.

\[
M_r = k_1 \cdot Pa \cdot \left( \frac{\theta}{Pa} \right)^{k_2} \cdot \left( \frac{\tau}{Pa} + 1 \right)^{k_3}
\]

Eq. (5-2)

Where:

- \( M_r \): resilient modulus
- \( Pa \): atmospheric pressure
- \( \sigma_1, \sigma_2, \sigma_3 \): principal stresses
- \( \theta = \sigma_1 + 2\sigma_3 \): bulk stress
- \( \tau_{oct} = \frac{\sqrt{2}}{3} (\sigma_1 - \sigma_3) \): octahedral shear stress
- \( k_i \): regression constants
In the MEPDG model, $M_r$ is linearly influenced by $k_1$, while the exponents $k_2$ and $k_3$ define the rate of increase and decrease, respectively, of stiffness hardening and soil softening (Yau and Quintus, 2002) with respect to the confining and deviatoric stresses. However, as currently formulated, all three coefficients are independent of the stress state.

A model is proposed in this paper to calculate the low plasticity Piedmont residual subgrade soil resilient modulus from the DCP data. Piedmont residual soils are in-place weathered soils from igneous and metamorphic rock, as opposed to transported or compacted soils. Hence the existing empirical correlation might not be applicable for these soils (Borden et al. 1996). The model is developed based on the *insitu* DCP measurements and laboratory resilient modulus on the undisturbed specimens retrieved from Shelby tubes. The model is based on calculating $M_r$ by predicting the constitutive model coefficients ($k_1$, $k_2$, and $k_3$) from *insitu* DCP data. By utilizing *insitu* measured DCP data in predicting the constitutive model coefficients, the proposed approach allow for taking into account the stress dependency of the resilient modulus, as well as properties of the soil in its natural state. The validity of the proposed model is examined with the portion of data set not used in the model development, as well as reported data in the literature.

5.2 Background

The dynamic cone penetrometer (DCP) is a portable instrument that has been used widely in geotechnical and pavement design for estimating the shear strength and stiffness properties of soils (AASHTO, 1993; Gabr et al., 2000; Gabr et al., 2001; Chen et al., 2005). As shown in Fig. 5-1, and presented in ASTM D6951, the DCP consists of an 8 kg sliding hammer, with a 57.5 cm (22.6 in) drop height, an 111 cm (44 in) driving shaft and a $60^\circ$ angle cone tip. During the DCP test, the sliding hammer falls 57.5 cm vertically and drives the cone tip attached to the bottom of the DCP rod into the ground. The penetration depth is recorded after each drop (blow) on a vertical stake.
positioned next to the DCP rod. Dynamic cone penetration index (DCPI) is expressed as inch or mm per blow.

Several correlations have been proposed in the literature between DCPI and soil shear strength and stiffness properties, such as those for: the California Bearing Ratio, CBR, (NCDOT, 1998; Gabr et al., 2000); the undrained shear strength, Su, (Ayres, 1989); the elastic modulus, E, (Chai and Roslie, 1998; Abu-Farsakh et al., 2004; Chen et al., 2005); and the resilient modulus, Mr, (AASHTO, 1993; Hassan, 1996; Herath et al., 2005).

Figure 5-1. Dynamic cone penetrometer sketch (after ASTM D6951).

Existing empirical correlations, which correlate DCPI to Mr, are summarized in Table 5-1. These models are capable of providing an estimate of site stiffness properties soils; however, they are restricted to a confining pressure of 13.79 kPa (2 psi) and a deviatoric stress of 41.37 kPa (6 psi).
Table 5-1. Previous direct DCP models.

<table>
<thead>
<tr>
<th>Researcher</th>
<th>Correlation equation</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hassan (1996)</td>
<td>( M_t = 7013.065 - 2040.783 \ln(DCPI) )</td>
<td>( M_t ): psi, DCPI: in/blow</td>
</tr>
<tr>
<td>George and Uddin (2000)</td>
<td>( M_t = 235.3(DCPI)^{-0.48} )</td>
<td>( M_t ): MPa, DCPI: mm/blow</td>
</tr>
<tr>
<td>MEPDG (Powel) (2004)</td>
<td>( CBR = \frac{292}{DCPI^{1.12}} ) (Webster 1994)</td>
<td>( M_t ): MPa, DCPI: mm/blow</td>
</tr>
<tr>
<td></td>
<td>( M_t = 17.58(CBR)^{0.64} )</td>
<td></td>
</tr>
<tr>
<td>Herath et al. (2005)</td>
<td>( M_t = 16.25 + \frac{928.24}{DCPI} )</td>
<td>( M_t ): MPa, DCPI: mm/blow</td>
</tr>
<tr>
<td>Mohammad et al. (2008)</td>
<td>( M_t = \frac{1045.9}{DCPI^{0.96}} )</td>
<td>( M_t ): MPa, DCPI: mm/blow</td>
</tr>
</tbody>
</table>

5.3 Experimental Program

The experimental program included a series of laboratory resilient modulus and insitu DCP tests. The sampling and field testing program were performed at four 4.88-m (16-ft) wide by 15.24-m (50-ft) long test sections located in the Piedmont area, North of Greensboro, North Carolina. The DCP tests were performed at four locations in each test section, as shown in Fig. 5-2. The laboratory testing, including the resilient modulus test and index properties, was performed on undisturbed soil specimens retrieved from Shelby tubes. These tubes were taken from boreholes located between each pair of DCP tests, as indicated in Fig. 5-2.
5.3.1 Materials tested

The physical property tests included specific gravity, grain size distribution, Atterberg limits and standard compaction by following ASTM D854, D2216, D4318, and D698, respectively. These tests were conducted on the specimens after the resilient modulus tests were completed. The grain size distributions and properties of soil specimens are summarized in Fig. 5-3 and Table 5-2. The site soils were classified as A-4. From standard compaction tests, the optimum water contents ($w_{opt}$) and maximum dry densities ($\gamma_{d,max}$) were determined as, 11% and 20 kN/m$^3$, respectively.
Figure 5-3. Range of grain size distributions of materials tested.

Table 5-2. Engineering properties of resilient modulus test specimens

<table>
<thead>
<tr>
<th>Classification</th>
<th>Specimen Number</th>
<th>Depth (^1)</th>
<th>(\gamma_{\text{mois}}) (^2)</th>
<th>Gs</th>
<th>w %</th>
<th>e</th>
<th>LL</th>
<th>PL</th>
<th>PI</th>
<th>(P_{200}) %</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-4</td>
<td>H2-1</td>
<td>18</td>
<td>19.6</td>
<td>2.64</td>
<td>14</td>
<td>0.51</td>
<td>15</td>
<td>12</td>
<td>4</td>
<td>47</td>
</tr>
<tr>
<td></td>
<td>H3-1</td>
<td>8</td>
<td>18.9</td>
<td>2.65</td>
<td>17</td>
<td>0.61</td>
<td>20</td>
<td>17</td>
<td>3</td>
<td>49</td>
</tr>
<tr>
<td></td>
<td>H3-2</td>
<td>23</td>
<td>19.7</td>
<td>2.62</td>
<td>16</td>
<td>0.51</td>
<td>18</td>
<td>17</td>
<td>2</td>
<td>51</td>
</tr>
<tr>
<td></td>
<td>H3-3</td>
<td>61</td>
<td>19.1</td>
<td>2.63</td>
<td>19</td>
<td>0.63</td>
<td>14</td>
<td>10</td>
<td>4</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>H4-1</td>
<td>0</td>
<td>19.5</td>
<td>2.60</td>
<td>22</td>
<td>0.63</td>
<td>17</td>
<td>20</td>
<td>3</td>
<td>56</td>
</tr>
<tr>
<td></td>
<td>H4-2</td>
<td>15</td>
<td>20.2</td>
<td>2.62</td>
<td>15</td>
<td>0.47</td>
<td>16</td>
<td>19</td>
<td>3</td>
<td>55</td>
</tr>
<tr>
<td></td>
<td>H4-3</td>
<td>61</td>
<td>18.4</td>
<td>2.67</td>
<td>21</td>
<td>0.74</td>
<td>10</td>
<td>14</td>
<td>4</td>
<td>44</td>
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<td></td>
<td>H5-1</td>
<td>0</td>
<td>20.4</td>
<td>2.61</td>
<td>13</td>
<td>0.46</td>
<td>22</td>
<td>19</td>
<td>2</td>
<td>51</td>
</tr>
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<td></td>
<td>H6-1</td>
<td>8</td>
<td>19.2</td>
<td>2.65</td>
<td>17</td>
<td>0.58</td>
<td>13</td>
<td>10</td>
<td>3</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>H6-2</td>
<td>23</td>
<td>19.6</td>
<td>2.61</td>
<td>21</td>
<td>0.59</td>
<td>19</td>
<td>15</td>
<td>3</td>
<td>51</td>
</tr>
<tr>
<td></td>
<td>H6-3</td>
<td>61</td>
<td>18.8</td>
<td>2.63</td>
<td>21</td>
<td>0.7</td>
<td>18</td>
<td>12</td>
<td>5</td>
<td>49</td>
</tr>
<tr>
<td></td>
<td>H8-1</td>
<td>8</td>
<td>19.6</td>
<td>2.61</td>
<td>17</td>
<td>0.52</td>
<td>19</td>
<td>17</td>
<td>2</td>
<td>54</td>
</tr>
</tbody>
</table>

\(^1\) cm

\(^2\) kN/m\(^3\)

\(^3\) Hi-j: i: number of borehole, j: number of sample
5.3.2 Laboratory testing

The resilient modulus tests were performed following the AASHTO T-307 protocol on twelve 15.24-cm (6-in) tall, undisturbed specimens retrieved from Shelby tubes at their natural water content, as shown in Fig. 5-4(a). The resilient modulus test performed at 15 stress combinations that include five deviatoric stress levels 13.79, 27.58, 41.37, 55.16 and 68.95 kPa (2, 4, 6, 8 and 10 psi) at each of three applied confining pressures of 41.37, 57.58 and 13.79 kPa (6, 4 and 2 psi); at resilient modulus test apparatus, Fig. 5-4(b).

Figure 5-4. a) Undisturbed resilient modulus specimen b) Resilient modulus test apparatus.

Figs. 5-5 (a,b) shows the laboratory-resilient modulus test results for two specimens H4-1 and H2-1, which are representation of the upper and lower range of the grain size distribution. As shown in Figs. 5-5, hardening and softening effects of confining pressure and deviatoric stress,
respectively, can be observed. Fig 5-6. Shows the range of the resilient modulus values at different deviatoric stress levels. From Figs. 5-5 and 5-6, it can be seen that the hardening effect of the confining pressure on the resilient modulus is more pronounced at smaller deviatoric stress level. As shown in Fig. 5-6 the resilient modulus values can vary between 15 to 80 MPa depending on confining and deviatoric stress levels.
Figure 5-5. Laboratory resilient modulus test results a) specimen H4-1 b) specimen H2-1.
The laboratory-measured resilient moduli were analyzed in the context of the NCHRP 1-28A (MEPDG) constitutive model as presented in Equation 5-2. Figs. 5-7 (a,b) shows the performance of Equation 5-2 in back-calculating the laboratory-measured \( \text{Mr} \) using curve-fitted \( k_1, k_2, \) and \( k_3 \) for specimens H4-1 and H2-1. As shown in Figs. 5-7 and Table 5-3, the laboratory resilient modulus test results can be fitted in MEPDG correlation with an general \( R^2 \) higher than 0.94. The \( k_1, k_2 \) and \( k_3 \) coefficients vary between 500 to 800, 0.673 to 0.970, and -2.11 to 4.58, respectively, which are compatible with reported data by Yao and Van (2002), and Titi and English (2011) for A-4 soils.
Figure 5-7. Laboratory-measured vs Curve fitted Mr. from MEPDG correlation a) specimen H4-1 b) specimen H2-1.
5.3.3 *In-situ testing: Dynamic Cone Penetrometer*

The DCP tests were performed at the four locations on the centerline of each test section, as shown in Fig. 5-2. The DCPI values, corresponding to the location of resilient modulus specimens and the MEPDG coefficients (k₁, k₂, and k₃) calculated from resilient modulus laboratory results are summarized in Table 5-3.

Table 5-3. DCP measurement and resilient modulus model parameters

<table>
<thead>
<tr>
<th>Classification</th>
<th>Sample Number</th>
<th>DCPI¹</th>
<th>k₁</th>
<th>k₂</th>
<th>k₃</th>
<th>R²</th>
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<tbody>
<tr>
<td>A-4</td>
<td>H2-1</td>
<td>52</td>
<td>634</td>
<td>0.882</td>
<td>-2.50</td>
<td>0.94</td>
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<tr>
<td></td>
<td>H3-1</td>
<td>46</td>
<td>513</td>
<td>0.970</td>
<td>-3.08</td>
<td>0.96</td>
</tr>
<tr>
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<td>H3-2</td>
<td>57</td>
<td>593</td>
<td>0.874</td>
<td>-3.09</td>
<td>0.95</td>
</tr>
<tr>
<td></td>
<td>H3-3</td>
<td>57</td>
<td>488</td>
<td>0.897</td>
<td>-4.25</td>
<td>0.86</td>
</tr>
<tr>
<td></td>
<td>H4-1</td>
<td>82</td>
<td>666</td>
<td>0.879</td>
<td>-3.28</td>
<td>0.94</td>
</tr>
<tr>
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<td>H4-2</td>
<td>55</td>
<td>646</td>
<td>0.895</td>
<td>-2.44</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td>H4-3</td>
<td>55</td>
<td>803</td>
<td>0.698</td>
<td>-3.86</td>
<td>0.96</td>
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<td></td>
<td>H5-1</td>
<td>44</td>
<td>680</td>
<td>0.910</td>
<td>-2.82</td>
<td>0.96</td>
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<tr>
<td></td>
<td>H6-1</td>
<td>39</td>
<td>567</td>
<td>0.808</td>
<td>-2.11</td>
<td>0.96</td>
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<td>H6-2</td>
<td>49</td>
<td>717</td>
<td>0.673</td>
<td>-4.39</td>
<td>0.86</td>
</tr>
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<td>H6-3</td>
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<td>812</td>
<td>0.679</td>
<td>-4.58</td>
<td>0.95</td>
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<td></td>
<td>H8-1</td>
<td>46</td>
<td>554</td>
<td>0.924</td>
<td>-2.82</td>
<td>0.96</td>
</tr>
</tbody>
</table>

¹ mm/blow

5.4 *Applicability of Previous Models*

Empirical correlations for estimating the resilient modulus are grouped into two categories: 1) correlations which directly predict the Mr from DCPI and 2) correlations that predict the resilient modulus indirectly from the universal constitutive model coefficients (k₁, k₂ and k₃), where these coefficients are estimated on the basis of basic physical properties of soils.

5.4.1 *Empirical DCP models*

In order to evaluate the ability of the existing models to predict the Mr values measured in the current study, the models in Table 5-1 were used with the test-site measured DCPI data. The results
are plotted in Fig. 5-8, and show that the existing models have generally over-predicted the laboratory-measured resilient modulus values, at a confining pressure 13.79 kPa (2 psi) and a deviatoric stress 41.37 kPa (6 psi), with the exception of Mohammad et al.’s correlation, which consistently underestimated the resilient modulus. Fig. 5-9 shows the performance of these empirical models in predicting the resilient modulus of the laboratory-measured resilient modulus data set presented by Cowell et al. (2012). No consistent trend is observed in performance of these models. A summary of the coefficient of determination, $R^2$, and root mean squared error, RMSE, of the performance of the existing model in estimating the laboratory-measured resilient modulus values of both data set is presented in Table 5-4. The inconsistency in predicting the measured Mr values might be attributed to the fact that these correlations are empirical in nature, and they are most applicable to soil types similar to those for which the models were developed.
Figure 5-8. Laboratory-measured $M_r$ (this study) vs that predicted from existing direct DCP models.
Figure 5-9. Laboratory-measured Mr (Cowell et al 2012) vs that predicted from empirical DCP models

Table 5-4. Performance of the existing empirical DCP models

<table>
<thead>
<tr>
<th></th>
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<th></th>
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</tr>
</thead>
<tbody>
<tr>
<td>This Study</td>
<td>R² 0.16</td>
<td>&lt; 0</td>
<td>&lt; 0</td>
<td>0.34</td>
<td>&lt; 0</td>
</tr>
<tr>
<td></td>
<td>RMSE 9.1</td>
<td>20.2</td>
<td>13.3</td>
<td>8.1</td>
<td>12.5</td>
</tr>
<tr>
<td>Cowell et al. 2012</td>
<td>R² -</td>
<td>&lt; 0</td>
<td>-</td>
<td>&lt; 0</td>
<td>&lt; 0</td>
</tr>
<tr>
<td></td>
<td>RMSE -</td>
<td>15.1</td>
<td>-</td>
<td>18.7</td>
<td>15.6</td>
</tr>
</tbody>
</table>

5.4.2 Models that account for stress level

In order to develop a model to predict Mr at any stress level, several studies have been undertaken to develop correlations that estimate the universal constitutive model fitting coefficients (k₁, k₂,
and $k_3$) from basic physical properties of soils. Yao and Van (2002) and Elias and Titi (2006), proposed different correlations to predict $k_1$, $k_2$, and $k_3$ to calculate Mr for cohesive and cohesionless subgrade soils. Nazzal and Mohammad (2010) proposed a correlation for predicting $k_1$, $k_2$, and $k_3$ from index properties of A-4, A-6, A-7-5, and A-7-6 soils. These models are summarized in Table 5-5. Fig. 5-10(a-c) shows the generally unsatisfactory performance of these models in predicting the fitting coefficients. Negative $k_1$, and $k_2$ values were predicted from Nazzal’s correlations, which are not shown in Figs. 5-10(a-b). The better performance of the other models can be explained by noting that these researchers proposed separate equations for high plasticity soils and cohesionless soils, which resulted in different, and more appropriate, Mr predictions for a given DCPI value.

The predicted Mr values by the Yao and Van (2002) and Elias and Titi (2006) models are presented in Fig. 5-11(a-b). As shown in Fig. 5-11(a), resilient modulus values predicted by the Elias and Titi (2006) equation are significantly greater than the laboratory-measured Mr values. Fig. 5-11(b) shows that the Yao and Van (2002) model on average overestimated the Mr values by 34% with an $R^2=0.56$. Due to Nazzal and Mohammad’s model predicting negative $k_1$ and $k_2$ values, resilient modulus values were not able to be predicted.
Table 5-5. Published models for predicting \( k_1, k_2 \) and \( k_3 \) from material properties.

<table>
<thead>
<tr>
<th>Researchers</th>
<th>Correlation</th>
<th>Soil type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yao and Van (2002)</td>
<td>( k_1 = 1.3577 + 0.0106(%Clay) - 0.0437W_s )</td>
<td>Fine-Grained subgrade soils</td>
</tr>
<tr>
<td></td>
<td>( k_2 = 0.5193 - 0.0073P_4 + 0.0095P_{40} - 0.0027P_{200} - 0.0030LL )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(-0.0049W_{opt})</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( k_3 = 1.4258 - 0.0288P_4 + 0.0303P_{40} - 0.0521P_{200} + 0.0251(%Silt))</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(+0.0535LL - 0.0672W_{opt} - 0.0026\gamma_{opt} + 0.0025\gamma_s - 0.6055(\frac{W_s}{W_{opt}}))</td>
<td></td>
</tr>
<tr>
<td>Elias and Titi (2006)</td>
<td>( k_1 = 8642.873 + 132.643P_{200} - 428.067(%Silt) - 254.685PI + 197.23 \gamma_d )</td>
<td>Plastic coarse-grained soils</td>
</tr>
<tr>
<td></td>
<td>(-381.4(\frac{W_s}{W_{opt}}))</td>
<td>(P&lt;50% and small PI)</td>
</tr>
<tr>
<td></td>
<td>( k_2 = 2.325 - 0.00853P_{200} + 0.02579LL - 0.06224PI - 1.7338(\frac{\gamma_d}{\gamma_{d-max}}))</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(+0.20911(\frac{W_s}{W_{opt}}))</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( k_3 = -32.5449 + 0.7691P_{200} - 1.137(%Silt) + 31.5542(\frac{\gamma_d}{\gamma_{d-max}}))</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(-0.4128(\frac{W_s}{W_{opt}}))</td>
<td></td>
</tr>
<tr>
<td>Nazzal and Mo.</td>
<td>( \ln(k_1) = 1.334 + 0.0127P_{200} + 0.016LL - 0.036\gamma_{d-max} - 0.011MCCL )</td>
<td>A-4, A-6, A7-5 and A7-6</td>
</tr>
<tr>
<td>Mo. (2010)</td>
<td>(+0.001MCDDmaxPI)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(+0.00324(MCDDP)^{1.28} - 0.875P_{200})</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( k_2 = 0.722 + 0.0057LL - 0.00454(MCDDmaxPI)^{0.641} )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(+0.0324(MCDDP)^{1.28} - 0.875P_{200})</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( k_3 = -7.48 + 0.038LL + 0.235(\frac{\gamma_d}{W_s}) - 0.0008(MCPI) + 0.033\gamma_{d-max})</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(-0.016(MCDDP))</td>
<td></td>
</tr>
</tbody>
</table>
Figure 5-10. Computed vs predicted the constitutive model coefficients by previous models: a) $k_1$; b) $k_2$; and c) $k_3$
Figure 5-11. Laboratory-measured vs predicted Mr from a) Elias ant Titi (2006), and b) Yao and Van (2002) equations.
5.5 Proposed DCP Correlation

Multilinear statistical analyses were performed to develop an approach to calculate the resilient modulus by predicting \( k_1, k_2, \) and \( k_3 \) from the \textit{in situ} DCP test data. The multilinear regression analyses were performed on three quarters of the data set to develop a model that indirectly computes the resilient modulus at any desired stress state.

The proposed model is presented in Equation 5-3, with the model constants presented in Table 5-6. As shown in Fig. 5-12, the calculated \( M_r \) values by the proposed model and laboratory-measured resilient modulus are correlated with a coefficient of determination, \( R^2 \), equal to 0.70.

\[
k_i = C_1 + C_2 \ln(DCPI), \quad i:1,2,3 \tag{5-3}
\]

<table>
<thead>
<tr>
<th>Coefficients</th>
<th>( C_1 )</th>
<th>( C_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( k_1 )</td>
<td>2310 (911)(^1)</td>
<td>-377</td>
</tr>
<tr>
<td>( k_2 )</td>
<td>-0.3 (0.7)(^1)</td>
<td>0.3</td>
</tr>
<tr>
<td>( k_3 )</td>
<td>-10 (-4.5)(^1)</td>
<td>1.7</td>
</tr>
</tbody>
</table>

\(^1\) Constants for English Units
SI Units: DCPI: mm/blow,
English Units: DCPI: in/blow,
5.5.1 Proposed model validation

The validity of the proposed model was examined using the quarter of the data set which was not used in the statistical analyses and was selected arbitrarily, as well as additional data from the literature. The performance of the proposed model in predicting the resilient modulus of the quarter of the data is shown in Fig. 5-13. The line of equality is added for clarity. It can be seen that the proposed model slightly underestimate the resilient modulus by 4% and the data are correlated with an $R^2 = 0.73$. 

Figure 5-12. Laboratory-measured vs calculated Mr by the proposed model.
Figure 5-13. Laboratory-measured vs predicted $\text{Mr}_t$ by the proposed model for the quarter of the data set.

The data set from Cowell et al. (2012) was also used to test the proposed model. The subgrade soil for this project consisted of low plasticity SM and SC (A-4). The data set by Cowell et al. (2012) included $\text{Mr}$ values from tests on undisturbed specimens collected from the Coastal Plain of North Carolina, and insitu DCP measurements, summarized in Table 5-7. As shown in Fig. 5-14, the predicted $\text{Mr}$ by the proposed model show reasonably good agreement with the laboratory-measured $\text{Mr}$ values.

The performance of the proposed model was also investigated through the use of data presented by Mohammad et al. (2007, 2008). The reported data include laboratory and field DCP measurements, summarized in Table 5-7, and laboratory $\text{Mr}$ test data on the low plasticity soil specimens tested at a confining pressure of 13.8 kPa (2 psi) and deviatoric stress of 41.4 kPa (6 psi).
The data plotted in Fig. 5-14 shows that the proposed model underestimates Mr of this data set by 8% with an $R^2$ of 0.53. By comparing the performance of the proposed model to that of existing Mr predicting correlations, presented in Table 5-4 and Fig. 5-11, it can be seen that the proposed model provides significantly improved predictive capability, with values slightly less than laboratory-measured values and with a higher coefficient of determination.

Table 5-7. DCP data in the literature with corresponding predicted coefficients by proposed model

<table>
<thead>
<tr>
<th>Authors</th>
<th>Sample ID</th>
<th>DCPI(mm/blow)</th>
<th>$k_1$</th>
<th>$k_2$</th>
<th>$k_3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cowell et al. (2012)</td>
<td>ST7</td>
<td>24</td>
<td>933</td>
<td>0.682</td>
<td>-4.62</td>
</tr>
<tr>
<td></td>
<td>ST6</td>
<td>17</td>
<td>1063</td>
<td>0.578</td>
<td>-5.20</td>
</tr>
<tr>
<td></td>
<td>ST5</td>
<td>36</td>
<td>780</td>
<td>0.804</td>
<td>-3.94</td>
</tr>
<tr>
<td></td>
<td>ST4</td>
<td>77</td>
<td>494</td>
<td>0.995</td>
<td>-2.66</td>
</tr>
<tr>
<td></td>
<td>ST3</td>
<td>37</td>
<td>770</td>
<td>0.813</td>
<td>-3.89</td>
</tr>
<tr>
<td></td>
<td>ST2</td>
<td>65</td>
<td>558</td>
<td>0.983</td>
<td>-2.94</td>
</tr>
<tr>
<td>Mohammad et al. (2007)</td>
<td>Clayey silt-1</td>
<td>26.1</td>
<td>902</td>
<td>0.707</td>
<td>-4.48</td>
</tr>
<tr>
<td></td>
<td>Clayey silt-2</td>
<td>18.8</td>
<td>1025</td>
<td>0.609</td>
<td>-5.04</td>
</tr>
<tr>
<td></td>
<td>Clayey silt-3</td>
<td>27</td>
<td>889</td>
<td>0.718</td>
<td>-4.42</td>
</tr>
<tr>
<td></td>
<td>Clayey silt (ALF)</td>
<td>29</td>
<td>862</td>
<td>0.739</td>
<td>-4.30</td>
</tr>
<tr>
<td></td>
<td>LA-182</td>
<td>36</td>
<td>780</td>
<td>0.804</td>
<td>-3.94</td>
</tr>
<tr>
<td></td>
<td>LA-334C(2)</td>
<td>18.2</td>
<td>1037</td>
<td>0.599</td>
<td>-5.09</td>
</tr>
<tr>
<td></td>
<td>LA-334C(5)</td>
<td>19.3</td>
<td>1015</td>
<td>0.616</td>
<td>-4.99</td>
</tr>
<tr>
<td></td>
<td>LA-334C(8)</td>
<td>18.6</td>
<td>1029</td>
<td>0.605</td>
<td>-5.05</td>
</tr>
</tbody>
</table>
Summary and Conclusion

A laboratory testing program, including resilient modulus and index property tests, and \emph{insitu} Dynamic Cone Penetrometer (DCP) tests were performed to establish a model for estimating Mr model parameters. Comprehensive statistical analyses were conducted and a new model is proposed for calculating the Mr of subgrade soil at any desired stress level. This model uses \emph{insitu} DCP data indirectly, by predicting the MEPDG universal constitutive model fitting coefficients $k_1$, $k_2$, and $k_3$. Based on the results presented in this paper, the following conclusions are advanced:

- Good agreement was obtained between calculated Mr values from the proposed model and the laboratory-measured resilient modulus data, with a coefficient of determination of 0.70.

Figure 5-14. Laboratory-measured vs predicted Mr by the proposed model for data presented in the literature.
• Examination of the performance of the proposed model with a quarter of the data set which was not included in the statistical analyses, indicated that, on average, the proposed model underestimated the Mr by 4% with a coefficient of determination of 0.73.

• Predicted Mr values by the proposed model were seen to be in reasonably good agreement with the laboratory-measured Mr presented in the literature, however with a lower coefficient of determination of 0.53.

• The evaluation of existing models which directly estimate the Mr of soils from the DCP measurements showed that they overestimated measured Mr values. In addition, the validity of these models only at one determined stress level limits their applicability to one particular pavement structure.

• The assessment of existing empirical models that predict the universal constitutive model fitting coefficients from basic physical properties of soils, yielded poor predictions of Mr for the soils tested in this study.

• The proposed model is capable of predicting the resilient modulus of low plasticity Piedmont residual soils (A-4), with PI<5, and 40%<P200<55%; at any stress state. Further work will need to be performed to evaluate its applicability to other soils.
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Vanapalli, S. K. and Han, Z., 2010, “Prediction of the Resilient Modulus of Unsaturated Fine-Grained Soils”, Proc. of Int. Conf. on Advances in Civil Engineering, AETACE.


CHAPTER 6: CORRELATION OF DCPI TO PROOFROLLER TEST TO ASSESS SUBGRADE SOILS STABILIZATION CRITERION

S. Hamed Mousavi, Corresponding Author

North Carolina State University
Graduate Research Assistant, Department of Civil, Construction, and Environmental Engineering, North Carolina State University, Raleigh, NC 27695-7908;
Tel: 919-995-8792; Email: smousav3@ncsu.edu

Mohammed A. Gabr

North Carolina State University
Professor, Department of Civil, Construction, and Environmental Engineering, North Carolina State University, Raleigh, NC 27695-7908;
Tel: 919-515-7904 FAX: 919-515-7908; Email: gabr@eos.ncsu.edu

Roy H. Borden

North Carolina State University
Professor, Department of Civil, Construction, and Environmental Engineering, North Carolina State University, Raleigh, NC 27695-7908;
Tel: 919-515-7630 FAX: 919-515-7908; Email: borden@ncsu.edu
ABSTRACT
The proof roller test has been traditionally carried out as a technique for subgrade quality assessment in road construction and the induced pumping, and permanent deformation are used as the basis subgrade’s pass/fail criteria. The objective of the study presented herein is to assess the feasibility of using the DCPI to discern the need for undercut and the quality of subgrade. Deformation behavior of the subgrade soil layer was evaluated under cyclic proof roll loading in 3D FEM models with the analysis taking into account the effect of the superposition of the multi-loaded areas. It was determined that the failure deformation occurs at DCPI value of 38 mm/blow for single layer subgrade soil with the Mr/E₅₀ of 6; which is consistent with NCDOT subgrade undercut criteria. A subgrade stabilization recommendation chart was developed based on the criterion of 25 cm plastic rut depth on top of the subgrade after two passes proof roller and incorporation of in-situ DCP measurement and numerical proof roll test results. Strong agreement were obtained with the 38 mm/blow NCDOT undercut criteria. The numerical results indicated that the subgrade with 30 cm stiff layer (DCPI < 20 mm/blow) on top does not require any subgrade soil stabilization, regardless of subgrade layers underneath, while subgrade needs to be stabilized if there is a very soft soil layer top 30 cm of subgrade (DCPI > 60 mm/blow), unless the sublayers underneath are composed of stiff soil (DCPI < 20 mm/blow).

Keyword: proof roll, subgrade, stabilization, FEM, DCP
6.1 Introduction

The proof roller test has been carried out as a technique for subgrade quality assessment in road construction for decades. A proof roller is a loaded single-axle, four-wheel trailer, as shown in Fig. 6-1, for compacting the roadbed and testing the roadbed for stability and uniformity of compaction. Based on NCDOT specifications (NCDOT 2012, Section 260), the maximum center-to-center spacing between adjacent wheels is 32 in., with tire pressure between 470, and 500 kPa (68 psi, and 72 psi). The load capacity of the trailer is from 427 to 445 kN (48 to 50 tons). A proof roll trailer is pulled over the subgrade soil embankment, and the induced pumping and permanent deformation are used as the subgrade’s pass/fail basis criteria. Generated pumping under proof roll loading is associated with the stiffness properties of the deep soft layers, while rutting is related to plastic deformation within shallow layers. The acceptable rutting depths varies between zero to 80 mm (3 in.), across the U.S., based on the proof roll trailer specification and state criteria.

Figure 6-1. Proof roll trailer (Borden et al., 2010)
Based on the North Carolina Department of Transportation specification, the induced surface deformation by proof roll test, less than 25 mm (1 in.) is considered as acceptable. Colenbrander and Smith (2007) performed numerical analyses and reported that tire pressure of proof roll trailer has a significant effect on deformation, and subgrades with CBRs greater than 6% are adequate to withstand proof-rolling and subsequent construction traffic.

Hambleton and Drescher (2008) developed two theoretical criterions based on analytical and FEM methods, to relate surface plastic deformation of single homogenous subgrade layer to wheel geometry, wheel load, and soil strength parameters (friction angle and cohesion), while the soil stiffness plays a secondary role. White et al. (2009) performed a series of in-situ dynamic cone penetrometer, DCP, test conjunction to the proof roll test. The observation showed the excessive surface rutting corresponding to the deep soft layer. A chart solution was proposed from analytical analyses of bearing capacity of two layers subgrade to determine acceptable shear strength properties of layered soil.

Validity of existing stabilization criteria in the literature were examined for the Residual Piedmont subgrade soil, by utilizing field measured DCP data. A new subgrade stabilization recommendation chart was developed for layered subgrade profile, based on the incorporation of in-situ DCP measurement and computed deformation response of subgrade soil from advanced 3D numerical analyses, under cyclic loading corresponding to the proof roller passes with taking into account the effect of the superposition of the multi-loaded areas.

6.2 Background

NCDOT has been conventionally employed DCP in-situ method for subgrade and stabilization quality control. Based on the NCDOT criteria, subgrade soil with a DCPI value less than 38 mm/blow needs to be undercut or stabilized. The DCP is a portable instrument; which consists of
an 8-kg sliding hammer, 57.5-cm (22.6-in.) drop height, 111-cm (44-in.) drive shaft, and a cone tip with a 60° angle, in accordance with ASTM D6951 specifications. During DCP testing, the sliding hammer falls 57.5 cm vertically and drives the cone tip attached to the bottom of the DCP rod into the ground. As previously mentioned White et al. (2009), developed a stabilization design graph from analytical analyses, which validated by experimental data. The stability of the two layered subgrade soils was evaluated for different thickness of the first layer (0 to 0.5 m) and a given range of DCPI value of the first and second layer. Borden et al. (2010) proposed criteria to indicate the need for undercutting based on the results of prototype laboratory-scale tests and extensive numerical analyses. They used plane strain and axisymmetric numerical models to evaluate the effects of the stiffness of deep layers on pumping and the influence of the shear strength properties of shallow layers on surface rutting. The undercut criteria proposed by Borden et al. (2010) are based on an acceptable rut depth of less than 25.4 mm (1 in.), and an acceptable performance capacity ratio that is equal to 1.5, where the performance capacity ratio is defined as:

$$\xi = \frac{\text{Performance Capacity}}{70 \text{ psi}}$$

The performance capacity is defined as the pressure that corresponds to the asymptotic value of the pressure-deformation curve under wheel loading. Borden et al. (2010) proposed two undercut criteria charts: one for axisymmetric loading and the second for the plane strain condition, thereby simulating the conditions of local bearing capacity failure and deep layer pumping, respectively. Equation 6-1 estimates the normalized settlement at the center of the loaded area for the axisymmetric loading condition. Details can be found in the final report by Borden et al. (2010).

$$\left(\frac{\delta}{B}\right) = \left\{-2.13 \times 10^{-6} + 3.61 \left(\frac{E}{p_a}\right)^{-1}\right\} + \left(0.085 + 1.32 e^{-0.25\phi}\right) \left(\frac{E}{p_a}\right)^{-1} \left(\frac{C}{p_a}\right)^{0.22-7.75e^{-0.046\phi}}$$

Eq. (6-1)
6.3 Experimental Program

The experimental program included constructing three 4.88-m (16-ft) wide by 15.24-m (50-ft) long test pads on comparable subgrade conditions in the Piedmont area, North of Greensboro, North Carolina, using different subgrade stabilization measures, including select fill material and geosynthetic reinforcement together with a relatively thin layer of aggregate base course (ABC). Field loading was applied using 1,000 passes of a loaded construction truck. Deformation response of stabilized section were monitored by using LIDAR Scan survey at different stage of the full scale test. Detailed results are presented in the report by Mousavi et al. (2016). A series of \textit{in-situ} DCP, laboratory resilient modulus tests were performed to evaluate shear strength and stiffness properties of subgrade and stabilized sections. The DCP tests were performed at four locations in each test section, as shown in Fig. 6-2. The laboratory testing, including the resilient modulus test was carried out on undisturbed soil specimens retrieved from Shelby tubes.

![Diagram of test sections configuration, locations of DCP tests and boreholes, (dimensions in cm).](image-url)

Figure 6-2. Test sections configuration, locations of DCP tests and boreholes, (dimensions in cm).
6.3.1 DCP tests measurements

As previously stated, DCP tests were performed during the project at different time steps, before excavation and traffic loading. To establish the interfaces between the soil layers using the DCP data, ASTM D6951 recommends plotting the cumulative blow counts versus the penetration depth and finding the intersection of the tangent lines. After locating the interfaces of the layers, the weighted average DCPI value of each soil layer was calculated using Equation 6-2.

\[
DCPI_{wt, avg.} = \frac{1}{H} \sum_{i=1}^{n} (DCPI_i \times z_i)
\]

Eq. (6-2)

Which,

\[ z = \text{Depth of penetration per blow (mm or in.)} \]

\[ H = \text{Total depth of the soil layer (mm or in.)} \]

The DCP measurements prior to excavation and before starting the traffic, are summarized in Tables 6-1 and 6-2. The CBR value of subgrade soils was estimated from NCDOT recommended correlation equation (NCDOT, 1998), as expressed in Equation 6-3. The undrained shear strength of subgrade soils was assumed to be 11 times of CBR values, Equation 6-4, as recommended by Cross and Gregory (2007):

\[
\log(CBR) = 2.60 - 1.07 \times \log(DCPI)
\]

Eq. (6-3)

\[
S_{u, subgrade\ soil} = 11 CBR
\]

Eq. (6-4)
Table 6-1. DCP test results prior to excavation.

<table>
<thead>
<tr>
<th>Test Section</th>
<th>Station</th>
<th>Start(^1)</th>
<th>End</th>
<th>DCPI(^2)</th>
<th>CBR(^3)</th>
<th>(S_u)^(^4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S11</td>
<td>0</td>
<td>533</td>
<td>60</td>
<td>5</td>
<td>13</td>
<td>55</td>
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<tr>
<td></td>
<td>533</td>
<td>864</td>
<td>25</td>
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<td>140</td>
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<td>S12</td>
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<td>33</td>
<td>9</td>
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<td></td>
</tr>
<tr>
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<td>533</td>
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<td>7</td>
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</tr>
<tr>
<td></td>
<td>533</td>
<td>889</td>
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<td>18</td>
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</tr>
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<td>54</td>
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<td>61</td>
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<tr>
<td></td>
<td>711</td>
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<td>37</td>
<td>8</td>
<td>92</td>
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<tr>
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<td>838</td>
<td>17</td>
<td>19</td>
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</tr>
</tbody>
</table>

\(^1\)mm
\(^2\)mm/blow
\(^3\)Eq. (6-3)
\(^4\)Eq. (6-4), kPa
Table 6-2. DCP test results prior to traffic loading.

<table>
<thead>
<tr>
<th>Test Section</th>
<th>Station</th>
<th>Start</th>
<th>End</th>
<th>DCPI</th>
<th>CBR</th>
<th>Su</th>
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</tr>
</tbody>
</table>

1 mm
2 mm/blow
3 Eq. (6-3)
4 Eq. (6-4), kPa

6.4 Evaluating Existing Subgrade Stabilization Criteria

6.4.1 NCDOT Undercut Criteria

The obtained DCP data prior stabilization, in sections 1, 2, and 3, were plotted in terms of cumulative penetration resistance as a function of the number of blows and shown in Figs. 6-3 (a-c) respectively; and compared to the NCDOT’s cut-off value of 38 mm/blow, indicating the need for undercutting the weak subgrade soil and replacing it with select fill. The NCDOT undercut criterion is indicated by the shaded triangle shown in each of the three figures. The data indicate that Sections 1, 2, and 3 need improvement to a varying degree and to different depths.
6.4.2 Borden et al. (2010)

6.4.2.1 Undercut criteria results - subgrade

The laboratory resilient modulus test results and measured DCP data were utilized to determine the subgrade soil stiffness and shear strength properties. The resilient modulus value depends on both confining and deviatoric stress. Hence, the resilient modulus value at the confining pressure of 2 psi and deviatoric stress of 6 psi was selected as a representative resilient modulus value for the subgrade (Mohammad et al. 2008, Rahim 2004, Asphalt Institute 1989). The resulting values of $\delta/B$ are plotted in Figure 6-4. In this case, the proposed criteria by Borden et al. (2010) are in good agreement with the NCDOT’s criterion of DCPI > 38 mm/blow to indicate the need for undercutting.
6.4.2.2 Undercut criteria results - stabilized materials

The results of stabilized section 1, 2, and 3 are plotted in Figure 6-5. It can be seen that all the data points are plotted within the stable zone of the chart. These results are consistent with the field observations discussed in the report by Mousavi et al. (2016). In this case, after 1,000 traffic passes, the maximum rut depth values in the three stabilized sections were lower than the failure criterion (75 mm). Hence, it can be concluded that the charts can accurately gauge the effectiveness of the stabilization measure when the section is subjected to fewer than 1,000 truck passes.
6.4.3 White et al. (2009)

The stability of in-situ subgrade soils and stabilized select fill and ABC sections were investigated by using proposed stabilization chart by White et al. (2009). It was determined, for the in-situ DCP measured data, presented in Table 6-1, the subgrade soils requires to be stabilized and will be failed under proof roll test, which is consistent with the results from NCDOT and Borden et al. (2010) undercut criteria. The performance of the stabilized select fill section and ABC sections under proof roll test were also predicted by implementing the filed measured DCPI values, summarized in Table 6-2., into the White et al. (2009)’s criterion. The analyses results indicated that the select fill materials with a DCPI value of ~13 mm/blow, will not pass the proof roll test, which is not in agreement with the field observation, 38 mm/blow NCDOT, and Borden et al. (2010)’s criteria.
The results for section 2 and 3 are plotted in Fig. 6-6. It was determined, that the stabilized ABC sections could be failed under proof roll test, in some area of the test sections, which is not consistent with field observation that indicated less than 25.4 mm (1 in.) plastic deformation after 2500 ESAL, and the results from Borden et al. (2010)’s criteria.

Figure 6-6. Application of undercut criteria (White et al. 2008) to the stabilized materials
6.5 Develop Stabilization Chart

Subgrade stabilization chart was developed based on the incorporation of in-situ DCP measurement and numerical proof roll test results in Plaxis 3D with taking into account the effect of superposition of the loaded areas. The geometry of the model domain is shown in Fig. 6-7 and consists of three sublayers, each having a thickness of 30.5 cm (12 in.), underlain by a uniform 36.55 cm (144 in.) thick layer. The assumption in this case is the 100 cm (3-ft) profile thickness is sufficient to include the zone of stress bulb. DCPI values of 20, 38, and 60 mm/blow, were chosen and assumed to represent, good, marginal, and poor subgrade sublayers, respectively. The DCPI of 38 mm/blow is the currently recommended criterion for undercutting the soft subgrade soils. Given the three sublayers, there are 27 possible combinations for the subgrade profile.

![Subgrade layers configuration](image)

Figure 6-7. Subgrade layers configuration (dimension in mm)

6.5.1 Soil constitutive model

The Harding Soil Small strain constitutive model was implemented in this study, in order to take into account the soil stiffness’s dependency on the stress and strain level. The Hardening Soil small
strain constitutive model requires several stiffness properties, to formulate stress and strain dependency of the soil stiffness.

\( m \): Stress dependent stiffness according to a power law.

\( E_{50}^{\text{ref}} \): Plastic straining due to primary deviatoric loading

\( E_{\text{sed}}^{\text{ref}} \): Plastic straining due to primary compression

\( E_{\text{ur}}^{\text{ref}} \): Elastic unloading/reloading

\( \nu_{\text{ur}} \): Unloading/reloading Poisson’s ratio

\( G_0 \): Initial shear modulus.

\( \gamma_{0.7} \): Shear strain level at which the \( G_s \) is reduced to 70 percent of the \( G_0 \) value

Benz (2007) defined the stress-strain relationship in the context of the HS Small Strain model, as expressed in Equation 6-5:

\[
\tau = G_s \gamma = \frac{G_0}{1 + 0.385 \left( \frac{\gamma}{\gamma_{0.7}} \right)}
\]

Eq. (6-5)

Equation 6-6 expresses stress dependency of the shear modulus:

\[
G_0 = G_0^{\text{ref}} \left( \frac{c \cos \varphi - \sigma_3 \sin \varphi}{c \cos \varphi + p^{\text{ref}} \sin \varphi} \right)^m
\]

Eq. (6-6)

6.5.2 Materials properties

The CBR and undrained shear strength of the subgrade layers were estimated by suing the Equations 6-3 and 6-4, as previously mentioned. The modulus properties of the subgrade soils were estimated using the DCPI values following the model by Mousavi et al. (2016), as presented in Equation 6-7. In this approach, the DCPI values are used to predict the coefficients of MEPDG model, \( k_1 \), \( k_2 \), and \( k_3 \), as presented in Eq. 3. The \( E_{\text{ur}} \) value is assumed to be equal to the resilient
modulus value at applied deviatoric stress of 41.4 kPa (6 psi) and confining pressure 13.8 kPa (2 psi)

\[ k_i = C_1 + C_2 \ln(DCPI), \quad i:1,2,3 \]  

\[ \text{Eq. (6-7)} \]

Where \( C_1, C_2 \) are summarized in Table 6-3.

<table>
<thead>
<tr>
<th>Coefficients</th>
<th>( C_1 )</th>
<th>( C_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( k_1 )</td>
<td>2310 (911)(^1)</td>
<td>-377</td>
</tr>
<tr>
<td>( k_2 )</td>
<td>-0.3 (0.7)(^1)</td>
<td>0.3</td>
</tr>
<tr>
<td>( k_3 )</td>
<td>-10 (-4.5)(^1)</td>
<td>1.7</td>
</tr>
</tbody>
</table>

\(^1\) English Unit  
SI Unit: DCPI: mm/blow,  
English Unit: DCPI: in./blow,

As a baseline case, the Mr/E\(_{50}\) value of 6 was selected, to reach to 25 mm (1 in.) surface deformation after two proof roller passes for subgrade with DCPI value of 38 mm/blow. This is consistent with the 38 mm/blow NCDOT undercut criterion; which has been validated for Coastal Plain and Piedmont residual soil by Cowell et al. (2012) and Mousavi et al. (2016), respectively. The properties of subgrade layers implemented in these numerical analyses are presented in Table 6-4.

<table>
<thead>
<tr>
<th>layer</th>
<th>DCPI (mm/blow)</th>
<th>CBR</th>
<th>( Su ) (kPa)</th>
<th>Mr. (MPa)</th>
<th>( G_{max} ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>20</td>
<td>16</td>
<td>178</td>
<td>44</td>
<td>18</td>
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<tr>
<td>C2</td>
<td>38</td>
<td>8</td>
<td>89</td>
<td>41</td>
<td>17</td>
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<tr>
<td>C3</td>
<td>60</td>
<td>5</td>
<td>55</td>
<td>37</td>
<td>16</td>
</tr>
</tbody>
</table>

Table 6-4. Subgrade layers properties.
It was assumed that $E_{ode}$ is equal to $E_{50}$, and $m$ value, stress dependency parameter, of 0.5. The maximum shear modulus is calculated using Equation 6-7 (NCHRP 2008), and assuming Poisson’s ratio ($\nu$) of 0.2 under cyclic loading.

$$G_{\text{max}} = \frac{M_r}{2(1+\nu)}$$  
Eq. (6-7)

### 6.5.3 Model geometry

For the numerical analysis in this study, the gross weight of the trailer was assumed to be 445 kN (50 tons) equally distributed among the four wheels with a tire pressure of 483 kPa (70 psi) and center-to-center spacing between adjacent wheels of 813 mm (32 in.) Therefore, the radius of the contact area was calculated to be 270 mm (10.6 in.). As previously mentioned the subgrade response under proof roller load was investigated in 3D FEM models with the analysis taking into account the effect of the superposition of the multi-loaded areas. The surface deformation of the subgrade was also evaluated after two proof roller passes, in PLAXIS 3D using 10-node tetrahedral elements. Four wheel loads were considered, as shown in Fig. 6-8, with a radius of 270 mm (10.6 in.), corresponding with a tire pressure of 483 kPa (70 psi) and an axial load of 445 kN (50 tons), and center-to-center spacing of 813 mm (32 in.). The boundaries were extended 5588 mm (220 in.) in the X direction, 6096 mm (240 in.) in both Y directions and 4572 mm (180 in.) in the Z direction. Absorbent boundaries also were imposed on the $X_{\text{max}}$, $Y_{\text{max}}$, $Y_{\text{min}}$, and $Z_{\text{min}}$ planes.

### 6.5.4 Cyclic loading

The tire pressure of 483 kPa (70 psi) was assumed for the cyclic loading analysis. Two load cycles, which represent two proof roller passes, were considered, as shown in Fig. 6-9.
Figure 6-8. Proof roller test model geometry. PLAXIS 3D, (dimensions in inches, 1 in.=25.4 mm)

Figure 6-9. Proof roller load cycles.
6.5.5 Analyses results

6.5.5.1 Deformation results

Fig. 6-10 (a-c) shows the computed permanent surface deformation of the analyses cases when the top 30 cm (12 in.) subgrade soil, L1, consists of soil type C1, C2, and C3, respectively, (as defined in Table 2). The stable subgrade zone is designated by the shaded rectangular area in Fig. 6-10, based on the criterion of 25 cm (1 in.) plastic rut depth on top of the subgrade after two proof roller load passes. The subgrade stability on the basis of the proof roller test for any other cases with DCPI value between 20 to 60 mm/blow, can be approximated by interpolating from the results of the computed cases. It should also be noted that while the numerical deformation criterion is 25 cm (1 in.) for an acceptable subgrade profile, marginal cases are expected for cases where the deformation is slightly less than the 25 cm (1 in.) value.
a) L3: DCPI (in./blow)

(b) L3: DCPI (in./blow)
6.5.5.2 Subgrade stabilization chart

Figure 6-11 presents the developed “Subgrade Stabilization Recommendation Chart”, based on the criterion of 25 cm (1 in.) plastic rut depth on top of the subgrade after two passes of the proof roller load. The results indicate that a subgrade with a 30 cm (12 in.) C1 layer (with DCPI value of 20 mm/blow) on top does not require a stabilization measure, given the assumed properties of the lower subgrade layers. If the subgrade has a 30 cm (12 in.) thick layer of soil type C2 on top, subgrade stabilization would be required if the second 30 cm sublayer is composed of either soil types C2 or C3. If the top 30 cm layer of the subgrade consists of soil type C3, subgrade stabilization would be required unless the second 30 cm sublayer is composed of stiff soil, type C1. The chart shown in Figure 6-11 summarizes the study findings and can be utilized by a field
engineer to discern the need for undercut when the subgrade profile is composed of sublayers having different properties.

6.5.5.3 Evaluating Subgrade quality

The presented field measured DCP data in Table 6-1, were employed into the proposed subgrade stabilization chart to evaluate the stability of the untreated subgrade soils, as summarized in Table 6-5. It was determined that the results are in a good agreement with 38 mm/blow NCDOT undercut criteria, and generally subgrade stabilization needs to be enforced in order to sustain under traffic load. It was observed that proposed criteria by Borden et al. (2010), is more conservative compared with developed design chart in this study. The DCP measured data of the stabilized sections were also implemented into the proposed model. It was concluded that in all the cases, the first layer is identified as the Case 1, with a good quality material, and will be passed proof roll test, which is consistent with the field observation that stabilized sections were experienced plastic surface deformation between 38 to 13 mm (1.5 to 0.5 in.) after 2500 ESAL, and the results obtained from Borden et al. (2010)’s criteria.
Figure 6-11. Subgrade stabilization recommendation chart.
Table 6-5. Quality untreated subgrade soils based on proposed stabilization chart.

<table>
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<th>Test Section</th>
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<th>End</th>
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<th>Quality</th>
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<td></td>
<td></td>
<td>533</td>
<td>864</td>
<td>25</td>
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<tr>
<td></td>
<td>S12</td>
<td>0</td>
<td>152</td>
<td>33</td>
<td>Marginal</td>
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<tr>
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<td>152</td>
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<td>43</td>
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</tr>
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<td></td>
<td>533</td>
<td>889</td>
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<td></td>
<td>635</td>
<td>838</td>
<td>17</td>
<td></td>
</tr>
</tbody>
</table>

\(^1\)mm
\(^2\)mm/blow
Conclusion

Extensive FEM analyses were performed to investigate the stability of subgrade soil under proof roller test. From the numerical analyses the following conclusions can be advanced:

- A subgrade stabilization recommendation chart was developed based on criterion of 25 cm (1 in.) plastic rut depth on top of the subgrade after two pass proof roller load and incorporation of in-situ DCP measurement and numerical proof roll test results.

- The failure deformation occurs at DCPI value of 38 mm/blow for single layer subgrade soil with the Mr/E$_{50}$ of 6; which is consistent with NCDOT subgrade undercut criteria.

- Strong agreement were obtained with the 38 mm/blow NCDOT undercut criteria. It was observed that proposed criteria by Borden et al. (2010), is more conservative compared to developed design chart in this study

- Subgrade with 30 cm stiff layer (DCPI < 20 mm/blow) on top does not require any subgrade soil stabilization, regardless of subgrade layers underneath, while subgrade needs to be stabilized if there is a very soft soil layer top 30 cm of subgrade (DCPI > 60 mm/blow), unless the sublayers underneath are composed of stiff soil (DCPI < 20 mm/blow).
References


CHAPTER 7: CONCLUSIONS AND FUTURE WORKS

This study investigates the performance of the geosynthetics-reinforced unsaturated subgrade soils and ABC, under cyclic loading, through field testing, laboratory measurements, and numerical analyses. Three test pads with comparable subgrade conditions were constructed in the Piedmont geologic area of North Carolina, using different subgrade stabilization measures, including select fill material, and geosynthetic reinforcement (Geogrid BX1200 and Geotextile HP570) together with a relatively thin layer of aggregate base course (ABC). Field loading was applied using 1,000 passes of a loaded construction truck. Several parameters were monitored during loading, including surface deformation, stress, deformation, and moisture and suction levels. Numerical analyses were performed using PLAXIS 3D software to study the deformation and stress distribution performance in the reinforced unsaturated subgrade soil under cyclic traffic loading within the context of advanced soil constitutive model. A novel approach for predicting resilient modulus of subgrade soils at various stress level based on light weight deflectometer (LWD), and dynamic cone penetrometer (DCP) data is introduced and validated by field measured data, and independent data from other studies reported in the literature. Validity of existing stabilization criteria in the literature were examined for the Residual Piedmont subgrade soil, by utilizing field measured data. A new subgrade stabilization recommendation chart was developed for layered subgrade profile, based on the incorporation of in-situ DCP measurement and computed deformation response of subgrade soil from advanced 3D numerical analyses, under cyclic loading corresponding to the proof roller passes with taking into account the effect of the superposition of the multi-loaded areas.

Based on the work conducted in this study, the following observations and conclusions are advanced.
7.1 Numerical and Experimental Investigation of Geosynthetics Reinforced Unsaturated Unpaved Road

- The vertical stress values measured near the interface of the subgrade for the geosynthetic-reinforced sections (Sections 2 and 3) decreased with traffic, from 138 to 69 kPa for Section 2 (geogrid-reinforced), and from 110 kPa to 55 kPa for Section 3 (geotextile-reinforced), whereas the pressure recorded for Section 1 (select fill) was almost constant, 27.5 +/- 13.8 kPa during traffic. This decrease in measured stress for the geosynthetic-reinforced sections is attributable to the mobilization of the reinforced layer tensile strength, an increase in the matric suction in the subgrade layer from 25 kPa to 35 kPa as a result of the hot summer weather during which the project was conducted, as well as variation of modulus ratio of ABC to subgrade implied by the different induced strain levels.

- Because each driver was to drive over the EPCs that were located on the driver’s side of the vehicle, the EPCs on the other (passenger) side was off between 13 to 38 mm from the truck tires (loaded area). It was observed that the lateral wander of the truck tires had a significant effect on the stress that was measured in the shallow depth of the subgrade layer (in Sections 2 and 3). Hence, the monitored pressure was categorized according to traffic direction, i.e., North to South and South to North. The variation in the recorded pressure reduced from 13.8 to 6.9 kPa, 69 to 34.5 kPa, and 55 to 27.5 kPa in Sections 1, 2, and 3, respectively.

- The geosynthetic-reinforced sections experienced up to 17 mm (0.67 in.) and 21 mm (0.83 in.) of surface deformation for the geogrid- and geotextile-reinforced sections, respectively.
The select fill-stabilized section showed 37.5 (1.47 in.) of deformation after 1,000 truck passes.

- The rut depth values (defined as the vertical displacement from the original ground surface) increased with the number of truck passes until they reached a relatively constant value within 1,000 truck passes. The rut depth value increased up to 500 truck passes in both the geotextile and geogrid sections (Sections 2 and 3, respectively). In Section 1 (select fill), the rut depth reached a limiting value after 700 passes.

- FEM analyses results indicated that the matric suction state of the ABC layer was shown to have a notable effect on the surface deformation of un/reinforced unsaturated subgrade under cyclic loading. A 120-kPa increase in matric suction in the ABC layer (from 0 kPa to 120 kPa) was associated with a 85 percent reduction in the surface deformation of the reinforced subgrade under cyclic loading.

- FEM results indicated that the mobilized force increased by a number of load cycles, as results of cumulative plastic deformation under cyclic loading, and an increase in ABC suction, increases the mobilized force until a certain suction level, in this study 40 kPa, and reduces after on.

- The numerical results showed that ABC layer behaves increasingly as a beam as the suction increases due to an increase in both the strength and stiffness. The more the layer acts like a beam, the location that the maximum horizontal deformation takes place; moves to the bottom of ABC, where the reinforcement layer is located.
7.2 Optimum Location of the Reinforcement in Unpaved Road

- Regardless of ABC layer thickness, the surface deformation is minimized, when the reinforcement is embedded at the depth of about half of the radius of loaded area, \( D/r = 0.5 \).

- The inclusion of reinforcement can lead to even smaller surface deformation if it is placed at the depth at which maximum vertical strain occur, compared with the case with greater ABC thickness and reinforcement is located at the interface of the base and subgrade, conventionally.

- Computed mobilized force in geogrid in Plaxis 3D, under vertical load, is independent of the vertical deformation, which is the main component that mobilizes force in reinforcement, and contingent only on the horizontal deformation (in XY plane) of the geogrid node.

- The Higher tension force is mobilized in reinforcement element when it is placed at the depth that maximum vertical strain take place, \( D= 0.5r \), thus reinforcement inclusion is more beneficial in the reduction of surface deformation.

- FEM results confirmed the literature, which reinforcement produce a higher reduction in surface deformation with thinner ABC layer. It can be seen that the efficiency of the geogrid can significantly increase if the reinforcement located at a depth equal to half of the radius of the loaded area.

7.3 Subgrade Resilient Modulus Prediction from Light Weight Deflectometer

- The proposed model is capable of predicting \( M_r \) at various stress combinations with a coefficient of determination of, \( R^2 \)0.83. Good agreement was obtained between the
LWD-predicted $M_r$ and laboratory-measured data presented in other studies, with an 11% average underprediction of $M_r$ values with $R^2 = 0.96$.

- Examination of the ability of two existing models to predict $M_r$ from in-situ LWD data, showed a general trend toward overprediction, except for the higher plasticity soils tested in this study. For this soil the Mohammad et al. correlation was seen to underestimate the measured $M_r$ values.

- Input parameters needed to calculate elastic modulus by the LWD, such as Poisson’s ratio and shape factor, can have a significant effect on the calculated $E_{LWD}$. At a given Poisson’s ratio, the $E_{LWD}$ can change by a factor of 2 depending on the shape factor selected. The effect of Poisson’s ratio was shown to increase with increasing shape factor.

- The proposed model was demonstrated to predict the resilient modulus of A-1-b (SP), A-4 (ML, SM, CL), A-6 (CL-ML), and A-7-5 (MH) soil types using the measured ratio of applied stress to surface deflection from a Prima 100 LWD device, employed in this study. Further work will need to be performed to evaluate the applicability of the proposed approach to other soil types as well.

### 7.4 Resilient Modulus Predication of Soft Low Plasticity Piedmont Residual Soil from Dynamic Cone Penetrometer

- Good agreement was obtained between calculated $M_r$ values from the proposed model and the laboratory-measured resilient modulus data, with a coefficient of determination of 0.70.
• Examination of the performance of the proposed model with a quarter of the data set which was not included in the statistical analyses, indicated that, on average, the proposed model underestimated the Mr by 4% with a coefficient of determination of 0.73.

• Predicted Mr values by the proposed model were seen to be in reasonably good agreement with the laboratory-measured Mr presented in the literature, however with a lower coefficient of determination of 0.53.

• The evaluation of existing models which directly estimate the Mr of soils from the DCP measurements showed that they overestimated measured Mr values. In addition, the validity of these models only at one determined stress level limits their applicability to one particular pavement structure.

• The assessment of existing empirical models that predict the universal constitutive model fitting coefficients from basic physical properties of soils, yielded poor predictions of Mr for the soils tested in this study.

• The proposed model is capable of predicting the resilient modulus of low plasticity Piedmont residual soils (A-4), with PI<5, and 40%<P200<55%; at any stress state. Further work will need to be performed to evaluate its applicability to other soils.

7.5 Correlation of DCPI to Proof Roller Test to Assess Subgrade Soils Stabilization Criterion

• A subgrade stabilization recommendation chart was developed based on criterion of 25 cm (1 in.) plastic rut depth on top of the subgrade after two pass proof roller load and incorporation of in-situ DCP measurement and numerical proof roll test results.
• The failure deformation occurs at DCPI value of 38 mm/blow for single layer subgrade soil with the Mr/E$_{50}$ of 6; which is consistent with NCDOT subgrade undercut criteria.

• Strong agreement were obtained with the 38 mm/blow NCDOT undercut criteria. It was observed that proposed criteria by Borden et al. (2010), is more conservative compared to developed design chart in this study.

• Subgrade with 30 cm stiff layer (DCPI < 20 mm/blow) on top does not require any subgrade soil stabilization, regardless of subgrade layers underneath, while subgrade needs to be stabilized if there is a very soft soil layer top 30 cm of subgrade (DCPI > 60 mm/blow), unless the sublayers underneath are composed of stiff soil (DCPI < 20 mm/blow).

7.6 Recommendations for Future Research

I. Validating the proposed LWD and DCP resilient modulus model for wider range of soil classification.

II. Evaluating effect of reinforcement layer location within unsaturated ABC layer, at different suction levels, on deformation response and mobilized force in reinforcement.

III. Develop a comprehensive stress distribution angle model which embraces impacts of the geosynthetic reinforcement properties, number of traffic, and modulus ratio of ABC into the subgrade soil with traffic.

IV. Investigate effect of matric suction of ABC layer for materials with different shear strength and stiffness properties.