

DYNAMIC FINITE ELEMENT SIMULATIONS OF VIBRATORY ROLLER SOIL
STIFFNESS USING ITERATIVE STRESS-DEPENDENT RESILIENT MODULI

by

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ABSTRACT

This thesis presents the application of the stress-dependent National Cooperative Highway Research Program (NCHRP) Mechanistic Empirical Pavement Design Guide (MEPDG) model for resilient modulus, M_r , in dynamic finite element analyses of pavement systems loaded by vibratory rollers. The MEPDG model has been well established as an appropriate means for describing the stress-dependent M_r of a soil material. However, it has not been previously used in dynamic analyses of pavement systems and has been limited to static applications.

In this study, an iterative Matlab-Abaqus algorithm is developed based on equivalent linear strain dependent dynamic analysis procedures used in seismic analyses to determine stress-dependent M_r values for soil foundations undergoing dynamic vibratory loading from a vibratory roller compactor. The soil materials and lift thicknesses were modeled based upon previously compiled field data from sites in Florida. Iterative dynamic finite element analyses were performed using resulting stress values and iteratively updated M_r values that could spatially vary in the soil foundation to determine the force-deflection behavior of a vibratory roller on the soil. The force-deflection behavior results from the finite element analyses were then compared to the field data measured during mapping of compacted lifts in Florida by an instrumented vibratory compactor. The results of this study show similarities to results obtained from the field data, indicating that the proposed algorithm may be a feasible modeling procedure for dynamic loading of pavement systems. However, further investigations are required to develop the proposed model for use in the field.

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CHAPTER 1

INTRODUCTION

The objective of this study is to model the relationships between continuous compaction control (CCC) roller soil stiffness measurements and stress-dependent resilient moduli of subgrade and base lifts using dynamic finite element (FE) analyses. Vibratory roller compactors are commonly used in earthwork projects as a means of soil compaction. They are also used after the compaction process in mapping procedures to evaluate the efficacy of the compaction procedures. The dynamic behaviors of the vibratory roller during the mapping procedures can be interpreted to determine the stiffness of the underlying soil. This process is commonly referred to as intelligent compaction (IC), or continuous compaction control (CCC).

The resilient modulus (M_r) was developed in 1962 as a means to characterize the resilient (elastic) behavior of soils under repeated loading (Seed et al. 1962). M_r is described as the elastic modulus under repeated loading, and is defined as the ratio of deviator stress to recoverable strain (Putri 2010). Typically, M_r is determined experimentally for a soil material by conducting a series of triaxial tests (Seed et al. 1967). In the literature (e.g. Taciroglu 1998, Kim 2000, Li and Selig 1994, Uzan 1992), M_r has been studied extensively for use in the analysis of pavement structures under repeated wheel loading. Similarly, M_r can be applied for use with IC due to the fact that the vibratory roller exhibits a repeated loading on the underlying soil during the compaction process (Rinehart et al. 2009). M_r values, however, are typically measured in the laboratory following the compaction process, and therefore do not typically provide real-time in-situ results that could be used in quality assurance/quality control (QA/QC) purposes. Due to this fact, there have been numerous studies conducted (e.g. Taciroglu 1998,

Kim 2000) in an attempt to predict the behavior of pavement systems based on calculated M_r values. This is done through the use of generally available FE programs such as ABAQUS, and used in the design and analysis of pavement structures. Many of these studies employ a solely static approach to the determination of M_r . This method is not adequate to fully describe the behaviors of the vibratory roller compaction process. Therefore, since M_r is stress-dependent, there is a need to develop a procedure in which M_r may be updated to reflect the spatial and temporal variations in stresses expected in dynamic applications such as IC. The following thesis presents the results of a study that addresses this need through the development and implementation of an iterative procedure to determine M_r based upon the stresses experienced by the soil materials during a dynamic FE analysis. This procedure is used to model a vibratory roller compactor passing over stratified soil layers. The vibratory roller compactor and soil layer properties are based upon data from a field site obtained from a previous Colorado School of Mines student's graduate work. The FE results are compared to the field soil stiffness values as measured by the vibratory compactor roller, demonstrating the efficacy of the developed method.

1.1 Motivation for Work

Typical QA/QC methods used in traditional earthwork construction procedures involve evaluating in-situ dry density values at various spot locations along the compacted soil beds and comparing them to the required relative compaction determined in the site design. The relative compaction is defined as the ratio of dry density to maximum dry density of the soil determined via a Standard or Modified Proctor test (ASTM D698-12 and ASTM D1557-12). If the in-situ dry densities do not meet the relative compaction requirements, the soil layer is recompacted, and the new in-situ dry densities are measured and evaluated against the required relative

compaction. Although this QA/QC method is generally acceptable for standard operating field procedures, the method has one major drawback: spot testing of dry densities typically quantifies less than 1% of the entire site, which does not necessarily provide a representative description of the soil layer (e.g. Mooney and Adam 2007). Because of this there is a need for a QA/QC method that provides more spatially representative quantification of compaction. This study focuses on the development and implementation of an iterative, stress-dependent M_r model that can be used in dynamic finite element analyses to model vibratory roller compactors on layered soil materials. This contributes to civil engineering knowledge by allowing for M_r to reflect the time-varying stress distributions found in a dynamic analysis instead of M_r remaining a static quantity in static FE analyses. The soil stiffness values from the finite element results are compared to those measured in the field, showing close agreement.

1.2 Summary

This thesis is divided into four separate chapters. An introduction of the topic, the motivation for work, and the overall objective are presented in Chapter 1. Chapter 2 is comprised of a comprehensive literature review detailing the current IC procedures as well as a description of the soil material property M_r . The results from finite element analyses are presented and discussed in Chapter 3. Chapter 4 presents the conclusions drawn from this study as well as recommendations for future research.

In Chapter 2, the literature review provides necessary background information concerning IC procedures used in practice as well as a detailed review of M_r . The literature review of the IC procedures used in practice includes information on how the vibratory roller compactor is instrumented to collect soil property data, how roller measured values (MVs) are

calculated, and previous research performed on IC rollers. The literature review of M_r includes the definition and standard experimental procedures for determining M_r , previous numerical models used in the literature to predict M_r , as well as a review of previously implemented FE models incorporating M_r .

Detailed FE analyses based upon field conditions and data collected by previous Colorado School of Mines graduate students are presented in Chapter 3. The goal of this chapter is to describe the development and implementation of a stress-dependent iterative model for M_r and to use that to simulate expected roller measured soil stiffness values. FE models of the vibratory roller and multiple underlying soil conditions were created, and analyses were conducted in which the stress-dependent M_r values for multiple soil layers were iteratively determined to calculate the respective soil stiffness values. These finite element soil stiffnesses are then compared to those measured in the field.

Conclusions and recommendations drawn from the completion of this study are presented in Chapter 4.

CHAPTER 2

LITERATURE REVIEW

Vibratory roller compactors are commonly equipped with instrumentation and on-board computing technology that allows for real-time measurement of soil property values such as soil stiffness, k_s (Rinehart 2008). Continuous monitoring of roller compaction measurements through the use of IC has been widely used in the construction industry for over three decades.

This chapter presents a literature review, designed to present the practical motivation for the work as well as current methods used in representing M_r in finite element simulations of pavement systems. Specifically, the literature review aims to: (1) present background on common MVs used in IC practice, particularly k_s , (2) present a discussion of M_r and its evolution through the literature, and (3) summarize common FE models currently used in conjunction with M_r in the design and analysis of pavement structures. The thesis work described in further chapters builds upon previous research and to develop and implement a dynamic, time-varying FE model that utilizes a stress-dependent iterative procedure for the specification of M_r to help predict k_s for IC procedures.

2.1 Roller Measured Stiffness, k_s

This section provides background on the practical context of the work. Typical roller measured values (MV) used in the construction industry for IC procedures include: compaction control value (CCV), compaction meter value (CMV), vibration modulus (E_{vib}), and force-displacement stiffness (k and k_s) (Mooney et al. 2010). These MVs are typically determined through the instrumentation of the roller with an accelerometer and a Hall effect sensor, which

monitors the position of the eccentric masses on the vibratory roller compactor drum (Facas 2010).

Currently, there are two common definitions for roller MV force-displacement stiffness, k and k_s , which are used by different IC roller manufacturers (Bomag and Case/Ammann, respectively). Both values are interpreted from force-displacement loops recorded by the roller as it passes over a soil layer, where the force is represented as the time-varying contact force (F_c) of the drum, and the displacement is represented as the time-varying displacement (z_d) of the drum (see Figure 2.1). F_c may be calculated by utilizing the vertical force equilibrium as seen in equation 2.1:

$$F_c = m_0 e_0 \omega^2 \sin(\omega t + \phi) + m_d g + m_f g - m_d \ddot{z}_d \quad (2.1)$$

where $m_0 e_0$ = eccentric mass moment of the eccentric masses, ω = rotating frequency of the eccentric masses, ϕ = eccentric force phase shift between the inertia force of the drum ($m_d \ddot{z}_d$) and the excitation force (F_{ecc}), m_d and m_f = mass of the drum and frame respectively, g = acceleration due to gravity, and \ddot{z}_d and \ddot{z}_f = acceleration of the drum and frame respectively (Mooney and Adam 2007). z_d can be determined by taking the double integration, with respect to time, of the measured \ddot{z}_d data. The force-displacement loops may then generated by plotting F_c vs. z_d for the time-history. There are two primary modes of vibratory roller operation behavior: contact, and loss of contact as seen in Figures 2.2 (a) and (b) respectively (Neff 2013). The contact mode occurs when the roller maintains constant contact with the soil during the mapping procedure, whereas the loss of contact occurs when the roller loses contact with the soil once per vibration cycle. Bomag soil stiffness, k , value can be determined via these force-

displacement loops by locating lines that pass through points that pass through 20% and 80% of the difference between the maximum and minimum F_c (see Figure 2.2). k is the slope of this line. Alternately, the Case/Ammann soil stiffness, k_s , is determined by calculating the gradient of the line that passes through zero displacement and the point of maximum drum displacement (see Figure 2.2). It has been shown that these roller MVs are influenced by 1.0 - 1.2 m of the underlying soil (e.g., Mooney and Facas 2012; Rinehart and Mooney 2009). In most cases, this depth will consist of various layers of soil. In earthwork construction, these layers typically consist of 15 – 20 cm each of subgrade, subbase, and base materials (Mooney and Facas 2012). Because of this, the vibratory roller MVs result in composite values of each lift up to 1.0 - 1.2 m deep.

Both of these stiffness measurements are intended to give a somewhat “mechanistic” measure of the underlying soil material’s stiffness properties; it is important to note, however, that this is not a true mechanistic measure that can as of yet be explicitly related to the Young’s modulus or shear modulus of the underlying soil. k_s will be used in this work, but the alternate measure of k could equally be evaluated as well.

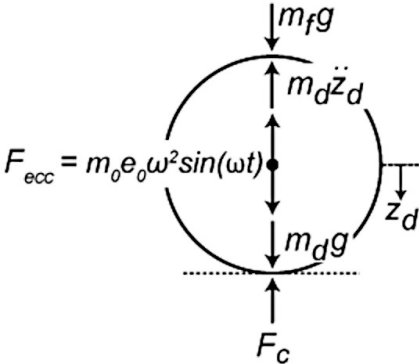


Figure 2.1 Free Body Diagram of Forces Acting on the Vibratory Roller (Mooney and Rinehart 2009)

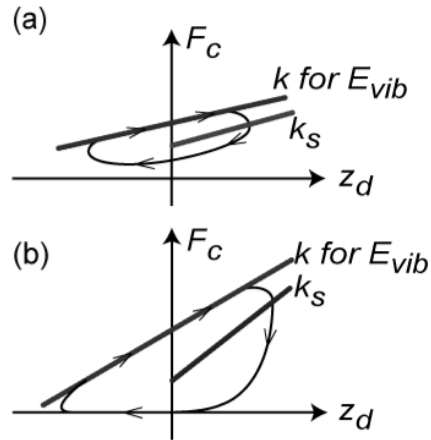


Figure 2.2 Force-displacement loop during (a) contact and (b) loss of contact (Mooney and Facas 2012)

2.2 Resilient Modulus

In this thesis work, an iterative stress-dependent algorithm is developed such that the vibratory roller behavior and the resulting measured force-displacement loops can be modeled in a dynamic finite element analysis. This thesis work contributes to the state of knowledge by providing a means for iteratively updating the resilient modulus of the soil in the finite element model based upon the observed time-varying stresses in the underlying soil. A review of the resilient modulus property is presented next.

Granular materials “shake down to resilient (elastic) behavior under repeated loading,” as illustrated in Figure 2.3 (e.g. Huang 1993, Hjelmstad and Taciroglu 2000), and it has been concluded that “nonlinear elasticity is a reasonable model of the behavior of granular materials” (Hjelmstad and Taciroglu 2000). This is due to the fact that the compaction of the soil “is thought to provide a sufficient excitation to shake down the material relative to the loads” seen in service (Hjelmstad and Taciroglu 2000). In this work, the modeled vibratory roller compaction

procedures represent “mapping” procedures done after the compaction processes. Mapping involves driving the roller over soil lifts that have already been compacted to assess the quality of the compaction processes via k_s . Therefore, elastic material models are used in the finite element simulations.

The elastic behavior of compacted granular soils may be commonly represented by the material property resilient modulus, M_r (see Figure 2.3), because it “applies even for stress paths that approach failure” and can therefore capture the stress and strain dependence of the soil’s elastic properties (Brown and Pappin 1981; Kim 2007). M_r has been well established in literature as a “key design and performance-related parameter for unbound subgrade, subbase and base materials” and is often used in place of the Young’s modulus to describe the soil (Hjelmstad and Taciroglu 2000, Rinehart et al. 2009). First introduced by Seed et al. (1962), M_r is typically determined experimentally by conducting a series of triaxial tests, described in AASHTO T307-99 (2003), and is defined as:

$$M_r = \frac{\sigma_d}{\epsilon_r} \tag{2.2}$$

where σ_d = deviator stress ($\sigma_1 - \sigma_2$) and ϵ_r = recoverable strain.

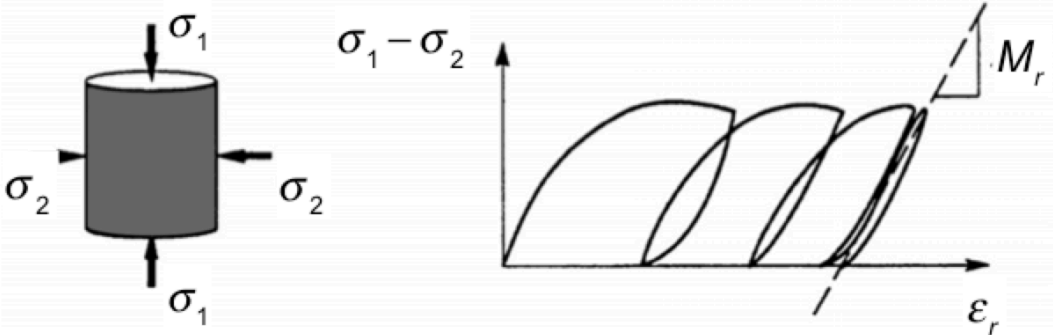


Figure 2.3 Resilient behavior of a soil during triaxial testing (Hjelmstad and Taciroglu 2000)

M_r of subgrade soils may also be experimentally predicted in practice by utilizing an established relationship between the California Bearing Ratio (CBR) and M_r (Kim 2007). This relationship, which has been verified in the literature through the use of laboratory testing, is defined as:

$$M_r = K_1(CBR)^{K_2} \tag{2.3}$$

where K_1 and K_2 are constants determined in literature. Some typical values for K_1 and K_2 are summarized in Table 2.1. The NCHRP (2004) recommends the values of $K_1 = 2555$ and $K_2 = 0.64$ due to the correlation of the strength and stiffness of the subgrade (Kim 2007).

Table 2.1 Typical values of K_1 and K_2 to be used in Equation 2.3

K_1	K_2
1500	1.0
2555	0.64
3000	0.65
5409	0.711

It has been shown that M_r is dependent upon the load and stress state of the soil and is therefore not a constant stiffness property (Li and Selig 1994). Because of this, M_r is highly influenced by the deviator and confining stress of the soil. Additionally, it has been shown that M_r can also be influenced by the soil type, grain size and shape, and moisture content of the soil material (George 2004, Kim 2007).

Various empirical models for M_r have been developed to be used in the design and analysis of pavement systems when experimental measurements of M_r are unavailable. Each of these models was developed to include the stress-dependencies of M_r (NCHRP 2004). One of the earliest empirical models to approximate M_r was developed by Hicks and Monismith (1971) and is commonly referred to as the K - θ model:

$$M_r = K_1(\theta)^{K_2} \quad (2.4)$$

where $\theta = \text{bulk stress} = \sigma_1 + \sigma_2 + \sigma_3$ and K_1 and K_2 are material specific constants determined through linear regression of experimental data.

Uzan (1985) advanced modeling of M_r by determining that the K - θ model could not accurately describe M_r if significant shear stresses were present. Because of this, the K - θ model often predicts an increasing M_r with increasing axial strain, which contradicts experimental data (Kim 2007). In order to account for the shear stresses, Uzan (1985) proposed a modification of the K - θ model:

$$M_r = K_1(\theta)^{K_2}(\sigma_d)^{K_3} \quad (2.5)$$

where K_1 , K_2 , and K_3 are material specific constants determined through linear regression of experimental data. Despite Uzan's work, however, the K - θ model has been widely used in practice for mathematical and experimental convenience.

By 2004, a generalized empirical model for the approximation of M_r was defined by the National Cooperative Highway Research Program (NCHRP). This model, generally referred to as the NCHRP Mechanistic Empirical Pavement Design Guide (MEPDG) model, is defined as:

$$M_r = K_1 (P_a) \left(\frac{\theta}{P_a} \right)^{K_2} \left(\frac{\tau_{oct}}{P_a} + 1 \right)^{K_3} \quad (2.6)$$

where K_1 , K_2 , and K_3 are material specific constants, θ = bulk stress, τ_{oct} = octahedral shear stress = $\frac{1}{3} \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}$, and P_a = atmospheric pressure in consistent units (NCHRP 2004).

This modification was developed to incorporate both the stiffening and softening effects of a material through the use of the bulk and shear stresses, respectively, and therefore can better capture the deformation behavior of granular materials (Kim 2007). By establishing that $K_2 > 0$, the model ensures that appropriate stress-dependences are represented, as an increasing bulk stress causes a stiffening of the soil (Kim 2007). Similarly, establishing that $K_3 < 0$ ensures that the softening of the soil due to shear stresses is represented (Kim 2007). The NCHRP has presented a generalized form of the M_r models (equation 2.7) that incorporates all of the previous mentioned M_r models by selectively assigning various K values to be equal to specific values (NCHRP 2004).

$$M_r = K_1 (P_a) \left(\frac{\theta - 3K_4}{P_a} \right)^{K_2} \left(\frac{\tau_{oct}}{P_a} + K_5 \right)^{K_3} \quad (2.7)$$

It is important to note that the MEPDG model, shown in equation 2.6, can be obtained from equation 2.7 by setting $K_4 = 0$ and $K_5 = 1$ (NCHRP 2004).

2.3 Previous FE Implementations

Several previous researchers (e.g. Taciroglu 1993, Kim 2007, and Mun 2003) have utilized the aforementioned empirical models to determine M_r for finite element simulations of

pavement systems that are statically loaded. These FE models are typically implemented through the use of the ABAQUS user material subroutine (UMAT) to perform the necessary calculations to determine M_r of underlying soil layers. These models update M_r values based upon *statically* calculated in-situ conditions. These statically calculated conditions, however, do not accurately describe the behavior of the vibratory roller on the underlying soil and therefore do not allow for accurate QA/QC methods. FE algorithms that update M_r based upon *dynamically* calculated in-situ conditions have not been explored prior to this thesis work.

2.4 Research Potential

Due to the fact that the mapping and compaction processes during earthwork projects are dynamic processes, in order to more accurately model force-deflection behavior and the associated roller-measured k_s , there is a need to develop an algorithm which can quantify M_r of the underlying layers considering the *dynamic*, time varying stress histories. This study will address this issue by: (1) developing an algorithm to iteratively update M_r values for each of the elements, allowing for spatial variation in the soil properties while considering the dynamic, time varying stress histories; (2) implementing this algorithm in a dynamic FE simulation that incorporates this algorithm to predict appropriate M_r values for underlying soil layers based upon the dynamic loading, and (3) using the predicted M_r values obtained from (2) to compute predicted k_s values. These predicted k_s values. From the finite element simulations are compared to actual k_s values measured in the field to evaluate the efficacy of the FE model.

CHAPTER 3

FE ANALYSES WITH ITERATIVE STRESS-DEPENDENT M_R

3.1 Finite Element Model

A 2D, plane strain, finite element model was developed for this study using ABAQUS 6.12. This model consists of a vibratory roller, which was modeled after a Sakai SV510D roller, and an underlying soil region. The construction of this model was developed to simulate the time-varying interactions developed during the dynamic loading of the roller on the soil (see Figure 3.1).

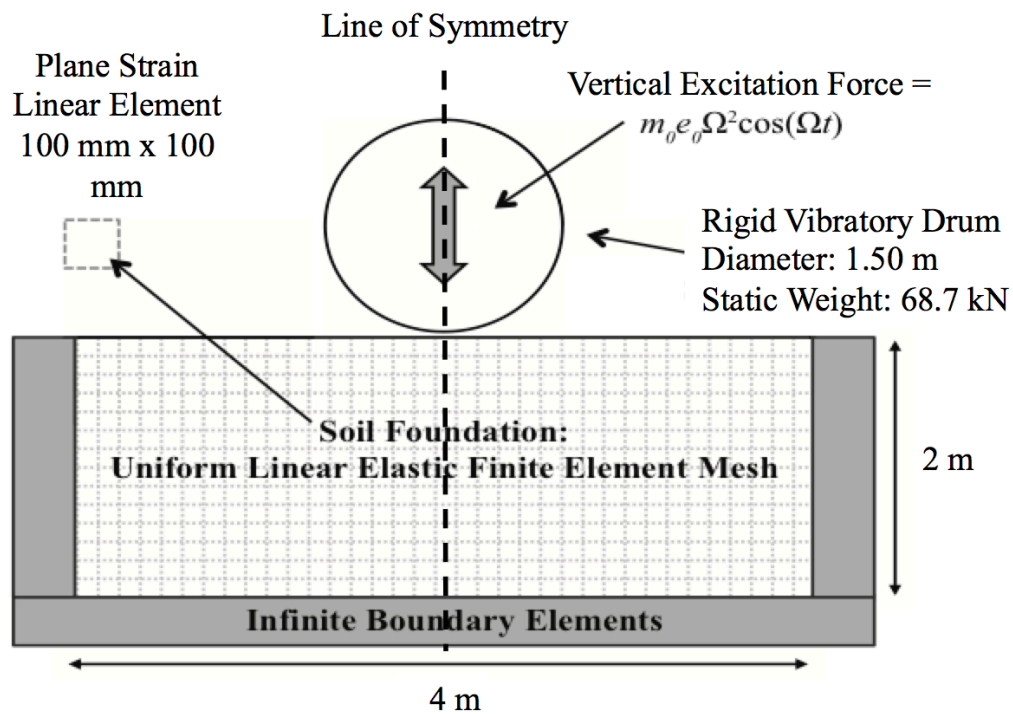


Figure 3.1 Finite Element Model Schematic (Kenneally et al. 2014)

The ABAQUS model consists of two parts: the roller and the underlying soil region. The drum is modeled using a rigid cylinder with a diameter of 1.5 m. The weight of the vibratory roller (drum plus frame) is applied to the cylinder through the use of downward forces applied at

the center of mass of the cylinder with magnitudes equal to the weight of the drum and the frame. The eccentric loading caused by the rotating drum masses is modeled as a vertical harmonic excitation force equal to $m_0 e_0 \Omega^2 \cos(\Omega t)$, which is applied at the center of the cylinder. The values for the excitation frequency, $\Omega = 30\text{Hz}$, and the eccentric mass moment, $m_0 e_0 = 3.0$ kg-m, are chosen from the respective ranges found in Table 3.1. These values were chosen based on the results from previous studies as to avoid bifurcation, or chaotic motion, of the roller on the soil foundation (Kenneally et al. 2014). A summary of the roller parameters can be found in Table 3.1. In order to simplify the resulting computations, the symmetry of the problem was exploited, and the underlying soil was modeled as a 2.0 m by 2.0 m finite element region consisting of 100 mm square CPE4R type elements. The FE mesh dimensions were based on previously documented mesh refinement studies and studies of vibratory roller MV sensitivity depth (Musimbi 2010, Mooney and Facas 2012). CPE4R are 4-node plain strain, bilinear, elements with reduced integration and hourglass control (ABAQUS 2013). In addition, infinite elements were applied to the outside boundaries of the region to simulate radiation damping via the surrounding soil and a one degree of freedom boundary condition was applied to the inside boundary (see Figure 3.2).

Table 3.1 Sakai SV510D Roller Parameters (Kenneally et al. 2014)

Parameter	Magnitude
Drum Mass, m_d	4466 kg
Frame Mass, m_f	2534 kg
Mass Moment, $m_0 e_0$	1.0 – 4.25 kg-m
Excitation Frequency, Ω	20 – 35 Hz
Drum/frame Stiffness, $k_{d/f}$	6.02 MN/m
Drum/frame Damping, $c_{d/f}$	4000 kg/s

The material damping in the model is approximated using the Rayleigh damping parameters α and β (see equation 3.1) and were assumed to be constant for all soil types (Kenneally et al. 2014).

$$[C] = \alpha[M] + \beta[K] \quad (3.1)$$

where $[C]$ is the global damping matrix, $[M]$ is the global mass matrix, and $[K]$ is the global stiffness matrix. The values of α and β were determined in a previous study, by a Colorado School of Mines graduate student, by comparing experimental data from two separate field sites (Kenneally et al. 2014) as 25 s^{-1} and 0.002 s respectively.

The soil types used in this study were based on those found in field data collected at a Florida earthwork construction site (see Figure 3.2). Two layers from this site, 60 cm of subgrade and the 15 cm limerock base layer were chosen for analysis with the proposed M_r model (see Figure 3.3). Based on the AASHTO classifications determined in a previous study (see Table 3.2), K_1 , K_2 , and K_3 were chosen for each soil layer and applied to the FE model. These values, see Table 3.3, were presented in the MEPDG (2004) as typical values for commonly used soil types.

Table 3.2 Soils used at the Florida earthwork site (Neff 2013)

Soil	AASHTO Classification	USCS Classification	C_u	C_c
Embankment Fill	N/A	N/A	N/A	N/A
Subgrade	A-3	SP	2.1	0.88
Stabilized Subgrade	23 cm of A-3	N/A	N/A	N/A
	7 cm of ash			
Limerock Base	A-1-b	GW	90	0.90

Table 3.3 Typical Values for K_1 , K_2 , and K_3 (NCHRP 2004)

Soil Type	K_1	K_2	K_3
A-1-a (SW-SM)	1252.156	1.046	-0.610
A-3 - SM	1745.161	0.356	-5.157
A-1-b – SM	697.824	1.431	-1.064
A-4(1) – SM	4288.471	1.121	-3.785
A-6(9) – CL	3425.160	0.091	-1.299
A-1-b – SM	697.824	1.431	-1.064
A-6(7) - CL	3535.760	0.112	-1.538

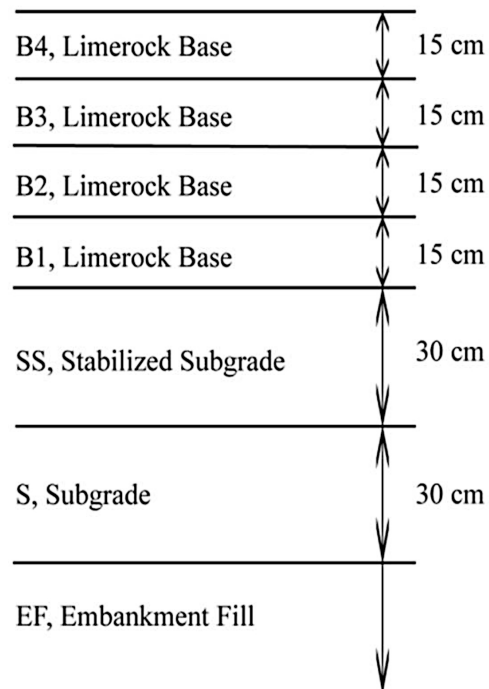


Figure 3.2 Soil layering of the Florida earthwork site

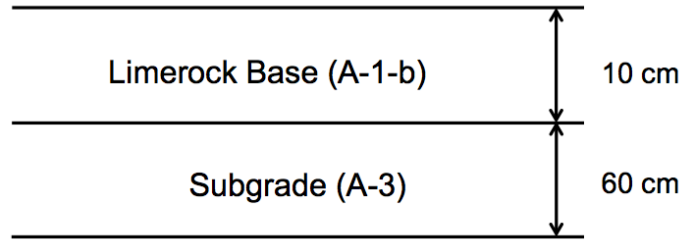


Figure 3.3 Soil Layers used in M_r FE analysis

3.2 Iterative Equivalent Linear Model for M_r

Prior to performing the dynamic analyses on the finite element models previously described, an algorithm newly developed in this thesis work performs an iterative loop to determine the appropriate stress-dependent resilient moduli for each element. A MATLAB script was created to interact with ABAQUS in performing this iterative procedure. The iterative procedure is based on Idriss's strain dependent, iterative equivalent linear model for shear modulus degradation for seismic applications described in detail in the literature (e.g., Kramer 1996) and was adapted for the stress-dependent resilient modulus model for this study.

The algorithm's logic follows the flow chart in Figure 3.4. The iterative procedure begins with the input of constant material properties, as summarized in Table 3.4, and user-supplied initial estimates of M_r to all of the elements in the finite element mesh. An ABAQUS input file is then created using these values and then read by ABAQUS to begin a dynamic analysis of the model. Upon completion, the ABAQUS outputs the time varying principal stresses that occur within each element. The time varying principal stresses, $\sigma_i(t)$, were then simplified to scalar, representative, effective values as:

$$(\sigma_i)_{eff} = 0.65 \left| \sigma_i(t) \right|_{max} \quad (3.2)$$

where i = principal stress component (σ_1 , σ_2 , or σ_3) and $(\sigma_i)_{eff}$ = effective principal stress values. This simplification of the time varying principal stresses to effective scalar values is based upon Idriss's method for simplifying time varying shear strains to effective scalar value (e.g. Kramer 1996). The effective principal stress values are then used to update the resilient moduli of each element based on equation 2.6. In order for the program to converge smoothly, a damping factor, λ is applied to the determine the predicted resilient modulus for the next iteration (Kim 2007):

$$M_r^j = (1 - \lambda) M_r^{j-1} + \lambda M_{r-calc}^j \quad (3.3)$$

where j = current iterative loop, λ = damping factor = 0 - 1.0, $M_r^{j-1} = M_r$ of the previous loop, and $M_{r-calc}^j = M_r$ determined via equation 2.6. As suggested by Kim (2007), a small value of $\lambda = 0.2$ was selected for the purposes of this study to promote the convergence of the model. The predicted M_r from the current iterative loop is compared to the M_r from the previous loop in the following equation:

$$\% \text{ convergence} = \left(1 - \left| \frac{M_r^j}{M_r^{j-1}} \right| \right) 100\% \quad (3.4)$$

If the % convergence from equation 3.4 is below 5% the program concludes; however, if it is not below 5%, the ABAQUS input file is updated with the new calculated M_r^j values, and the iterative procedure repeats until 5% convergence is achieved or specified maximum iterations is reached. A maximum iteration value of 100 was applied to the FE program such that run-time constraints are not exceeded. In this manner, each element may be assigned its own resilient modulus, allowing for spatial variation in soil properties based upon the stress fields induced by the dynamic loading.

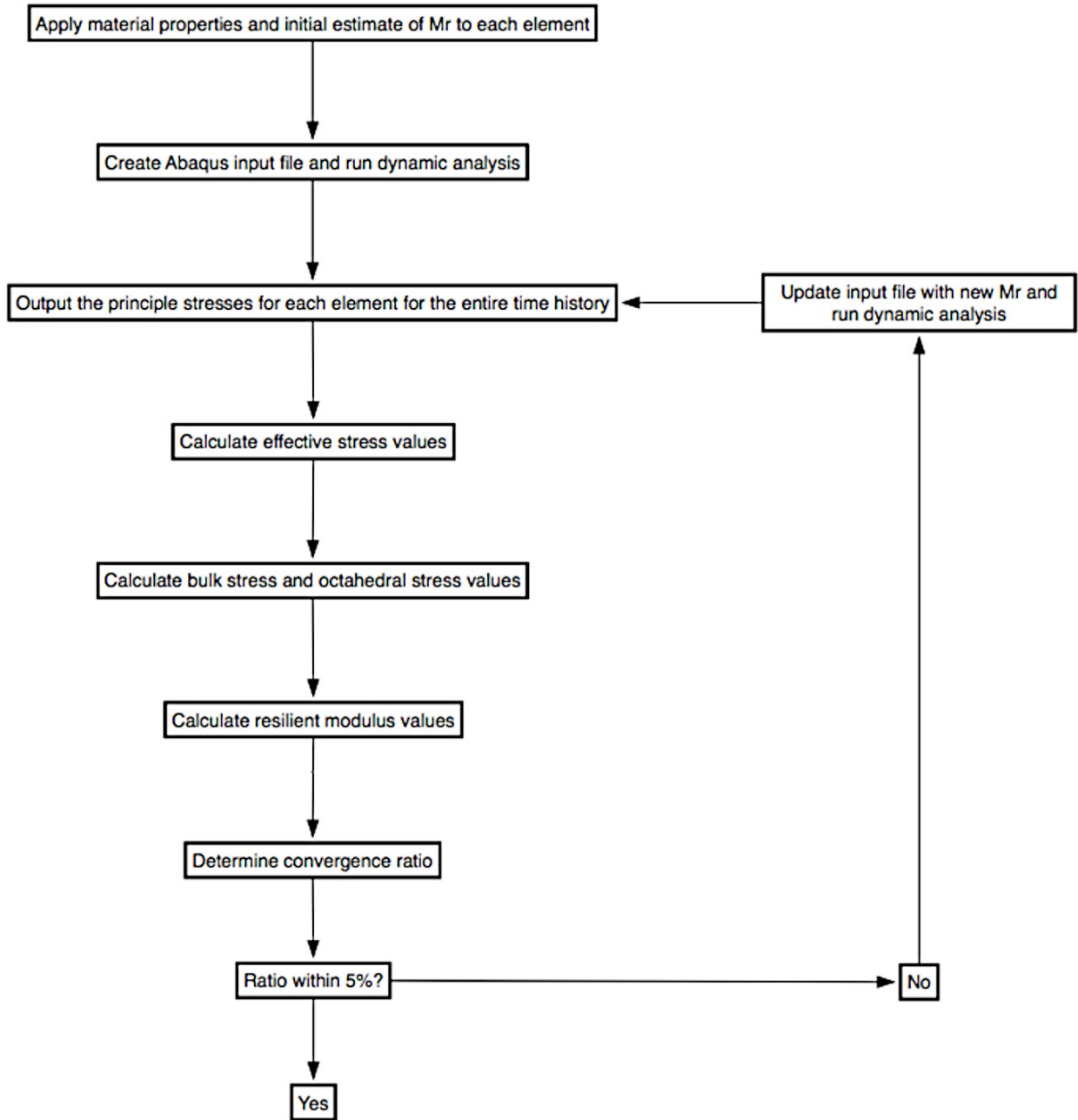


Figure 3.4 Flow chart showing iterative procedure

Table 3.4 Material properties applied to the FE model

Soil Property	Value
Density	2000 kg/m ³
Poisson's Ratio	0.35
α	25 s ⁻¹
β	0.002 s

3.3 Determination of k_s Using Dynamic Time Domain FE Analysis

The results of the full, iterative dynamic analyses are then used to determine the soil stiffness values through the force-deflection behavior of the roller. The implementation of this analysis is presented in Figure 3.5.

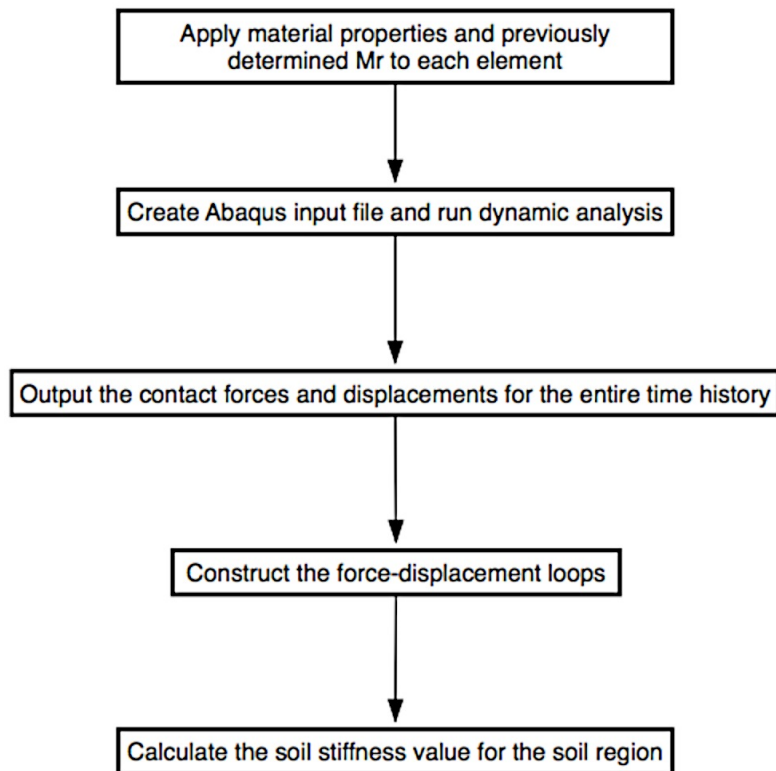


Figure 3.5 Flow chart detailing the dynamic FE analysis procedure

Upon completion, ABAQUS outputs the time varying F_c and z_d results. These values can then be used to develop the associated force-deflection loops for the full time history. From these loops, the roller measured stiffness values, k_s , can be determined by calculating the gradient of the line that passes through the point of zero displacement and the point of maximum displacement (see Figure 3.6). This value can then be compared to the associated k_s obtained from the field data.

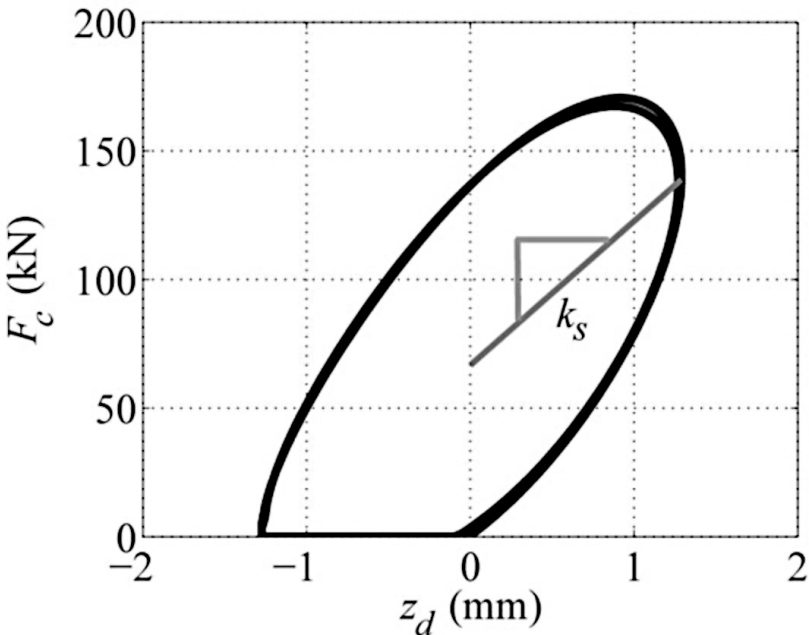


Figure 3.6 Force-displacement loop showing soil stiffness, k_s (Neff 2013)

3.4 FE Modeled Subgrade Layer

In order to compare the FE results with the collected field data, the material properties used in the model were selected based on the material properties obtained in Table 3.3. The subgrade layer was determined on site to have an AASHTO Classification of A-3. Based on this, the values for K_1 , K_2 , and K_3 were chosen according to Table 3.2 and applied to the iterative M_r ,

model. The iterative procedure converged in 28 loops and resulted in a convergence value of 3.87%. The resulting field of M_r values, see Figure 3.7, was then applied to dynamic time domain FE analysis as described above. The resulting force-displacement behavior, as seen in Figure 3.8, can then be found by applying the M_r field to the dynamic FE model.

By determining the maximum and minimum forces and displacements (see Table 3.5), the resulting stiffness of the soil continuum can be calculated as described above. The force-displacement behavior in Figure 3.8 resulted in a material stiffness value of 50.5 MN/m.

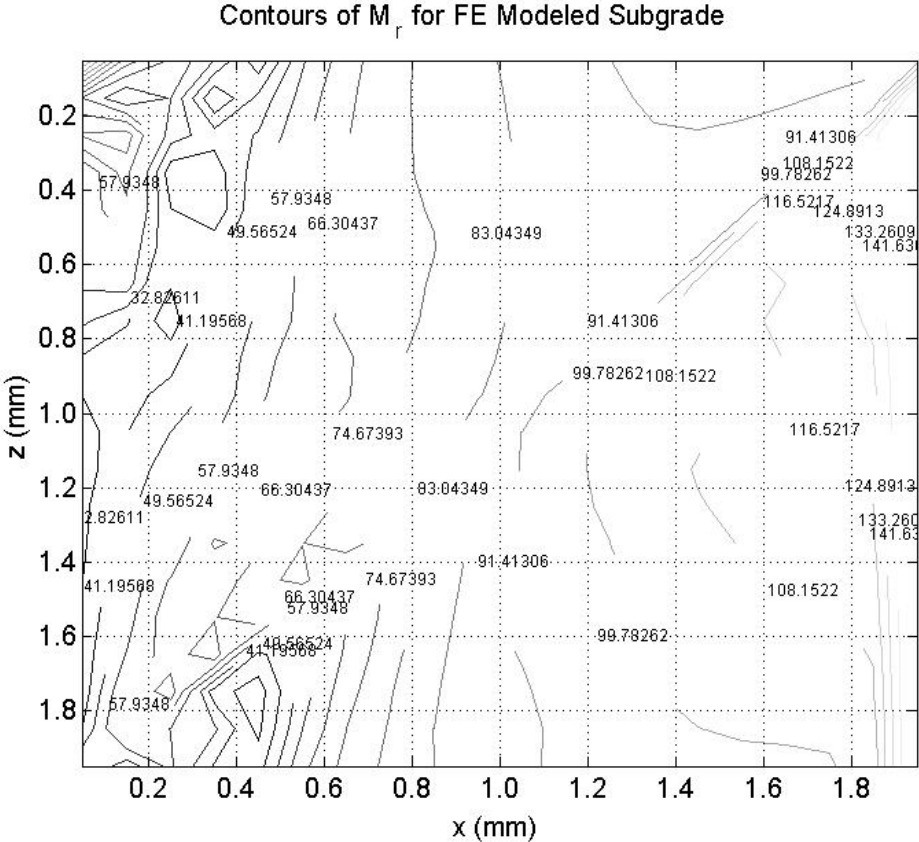


Figure 3.7 M_r (MPa) field for FE modeled subgrade

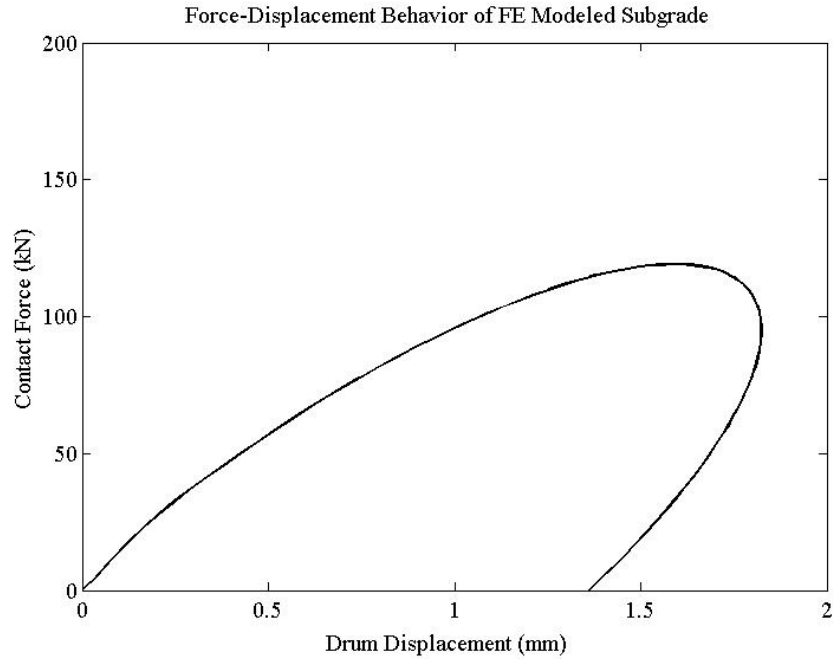


Figure 3.8 Force Displacement Behavior of the FE Modeled Subgrade Material

Table 3.5 Maximum/minimum forces and displacements for FE subgrade

$F_{c-\min}$	$F_{c-\max}$	$Z_{d-\min}$	$Z_{d-\max}$
0 kN	89.7 kN	0 mm	1.8 mm

3.5 FE Modeled Limerock Layer

The addition of the limerock layer was modeled as a layer of A-1-b soil determined via Table 3.3 and the corresponding values of K_1 , K_2 , and K_3 chosen from Table 3.2. The iterative procedure converged in 100 loops and resulted in a convergence ratio of 14.66%. This ratio was permitted to be greater than the specified 5% convergence ratio due to program run time constraints. The resulting field of M_r values, see Figure 3.9, was then applied to dynamic time domain FE analysis as described above to determine the force-deflection behavior of the roller on the limerock layer. The results of this analysis can be found in Figure 3.10.

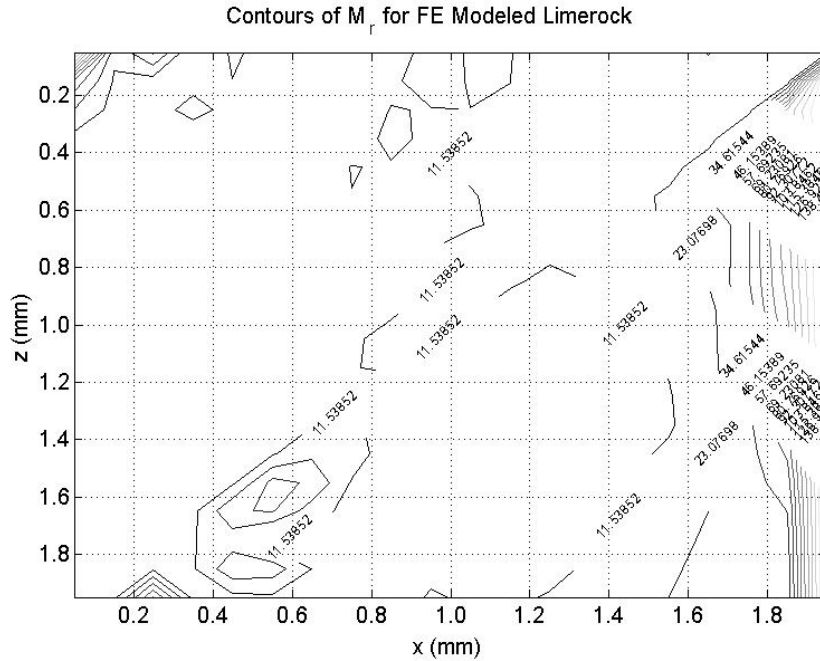


Figure 3.9 M_r (MPa) field for FE modeled limerock

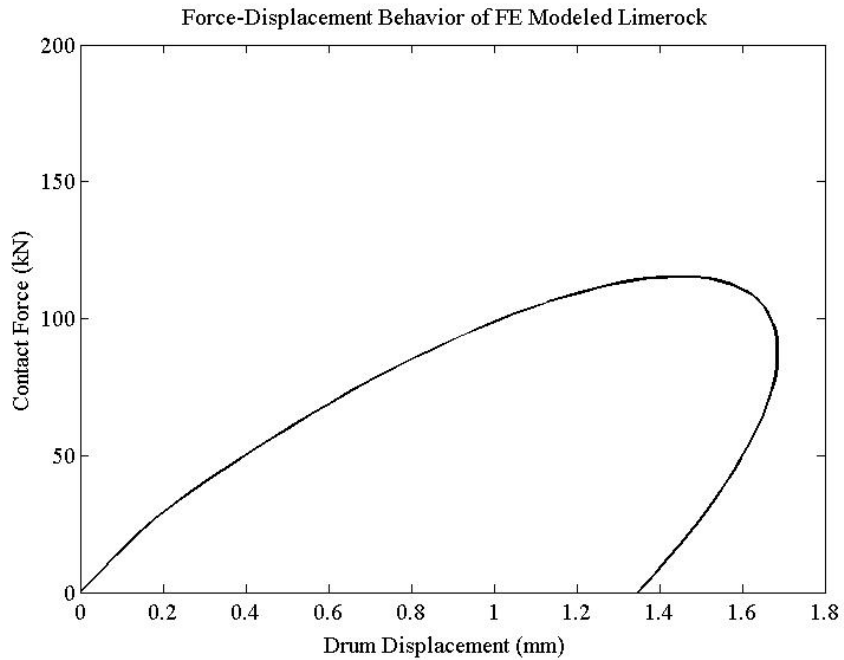


Figure 3.10 Force-Displacement Behavior of FE Modeled Limerock

By determining the maximum and minimum forces and displacements (see Table 3.6), the resulting stiffness of the soil continuum can be calculated as described above. The force-displacement found in Figure 3.10 resulted in a soil stiffness value of 53.1 MN/m.

Table 3.6 Maximum/minimum forces and displacements for FE limerock

$F_{c-\min}$	$F_{c-\max}$	$z_{d-\min}$	$z_{d-\max}$
0 kN	89.6 kN	0 mm	1.7 mm

3.6 FE Model Validation

In order to validate the result of the proposed M_r model, the maximum and minimum contact forces and displacements as well as the resulting k_s values from the FE M_r model were compared with the values from field data obtained from Florida test beds. The force-displacement behavior for the subgrade layer and the limerock layer can be found in Figures 3.11 and 3.12 respectively. Soil stiffness values for the subgrade and limerock layers were calculated from the field-measured force-displacement behavior (see Table 3.7) to be 55.7 MN/m and 74.3 MN/m, respectively.

Table 3.7 Maximum/minimum forces and displacements for Florida field data

Layer	$F_{c-\min}$	$F_{c-\max}$	$z_{d-\min}$	$z_{d-\max}$
Subgrade	0 kN	147.4 kN	-1.3 mm	1.3 mm
Limerock	0 kN	223.0 kN	-1.5 mm	1.5 mm

The subgrade soil stiffness values calculated from the force displacement loops from the FE model are relatively close to those calculated from field data with a difference of 5.95%. The limerock k_s values however, differed by 28.5%. This large difference is most likely due to the fact that the limerock M_r iterative process was halted at a convergence of 14.66% rather than the suggested 5%. A summary of the soil stiffness values can be found in Table 3.8. Although

the resulting k_s values from FE analysis and field data were relatively comparable, the resulting maximum forces and displacements differed considerably in the limerock layer and slightly in the subgrade layer. This may be due to slight differences between the described soil properties and the soil properties experienced in the field.

Table 3.8 Comparison of Soil Stiffness (k_s) Values

Layer	M_r FE Model	Field Data
Subgrade	50.5 MN/m	53.7 MN/m
Limerock	53.1 MN/m	74.3 MN/m

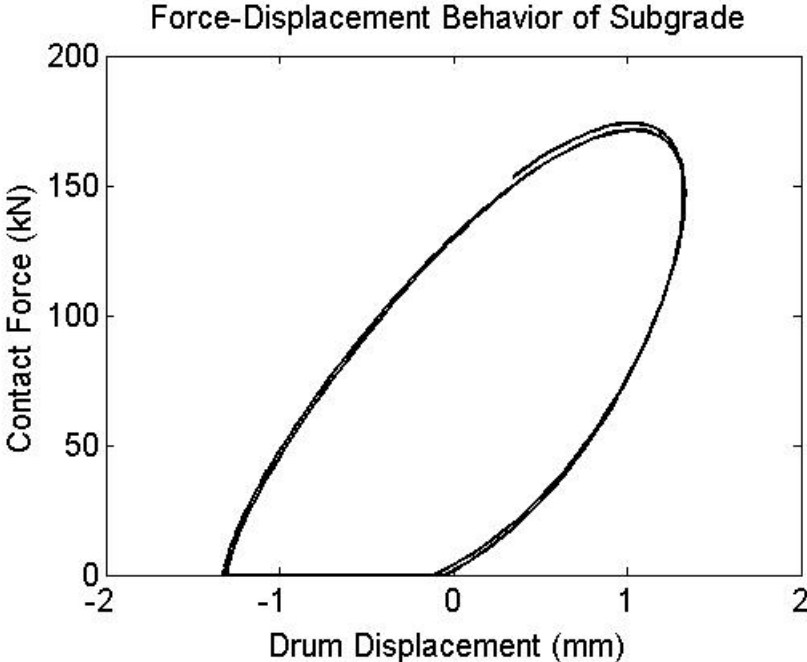


Figure 3.11 Force-Displacement Behavior of the Subgrade Layer from Field Data (Neff et al. 2014)

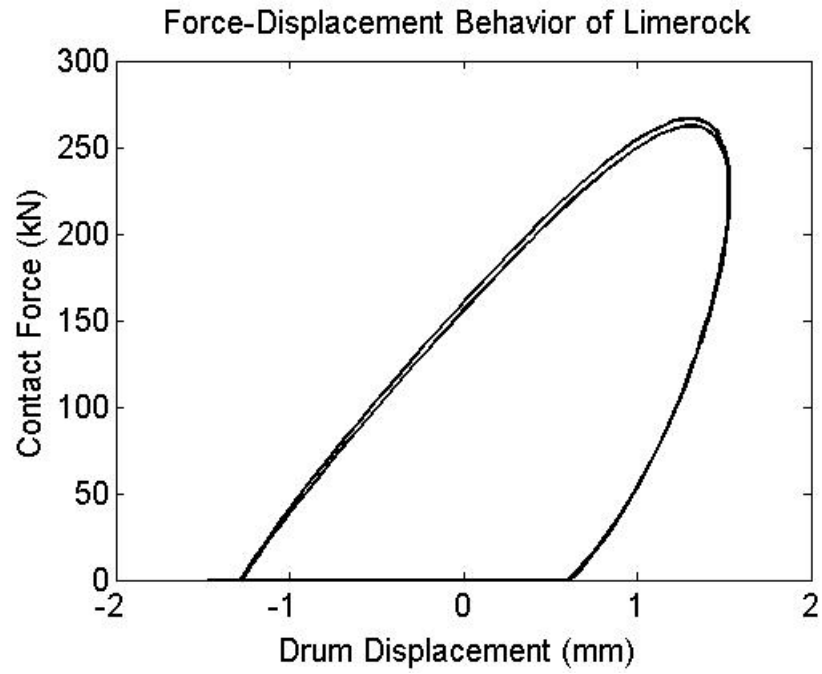


Figure 3.12 Force-Displacement Behavior of the Limerock Layer from Field Data (Neff et al. 2014)

CHAPTER 4

CONCLUSION AND RECOMMENDATIONS

4.1 Conclusions

This study presents of the development and implementation of an iterative stress-dependent procedure to specify spatially varying M_r values of soil foundation in a dynamic FE analysis. This procedure was used to model vibratory dynamic roller mapping operations and the resulting force-deflection behaviors and roller soil stiffness measurements. The finite element analysis results show that the k_s values determined using the developed iterative M_r model are somewhat similar to the k_s values found from the field data. The differences between the FE results and the field results for k_s can be explained by the drawbacks and limitations of the proposed M_r model. A major drawback of the proposed M_r model is that although the model predicts relatively similar k_s values, the resulting force-displacement maxima do not necessary relate to observed field conditions. The primary limitation of the proposed M_r model is the simulation run-time. Run-times for the iterative process were on average one day per iterative loop. For softer soils such as the A-3 subgrade layer the larger program run-time was not a concern due to the small amount of iterations required for convergence. However, for stiffer soils, such as the A-1-b limerock base layer, the required number of iterations required for convergence became a concern with respect to program run-time. In order to manage this limitation, a maximum iteration value of 100 was applied to the program. However, it was determined in this study that the accuracy of the FE model determined k_s values is highly

dependent on the full convergence of the iterative M_r model (i.e. % convergence < 5%).

Additionally, the proposed model is limited by the knowledge of the analyzed soil in that it is highly dependent on the accurate description of soil type and density. Due to the fact that the material properties for the embankment fill were not determined at the Florida field site, these material properties were assumed in the FE model. Because of this, it is likely that the differences between the field data and FE results are due to an inaccurate assumption of material properties.

4.2 Recommendations

Further research is required to fully develop this model to predict the soil behavior during dynamic IC procedures. It is recommended that further research be conducted to reduce the program run-times. This may be done through the use of a graded FE mesh that utilizes smaller mesh elements near the application of the dynamic loading and larger element near the boundaries. In addition it is recommended that further research be conducted to apply the proposed model to the IC procedure such that real-time M_r and associated k_s values are supplied to the vibratory roller operator during the mapping process for QA/QC.

LIST OF ACRONYMS, ABBREVIATIONS, AND SYMBOLS

AASHTO:	American Association of State Highway and Transportation Officials
ASTM:	American Society for Testing and Materials
B1:	First base lift
B2:	Second base lift
B3:	Third base lift
B4:	Forth base lift
CCC:	Continuous Compaction Control
CCV:	Continuous Control Value
CMV:	Compaction Meter Value
CPE4R:	4-node bilinear, reduced integration with hourglass control
EF:	Embankment Fill
FE:	Finite element
IC:	Intelligent Compaction
LWD:	Lightweight Deflectometer
MEPDG:	Mechanistic-Empirical Pavement Design Guide
MV:	Measurement value
NCHRP:	National Cooperative Highway Research Program
QA:	Quality Assurance
QC:	Quality Control
S:	Subgrade lift
SS:	Stabilized subgrade lift
USCS:	Unified Soil Classification System

$[C]$:	Global Damping Matrix
C_c :	Compression Index
C_u :	Uniformity Coefficient
D_{50} :	Mean grain size through which 50% of the soil passes through
E_{vib} :	Vibration Modulus
F_c :	Contact Force
F_{ecc} :	Eccentric Excitation Force
g :	Acceleration due to gravity
h :	Lift thickness
h_{total} :	Total nominal lift thickness
$[K]$:	Global Stiffness Matrix
k :	Bomag roller measured soil stiffness
k_s :	Ammann and Case roller measure soil stiffness
$[M]$:	Global Mass Matrix
m_d :	Mass of the roller drum
m_f :	Mass of the roller frame
$m_0 \mathbf{e}_0$:	Eccentric mass moment
M_r :	Resilient Modulus
M_{r-calc}^j :	Resilient Modulus calculated in iterative loop j

P_a :	Atmospheric pressure
z_d :	Drum displacement
\ddot{z}_d :	Drum acceleration
α / β :	Rayleigh damping parameters
ε_r :	Recoverable strain
θ :	Bulk stress
λ :	Iterative damping factor
ν :	Poisson's Ratio
ρ :	Density
$\sigma_1 / \sigma_2 / \sigma_3$:	Principal Stress values
σ_d :	Deviator stress
$(\sigma_i)_{eff}$:	Effective principal stress value ($i = 1, 2, \text{ or } 3$)
$\sigma_i(t)$:	Time-varying principal stress values ($i = 1, 2, \text{ or } 3$)
τ_{oct} :	Octahedral Shear Stress
ϕ :	Phase shift between the eccentric excitation force and the drum displacement
Ω :	Excitation frequency
ω :	Roller operating frequency

REFERENCES CITED

- AASHTO. (2003). "Standard Method of Test for Determining the Resilient Modulus of Soils and Aggregate Materials." SHRP Protocol P46. American Association of State and Highway Transportation, Washington, D.C.
- ABAQUS. (2013). ABAQUS Documentation. Dassault Systèmes. Providence, RI.
- Brown, S. F., & Pappin, J. W. (1981). "Analysis of Pavements with Granular Bases." *Transportation Research Record*. (810): 17-23.
- Facas, N.W. (2010). "Characterization, Modeling, and Specifications of Vibratory Roller-Measured Soil Properties." PhD Dissertation, Division of Engineering, Colorado School of Mines.
- George, K.P. (2004). "Prediction of Resilient Modulus from Soil Index Properties." Rep. No. FHWA/MS-DOT-RD-04-172. Federal Highway Administration, U.S. Department of Transportation, Washington D.C.
- Hicks, R.G., Monismith, C.L. (1971). "Factors Influencing the Resilient Response of Granular Materials." *Highway Research Record*. 345: 15-31.
- Hjelmstad, K.D., Taciroglu, E. (2000). "Analysis and Implementation of Resilient Modulus Models for Granular Solids." *Journal of Engineering Mechanics*. 126(8): 821-830.
- Huang, Y.H. (1993). "Pavement Analysis and Design." Prentice Hall, Englewood Cliffs, N.J.
- Kenneally, B., Musimbi, O.M., Wang, J.C., Mooney, M.A. (2014). "Finite Element Analysis of Vibratory Roller Response on Layered Soil Systems." *International Journal for Numerical and Analytical Methods in Geomechanics*. *Submitted May 2014*.

- Kim, M. (2007). "Three-Dimensional Finite Element Analysis of Flexible Pavements Considering Nonlinear Pavement Foundation Behavior." PhD Dissertation, University of Illinois at Urbana-Champaign.
- Kramer, S.L. (1996). "Geotechnical Earthquake Engineering." Prentice Hall, Englewood Cliffs, N.J.
- Li, D., Selig, E.T. (1994). "Resilient Modulus for Fine-Grained Subgrade Soils." *Journal of Geotechnical Engineering*. 120(6): 939-957.
- Mooney, M.A., Adam, D. (2007). "Vibratory Roller Integrated Measurement of Earthwork Compaction: An Overview." *Proceedings of 7th International Symposium on Field Measurements in Geomechanics-FMGM*.
- Mooney, M.A., Facas, N.W. (2012). "Extraction of Layered Properties from Intelligent Compaction Data." *National Cooperative Highway Research Program*, Washington, D.C.
- Mooney, M.A., Rinehart, R.V., White, D., Vennapusa, P., Facas, N.W., Musimbi, O.M. (2010). "Intelligent Soil Compaction Systems." *NCHRP Report 676*. Transportation Research Board, Washington, D.C.
- Mun, S. (2003). "Nonlinear Finite Element Analysis of Pavement of its Application to Performance Evaluation." PhD Dissertation. Department of Civil Engineering, North Carolina State University.
- Musimbi, O.M. (2011). "Experimental and Numerical Investigations of a Vibratory Roller Compactor and Layered Soil Interactions." PhD Dissertation, Division of Engineering, Colorado School of Mines.

- NCHRP. (2004). "Guide for Mechanistic Empirical Design of New and Rehabilitated Pavement Structures (Final Report)." Transportation Research Board, National Research Council, Washington D.C.
- Neff, A. (2013). "Analysis of Intelligent Compaction Field Data on Layered Soil." Master's Thesis, Division of Engineering, Colorado School of Mines.
- Neff, A., McAdams, M., Wang, J.C., Mooney, M.A. (2014). "*Analysis of Center of Gravity Roller Drum Soil Stiffness on Compacted Layer Earthwork*." Canadian Geotechnical Journal.
- Rinehart, R.V. (2008). "Characterizing Soil Stiffness Measured by a Vibratory Roller Compactor and its Relationship to In-Situ Stress Strain Response." PhD Dissertation, Division of Engineering, Colorado School of Mines.
- Rinehart, R.V., Berger, J.R., Mooney, M.A. (2009). "Comparison of Stress States and Paths: Vibratory Roller-Measured Soil Stiffness and Resilient Modulus Testing." Transportation Research Record. 2116: 8-15.
- Rinehart, R.V., Mooney, M.A. (2008). "Measurement Depth of Vibratory Roller-Measured Soil Stiffness." Géotechnique. 59(7): 609-619.
- Putri, E.E., Rao, N.S.V, Mannan, M.A. (2010). "Evaluation of the Modulus of Elasticity and Resilient Modulus for Highway Subgrades." EJGE. 15: 1285-1293.
- Seed, H.B., Mitry, F.G., Monismith, C.L., Chan, C.K. (1967). "Factors Influencing the Resilient Deformation of Untreated Aggregate Base in Two Layer Pavements Subjected to Repeated Loading." Highway Research Record. (190).

- Seed, H.B., Chan, C.K, Lee, C.E. (1962). "Resilience Characteristics of Subgrade Soils and Their Relation to Fatigue Failures in Asphalt Pavements." Proceeding of the International Conference on the Structural Design of Asphalt Pavements.
- Taciroglu, E. (1998). "Constitutive Modeling of the Resilient Response of Granular Solids." PhD Dissertation, University of Illinois at Urbana-Champaign.
- Taciroglu, E., Hjelmstad, K.D. (2002). "Simple Nonlinear Model for Elastic Response of Cohesionless Granular Materials." *Journal of Engineering Mechanics*. 128(9): 969-978.
- Uzan, J. (1992). "Resilient Characterization of Pavement Materials." *International Journal for Numerical and Analytical Methods in Geomechanics*. 16: 453-459.
- Uzan, J. (1985). "Characterization of Granular Material." *Transportation Research Record*. 1022: 52-59.