University Transportation Research Center - Region 2

Final Report

Nondestructive Evaluation of Pavement Structural Condition for Rehabilitation Design

Performing Organization: Rutgers University

May 2016

Sponsor:
University Transportation Research Center - Region 2

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The Region 2 University Transportation Research Center (UTRC) is one of ten original University Transportation Centers established in 1987 by the U.S. Congress. These Centers were established with the recognition that transportation plays a key role in the nation's economy and the quality of life of its citizens. University faculty members provide a critical link in resolving our national and regional transportation problems while training the professionals who address our transportation systems and their customers on a daily basis.

The UTRC was established in order to support research, education and the transfer of technology in the field of transportation. The theme of the Center is "Planning and Managing Regional Transportation Systems in a Changing World." Presently, under the direction of Dr. Camille Kamga, the UTRC represents USDOT Region II, including New York, New Jersey, Puerto Rico and the U.S. Virgin Islands. Functioning as a consortium of twelve major Universities throughout the region, UTRC is located at the CUNY Institute for Transportation Systems at The City College of New York, the lead institution of the consortium. The Center, through its consortium, an Agency-Industry Council and its Director and Staff, supports research, education, and technology transfer under its theme. UTRC's three main goals are:

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16. Abstract

Falling Weight Deflectometer (FWD) is the common non-destructive testing method for in-situ evaluation of pavement condition. This study aims to develop finite element (FE) models that can simulate FWD loading on pavement system and capture the complexity in material properties, layer interface, and boundary conditions. Parametric analysis was conducted considering the effects of dynamic analysis, temperature gradient, bedrock depth, asphalt layer delamination, viscoelasticity, and unbound material nonlinearity on pavement surface deflections and critical strain responses. Although the parametric analysis findings vary depending on the specific pavement response, the study results illustrate the appropriate selection of analysis type, constitutive models of pavement material, and layer boundary conditions on the accuracy of FE modeling results. In particular, the analysis findings show that delamination in asphalt layers induces the greater strain responses; while neglecting bedrock effect overestimates surface deflections. The developed FE models can directly benefit the use of FWD testing for in-situ pavement condition evaluation, such as pavement performance prediction and/or backcalculation of layer moduli.

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EXECUTIVE SUMMARY

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1. INTRODUCTION

1.1 Background

Falling Weight Deflectometer (FWD) is the common non-destructive testing method for in-situ evaluation of pavement condition. The surface deflections measured under FWD testing are used to assess structure capacity of pavement system and backcalculate the moduli of pavement layers. Therefore, accurate mechanistic models are needed to analyze FWD testing results, considering the interaction between loading, environmental, and pavement structure. Analytical and numerical models have been used for years to simulate FWD testing and predict pavement responses. In the early studies, multi-layer models with assumed linear elastic moduli for all layers were developed for analysis of pavement structural behavior and backcalculation of layer moduli through iterations. To overcome the limitations pertaining to simple modeling assumptions, finite element (FE) models were developed considering more realistic loading and materials assumptions in simulation of FWD testing.

Previous researches mainly focused on one or two factors among the interaction between environmental, material, and structural behavior under impulsive dynamic loading applied in FWD testing. The dynamic analysis approach was used with consideration of viscoelastic asphalt layer to simulate the impulsive FWD loading and wave propagation in the pavement structure (Al-Qadi et al. 2010; Xu and Prozzi 2013; Xu and Prozzi 2015). The simulation results showed better agreements in matching the field FWD measurements as compared to non-dynamic analysis or elastic layer assumption. The dynamic behavior of FWD can be magnified with the existence of bedrock as a rigid layer underlying the subgrade (Broutin 2010). Few researches have considered the effect of boundary condition due to bedrock and investigated the threshold depth after that there was no negative effect of bedrock on pavement surface deflections.

The correlation of asphalt concrete layer modulus to the temperature along the layer depth was investigated. The results indicated that the mid-depth temperature can be considered representing the in-depth temperature distribution for the prediction of elastic modulus (Salem et al. 2004). However, in consideration of viscoelastic property of asphalt layer, temperature profile in positive or negative distributions can cause different stress and strain distributions as compared to the constant temperature profile (Lu et al. 2009). The FWD testing results were also affected by the behavior of base layer and subgrade. It was reported that neglecting nonlinearity of unbound material properties produced substantial errors in calculating the responses of flexible pavement (Schwartz 2002; Kim et al. 2009).

Another significant issue related to the existing pavement condition is the degradation of interlayer bonding between asphalt layers. An investigation on several newly constructed asphalt pavements was carried out to study the effect of interlayer debonding behavior on back-calculation of pavement layer moduli (Hakim et al. 2000). The effect of layer debonding could become more serious in the composite pavement. It was found that the broken bonds between asphalt overlay and existing concrete slabs could cause an oscillation of the deflection-time history due to the vibrating behavior experienced by the asphalt overlay (Shoukry et al. 1997). Therefore, to better understand pavement behavior under FWD loading, all the affecting factors in FWD testing need to be considered in the simulation models in order to enhance the accuracy modeling results.

1.2 Objective

The objectives of this study have two folds. The first objective is to develop finite element (FE) models that can simulate FWD loading on pavement system and capture the complexity in material properties, layer interfaces, and boundary conditions. The second objective is to analyze the important factors that should be considered in the FE analysis, including dynamic analysis, temperature gradient, depth to bedrock, asphalt layer delamination, viscoelasticity of asphalt layer, and nonlinearity of unbound material. The development of FE models can directly benefit the use of FWD testing for in-situ pavement condition evaluation, such as pavement performance prediction and backcalculation of layer moduli.

2. FINITE ELEMENT MODELLING

2.1 Viscoelastic Modulus of Asphalt Layer

Constitutive models of each pavement layer are critical for mechanistic analysis of pavement responses. The time- or frequency- and temperature-dependent behavior of asphalt layer was considered with the viscoelastic model in this study. The relaxation modulus of asphalt mixture was modeled as a generalized Maxwell solid model in terms of Prony series, as shown in Eq. 1 and 2. The relaxation modulus can be obtained from laboratory-tested creep compliance or dynamic modulus by means of an inter-conversion relationship (Park and Kim 1999). The temperature dependency of AC modulus is characterized by time-temperature superposition principle. This behavior introduces the horizontal shifting of the material property to form a single characteristic master curve as a function of reduced time (or frequency) at a desired reference temperature. The relationship between the shift factor and the temperature can be approximated by the WLF (Williams-Landell-Ferry) function (ABAQUS 2010).

$$G(t) = G_0[1 - \sum_{i=1}^{n} G_i(1 - \mathrm{e}^{-t/\tau_i})] \tag{1}$$

$$K(t) = K_0[1 - \sum_{i=1}^{n} K_i(1 - \mathrm{e}^{-t/\tau_i})] \tag{2}$$

Where,

G is shear modulus;

K is bulk modulus;

t is relaxation time;

G_0 and K_0 are instantaneous shear and volumetric elastic moduli; and

G_i, K_i, and τ_i are Prony series parameters.

The BELLS3 equation, which was validated with measurements from field sections in the Long Term Pavement Performance (LTPP) study, was used to predict temperatures within asphalt layer, as shown in Eq. 3 (FHWA 2000). Compared to other temperature prediction models developed by Huber (Huber 1994) and Park (Park et al. 2001), the BELLS3 equation predicted the distributed temperatures that were closest to the measured ones (Gedafa et al. 2013).

$$\begin{aligned} T_d &= 0.95 + 0.892IR + [\log(d) - 1.25][-0.448IR + 0.621(1-\mathrm{day}) \\ &+ 1.83\sin(hr_{18} - 15.5)] + 0.042IR\sin(hr_{18} - 13.5) \end{aligned} \tag{3}$$

Where,

T_d is pavement temperature at depth d, °C;

IR is pavement surface temperature, °C;

d is depth at which mat temperature is to be predicted, mm;

1-day is average air temperature the day before testing, °C;

$\sin$ is sine function on an 18-hr clock system, with 2π radians equal to a 18-hr cycle; and

hr_{18} is time of day, in a 24-hr clock system, but calculated using an 18-hr AC temperature rise-and-fall time cycle.

2.2 Nonlinear Modulus of Unbound Material

The unbound layers including aggregate base and subgrade were both modeled as the nonlinear materials. A generalized model was introduced to express the vertical modulus of unbound layer material, as shown in Eq. 4 (ARA 2004). Moreover, the horizontal and shear modulus ratios (*n* and *m*) were introduced for the aggregate base layer for cross-anisotropic behavior. It was found that horizontal modulus ratio and shear modulus ratio have relatively small ranges of variation. Therefore, typical values of 0.15 and 0.34 for *n* and *m* in aggregate base layer were used in this study (Tutumluer and Thompson 1997).

$$M_r^v = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3} \tag{4}$$

with $\theta = \sigma_1 + \sigma_2 + \sigma_3$ and $\tau_{oct} = \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_1 - \sigma_3)^2}$

Where,

M_r^v is vertical resilient modulus (kPa) ;

θ is bulk stress (kPa);

τ_{oct} is octahedral shear stress (kPa);

σ_1, σ_2, and σ_3 are maximum, middle, and minimum principal stresses;

k_1, k_2, and k_3 are exponent parameters; and

p_a is atmospheric pressure (100kPa).

ABAQUS/Standard uses the iterative Newton-Raphson method to solve nonlinear equations. The applied load in this method is augmented incrementally, and at each increment the program solves a system of equations through iterations. The iterations continue on the basis of the previous solutions until it reaches a reasonable convergence (ABAQUS 2010). Because the modulus of the unbound material is a function of the total stress states, a modified Newton-Raphson approach with secant stiffness was used in this study and implemented in a user material subroutine (UMAT) (Wang and Al-Qadi 2013).

2.3 Dynamic Analysis

Static, quasi-static, and dynamic analysis have been employed in different studies for pavement analysis. Dynamic transient analysis allows for the inertia associated with the FWD loading and the dependency of viscoelastic material properties on loading frequency. For a nonlinear dynamic analysis problem, the direct integration method is commonly employed. Therefore, the implicit dynamic analysis was selected in this study because it provides better numerical stability than explicit analysis and is generally efficient for structural dynamic problems (Bathe 1982).

The equation of motion of a multi-degree of freedom system with damping is shown in Eq. 5. When using viscoelastic material behavior for an asphalt layer, it is not necessary to introduce additional structural or mass damping for that layer. A popular spectral damping scheme used in structure dynamic analysis is Rayleigh damping. For the dynamic loading on a pavement structure, the two frequencies in calculating the Raleigh coefficients may be taken as the lowest natural frequency of the structure and the highest loading frequency. The critical damping ratio of soil falls in the range of 2% to 5% depending on the confining pressure and shear strain level, which can be measured from the resonant column test or cyclic triaxial test (Zhong et al. 2002). Because the subgrade is considered as elastic material without any other energy dissipation sources (such as plastic), the maximum damping ratio, 5%, is used.

$$[M]\{\ddot{U}\}+[C]\{\dot{U}\}+[K]\{U\}=\{P\} \tag{5}$$

where,

$[M]$ is mass matrix;

$[C]$ is damping matrix;

$[K]$ is stiffness matrix;

$\{\ddot{U}\}$ is acceleration vector;

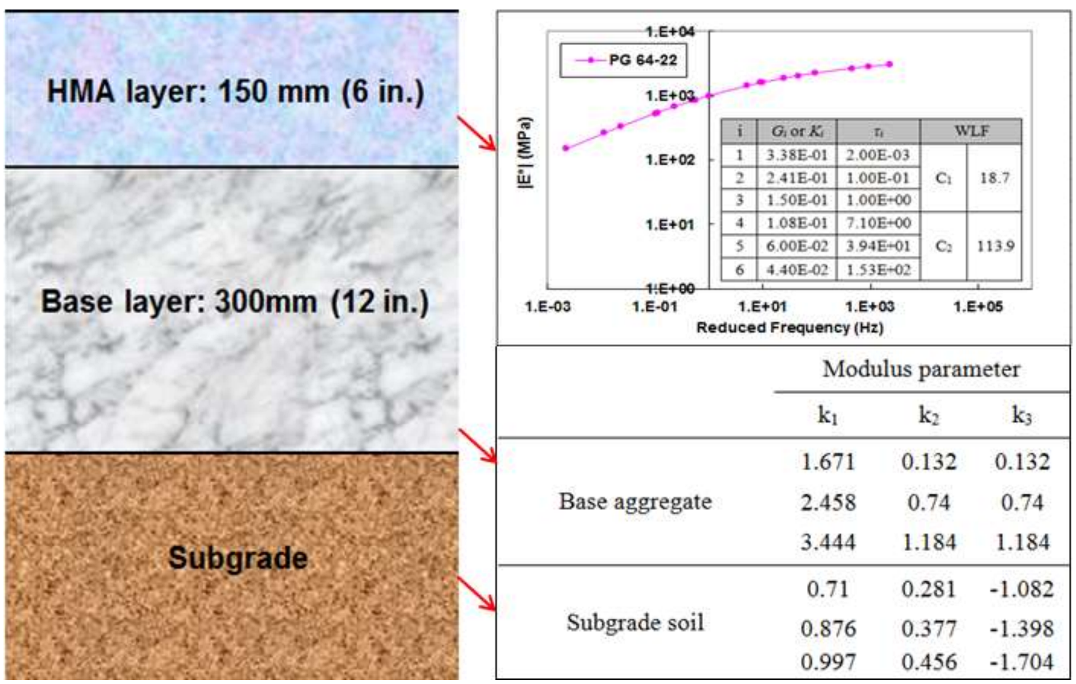
$\{\dot{U}\}$ is velocity vector;

$\{U\}$ is displacement vector; and

$\{P\}$ is displacement vector.

2.4 Finite Element Model Development

The modeled pavement structure was constructed with three structure layers, as shown in Fig. 1. It consists of a 150-mm asphalt pavement placed on a 300-mm aggregate base layer over subgrade. The master curve of dynamic modulus and temperature shift factors were obtained from the prediction model that was developed from the LTPP study using the asphalt binder type of PG 64-22 and 4% air void (Kim et al. 2011). The material parameters for base aggregate and soil subgrade were obtained from the laboratory-developed empirical equations that related the physical properties of unbound material to nonlinear modulus parameters (George 2004; Xiao and Tutumluer 2012; Xiao et al. 2011). Three groups of parameters were selected to represent the variations of nonlinear modulus. Pavement temperature profiles were characterized using the BELLS model and the climate data extracted from the LTPP database (FHWA 2016).



i	G_i or K_i	τ_i	WLF	
1	3.38E-01	2.00E-03	C_1	18.7
2	2.41E-01	1.00E-01		
3	1.50E-01	1.00E+00		
4	1.08E-01	7.10E+00	C_2	113.9
5	6.00E-02	3.94E+01		
6	4.40E-02	1.53E+02		

	Modulus parameter		
	k_1	k_2	k_3
Base aggregate	1.671	0.132	0.132
	2.458	0.74	0.74
	3.444	1.184	1.184
Subgrade soil	0.71	0.281	-1.082
	0.876	0.377	-1.398
	0.997	0.456	-1.704

Fig. 1 Flexible pavement structure with material properties of different layers

An axisymmetric 2-D FE model of the thin asphalt pavement was developed by applying the general-purpose software ABAQUS. The 2-D FE model is more appropriate for simulating the

FWD testing in comparison to the 3-D FE model. It can better emulate a real circular load on the FWD-pavement interface, because the axisymmetric elements are capable of shaping an absolute circular load other than the elements in 3-D FE model. In this study, four-node bilinear axisymmetric solid elements were used in the finite domain; while axisymmetric infinite elements were applied to reduce a large number of far-field elements without significant loss of accuracy and then to build a “silent” boundary for the dynamic analysis.

Fig. 2 presents an axisymmetric 2-D FE model that discretizes the pavement structure. The FWD loading was represented by a circular impulse loading applied on the surface of asphalt layer. The FE mesh is refined around the circular loading area; instead a relatively coarse mesh is applied far away from the loading area. The length of elements within the loading area is selected at 12.7mm in the radial direction. The element thickness is selected to be 8.5mm for the asphalt layer. The selection for the element sizes is based on the mesh convergence analysis conducted in the early study (Wang and Al-Qadi 2011).

A sensitivity analysis was performed to define the infinite boundary in both radial and vertical directions, as shown in Fig. 3. After comparing the maximum central and edge deflections in the asphalt layer, the locations of the infinite boundary in the two directions from the loading center are needed to be greater than 2.2m and 7.5m in order to obtain a stable solution (less than 5% changes). The eventual selected domain size as an axisymmetric 2-D model (finite + infinite) has to be of 2.5×8m to achieve the balance between computation cost and accuracy.

Contact conditions at layer interfaces are important parameters that could significantly affect pavement responses under FWD loading. It is expected that the layers within the pavement structure remains in contact with no gap-opening since the contact area is very large and compressive loading due to gravity and traffic loading. Therefore, it is reasonable to assume that both relative and absolute motions of contacting surfaces at layer interfaces are small. In this study, the Coulomb friction model was used with a friction coefficient of 1.0 for the HMA-base layer interface and 0.3 for the base-subgrade interface (Romanoschi and Metcalf 2001).

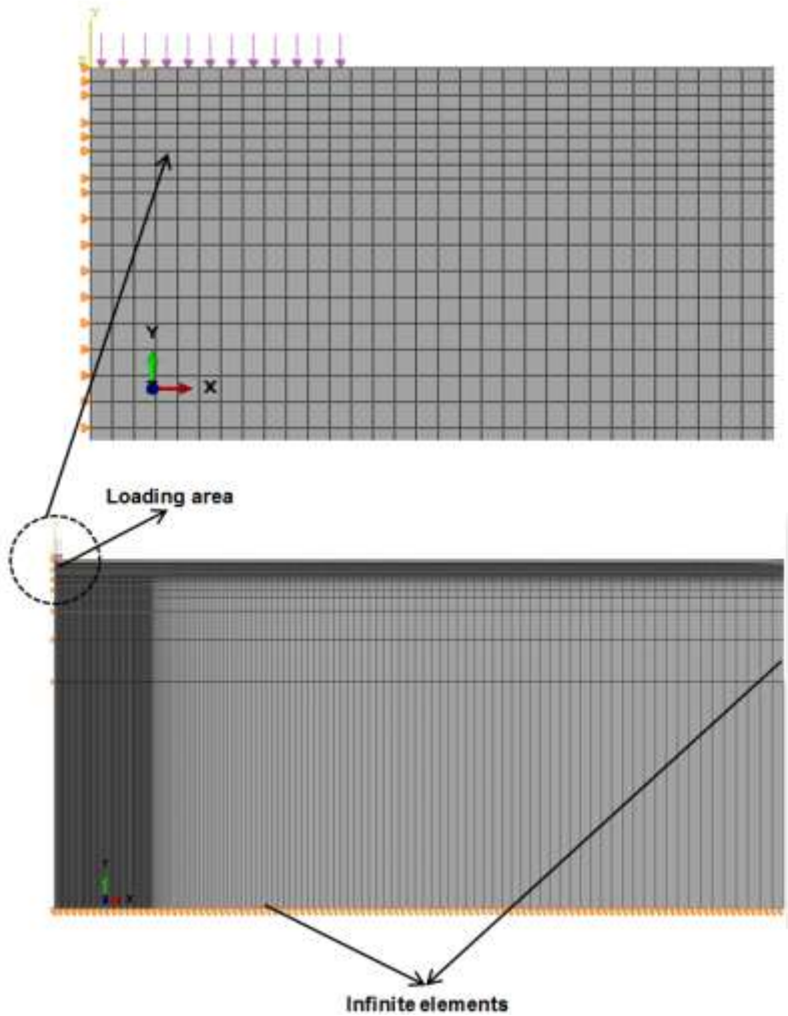
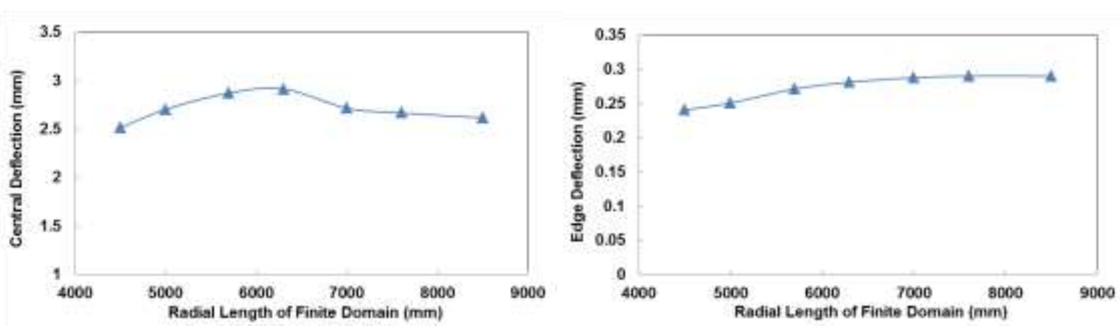
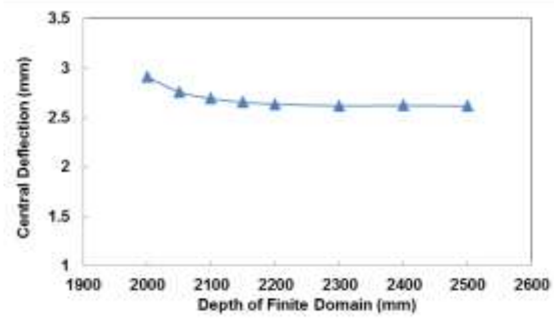
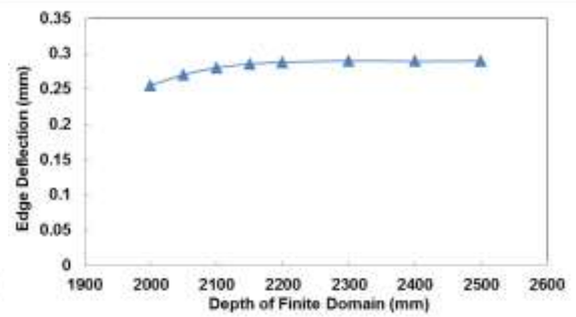


Fig. 2 FE model layout in axisymmetric 2-D domain



(a) (b)



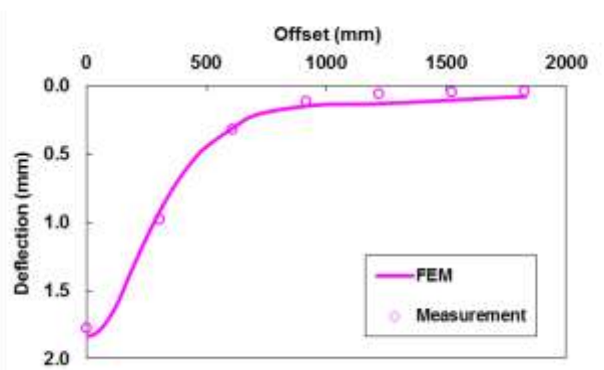


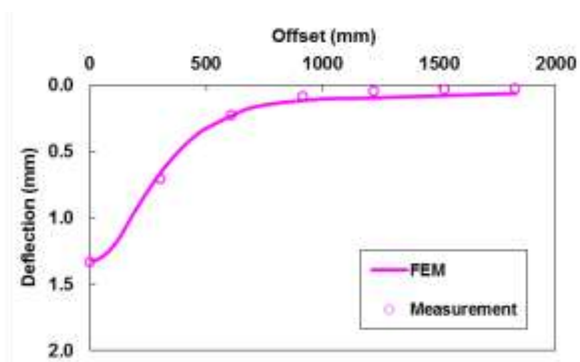
(c) (d)

Fig. 3 Sensitivity analysis for (a) central deflection vs. radial length of finite domain; (b) edge deflection vs. radial length of finite domain; (c) central deflection vs. depth of finite domain; and (d) edge deflection vs. depth of finite domain

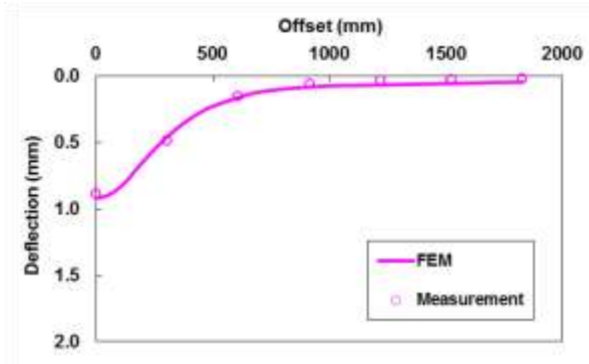
2.5 Model Validation

A validation study for the developed FE model was conducted with field measurement results under FWD loading. A 127-mm asphalt pavement with a 300-mm aggregate base layer that was used in the field study was modeled to simulate the in-situ condition. The material properties were obtained from laboratory testing and pavement temperature profiles were recorded in the asphalt layer (Kwon 2007; Wang and Al-Qadi 2011). Fig. 4 compares the measured and predicted surface deflections under three different loading levels. The results show that with the FE model results have good agreements with the field measurements. The Root Mean Squared Error (RMSE) between the measured and predicted surface deflections was calculated and the results were smaller than 0.1 mm for all deflections at different sensors.





(a) (b)



(c)

Fig. 4 Validation of FE model with measurement under FWD loading of (a) 53kN; (b) 40kN; and (c) 27kN

3. RESULTS AND ANALYSIS

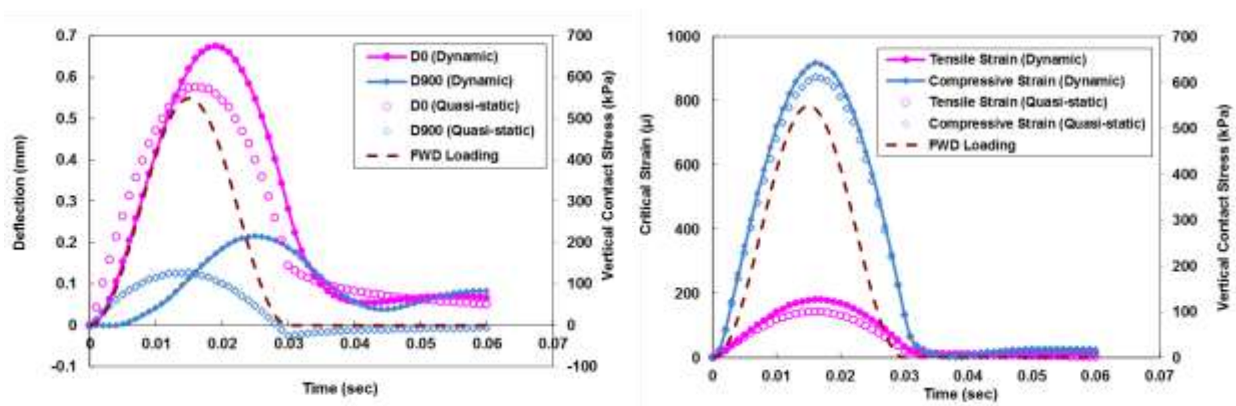
The effects of primary factors on pavement responses under FWD loading were investigated using the developed FE model. Both the surface deflections and critical strains were considered in the analysis. Two specific deflection indicators D0 (deflection under central load) and D900 (deflection 900mm away from central load) were primarily used in the analysis. The two typical deflections can represent structure behavior of the entire pavement system and the subgrade, respectively (Huang 1993). The critical strains analyzed included tensile strains at the bottom of HMA layer, compressive strains on the top of subgrade. It has been widely accepted that critical tensile strains in HMA layer are related to bottom-up fatigue cracking distress and compressive strains in subgrade are attributed to rutting failure. In addition, the load-deflection curve was used to indicate the energy dissipation of pavement system in terms of the hysteresis loop. The hysteresis loop was represented using the FWD loading-deflection (vertical displacement) rather than stress-strain loop, because the deflection can better represent the integrity of the pavement system than the critical strains (Ghuzlan and Carpenter 2000).

3.1 Effect of Dynamic Analysis

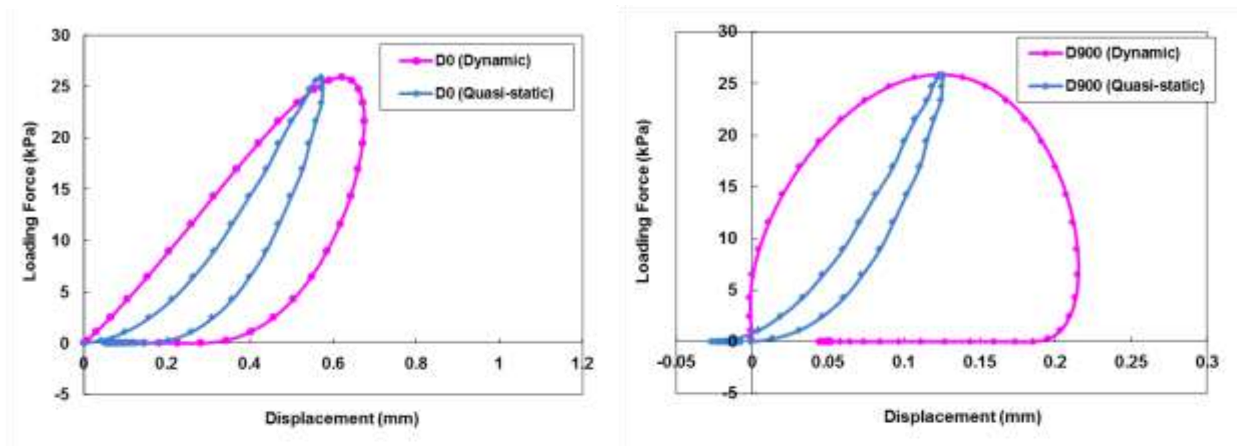
Fig. 5 shows the comparisons between dynamic analysis and quasi-static analysis through the deflection-time and strain-time histories as well as the hysteresis loops of D0 and D900. The load-time history is plotted to indicate the time shift between the applied load and resulting responses. It is believed that the time lag here is due to the viscoelastic and damping behaviors as well as the stress wave propagation from loading center. As expected, all the response-time history curves in Fig.5 (a) and (b) resemble the shape of the load-time history. It was found that the time lags to the load-time history were obvious in the D0- or D900-time histories under the dynamic analysis as compared to quasi-static analysis. Moreover, the magnitudes of D0 or D900 under dynamic analysis are larger than the ones under quasi-static analysis. It is worth mentioning that the D900 has a longer time lag than D0 in the displacement-time history. As mentioned before, the D0 represents the behavior of the entire pavement structure but D900 is uniquely attributed to the subgrade behavior. Therefore, the different time lags imply that the stress wave propagation plays a dominating role in addition to the viscoelastic behavior of asphalt layer and the damping effect of unbound layers.

On the other hand, as shown in Fig. 5 (b), no time lags were observed for critical strains under both analysis approaches. However, the critical strains under dynamic analysis are slightly greater than the ones under quasi-static analysis. However, the discrepancies of strains under two analysis methods were not as significant as the deflections. This indicates that the difference of both analysis approaches casts more effect on the back-calculation of the layer moduli through FWD deflections rather than on the assessment of pavement performance through critical responses.

Fig.5 (c) and (d) show the hysteresis loops in terms of D0 and D900 to indicate the energy dissipation resulted from the FWD loading. If the loading and unloading paths coincide, it means that all the strain energy caused by the load is recovered after unloading. If not, the area between the loading and unloading curves indicates the dissipated energy due to the viscoelasticity, damping, or material damage. Fig. 5 (c) shows that the dissipated energy in D0 using dynamic analysis is higher than the one using quasi-static analysis. It reveals that more energy is dissipated under the dynamic loading because of the damping behavior. The energy dissipation under quasi-static analysis can be attributed to the viscoelasticity of asphalt layer. In the case of Fig. 5 (d) shows that the discrepancy of energy dissipation using two analysis methods is more significant for D900. These findings emphasize that the influence of dynamic analysis cannot be ignored for FWD loading.



(a) (b)



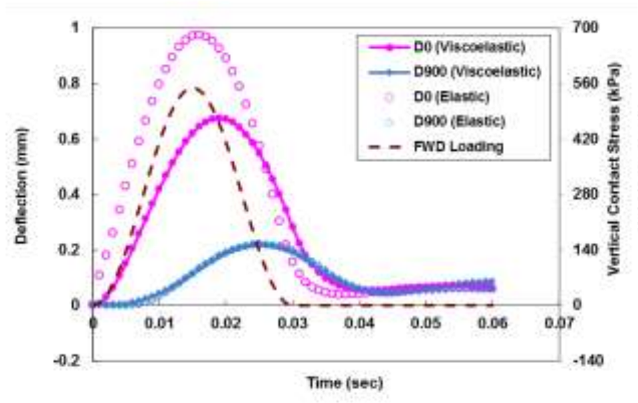
(c) (d)

Fig. 5 Comparison of dynamic and quasi-static analysis through (a) deflection-time histories; (b) strain-time histories; (c) D0 hysteresis loops; and (d) D900 hysteresis loops

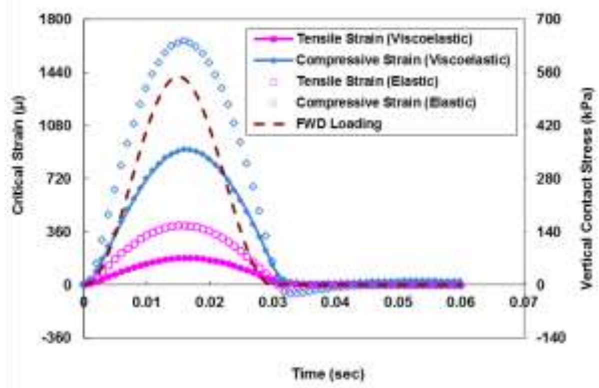
3.2 Effect of HMA Viscoelasticity

Fig. 6 shows the comparisons between the viscoelastic asphalt material and elastic asphalt material through the deflection- and strain-time histories as well as hysteresis loops of D0 and D900. The elastic modulus of asphalt layer was selected according to the dynamic modulus at the loading frequency of 30.3 Hz that is calculated from the pulse duration of FWD loading. In Fig. 6 (a), the longer time lag and greater magnitude of D0 were observed for viscoelastic material; while the time lag and magnitude of D900 were similar using both material models. As shown in Fig. 6 (b), no time lags were found for critical strains using both material models, although the magnitudes of critical strains in the elastic asphalt material are greater than the ones in the viscoelastic one. It is noted that the assumption of elastic asphalt layer may underestimate or overestimate pavement responses, depending on the selection of loading frequency; therefore, it cannot capture the frequency-dependent response of viscoelastic material.

Fig. 6 (c) and (d) show the hysteresis loops in terms of D0 and D900. It shows that the dissipated energy of D0 in the viscoelastic asphalt material is higher than the ones in the elastic asphalt material, while the discrepancy between the two material models in terms of D900 is negligible. This is reasonable since more energy is dissipated in the case of viscoelastic asphalt material due to the "dashpots" in the generalized Maxwell solid model (Eq. 1 and 2).



(a)



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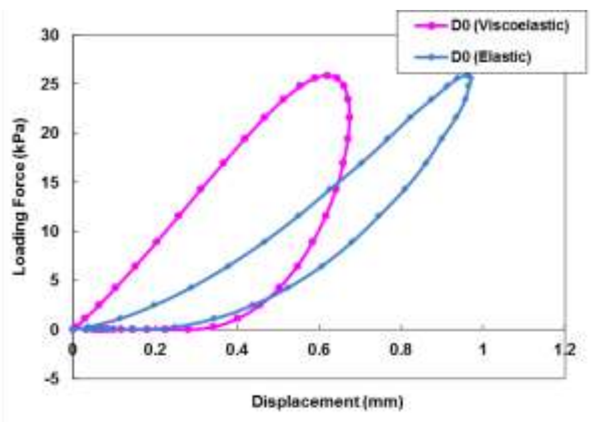
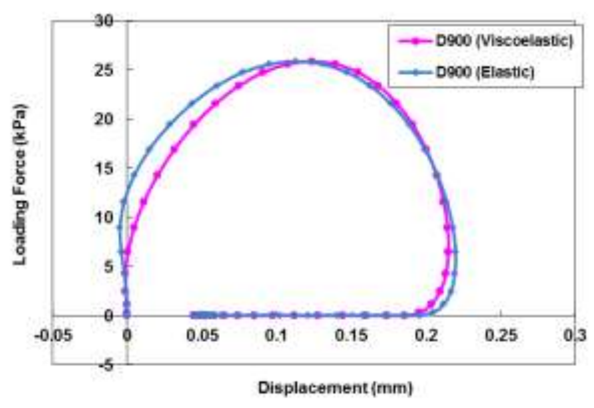


(c)



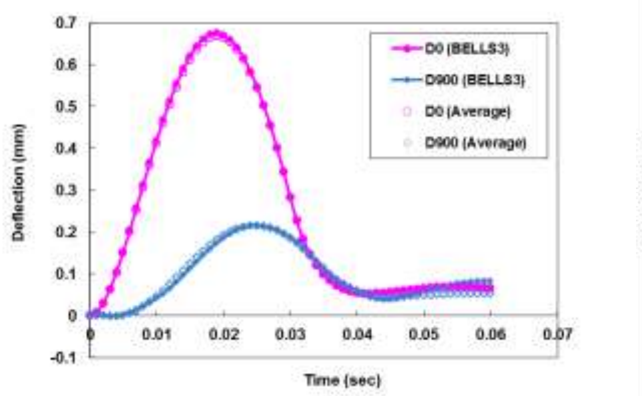
(d)

Fig. 6 Comparison of viscoelasticity and elasticity through (a) deflection-time histories; (b) strain-time histories; (c) D0 hysteresis loops; and (d) D900 hysteresis loops

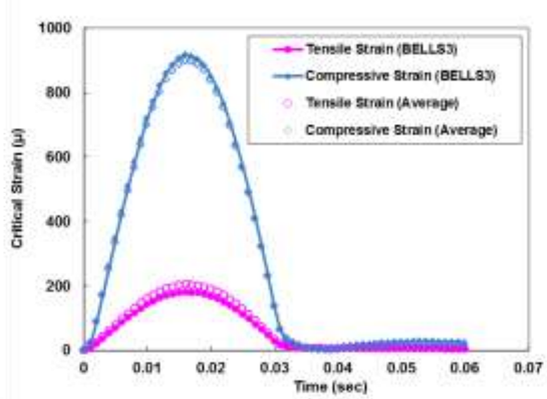
3.3 Effect of Temperature Gradient

The assumption of temperature profiiles in the asphalt layer can also affect viscoelastic modulus of asphalt layer and accodingly pavement resposnes. Two temperature conditions were considered to evalute the effect of in-depth nonlinear temperature gradient on pavement deflections and critical strains as compared to the constant temperature gradient using average temperature. The pavement surface temperature and average air temperature in the BELLS2 model (Eq. 5) were selected as 23.5°C (74.3°F) and 14.2°C (57.6°F) for the intermediate temperature case, and 37.5°C (99.5°F) and 25.2°C (77.4°F) for the high temperature case, respectively.

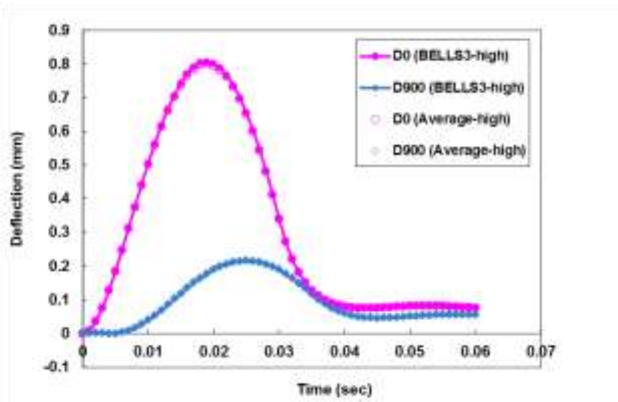
Fig. 7 illustrates the comparisons of the deflection-time and strain-time histories using two different temperature gradients and the corresponding average constant temperatures. In Fig. 7 (a) and 7 (c), the magnitudes of D0 or D900 under different temperature assumptions were found close to each other at both cases. Similarly, in Fig. 7 (b) and 7 (d), for the compressive strains on the top of subgrade, no significant discrepancies of the strain amplitudes were found with regard to the effect of temperature gradients. However, it was found that tensile strains at the bottom of asphalt layer under the constant average temperature gradient were slightly greater than the ones under the nonlinear temperature gradient. This is because the temperature at the bottom of asphalt layer is higher when the average temperature assumption is used. The effect of temperature gradient on tensile strains became more significant at the high temperature case. This indicates that the assumption of average constant temperature may be valid for calculating surface deflections but not for predicting bottom-up fatigue cracking potential of the relatively thin asphalt pavement.



(a)



(b)



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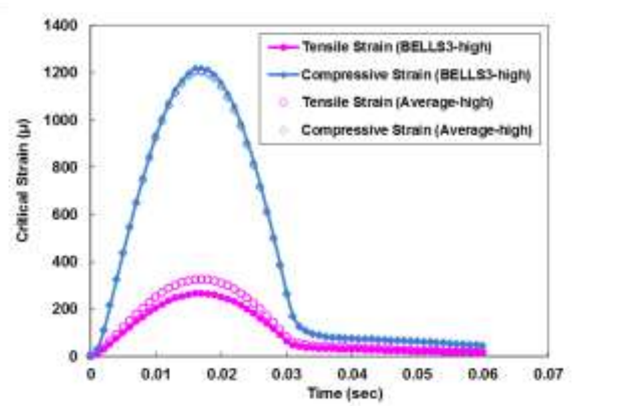


(d)

Fig. 7 Effect of temperature gradient on (a) deflection-time histories at the intermediate temperature; (b) strain-time histories at the intermediate temperature; and (c) deflection-time histories at the high temperature; and (d) strain-time histories at the high temperature

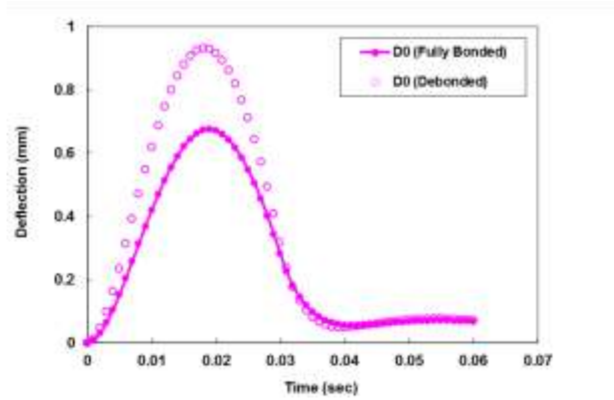
3.4 Effect of Asphalt Layer Delamination

Although the asphalt layer is designed to be full-bonded, layer delamination could happen in the field due to poor construction quality. Fig. 8 illustrates the effect of layer delamination within the asphalt layer on deflection- and strain-time histories as well as hysteresis loops. The debonded behavior was defined in the FE model by dividing the asphalt layer into two sub-layers (upper and lower) with frictional interaction at the mid-depth of 75 mm.

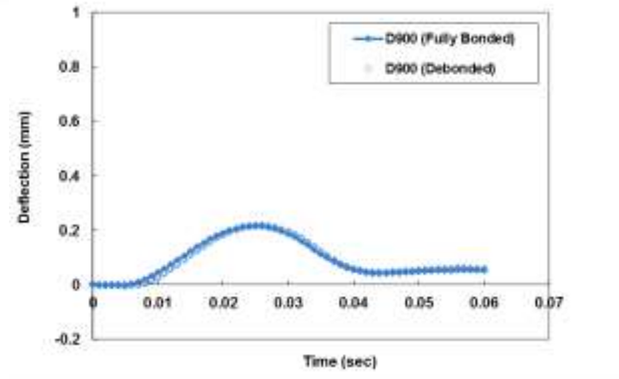
As expected, Fig. 8 (a) shows that deflection-time histories of D0 with fully-bonded asphalt layer are much smaller than the ones with debonded asphalt layer. However, Fig. 8 (b) indicates that no difference can be found in D900 between the two assumptions of bonding. It is reasonable because D0 represents the entire pavement behavior so that the layer debonding causes the larger magnitude of D0.

Fig. 8 (c) shows the tensile strain-time histories under two bonding assumptions of the asphalt layer. Four scenarios are focused on investigating the delamination effect in terms of HMA upper/lower bottoms along with debonded/fully bonded asphalt layers. The critical tensile strain at the bottom of upper layer with the deboned asphalt layer has the maximum magnitude among all scenarios. As expected, the strains at the bottom of upper layer with the fully-boned asphalt layer are around zero or have small compressions. This can be attributed to the reasons that the bottom of upper layer with the debonded asphalt layer bears the most flexural deformation; while the bottom of upper layer with the fully-bonded asphalt layer experiences approximately zero strain on the neutral axis of bending. The layer debonding also reduces the tensile strain at the bottom of lower layer as compared to the fully bonded case. Fig. 8 (d) presents that the compressive strains in the debonded case are much greater compared to the fully bonded case. Therefore, the delamination behavior can induce premature failure in fatigue cracking initiating at the bottom of asphalt upper layer or the permanent deformation in the subgrade. These observations signify that the asphalt layer bonding condition should be considered in FE modeling.

Figs. 8 (e) and (f) show the hysteresis loops in terms of D0 and D900 in consideration of the asphalt layer delamination. It shows that the energy dissipation of D0 in the full-bonded case is different from the one in the debonded case. However, the energy dissipation of D900 in both cases keeps approximately similar to each other. In general, the results indicate that the layer debonding could be detected through both the magnitude and energy dissipation of D0 if in-situ testing data are available.



(a)



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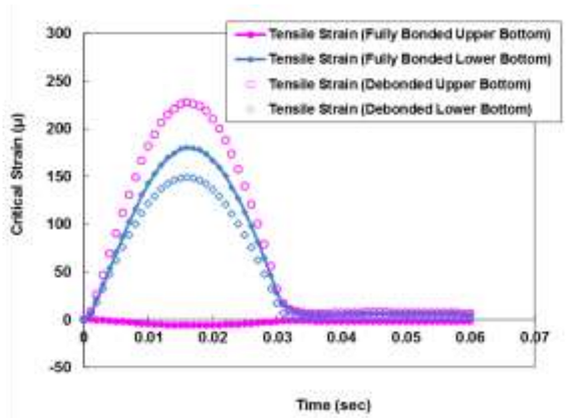
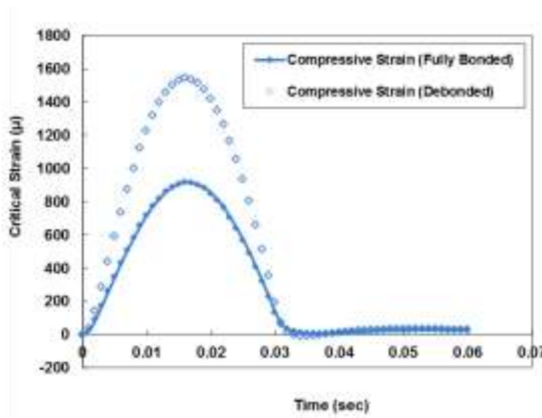
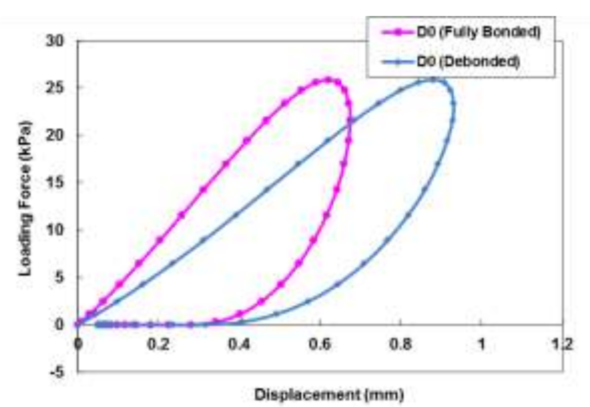


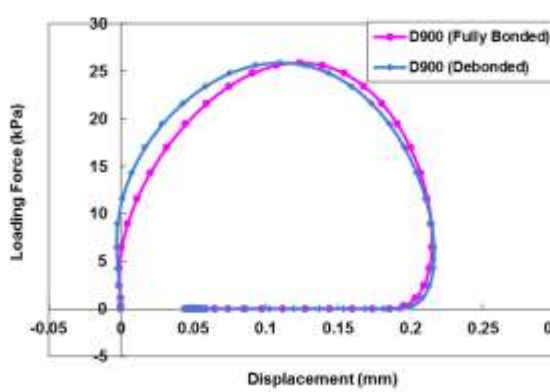
(c)



(d)



(e)



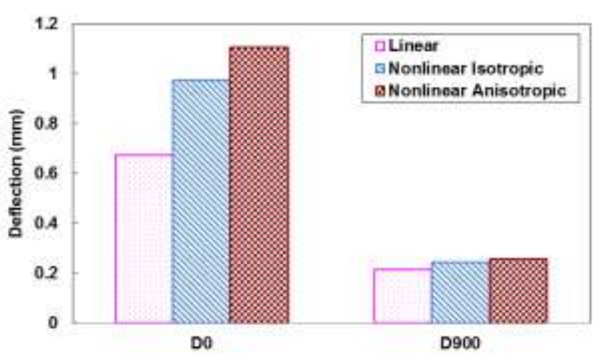
(f)

Fig. 8 Effects of layer delamination on (a) D0-time history; (b) D900-time history; (c) tensile strain-time history; (d) compressive strain-time history; (e) hysteresis loop of D0; and (f) hysteresis loop of D900

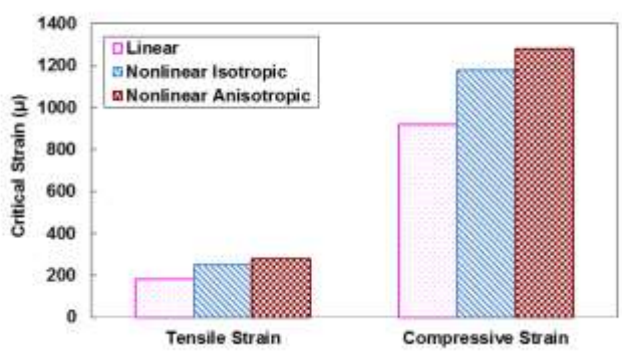
3.5 Effect of Unbound Material Nonlinearity

Fig. 9 and Fig. 10 show the comparison of different unbound material models through the deflections and critical strains as well as hysteresis loops of D0 and D900. In the reference case, the linear elastic moduli for both base aggregate layer and subgrade were calculated as the average moduli from the modulus distribution throughout the base layer and subgrade, respectively. As shown in Fig. 9 (a) and (b), the linear and nonlinear isotropic models for base aggregate underestimated the deflections and critical strains in comparison to the nonlinear cross-anisotropic model. This trend is similar for the effect of nonlinear subgrade model, as shown in Fig. 10 (a) and (b). Therefore, it is concluded that ignoring the nonlinearity of the unbound materials can lead to adverse effects on the modulus back-calculation and pavement performance assessment.

Fig. 9 and Fig. 10 (c) and (d) present the hysteresis loops using different unbound material models in terms of D0 and D900. The dissipated energy of D0 and D900 in the linear model is the lowest in general. It was found that the dissipated energy of D0 was mainly affected by the nonlinear model of aggregate base; while the dissipated energy of D900 was mainly affected by the nonlinear model of subgrade. Therefore, the linearity assumption for both unbound materials might result in obvious error if energy dissipation is used as an analysis indicator.



(a)



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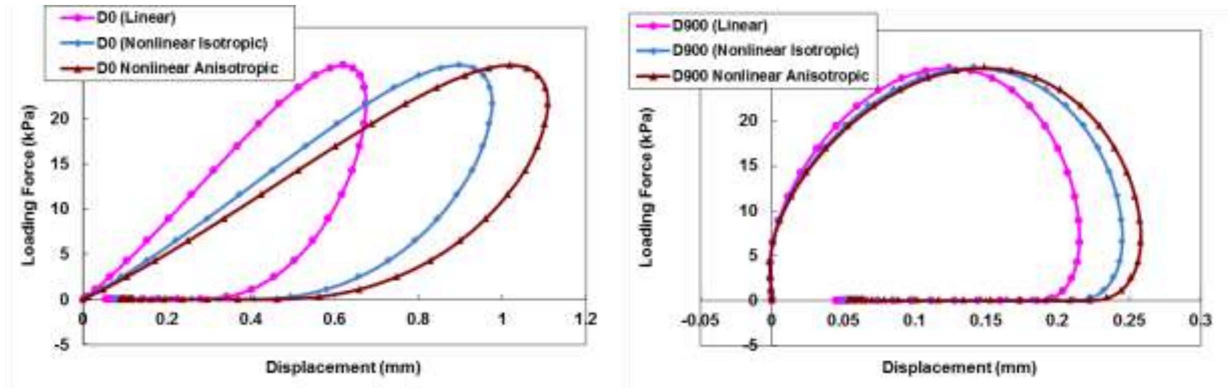
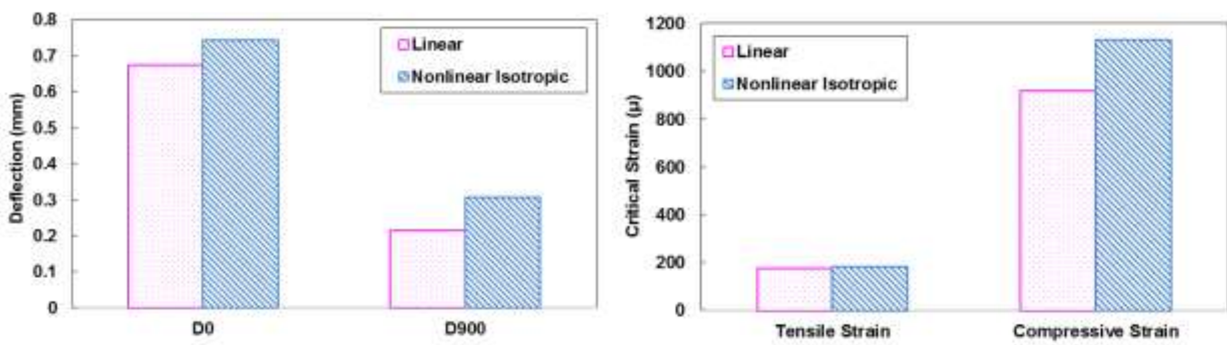
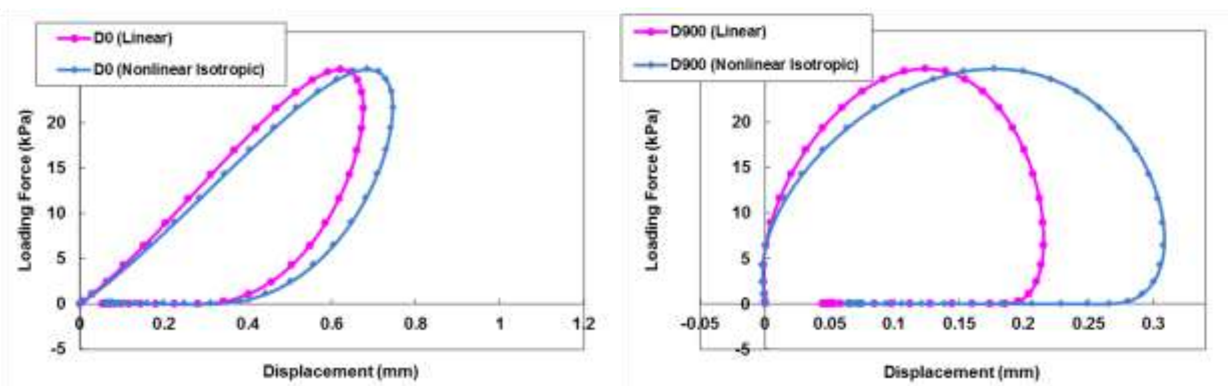


(c) (d)

Fig. 9 Effect of nonlinear model of aggregate base on (a) D0 and D900; (b) critical strains; (c) hysteresis loop of D0; and (d) hysteresis loop of D900



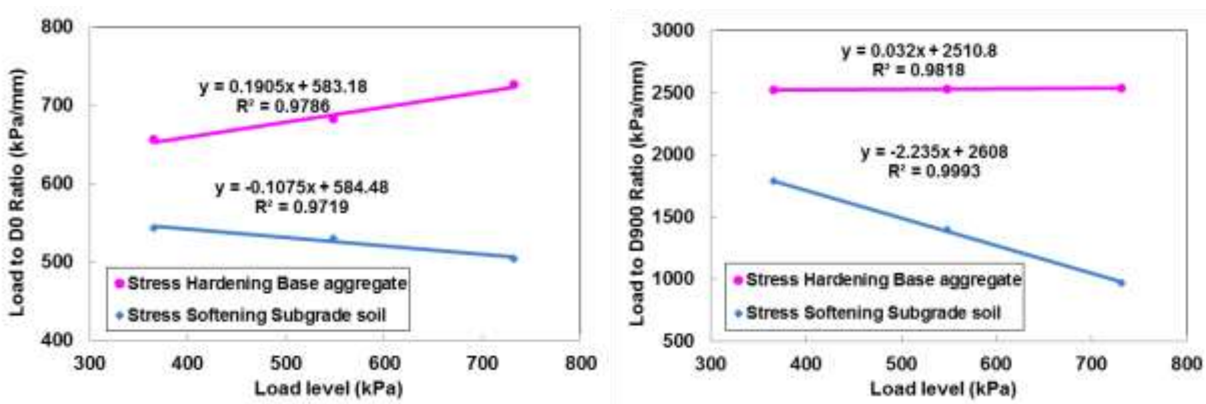
(a) (b)



(c) (d)

Fig. 10 Effect of nonlinear model of subgrade on (a) D0 and D900; (b) critical strains; (c) hysteresis loop of D0; and (d) hysteresis loop of D900

In order to investigate the stress-dependent behavior of unbound material, multiple loading levels were considered in the FE analysis. The stiffness sensitivity analysis to the loading levels was conducted using the ratio of loading to deflection (D0 and D900), as shown in Fig. 11. This ratio would be equal to one if the linear material properties are used in the FE model. The positive slope indicates that the nonlinear stress hardening behavior in dominant, while the negative slope indicates that the stress softening behavior of subgrade is dominant. However, the negative slope explains the stress softening behavior in the subgrade. The results indicate that D0 is equally affected by the nonlinearity in base layer and subgrade. However, D900 is mainly governed by the nonlinearity in subgrade that shows significant stress-softening behavior.



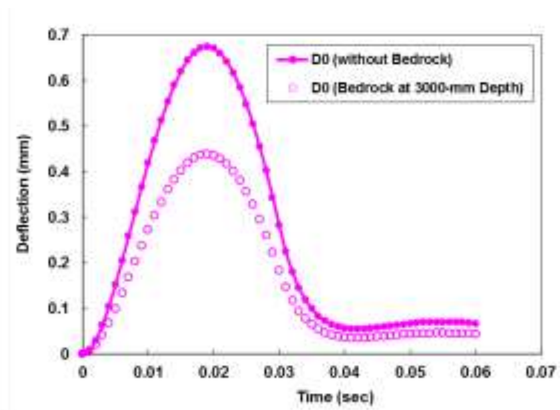
(a) (b)

Fig. 11 FWD deflection variations under multiple loadings considering nonlinear unbound material for (a) D0 and (b) D900

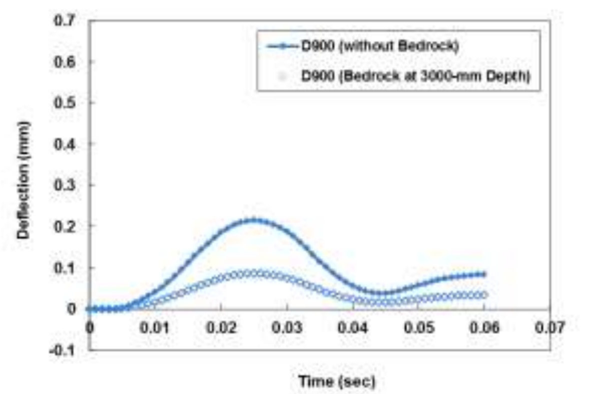
3.6 Effect of Bedrock Depth

Fig. 12 shows effect of stiff layer (bedrock) underlying subgrade on surface deflections through the deflection-time histories and the sensitivity of D0 and D900 to the depth to bedrock. The selected stiffness of bedrock is based on typical rock properties with an elastic modulus of 7000 MPa and a Poisson's ratio of 0.2. As shown in Fig. 12 (a) and (b), deflection-time histories of D0 and D900 sharply decreased if the bedrock exists at the depth of 3000 mm from pavement surface. It is noted that the variations of strain-time histories for tensile and compressive strains are insignificant (difference smaller than 3%); therefore they were not plotted here due to the reason of brevity. It indicates that the existence of bedrock casts more effect on the back-calculation of layer moduli through FWD deflections than on the assessment of pavement performance through critical strains.

Fig. 12 (c) and (d) shows the sensitivity of D0 and D900 to the depth to bedrock. It shows that the D0 is increased gradually as the location bedrock becomes deeper; and the variation of the D0 becomes stable (variation smaller than 5%) as the depth goes beyond 8000 mm from pavement surface. However, the stability for the D900 variation was not obtained until the depth reached to 10000 mm. Given that the bedrock is closer to and then has more effect on the subgrade behavior, this helps explain that the D900 is more sensitive than the D0 to the change in the depth to bedrock. The threshold range of 8000 mm and 10000 mm obtained from the sensitivity analysis were consistent with previous findings using field measured deflections (Broutin 2010).



(a)



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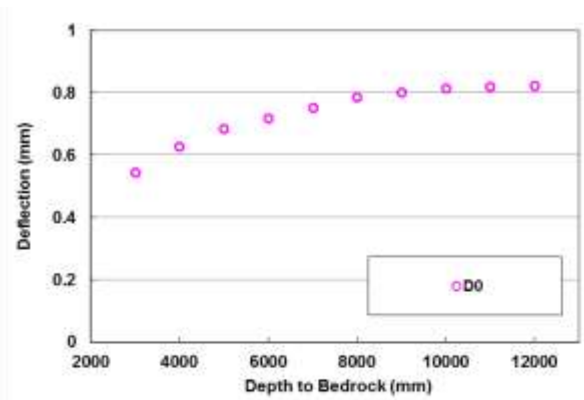
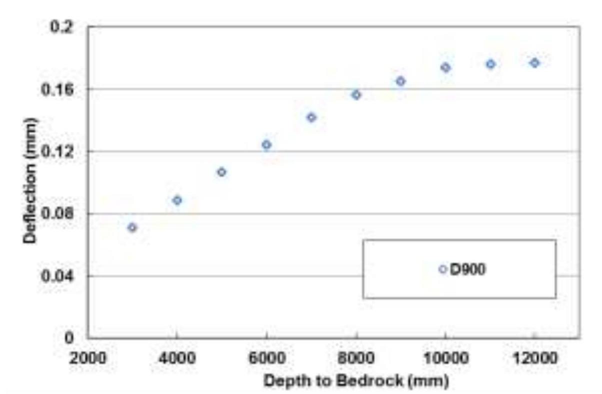


(c)



(d)

Fig. 12 Effects of bedrock on FWD deflections for (a) D0-time history; (b) D900-time history; (c) sensitivity of D0 to the depth to bedrock; and (d) sensitivity of D900 to the depth to bedrock

4. CONCLUSIONS

In this study, FE models were developed to simulate flexible pavement behavior under impulsive FWD loading. Parametric analysis was conducted to analyze the primary factors affecting model results, including dynamic analysis, temperature gradient, bedrock depth, asphalt layer delamination, viscoelasticity of asphalt layer, and nonlinearity of unbound materials. Although the parametric analysis findings vary depending on the response of interest, the study results illustrate the appropriate selection of analysis type, constitutive models of pavement material, and layer boundary conditions on the accuracy of FE modeling. In particular, the analysis findings show the delamination in asphalt layer induces the greater strain responses; while neglecting bedrock effect overestimates surface deflections. These two factors were usually neglected in the previous analysis work.

The developed FE model successfully captures the distinctive constitutive model for each pavement layer and the interaction between different layers and boundary conditions. With the proper development of FE models, further research will be conducted for backcalculation of layer moduli using the combination of synthetic database from the forward FE analysis and artificial intelligence techniques.

5. REFERENCES

ABAQUS. *ABAQUS/Standard User's Manual*, Version 6.10, Hibbitt, Karlsson & Sorenson, Inc., Pawtucket, RI, 2010.

Al-Qadi, I.L., H. Wang, and E. Tutumluer. Dynamic Analysis of Thin Asphalt Pavements by Using Cross-Anisotropic Stress-Dependent Properties for Granular Layer. *Transportation Research Record, No. 2154*, 2010, pp. 156-163.

ARA, Inc. ERES Division. *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures*, NCHRP 1-37A Final Report, TRB, Washington, DC, 2004.

Bathe, K.J. *Finite Element Procedures in Engineering Analysis*, Prentice-Hall, NJ, 1982.

Broutin, M. *Assessment of Flexible Airfield Pavements using Heavy Weight Deflectometers*. Ph.D. Dissertation, LCPC Paris, 2010

FHWA. *Temperature Predictions and Adjustment Factors for Asphalt Pavement*, FHWA-RD-98-085, Federal Highway Administration (FHWA), McLean, VA, 2000.

FHWA. LTPP InfoPave. https://infopave.fhwa.dot.gov/, 2016.

Gedafa, D.S., M.Hossain, and S.A. Romanoschi. Prediction of Asphalt Pavement Temperature. *Proceedings Airfield and Highway Pavement*, Los Angeles, CA, 2013, pp. 373-382.

George, K.P. *Prediction of Resilient Modulus from Soil Index Properties*, Final Report to Federal Highway Administration, Mississippi Department of Transportation Research Division, Jackson MS, 2004.

Ghuzlan, K.A. and S.H. Carpenter. Energy-Derived, Damage-Based Failure Criterion for Fatigue Testing. *Transportation Research Record: Journal of the Transportation Research Board, No. 1723*, TRB, National Research Council, Washington, DC, 2000, pp. 141-149.

Hakim, B.A., L.W. Cheung, and R.J. Armitage. Use of FWD Data for Prediction of Bonding between Pavement Layers. *International Journal of Pavement Engineering*, Vol. 1(1), 2000, pp.49-59.

Huber, G.A. *Strategic Highway Research Program Report SHRP-A-648A: Weather Database for the Superpave® Mix Design System*, Transportation Research Board, Washington, DC, 1994.

Huang, Y.H. *Pavement Analysis and Design*, 1st ed., Prentice Hall, Upper Saddle River, NJ, 1993.

Kim, M., E. Tutumluer, and J. Kwon. Nonlinear Pavement Foundation Modeling for Three-Dimensional Finite-Element Analysis of Flexible Pavements. *International Journal of Geomechanics*, Vol. 9, No. 5, ASCE, 2009, pp. 195-208.

Kwon, J. *Development of a Mechanistic Model for Geogrid Reinforced Flexible Pavements*. Ph.D. Dissertation, University of Illinois at Urbana-Champaign, 2007.

Lu, Y, P.J. Wright, and Y. Zhou. Effect of Temperature and Temperature Gradient on Asphalt Pavement Response. *Road & Transport Research: A Journal of Australian and New Zealand Research and Practice*, Vol. 18, No. 1, 2009, pp.19-30.

Park, S. W. and Y. R. Kim. Interconversion between Relaxation Modulus and Creep Compliance for Viscoelastic Solids. *Journal of Materials in Civil Engineering*, Vol. 11, No. 1, 1999, pp. 76–82.

Park, D.Y, N. Buch, and K. Chatti. Effective Layer Temperature Prediction Model and Temperature Correction via Falling-Weight Deflectometer Deflections. *Transportation Research Record*, No. 1764, TRB, National Research Council, Washington, DC, 2001. pp. 97-111.

Romanoschi, S.A. and J.B., Metcalf. Effects of Interface Condition and Horizontal Wheel Loads on the Life of Flexible Pavement Structures. *Transportation Research Record*, No. 1778, TRB, National Research Council, Washington D.C., 2001, pp. 123-131.

Salem, H. M., F. M. Bayomy, M. G. Al-Taher, and I. H. Genc. Using Long-Term Pavement Performance Data to Predict Seasonal Variation in Asphalt Concrete Modulus. *Transportation Research Record, No. 1896*, 2004, pp. 119-128.

Shoukry, S.N., D.R. Martinelli, and O.I. Selezneva. Dynamic Performance of Composite Pavements under Impact. *Transportation Research Record, No. 1570*, 1997, pp. 163-171.

Schwartz, C. W. Effect of Stress-Dependent Base Layer on the Superposition of Flexible Pavement Solutions. *International Journal of Geomechanics*, Vol. 2, No. 3, 2002, pp. 331–352.

Tutumluer, E. and M.R. Thompson. Anisotropic Modeling of Granular Bases In Flexible Pavement. *Transportation Research Record*, No. 1577, 1997, pp. 18-26.

Wang, H. and I.L. Al-Qadi. The Importance of Nonlinear Anisotropic Modeling of Granular Base for Predicting Maximum Viscoelastic Pavement Responses under Moving Vehicular Loading. *Journal of Engineering Mechanics*, ASCE, Vol. 139, No. 1, 2013, pp. 29-38

Wang, H. and I.L. Al-Qadi. Impact Quantification of Wide-base Tire Loading on Secondary Road Flexible Pavements. *Journal of Transportation Engineering*, Vol. 137. No.9, ASCE, 2011, pp. 630-639.

Xiao, Y.J. and E. Tutumluer. *Best Value Granular Material for Road Foundations*. Final report to Minnesota DOT, Minnesota DOT, Saint Paul, 2012.

Xiao, Y.J., E. Tutumluer, and J. Siekmeier, Mechanistic-Empirical Evaluation of Aggregate Base and Granular Subbase Quality Affecting Flexible Pavement Performance in Minnesota, *Transportation Research Record*, No. 2227. pp. 97-106

Xu, Q. and J.A. Prozzi. Static versus Viscoelastic Wave Propagation Approach for Simulating Loading Effects on Flexible Pavement Structure. *Construction and Building Materials*, Vol. 53, 2013, pp. 584-595.

Xu, Q. and J.A. Prozzi. A Time-Domain Finite Element Method for Dynamic Viscoelastic Solution of Layered-Half-Space Responses under Loading Pulses, *Computers & Structures*, No. 160, 2015, pp. 20-39

Zhong, X.Z., X. Zeng, and J.G. Rose. Shear Modulus and Damping Ratio of Rubber-Modified Asphalt Mixes and Unsaturated Subgrade Soils. *Journal of Materials in Civil Engineering*, Vol. 14, No. 6, ASCE, 2002, pp. 496-502.

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