JOURNAL OF TRAFFIC AND TRANSPORTATION ENGINEERING (ENGLISH EDITION) 2016; 3 (5): 427–437

Available online at www.sciencedirect.com

ScienceDirect

journal homepage: www.elsevier.com/locate/jtte

Original Research Paper

Dynamic analysis of falling weight deflectometer

Ainalem Nega [a,*], *Hamid Nikraz* [a], *Imad L. Al-Qadi* [b]

[a] *Department of Civil Engineering, Curtin University, Perth, WA 6845, Australia*
[b] *Illinois Center for Transportation, University of Illinois at Urbana-Champaign, Urbana, IL 61801, USA*

ARTICLE INFO

Article history:
Available online 3 October 2016

Keywords:
Flexible pavement
Layer modulus
Falling weight deflectometer
Deflection basin
Dynamic analysis
Analytical model

ABSTRACT

Falling weight deflectometer (FWD) testing has been used to evaluate structural condition of pavements to predict the layer moduli using backcalculation process. However, the predicted pavement layer moduli sometimes may not be accurate even if computed and measured deflection basin has fulfilled the standard and is in concurrence with certain tolerable limits. The characteristics of pavement structure, including pavement layer thickness condition and temperature variation, affect the predicted pavement structural capacity and back calculated layer modulus. The main objective of this study is to analyze the FWD test results of flexible pavement in Western Australia to predict the pavement structural capacity. Collected data includes, in addition to FWD measurements, core data and pavement distress surveys. Results showed that the dynamic analysis of falling weight deflectometer test and prediction for the strength of character of flexible pavement layer moduli have been achieved, and algorithms for interpretation of the deflection basin have been improved. The variations of moduli of all layers along the length of sections for majority of the projects are accurate and consistent with measured and computed prediction. However, some of the projects had some inconsistent with modulus values along the length of the sections. Results are reasonable but consideration should be taken to fix varied pavement layers moduli sections.

1. Introduction

FWD testing has been an integral part of flexible pavement's condition assessment for a decade. Various approaches and procedures for FWD deflection analysis have been developed in several studies (Xu et al., 2002a, b). Most of these procedures and guidances for FWD deflection do not take account either the dynamic loading effect or nonlinear material behavior. Although few procedures do take account for these effects, their implementations are very challenging because of their complexity and the large number of variable (Xu et al., 2002a, b).

Sebaaly et al. (1986) evaluated the dynamic analyses data from falling weight deflectometer by using a multi-degree of freedom elastodynamic analysis, which was based on a Fourier solution synthesis for periodic loading elastic or viscoelastic moduli layered strata. The results indicated that inertial effects were important in the pavement response

* *Corresponding author*. Tel.: +61 431548494; fax: +61 8 9266 2681.
E-mail address: ainalem.nega@curtin.edu.au (A. Nega).

Peer review under responsibility of Periodical Offices of Chang'an University.
http://dx.doi.org/10.1016/j.jtte.2016.09.010

prediction. Xu et al. (2002a, b) developed a mechanistic relationship between FWD deflection and asphalt pavement layers condition. From the results, deflection basin parameters (DBPs), effective layer moduli, and stresses and strain were identified as pavement layers condition indicators. Xu et al. (2002a, b) also presented a new condition assessment criterion for flexible pavement layers using FWD from field data. Nondestructive condition assessment criteria were developed for application in conjunction with the condition evaluation indicators that were estimated using on falling weigh deflectometer deflection.

Kim and Park (2002) developed a mechanistic empirical method for assessing pavement layer condition as well as estimated the remaining life of flexible pavement using multi-load level FWD deflection. Synthetic deflection database was generated using FWD and a stress-dependent soil model. Results showed that the base and subgrade pavement layer moduli condition can be estimated using multi-load level FWD deflection. AC layer moduli were found to be better indicators than deflection basin parameters. Appea and Al-Qadi (2000) also have assessed FWD data for stabilized flexible pavements. The performance and structural condition of nine flexible pavement test sections that were built in Bedford and Virginia, have been monitored for 5 years using FWD. The flexible pavements had three groups with aggregate base layer moduli thickness of 100, 150, and 200 mm. The deflection basins obtained from the flexible pavement testing were analyzed using the ELMOD back calculated program in order to find the pavement structural capacity and to defect changes in the aggregates resilient modulus. The analysis showed a 33% reduction in the back calculated resilient modulus for the 100 mm thick base layer over 5 years for non-stabilized as compared to the geosynthetically stabilized section.

A study was conducted to develop methods for using FWD measurements to determine moduli of onsite pavement material sands and compare FWD-estimated moduli with laboratory-measured values in order to achieve consistent input to thickness design procedures (Frazier, 1991). A three-layer pavement model was used to characterize flexible pavement and simple procedures were developed to account for seasonal variations and effective moduli values for granular base-subbase and subgrade soils from limited FWD measurement (Frazier, 1991). There were large differences between FWD moduli and laboratory moduli from triaxial testing (AASHTO T274). However, good agreement was demonstrated between FWD and laboratory values (AASHTO T274) moduli for subgrade soils. This was also seen when characterization of granular base-subbase was difficult.

FWD tests have been used in the evaluation of material properties of pavement system for decades. Load amplitudes and frequency content intend to provide pavement deformation levels similar to those induced by truck wheal loads in heavy urban traffic loading. Interpretation of the in situ measured data is normally based on elastic solutions and does not take into account the possible existence of localized nonlinearities. Chang et al. (1992) investigated the nonlinear effects in FWD using both a linear and nonlinear solution with the generalized cap model to reproduce the nonlinear soil behavior. The material nonlinearities were found to be important for FWD tests on flexible pavement where the subgrade is relatively soft and the pavement is thin. FWD tests are commonly considered to provide estimates of material properties for levels of loading, similar to those exerted by truck model as discussed by Uddin et al. (1985a, b).

The main objective of this study is to analyze the FWD test results for the strength of flexible pavement layer moduli character in Western Australia so that allowable loads for existing pavement structures can be determined. In addition, demonstration for a proper interpretation of FWD tests deflection data for the flexible pavement sections that have been experienced multiple milling operations and overlays. Design of typical FWD configuration, location of loading plate, geophones and measured deflection basin are shown in Fig. 1. D_j ($j = 0, 1, \cdots, 8$) is the measured deflection at pavement surface.

1.1. *Analytical model and approaches*

FWD testing has been extensively practiced in the past to assess structural condition and determine the model of flexible pavement layer. The set of modulus value for pavement layers obtained from the backcalculation may not be accurate even though the computed and measured deflection basin may match within tolerable limits (Mehta and Roque, 2003). Extensive data interpretation is involved in obtaining the layer moduli of these pavements. For example, guidelines and tools are provided for calculating layer moduli of flexible pavement. However, FWD interpretation has become challenging because more roads have experienced several milling operations and overlays. Flexible pavement structure characteristics (damage layers, variation in pavement thickness, and change of pavement temperature) can overwhelm the deflection data to show a more significant effect than those induced by structural layer moduli stiffness as summarized by Mehta and Roque (2003).

1.2. *Backcalculation of flexible pavement*

Several computer programs such as ADAM, BISDEF, BOUSDEF, CHEVDEF, COMDEF, DBCONPAS, ELMOD, ELSDEF, EVERCALC,

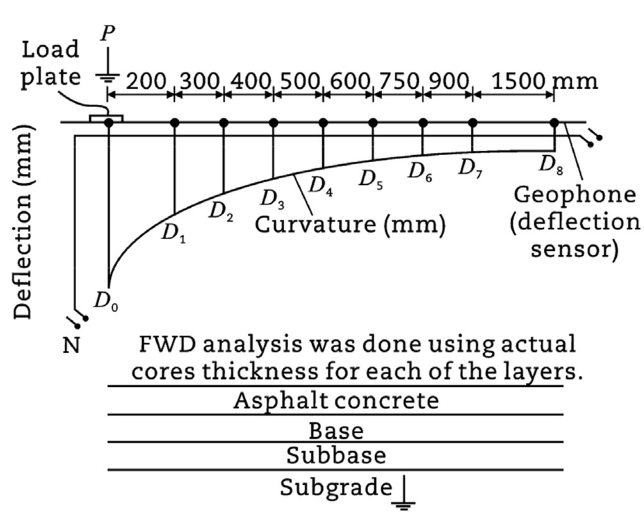


Fig. 1 – Design of typical FWD configuration, location of loading plate, geophones and measured deflection basin.

ILLI-BACK, ISSEM4, MODCOMP2, MODULUS, OAF, PADAL, and WESDEF, have been developed for backcalculation analyses (Kang, 1998). Each of this computer programs for backcalculation employs a particular forward model and a specific task of backcalculation scheme. Kang (1998) summarized that an error in backcalculation may occur due to several factors such as nonuniform pressure, distribution, temperature and moisture gradients, improper loading position to the edge of the pavement, variation of material properties and nonlinearity, selection of an improper forward model, and deflection-matching algorithms.

There were several sources of errors in the back calculated moduli besides the nonlinearity of the stress–strain relation of the material in pavement layers (Lytton, 1989). These errors, which were introduced by deflection calculation model and its presumed constitutive relations, were systematic and cannot be eliminated or reduced by repeated measurement or calculations (Lytton, 1989; Ullidtz and Coetzee, 1995). Lytton (1989) discussed those only random errors in computing layer moduli can be reduced or eliminated.

1.3. *Falling weight deflectometer*

A FWD is a device that applies on impulsive load to a pavement surface and the deflection response is recorded at a series of radial point. The level of impact load, loading duration, and area are adjusted in such a way that it corresponds to the actual loading by a standard truck moved on a in-service load as defined by Sharma and Das (2008). The problem of pavement layer moduli backcalculation of flexible pavement from FWD deflection data is truly complex and efforts are made to involve a generalized approach to impact analysis in order to accurately and efficiently back calculate the in situ layer moduli (May and Von Quintas, 1994; Sharma and Das, 2008).

Among all nondestructive testing (NDT) methods, the FWD method is the most wide or popular technique (Goktepe et al., 2006). FWD can successfully simulate traffic loads, and it can also produce a huge amount of deflection data in a short period of time as reviewed by Bianchini and Bandini (2010), Hoffman and Thompson (1982), Saltan et al. (2002). FWD can measure the time-domain deflection on numerous road sections, and has been used to back calculate mechanical pavement properties using specific software involving forward and backcalculation directions (Goktepe et al., 2006). Despite the fact that interpretation of deflection data are remain somewhat problematic. According to Sebaaly et al. (1985), the dynamic analysis of falling weight deflectometer comprised in two distinct parts: determination of FWD's dynamic motion and pavement response's evaluation.

2. Methods and materials

2.1. *Methods*

Various computer programs are available to perform backcalculation analysis. In this study, the BISDEF computer program is used for backcalculation analysis. Bush III and Alexander (1985) developed the BISDEF computer program to handle multiple loads and consider different interface layer condition. Burmister (1944) investigated the load (means a method to determine stresses, strains and displacement) in order to develop a flexible pavement layered moduli theory and found an exact solution for the boundary stresses in the center of a circular, which was uniformly distributed load acting on the surface of a three-layer and half-space (Appea, 2003; Appea and Al-Qadi, 2000). To obtain a Burmister (1944) type of solution, it is necessary to perform an integration using digital computers (Appea, 2003).

$$D = F\left[\int_0^{\infty} f\left(e^{2mh}, e^{-2mh}, h\right) J_0(mr) J_1(mr) \mathrm{d}m\right] \tag{1}$$

where D is the deflection, F is Bessel function of $J_0(mr)$, f is Bessel function of $J_1(mr)$, h is layer thickness, r is radial distance from the load axis.

The nonlinear least squares optimization method was then used to minimize the sum of the squared relative difference to solve the following problem (Sivaneswaran et al., 1991).

$$f(E,h) = \frac{1}{n}\sum_{i=1}^{n}\left[\frac{d_i^c(E,h) - d_i^m}{d_i^m}\right] \tag{2}$$

where c and m are parameters (refer to calculated and measured deflection).

The error location i is defined as follow

$$r_i(E,h) = \frac{d_i^c(E,h) - d_i^m}{d_i^m} \tag{3}$$

After multiplying by the constant n for any convenience and then, Eq. (3) can be expressed as follow

$$f(E,h) = \sum_{i=1}^{n}[r_i(E,h)]^2 = r^{\mathrm{T}}r \tag{4}$$

where $r = \{r_1, r_2, \cdots, r_n\}$ is the relative error, T is the transpose function.

The gradient of the criterion function is $\nabla f = 2Ar$, $A = \{\nabla r_1, \nabla r_2, \cdots, \nabla r_n\}$. The Hessian integral estimates equation can be written as follow

$$H = \nabla^2 f = 2AA^{\mathrm{T}} + 2\sum_{i=1}^{n} r_i \nabla^2 r_i \tag{5}$$

The gradient and Hessian are the respective multidimensional equivalents of the slope and curvature of a one-dimensional function. In this formulation, the first part of the Hessian is known as soon as the gradient ∇f has been evaluated. Since $r^{\mathrm{T}}r$ is minimized, the relative errors are often errors. A good approximation to the Hessian may be made by neglecting the second part (Appea, 2003).

$$H = 2AA^{\mathrm{T}} \tag{6}$$

All flexible pavement materials are assumed to be homogenous, isotropic, and linear–elastic except for the subgrade (Al-Qadi et al., 1994, 1997; Appea, 2003). The subgrade is assumed to be exhibit nonlinear response, and is defined as follow

$$E_0 = C_0\left(\frac{\sigma_1}{\sigma}\right)^n \tag{7}$$

where E_0 is the surface modulus, σ_1 is major principle stress, σ is reference stress, C_0 is constant.

The FWD deflection basin was also characterized by analyzing the centroid (r_x, r_y) of the deflection basin. It has been determined that a flexible pavement having a deflection basin with higher ratio of r_x to r_y would represent a pavement with better loading distribution ability and higher relative stiffness as discussed by Appea and Al-Qadi (2000). The r_x/r_y ratio can be calculated as follows

$$N = \frac{r_x}{r_y} \tag{8}$$

$$r_x = \frac{\sum_i^n A_i x_i}{A} \tag{9}$$

$$r_y = \frac{\sum_i^n A_i y_i}{A} \tag{10}$$

where A_i is the area of each element under deflection basin, x_i is centroid distance of each element from the first sensor along the x-axis, y_i is vertical centroid distance from each element along y-axis, A is the total area of the deflection basin, n is the sum number of elements under the deflection basin (Fig. 1).

To determine the modulus values, the flexible pavement system is modeled as a layered system. In the computer program, the modulus of the surface layer is assigned and then, the material is assumed as linear elastic. The elastic pavement layered moduli analysis is an approximation because all the flexible pavement layers are either nonlinear elastic or viscoelastic. The incremental advantages of conducting nonlinear or viscoelastic analysis over elastic analysis will be compromised by the inherent approximation involved in the backcalculation process (Appea, 2003; Mehta and Roque, 2003). BISAR can handle horizontal applied loads and also allows variation in strain transfer at pavement interacts.

2.2. *Materials*

FWD deflection data was collected from seven project sites, and each site has fifteen locations. The test sections were an approximation of 7 km and 105 locations that were investigated for the flexible pavement structural performance. Falling weight deflectometer was used to evaluate and identify pavement characteristics and pavement structural properties that have strongly been influenced pavement performance and functionality. Hard cores of the asphalt pavement concrete layer were taken from each of these locations during FWD testing. Inertial force was considered in the pavement structure analysis using the FWD test. And the asphalt density in situ was generally around 2400 kg/m^3, the roadbase was around 2150 kg/m^3, and the subbase was around 1850 kg/m^3. These densities were at Marshall or maximum dry density (MDD). Generally, the asphalt is compacted to around 97% of Marshall Density, the roadbase to around 96% of MDD and the subbase to around 96% of MDD according to the Australian Standard (Standards Australia, 2003). While after traffic ticking, density is expected to increase slightly. A summary of pavement material properties and thickness of the profile at seven project sites that were investigated is shown in Table 1 and the material property of each layer in flexible pavement model is shown in Table 2. The effective layer moduli backcalculation have been verified using the test result by Mehta and Roque (2003) and the comparison of the verification can be found in details in the result analysis.

3. Results and analysis

The range of asphalt concrete effective surface modulus values and pavement temperature are shown in Table 3. The FWD measurements were taken during a day time, and no pavement crack or visible possibility damage was observed on the mixed asphalt cores. From the FWD measurements data, it showed the variation of the modulus values of the project appeared to be an artifact of the backcalculation investigation process. The variation of deflection with all projects followed the same trend as that of the deflections

Table 2 – Material properties of each layer in flexible pavement model.

Pavement layer	Modulus (E) (GPa)	Poisson's ratio (υ)	Thickness (mm)	Density (ρ) (kg/m^3)
Asphalt concrete	1.80	0.30	50	2400
Unbound base	1.20	0.35	150	2150
Unbound subbase	0.50	0.35	250	1850
Compacted subgrade	0.07	0.40	75	1700
Natural subgrade	0.05	0.45	Infinite	1600

Table 1 – Summary of material properties and thickness of profile at sites 1–7.

Project/site number	Temperature during testing (°C)		Altitude	Route	Station (km)	Design thickness (cm)		
	Air	Surface				AC	Base	Subbase
1	19.8	17.8	39.0	Dongara Rd	0–1.15	20	30	40
2	30.5	35.6	8.7	Barker St	0–1.15	20	30	40
3	18.7	25.0	15.6	Armadale Rd	0–1.15	20	30	40
4	29.8	37.7	22.1	Burlington St	0–1.15	20	30	40
5	39.8	31.4	25.5	Nicholson Rd	0–1.15	20	30	40
6	28.7	39.9	12.9	Star St	0–1.15	20	30	40
7	25.7	27.2	12.9	Orrong Rd	0–1.15	20	30	40

Table 3 – Range of effective surface layer modulus values during first iteration for sites 1–7.

Project/site number	Effective surface layer modulus (GPa)			Pavement temperature (°C)
	Minimum	Maximum	Average	
1	0.20	0.65	0.38	34.9
2	0.35	5.86	3.17	35.5
3	0.61	14.93	8.20	35.3
4	0.35	5.78	3.13	35.2
5	1.14	16.06	8.70	34.9
6	0.45	12.35	5.21	35.2
7	1.05	16.18	8.70	35.1

away from pavement load. This showed that the deflection under the load was a representative of surface layer modulus, and this trend appeared to indicate that the response of the inflexible pavement was dominated by lower layer and then, made the pavement surface layer moduli along with the location to be identical. Thus, analyses suggest that stress-dependent nature of lower can be significant for overall behavior of the flexible pavement. Mehta and Roque (2003) evaluated FWD for determination of flexible pavement layer moduli using BISDEF computer program which was used for backcalculation analysis of seven projects. As what's shown from the results, the variation of deflection under the load versus project location has followed the same trend as that deflection away from the load and response was dominated by lower layer and surface layer modulus was also the same along with the location. De Almeida et al. (1994) also analyzed similar to the Mehta and Roque (2003).

The effective layer moduli of the combined base, subbase layer and the subgrade for project 1 are shown in Fig. 2. The FWD modeled data presented that the combined base, subbase layer modulus and the subgrade modulus of flexible pavement have followed the same trend of the deflection at various locations. This showed that the measured and computed deflection basins were approximately similar. These results clearly indicated that the FWD data interpretation of the comparative quality of the base and subbase, and subgrade were appropriate. An insignificant error between the measured and predicated basin might have occurred. This showed that the designed load on the structure had a capacity to resist deformation because it was designed that boundary condition of the flexible pavement layers should not affected by repletion of cyclic traffic loading. The average effective layer moduli for base and subbase were 0.39 GPa while 0.33 GPa for subgrade. Mehta and Roque (2003) evaluated the FWD data for deformation of pavement layer moduli, and results indicated that the average effective layer moduli for base and subbase were about 0.38 GPa and 0.33 GPa for subgrade layer.

Figs. 3 and 4 show the effective layer moduli of combined base and subbase layer, and subgrade layer for projects 2 and 3, respectively. Seen from the measured and computed FWD, the combined base and subbase layer moduli follow the same trend of the deflection at various locations in both projects while the subgrade layer moduli fluctuate to match the farthest deflection. This shows that the FWD data, a definitive interpretation quality of the base and subbase modulus, would have been inappropriate due to insignificant error on subgrade moduli because the measured and computed deflection basins remained approximately the same. The influence of overburden and pore pressures might have locked in the horizontal stresses on the in situ stiffness so that subgrade moduli were locked to match deflection. De Almeida et al. (1994) recommended the influence of overburden, pore pressures and a rigid bottom to be included during analysis

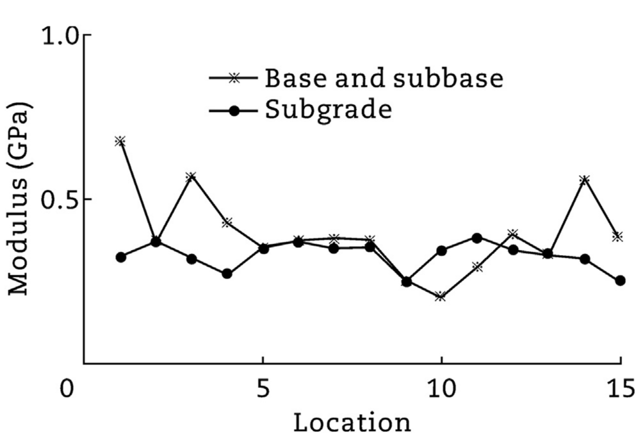


Fig. 2 – Effective layer moduli versus location for project 1.

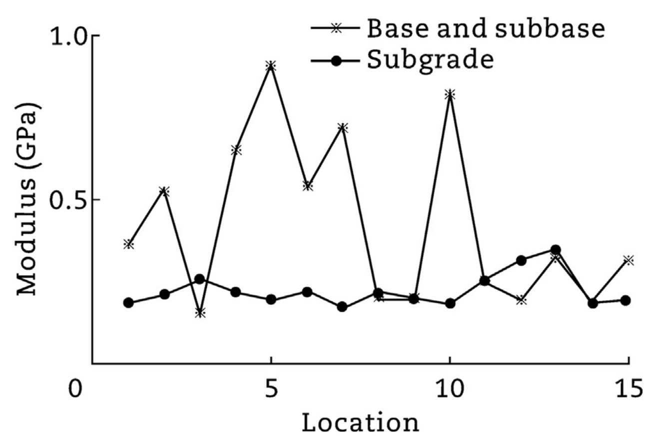


Fig. 3 – Effective layer moduli versus location for project 2.

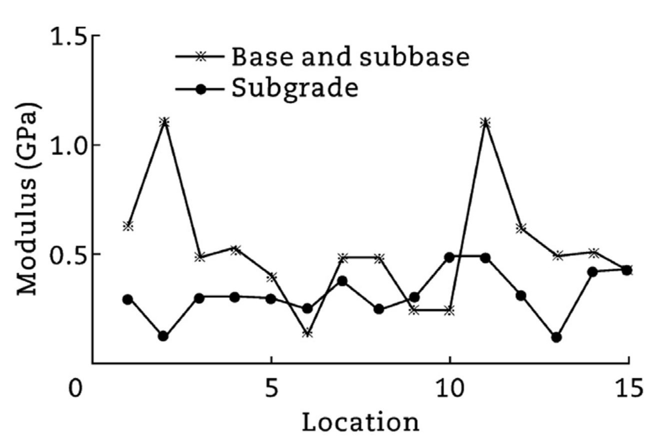


Fig. 4 – Effective layer moduli versus location for project 3.

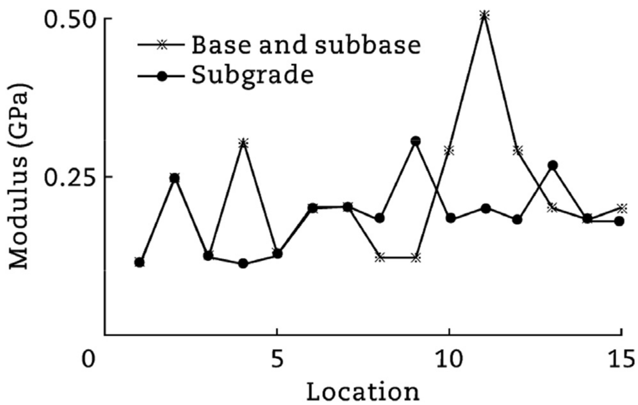


Fig. 5 – Effective layer moduli versus location for project 4.

because errors can influence the flexible pavement layer moduli from being progressing to match the furthest deflection with other layers.

Effective layer moduli of combined base and subbase layer and subgrade modulus for projects 4 and 5 are shown in Figs. 5 and 6. The FWD data and predicted model presented that both the combined base and subbase, and subgrade pavement layer modulus are on the same trend of deflection at various locations. This indicated the strain could continue to develop due to unloading stress distribution. A rest period, while small errors might have occurred on the subgrade, determined that neither the combined base and subbase layers nor the subgrade layers independently affected the deflection basin due to pavement dynamic load. However, subgrade layers usually contributed 60%–80% of the total center deflection, therefore small errors might have occurred in the moduli of the other layers. It appeared that thick and stiff asphalt concrete layers and stiff subgrade layers might have causing the deflection basin so that it would be insensitive either to the combined base and subbase layer or subgrade moduli (Appea, 2003; Ghadimi et al., 2015; Mehta and Roque, 2003). Ghadimi et al. (2015) and Appea (2003) discussed subgrade moduli that usually contributed 60%–80% of the total center deflection. Therefore, a small error in determination of subgrade moduli can lead to a very big error in the moduli of the other layers.

Fig. 7 shows the effective layer moduli of base and subgrade layer for project 6. The analysis was repeated with

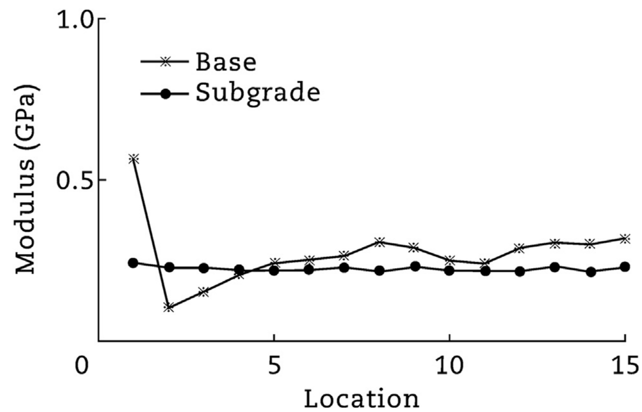


Fig. 7 – Effective layer moduli versus location for project 6.

the moduli values of particular two layers. From Fig. 7, it can be seen that the base layer modulus was computed with subgrade layer moduli to remark a similar trend of the deflection at the various locations. The average deflection for base layer modulus was 0.27 GPa while 0.23 GPa for subgrade layer. This shows that the modulus values of these layers were locked to enable the program to give a robust solution and multiple FWD test measurements and predictions were the key in ascertaining the consistency of the modulus values. Mehta and Roque (2003) and Ghadimi et al. (2015) repeated the analysis base and subgrade modulus values because of the subbase layer modulus was kept constant. Mehta's effective layer modulus for base layer was approximately 0.26 GPa while 0.22 GPa for subgrade at thirty locations of investigation.

The effective layer moduli of base, subgrade, and damage binder course for project 7 are shown in Fig. 8. Seen from the FWD analysis, the base and subgrade moduli were closer to each other in the computed deflection, which was 0.19 GPa for base layer and 0.20 GPa for subgrade layer, and had a similar variation trend at all locations. However, the binder course had two sections: damage and undamaged section. This difference in deflection could be due to high continuing stresses at a depth or lack of load transfer to the bottom layers due to the damage asphalt concrete layer. Oliveira et al. (2009) compared fatigue lines of damaged and

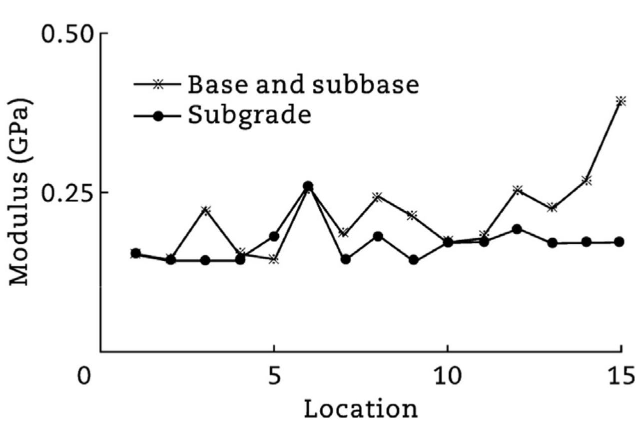


Fig. 6 – Effective layer moduli versus location for project 5.

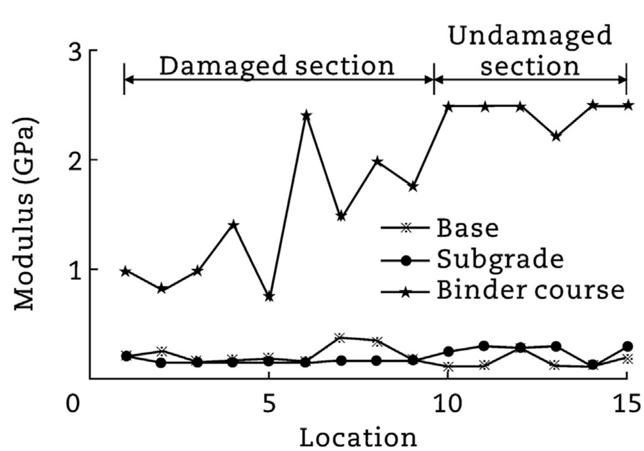


Fig. 8 – Effective layer moduli versus location for project 7.

undamaged binder course specimen of flexible pavement. Results indicated that the number of load applications required to cause a reduction in initial modulus by 50%, was defined as the number of cycles of failure. The main reason of the difference (damage and undamaged) behavior of the asphalt mixes is the strain and stress controlled testing. Similarly, Nega et al. (2013a, b) and Mehta and Roque (2003) observed cracking cores and matched reasonably well with effective layer moduli of the damaged binder course at the bottom of some cores.

The subgrade was modeled as a finite-thickness, homogenous, linear–elastic layer place on top of a bedrock and dynamic deflection basin were obtained by computing the deflection at the sixteen geophone locations using a variable subgrade depths with an average modulus of 236 MPa and 40 kN loads distributed over area of 0, 200, 300, 400, 500, 600, 750, 900 and 1500 mm. The thickness of the subgrade used was 0–3 m/0–0.15 ch. A Poisson's ratio of 0.30 was assumed for the subgrade (granular). The deflections obtained for each model were normalized to 700 kPa using the following equation.

$$D_k^* = \frac{L_a}{L_b} D_k \tag{11}$$

where D_k^* is the normalized deflection (mm) in sensor k (sensor located from the center of the applied load L), D_k is the deflection (mm) in sensor k, L_a is the load level in target a, L_b is the load level applied during test b. The load level in target means the estimated target load that is reached under predefined level of reliability, and the target load is achieved by increasing the test road.

$$\frac{L_a}{L_b} = \frac{1}{D_0} \tag{12}$$

where D_0 is the center load deflection (sensor 0).

Assuming a semi-infinite space, the theoretical pressure distribution under a rigid plate that is used FWD testing can be expressed as follow (Ullidtz, 1998).

$$q(r') = \frac{q_a}{2(a^2 - r'^2)^{0.5}} \tag{13}$$

where q is the applied pressure, a is radius of the plate, r' is the distance from the center of the plate.

From Eqs. (11) and (12), a simplify equation can be written as follow

$$D_k^* = \frac{D_k}{D_0} \tag{14}$$

If the solution for a point loads a homogenous half-space in integrated over the area of the rigid plate of the FWD with the distribution pressure that was given by Eq. (13), then the maximum deflection equation is given as follow (Appea, 2003).

$$D_0 = \frac{\pi(1-\mu^2)q_a}{2E} = \frac{(1-\mu^2)p}{20E} \tag{15}$$

where E is modulus of elasticity, υ is the Poisson's ratio, p is applied load.

The dynamic deflection basin for linear elastic and nonlinear analysis at different distance from the load center for location of project 1 is shown in Fig. 9. The analyses presented the maximum deflection ranges at D_0 (sensor 0) of all sections for project 1 were an average approximately from 320 to 880 μm for linear elastic behavior and 350 μm for nonlinear analysis. The dynamic deflection had similar trends for both linear elastic and nonlinear analysis but the nonlinear analysis had a lower dynamic deflection comparing with linear elastic behavior.

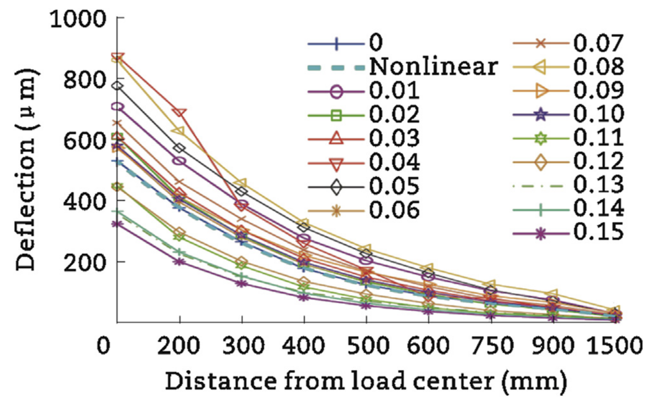


Fig. 9 – Dynamic deflection basin at various locations for project 1 from nonlinear analysis.

Fig. 10 shows the dynamic deflection basin for linear elastic behavior and nonlinear analysis at different distance from the load center for location of project 2. The data presented maximum deflection ranges at D_0 (sensor 0) of all sections for project 2 were approximately from 380 to 570 μm for linear elastic and 380 μm for the nonlinear analysis. The deflection basin had a similar trend as project 2. Similarly, Fig. 11 shows the dynamic deflection basin for linear elastic behavior and nonlinear analysis at different distance from the load center of project 3. From the data analysis, the maximum deflection ranges at D_0 (sensor 0) of all sections for project 3 were approximately from 500 to 650 μm for linear elastic behavior and about 500 μm nonlinear analysis. The trend of the dynamic deflection basin was similar as projects 1 and 2.

The dynamic deflection basin for linear elastic behavior and nonlinear analysis at different distance from the load center of project 4 is shown in Fig. 12. The analyses presented the maximum deflection ranges at D_0 (sensor 0) of all sections

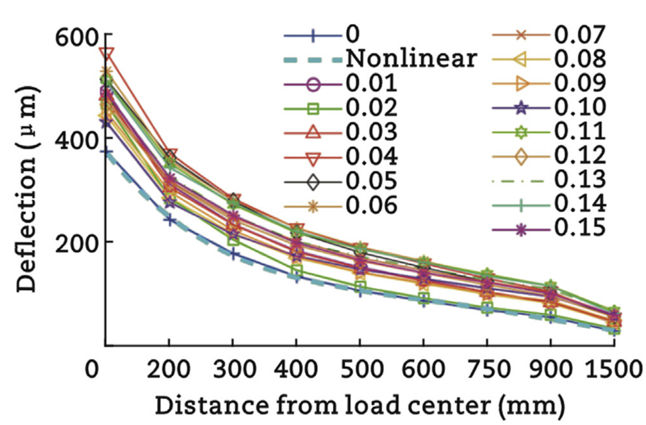


Fig. 10 – Dynamic deflection basin at various locations for project 2 from nonlinear analysis.

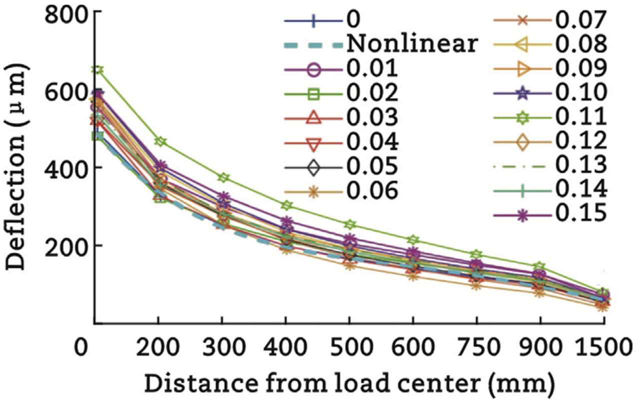


Fig. 11 – Dynamic deflection basin at various locations for project 3 from nonlinear analysis.

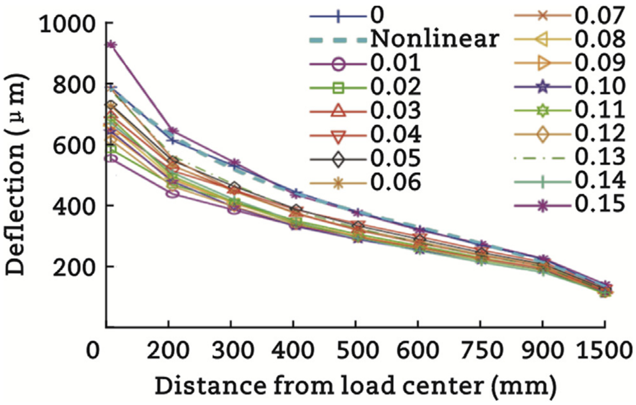


Fig. 12 – Dynamic deflection basin at various locations for project 4 from nonlinear analysis.

for project 4 were approximately between 500 and 930 μm for the linear elastic behavior and about 800 μm for the nonlinear analysis. The maximum deflection ranges were higher comparing to projects 1–3. However, the deflection trends were similar to both locations.

Fig. 13 shows the dynamic deflection basin for linear elastic behavior and nonlinear analysis at different distance from the load center of project 5. The analysis data presented the maximum deflection ranges at D_0 (sensor 0) of all sections for project 5 were approximately 300–800 μm for linear elastic behavior and 550 μm for nonlinear analysis. Though the deflection trends were similar to project 4, there were high gap in between at D_0 comparing to the other locations.

Fig. 14 shows the dynamic deflection basin for linear elastic behavior and nonlinear analysis at different distance from the load center of project 6. As it can be seen from the data, presented, the maximum deflection ranges at D_0 (sensor 0) of all sections for project 6 were approximately from 470 to 600 μm for linear elastic behavior whereas 550 μm for nonlinear analysis. The deflection trends were similar as other projects. Similarly, Fig. 15 shows the dynamic deflection basin for linear elastic behavior and nonlinear analysis at different distance from the load center of project 7. Seen from the analysis, the maximum deflection ranges at D_0 (sensor 0) of all sections for project 7 were approximately from 400 to 500 μm for the linear elastic behavior and 500 μm for the nonlinear analysis. The dynamic deflection trends were similar to project 6.

In general, the dynamic measured deflection basins from the load center at the different types of projects using the FWD linear elastic behavior and nonlinear analysis were dropped between the calculated deflection from the nonlinear and linear elastic behavior for all sections of all projects. In addition, it can be observed that the measured deflection basins were approximately followed a similar trends for all projects

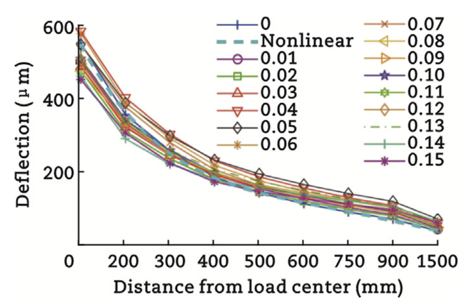


Fig. 14 – Dynamic deflection basin at various locations for project 6 from nonlinear analysis.

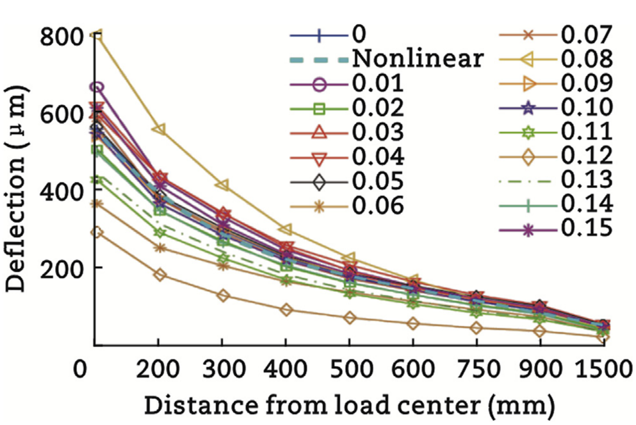


Fig. 13 – Dynamic deflection basin at various locations for project 5 from nonlinear analysis.

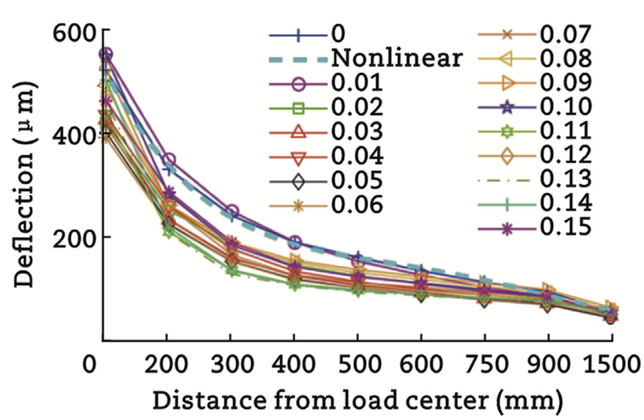


Fig. 15 – Dynamic deflection basin at various locations for project 7 from nonlinear analysis.

1–7. However, the maximum deflections at D_0 (sensor 0) of all sections of projects 1, 4 and 5 were higher (between range 320–880, 550 to 930 and 300–800 μm) and projects 2, 3, 6 and 7 ranged an average approximately from 400 to 650 μm.

The modular ration between the subgrade and the rigid foundation can create influence on deflection in some of the layers system. However, in this case, it is totally negligible because the variation in the subgrade modulus was lesser than the difference between the subgrade modulus and rigid foundation. It should be understood that some of the sections have the same modulus, which is included in the same calculation for linear–elastic and nonlinear analysis, and as the results of these, the measured deflection fall down between calculated linear and nonlinear values for most of the sections for all projects.

Appea (2003) analyzed two-layer system analysis of subgrade. The deflection results from seven sensor spacings (0, 305, 452, 609, 914, 1219, and 1524 mm) were calculated using the KENLAYER software for twelve sections (section A–L). Analysis has shown some of the sections had the same moduli and values were used to calculate both linear elastic as well as nonlinear analysis. The maximum deflection range at D_0 (sensor 0) for majority section was approximately between 200 and 500 μm with a similar deflection trends.

A summary of the subgrade analysis using invariance constraints approach and the procedure proposed by Ullidtz (1987) to determine the present of a stiff layer (depth to stiff layer) was investigated with ELSYM5 software (Eqs. (13) and (14)). The average subgrade modulus using different analysis method is shown in Fig. 16. From the subgrade analysis using different approach, it can be seen that results of the different approach are generally in agreement. Although differences were observed in some sections, in particular section D from linear elastic analysis method. The difference percentage for this section is 22% for the linear elastic analysis. The average result obtained from the layered elastic analysis does not differ much from the results of the apparent subgrade formulas (Fig. 16). They show a similar trend, with section D having the highest modulus and section G having a little bit lower modulus. But all sections have reasonable results. A stiffer layer is possible in sections B, C, E, F and G, and the depth to stiff layer is estimated at about 4.27 m (about 14 ft); this agrees with predictions from Ullidtz method, which is suggested the presence of a shallow stiffer at around 4.57 m (about 15 ft) (Ullidtz, 1987).

A comparison of average vertical strain at the top subgrade layer using WESLEA (WES), Finite Element Method (FEM) and Method of Equivalent Thickness (MET) is shown in Fig. 17. This influence lines for vertical strain at top of subgrade measured by four different gauges are also among as the predicated by three different response models: WES, FEM and MET. From the model presented, it can be seen that the vertical strain at the top of subgrade using WESLEA, Finite Element Method and Method of Equivalent Thickness multilayer computer program analysis were close to another in computed deflection with a similar trend of variation at various distance. This showed a reducing in vertical surface deflection and the critical tensile strain in the asphalt concrete layer. In this case, MET is seen to result in best prediction as compared to others prediction. However, all the predictions are quite reasonable; and different analyses methods are enable to give a robust solution.

Ullidtz et al. (1999) evaluated a pavement response and performance of models from instrumental tests in Danish Road Testing Machine. The pavement was instrumented with gauges for measuring stresses and strains at the critical positions. Layer moduli were determined from FWD testing using different backcalculation procedures. Then, stresses and strain were calculated at the position of the instruments, and compared with the values. Results indicated the vertical stress on the subgrade are overestimated by the linear elastic method, but underestimated for two nonlinear subgrades. The vertical strain in the subgrade, which is an important design parameter, is also underestimated by a factor of two linear elastic methods. However, the horizontal strain at the bottom of the asphalt layer is well reasonable. The influence lines for vertical strain at the top of subgrade were measured by gauges and predicted by different types of response models (i.e., WES, FEM, and MET). This also shows that the MET is the best predictor as compared to WES and FEM.

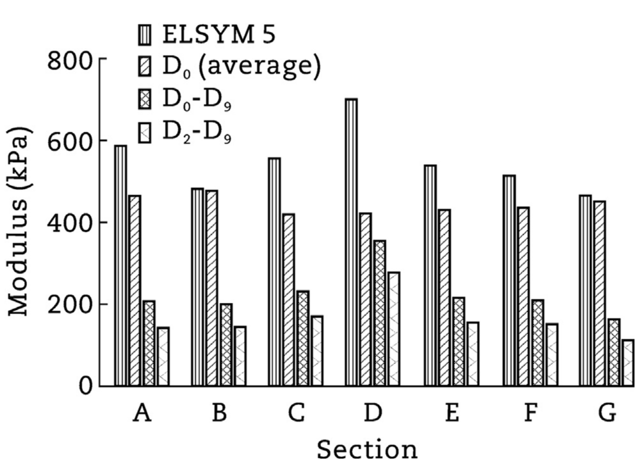


Fig. 16 – Average subgrade moduli determined using different analysis method.

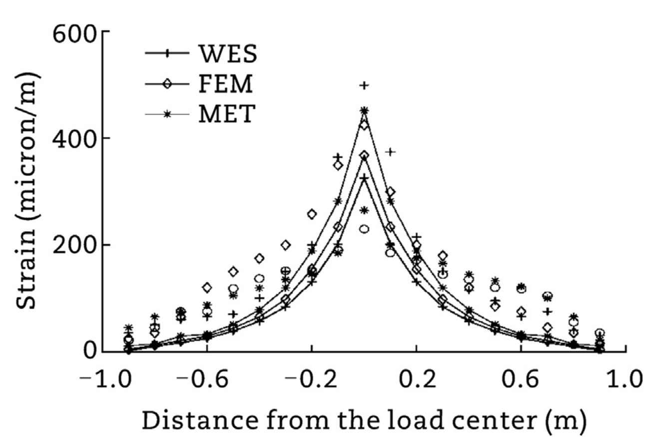


Fig. 17 – Measured and calculated vertical strains in subgrade.

4. Conclusions

The dynamic analysis of falling weight deflectometer test and predicting for the strength of character of flexible pavement layer moduli has been achieved and algorithms for interpretation of the deflection basin has been improved. The variations of moduli of all layers along the length of sections for a majority of the projects are accurate and consistent with measured and computed predicting. However, some of the projects have some inconsistent with modulus values along the length of the sections. Results are reasonable but consideration should be taken to fix variations along the pavement layers moduli sections.

The normalized dynamic deflection basins at the various distances from the load center of nonlinear and linear elastic behavior of different sections for all the projects were accurate between the measured and computed deflection, and also followed a similar trend. At the same time, consideration should be taken when the length of subgrade modulus section was locked or fluctuating in progress to match the deflection, for the reason the length of the section along its layer can create incompatible results and lack accuracy when measured or predicting of the layer modulus.

A stiffer layer of the subgrade moduli is possible in sections B, C, E, F and G, and the depth to stiff layer is estimated to be about 4.27 m (14 ft) which is in accordance with predictions from Ullidtz method, and it suggested the presence of a shallow stiffer at around 4.57 m (15 ft).

In general, analyzing FWD deflection data and the effect of layer modulus condition are difficult specially when predicting of layer moduli obtains a unique set of layer moduli that suitable to the field conditions. This is particularly true when backcalculation procedures such as BISDEF are exclusively used for layer moduli determination. Thus backcalculation procedures should enhanced with one and/or more of the layer moduli that can be accurately determined by other means or another multilayer computer program such as BISAR and WESLEA.

In addition to BISDEF, BISAR and WESLEA can be used to predict the effect of layer condition and layer modulus, which can also be used for verification by comparing with the BISDEF measure deflections data with BISAR and WESLEA. Badu-Tweneboah et al. (1989) investigated flexible pavement layer moduli from Dynaflect and FWD deflections using a linear elastic multilayer computer program (BISAR) to generate deflection for different combination of layer thickness and moduli. Badu-Twneboah's study concluded that it is often difficult to analyze Dynaflect and FWD deflection data that proves suitable to the field conditions as well as to obtain a unique set of layer moduli for flexible pavement even if the results are reasonable.

5. Recommendations

Analysis of flexible pavement layer moduli using FWD data based on backcalculation should not only be done in a single location because it would be very difficult to determine layer modulus values from one location, and results would be unrealistic and uncertain. Greater FWD load level deflection data is necessary in some more flexible pavement to yield more accurate and reliable prediction of pavement performance. Investigation is needed toward the effect of shift factor in Western Australia if it has used FWD for accuracy and effectiveness of flexible pavement. Proper interpretation of FWD data is needed and it should include the complete evaluation of all available data to minimize the FWD interpretation difficulties.

Air or surface pavement temperature should be included during FWD testing. Pavement temperature can contribute an impact to individual layer moduli of the asphalt concrete thickness or weakening strength of layer modulus of the flexible pavement. For example, specific distress, cracks or failure pavement can be caused as the result of temperature change. Thus understanding the change of pavement temperature will make it easier to make plan for the pavement maintenance and rehabilitation (M&R).

Acknowledgments

The research described in this paper is made possible through the Australia Postgraduate Award (APA) by the Australia Government and Curtin Reseach Scholarship Award (CRS). The financial support by Australia Government and Curtin University is gratefully acknowledged. The value of supervision and advice given by Professor Hamid Nikraz and Professor Imad L. Al-Qadi is also much appreciated. Mainroads Western Australia and Mr. Colin Leek (Engineer for the City of Canning) are acknowledged for providing FWD data.

REFERENCES

Al-Qadi, I.L., Brandon, T.L., Bhutta, S.A., 1997. Geosyntheically stabilized flexible pavement. In: Geosyntheics'97, Long Beach, 1997.

Al-Qadi, I.L., Brandon, T.L., Valentine, R.J., et al., 1994. Laboratory evaluation of geosynthetic reinforced pavement section. Transportation Research Record 1439, 25–31.

Appea, A., Al-Qadi, I., 2000. Assessment of falling weight deflectometer data for stabilized flexible pavements. Transportation Research Record 1709, 19–25.

Appea, A.K., 2003. Validation of FWD Testing Results at the Virginia Smart Road: Theoretically and by Instrument Responses (PhD thesis). Virginia Polytechnic Institute and State University, Blacksburg.

Badu-Tweneboah, K., Manzione, C.W., Ruth, B.E., et al., 1989. Prediction of flexible pavement layer moduli from Dyaflect and FWD deflections, in: Von Quintas, H.L., Bush III, A.J., Baladi, G.Y. (Eds.), Nondestructive Testing of Pavements and Backcalculation of Moduli. American Society for Testing and Materials (ASTM), Philadephia, pp. 245–275.

Bianchini, A., Bandini, P., 2010. Prediction of pavement performance through Neuro-Fuzzy reasoning. Computer-Aided Civil and Infrastructure Engineering 25 (1), 39–54.

Burmister, D.M., 1944. The theory of stress and displacements in layered systems and applications to the design of airport runways. In: The 23rd Meeting of the Highway Research Board, Chicago, 1943.

Bush III, A.J., Alexander, D.R., 1985. Pavement evaluation using deflection basin measurements and layered theory. Transportation Research Record 1022, 16–29.

Chang, D.W., Roesset, J.M., Stokoe II, K.H., 1992. Nonlinear effects in falling weight deflectometer tests. Transportation Research Record 1355, 1–7.

De Almeida, J.R., Brown, S.F., Thom, N.B., 1994. A pavement evaluation procedures incorporating material non-linearity, in: Von Quintas, H.L., Bush III, A.J., Baladi, G.Y. (Eds.), Nondestructive Testing of Pavement and Backcalculatioin of Moduli, second ed. American Society of Testing and Materials, Philadephia, pp. 218–232.

Frazier, P.J., 1991. Estimation of paving materials design moduli from falling weight deflectometer measurements. Transportation Research Record 1293, 42–51.

Ghadimi, B., Nega, A., Nikraz, H., 2015. Simulation of shakedown behavior in pavement's granular layer. International Journal of Engineering and Technology 7 (3), 198–203.

Goktepe, A., Agar, E., Lav, A., 2006. Role of learning algorithm in neural network-based backcalculation of flexible pavements. Journal of Computing in Civil Engineering 20 (5), 370–373.

Hoffman, M.S., Thompson, M.R., 1982. Backcalculating nonlinear resilient moduli deflection data. Transportation Research Record 852, 42–51.

Kang, Y.V., 1998. Multifrequency back-calculation of pavement-layer moduli. Journal of Transportation Engineering 124 (1), 73–81.

Kim, Y.R., Park, H., 2002. Use of Falling Weight Deflectometer Multi-load for Pavement Strength Estimation. FHWA/NC/2002-006. North Carolina Department of Transportation, Raleigh.

Lytton, R.L., 1989. Backcalculation of pavement layer properties, in: Von Quintas, H.L., Bush III, A.J., Baladi, G.Y. (Eds.), Nondestructive Testing of Pavement and Backcalculation Moduli. American Society for Testing and Materials, Philadelphia, pp. 7–38.

May, R.W., Von Quintas, H.L., 1994. The quest for a standard guide to NDT backcalculation, in: Von Quintas, H.L., Bush III, A.J., Baladi, G.Y. (Eds.), Nondestructive Testing of Pavements and Backcalculation of Moduli, second ed. American Society for Testing and Materials, Philadelphia, pp. 505–520.

Mehta, Y., Roque, R., 2003. Evaluation of FWD data for determination of layer moduli of pavements. Journal of Materials in Civil Engineering 15 (1), 25–31.

Nega, A., Nikraz, H., Leek, C., 2013a. Engineering characterization of hot-mix asphalt in Western Australia, in: Al-Qadi, I.L., Murrell, S. (Eds.), Airfield and Highway Pavement 2013: Sustainable and Efficient Pavement. American Society of Civil Engineers, Los Angeles, pp. 987–998.

Nega, A., Nikraz, H., Leek, C., et al., 2013b. Evaluation and validation of characterization methods for fatigue performance of asphalt mixes for Western Australia. Advanced Materials Research 723, 75–85.

Oliveira, J.R.M., Sangiorgi, C., Fattorini, G., et al., 2009. Investigating the fatigue performance of grouted macadams. Proceedings of the Institution of Civil Engineers – Transport 162 (2), 115–123.

Saltan, M., Tigdemir, M., Karasahin, M., 2002. Artificial neural network application for flexible pavement thickness modeling. Turkish Journal of Engineering and Environmental Science 26 (1), 243–248.

Sebaaly, B., Davis, T., Mamlouk, M., 1985. Dynamics of falling weight deflectometer. Journal of Transportation Engineering 111 (6), 618–632.

Sebaaly, B.E., Mamlouk, M.S., Davis, T.G., 1986. Dynamic analysis of falling weight deflectometer data. Transportation Research Record 1070, 63–68.

Sharma, S., Das, A., 2008. Backcalculation of pavement layer moduli from falling weight deflectometer data using an artificial neural network. Canadian Journal of Civil Engineering 35 (1), 57–66.

Sivaneswaran, N., Kramer, S.L., Mahoney, J.P., 1991. Advanced backcalculation using a nonlinear least squares optimization. Transportation Research Record 1293, 93–102.

Standards Australia, 2003. Methods of Testing Soils for Engineering Purposes – Method 5.2.1: Soil Compaction and Density Tests – Determination of the Dry Density/moisture Content Relation of a Soil Using Modified Compactive Effort. AS 1289.5.2.1-2003. Standards Australia, Sydney.

Uddin, W., Meyer, A.H., Hudson, W.R., et al., 1985a. Project-level structural evaluation methodology for pavements based on dynamic deflections. Transportation Research Record 1007, 37–45.

Uddin, W., Meyer, A.H., Hudson, W.R., et al., 1985b. A Structural Evaluation Methodology for Pavements Based on the Dynamic Deflection. FHWA/TX-86/17+387–1. Center for Transpotation Research, University of Texas, Austin.

Ullidtz, P., 1998. Modelling Flexible Pavement Response and Performance. Polyteknisk Forlag, Lyngby.

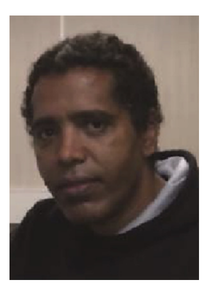
Ullidtz, P., 1987. Pavement Analysis. Elsevier Science Publication Co. Inc, Amsterdam.

Ullidtz, P., Coetzee, N.F., 1995. Analytical procedures in nondestructive testing pavement evaluation. Transportation Research Record 1482, 61–66.

Ullidtz, P., Zhang, W., Baltzer, S., 1999. Validation of pavement response and performance models. In: International Conference Accelerated Pavement Testing, Reno, 1999.

Xu, B., Ranjithan, S.R., Kim, Y.R., 2002a. New condition assessment procedure for asphalt layers using falling weight deflectometer deflection. Transportation Research Record 1806, 57–69.

Xu, B., Ranjithan, S.R., Kim, Y.R., 2002b. New relationships between falling weight deflectometer deflection and asphalt pavement layer condition indicators. Transportation Research Record 1806, 48–56.



Dr. Ainalem Nega is a civil, mining and environmental engineer with substantial experience in non-governmental organization (NGO's), industry and advanced research and teaching in academia. Nega completed his PhD in Civil Engineering at Curtin University in Perth, Australia in December 2015. His doctoral research evaluated the fundamental different types of asphalt mixtures and compared a pavement material characteristic of flexible pavement using numerical simulation and advanced experimental techniques. Nega completed the Master of Engineering Science (M.Eng.Sc) in Hydrology at the University of Western Australia, Perth, Australia in 2010. He also holds the Bachelor of Engineering (B.Eng) in Mining and Environmental Engineering at the Western Australia School of Mines, Curtin University and the Bachelor of Science (B.Sc.) in Civil Engineering at the University of Nairobi, Kenya.