

Backcalculation and sensitivity of NDT to temperature variations in flexible pavements



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ABSTRACT

The Light Weight Deflectometer is a smaller version of the Falling Weight Deflectometer, which is used to evaluate pavement-subgrade systems. Given the limited information obtained from the LWD, difficulties with back-calculation of properties exist. It is shown that elastostatic back-calculation provides a stiffness, which combined with the dynamic impedance of the Mindlin plate model allows one to estimate an effective subgrade modulus and elastic modulus of pavement or an equivalent asphalt thickness. Based on in-situ and simulated data, the sensitivity of the LWD test to variable properties and temperature is addressed.

RÉSUMÉ

Le Défectomètre à Masse Léger (DML) est une version réduite de Défectomètre à Masse Tombante utilisée pour évaluer le système de l'infrastructure des chaussées. Étant donné que les informations obtenues à partir du DML sont limitées, le retro-calcul des propriétés existe. Il est montré que le retro-calcul élastostatique donne une rigidité, qui combinée avec l'impédance dynamique obtenue à partir du modèle de la plaque de Mindlin, permet l'estimation du module effective de l'infrastructure, le module élastique des chaussées ou une épaisseur équivalente de l'enrobée. Sur la base des données in-situ et simulées, la sensibilité du test de LD vis-à-vis les variations des propriétés et de la température est examinée.

1 INTRODUCTION

Non-destructive test (NDT) techniques are important for estimating the integrity of pavement structures, as well as for determining the in-situ layer characteristics of pavement-subgrade systems. Given that mechanical properties of the pavement-subgrade layers are not uniform and depend on such factors as moisture content, temperature within pavement structure, and state of stress, one must accept that the back-calculated characteristics are system parameters that merely reflect the state of the system and may not correspond to actual properties. An important goal of research in the interpretation of NDT data has been to develop techniques that allow the engineer to estimate system parameters that more closely represent the values of the actual system characteristics such as elastic modulus.

The Falling Weight Deflectometer (FWD) has been popular for evaluating pavement-subgrade integrity. In the recent past, the Light Weight Deflectometer (LWD) has received increasing attention for the quality control of pavements. Both devices are based on measuring the response of a pavement to an impact load, which basically occurs by dropping a weight onto a plate. On the other hand, "state-of-the-practice" algorithms are mostly based on elastostatic models that do not reflect real dynamic response of a pavement to impact loading. An additional, important restriction is that the

interpretation of test data is more reliable for new pavements than for pavements that contain flaws such as cracks, ruts and potholes. Not accurately taking into account flaws leads to systematic errors in interpretation of data. On a positive note, since highway engineers are primarily interested in keeping track of the deterioration of strength and stiffness of pavement-subgrade systems an exact determination of system properties is not always necessary.

The objectives of this paper are to develop an approach that maximizes the information that one can extract from a LWD test and to compare its ability to capture in-situ properties relative to that of the FWD test. An important consideration is the influence of temperature on back-calculated parameters.

2 THEORETICAL BACKGROUND

The LWD is a scaled-down version of the falling weight deflectometer. It has a smaller load range and shorter load duration when compared to the FWD. In principle, for perfect elastic systems, one should be able to back-calculate similar system parameters from data generated by each device. On the other hand, since the FWD excites a larger volume of the pavement-subgrade system, in practice predictions by the two devices may be very different owing to the heterogeneity and pressure

sensitivity of layer properties. One could exploit these differences to discriminate between average local and more 'global' stiffness characteristics.

The most popular backcalculation techniques for FWD data interpretation are based on analyzing the peak deflections from the load-displacement history of each geophone located at specific offsets. Various algorithms have been developed to determine the layer moduli. Owing to space limitations only those most relevant to the paper are considered. There are two simplified backcalculation approaches for two-layered pavement-subgrade models: elastostatic and elastodynamic approaches; see, e.g., Stolle (1996). Both of them characterize the pavement structure by an equivalent asphalt thickness h_p and assume the subgrade layer to be an elastic half-space. Implicit in the approaches is the assumption that only two system parameters must be backcalculated to characterize the integrity of a pavement-subgrade system: effective elastic modulus of subgrade (E_s) and equivalent pavement thickness (h_p or h_e).

2.1 Effective Surface Modulus

The surface deflection of a pavement is related to the elastic modulus of each layer such that under the load the deflection is due to the straining of all pavement layers and subgrade, while at larger offsets it is mainly the subgrade that deforms with the pavement 'going along for the ride'. Deflections due to an applied surface load P may be estimated by using the Boussinesq-Odemark approach in which the surface displacement $w(r)$ at offset r is obtained by using

$$w(r) = \frac{3}{4\pi} \frac{P}{E_s h_e} \left(\frac{1}{R} + \frac{1}{R^3} \right) \quad [1]$$

where $R^2 = 1 + (r/h_e^2)$, and E_s and h_e are effective subgrade modulus and equivalent thickness of the pavement structure. System parameters E_s and h_e may be evaluated through back-calculation using surface measurements (Stolle and Jung 1992).

An effective asphalt thickness h_p of the pavement is approximated by

$$h_p = \beta \sqrt[3]{\frac{E_s}{E_p}} h_e \quad [2]$$

with β being a constant that is often assumed to be 0.9. The analyses completed in this paper assume a reference value of 4000 MPa for pavement modulus E_p .

Using Boussinesq theory for an elastic halfspace, Jung (1993) suggested that the effect of the pavement can also be captured via the 'effective or equivalent surface modulus' E_{SM}

$$E_{SM}(r) = q \cdot (1 - \nu^2) \cdot \left(\frac{a^2}{w(r)} \right) \cdot f(r) \quad [3]$$

where q is the contact pressure; ν represents the Poisson's ratio; and a is radius of the loading plate with f being the functional variation of measured deflection w that depends on r . The use of effective surface modulus is presented later.

2.2 Elasto-dynamic Model

Several researchers have alluded to the importance of taking into account inertial influences (Lytton 1989). To keep the presentation simple, a simplified procedure for FWD elastodynamic data interpretation developed by Stolle and Peiravian (1996), which takes into account the dynamic nature of the FWD test, is described in this section.

The dynamic response of the pavement directly under a dynamic load can be modelled using a single degree of freedom oscillator

$$M\ddot{w}(t) + C\dot{w}(t) + Kw(t) = P(t) \quad [4]$$

with M , C and K being the mass, damping coefficient and system stiffness, respectively. The dots as usual imply differentiation of the deflection w under the load with respect to time t . The impedance function $S(\omega)$ is derived by taking the Fourier transform of Eq. 4 and inverting to yield,

$$S(\omega) = \frac{P(\omega)}{w(\omega)} = K - \omega^2 M + i\omega C \quad [5]$$

where ω is the angular frequency, and $w(\omega)$ and $P(\omega)$ are the frequency domain equivalent of w and P , respectively (Wolf 1988). Stolle and Peiravian (1996) found that the real component of Eq. 5 does not vary greatly, at lower frequencies (0 – 30 Hz) due to the influence of the effective mass being small. Based on a Kirchhoff-Hogg model, they proposed

$$K \approx 0.6\pi E_s h_e \quad \text{or} \quad K = 6G_s \sqrt{0.81h_e^2 + a^2} \quad [6]$$

and

$$C \approx 0.73Kh_e \sqrt{\frac{3\rho_s}{E_s}} \quad [7]$$

with ρ_s ($\sim 2000 \text{ kg/m}^3$) being the density of the subgrade and G_s the shear modulus. First order approximations for the effective subgrade modulus and equivalent thickness may be obtained directly by curve fitting temporal load-displacement data from the time domain to Eq. 4 via central difference approximations for the time derivatives or by frequency domain analysis. Nezhentseva (2009) extended the approach to take into account shear deformation in the plate using Mindlin plate theory.

3 SENSITIVITY ANALYSIS

Analyses using both Kirchhoff and Mindlin plate models were performed with various modulus ratios E_p/E_s , pavement thicknesses h_p and with angular frequencies ω between 0 and 400 rad/s (64 Hz). As mentioned by Peiravian (1994), FWD loading contains frequencies that are usually less than 80 Hz with most energy in the 0 to 25 Hz range. E_s was varied from 75 to 250 MPa, E_p from 1000 to 6000 MPa, and h_p from 0.05 to 0.3 m. The computer simulations were performed for the material damping ratio $\xi=0.5\%$. A plot of the normalized imaginary components of dynamic impedance is shown in Figure 1. As one can see, the imaginary components $\omega C/K$ of the normalized dynamic impedance have an approximate linear relation with respect to kh_e in the frequency range of interest. The parameter k represents ω/v_s with v_s being the foundation soil's shear wave velocity $v_s = \sqrt{G_s/\rho_s}$. Radiation damping related to the wave propagation governs the imaginary component, not the material damping. The linear relationship indicates that the viscous damping of the SDOF model increases with increase of loading frequency ω or equivalent pavement thickness h_e . Quite a bit of scatter can be noted when $kh_e > 2$ for both cases. An important observation is the fact that the relation is sensitive to how the pavement is modelled.

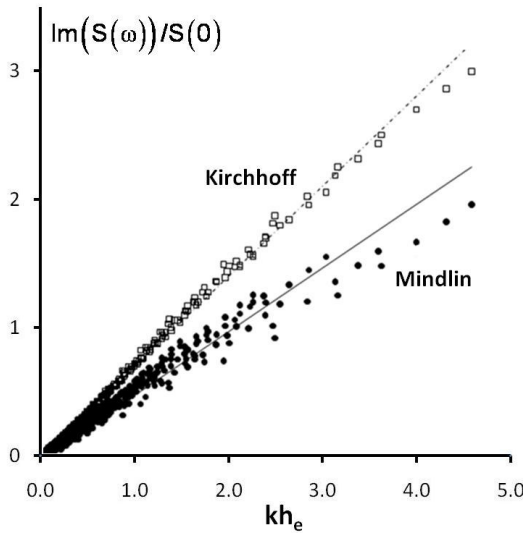


Figure 1. Normalized dynamic impedance

As for the real component of a SDOF, the relationship is quadratic in terms of ω . Up to $kh_e = 2$ the normalized real component of dynamic impedance is close to one for the Mindlin plate, but increases quickly when $kh_e > 2$. This fact shows that the mass of the SDOF system has a small value indicating that inertial effect cannot be properly captured by an equivalent mass M of SDOF oscillator. At high frequencies when the wavelengths become shorter and, thus, most of the energy is not

reaching the subgrade layer, the response is mainly dominated by the pavement properties (Peiravian, 1994).

4 DIMENSIONAL ANALYSIS

Given the sensitivity of the dynamic impedance to the pavement model selection, additional synthetic data was generated using finite element analyses based on 8-noded axis-symmetrical elements. The analyses were completed for $a = 0.075$ and 0.15 m, $h_p = 0.05$ to 0.3 m, $E_p = 2000$ to 6000 MPa, $E_s = 75$ to 250 MPa and frequencies up to 50 Hz. The boundaries were approximately 4 m away from the load and Poisson's ratio was maintained constant. The data was then examined within the framework of dimensional analysis (Buckingham 1914). Figures 2 and 3 summarize the findings for 'quasi-static' behaviour and the imaginary component of dynamic impedance. An important observation is that the stiffness varies approximately with the cube root of the modulus ratio (E_p/E_s), which is consistent with the Odemark (1949) transformation. The analysis however shows that the dependency on a/h_p is not properly taken into account when using the Odemark-Boussinesq approximation.

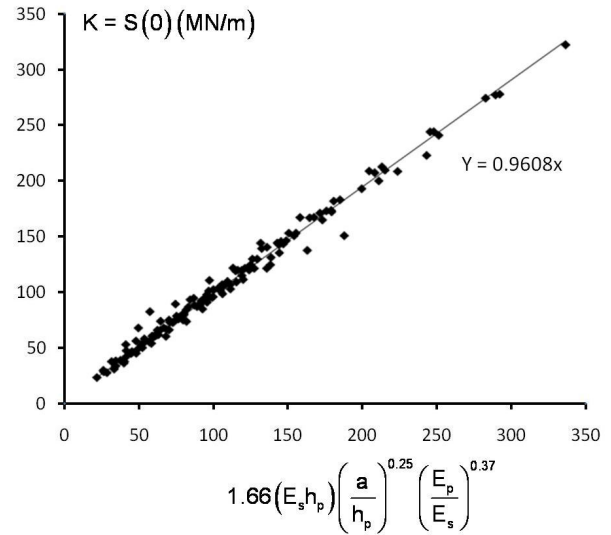


Figure 2. Dimensionless groups for static stiffness

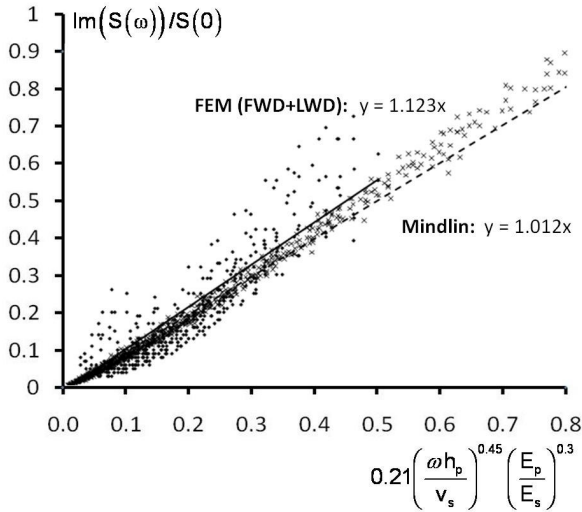


Figure 3. Dimensionless groups for imaginary component

By examination of Figure 3, it appears that the Mindlin plate theory is more realistic than the Kirchhoff theory with regard to modelling the pavement-system using two layers, given how it captures the trends from the finite element-generated data. Taking into account the scatter, it is nevertheless clear that the model is more suited as a screening tool to identify more significant changes in system properties.

5 INSITU TEST PROGRAM

LWD tests were carried out on four pavement pads, outside the Applied Dynamics Laboratory building at McMaster University during the Fall of 2007. A limited experimental program was also conducted to measure temperature profiles of the pavement-base system during August 2008 over 10 days at 10 minute intervals. Figure 4 shows the test layout as well as a LWD test. Each pad was 60x60 cm square with each having different asphalt-concrete thickness (50, 100, 150 and 200 mm). The location of bedrock near the ADL building is supposed to be a significant distance below the surface. All tests were performed with a Dynatest 3031 LWD according to standard LWD testing procedures with the 150 mm diameter loading plate. For each station 10 drops were made, with full load and displacement histories recorded. The recorded peak load value was approximately 7.8 kN.



Figure 4. Site layout showing base and LWD test set up.

Characteristic load and displacement histories are illustrated in Figure 5. As one can observe from this figure, the peak values of load and deflection are shifted as one would expect of the dynamic nature of LWD tests. There is a time lag of about 2-3 ms between the peak load and displacement measured by the central geophone. This time lag is due to inertial effects (radiation damping) typical of FWD and LWD tests. The deflection histories are not symmetrical and the displacement does not return to zero, which would cause problems when performing frequency domain analyses. This is attributed to LWD movement after the peak load. To avoid back-calculation difficulties, the deflection data after approximately 25 ms were eliminated from the analyses, which were performed in the time domain.

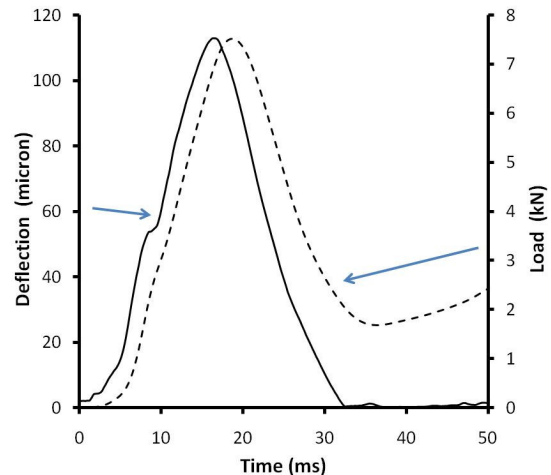


Figure 5. Representative load (solid) and displacement (dashed) histories

Back-calculation analysis was performed by recognizing that $C\omega/K = \alpha kh_e$ where α is the slope, which was found to be 0.7314 (Peiravian 1994) and 0.4976

(Nezhentseva 2009) for Kirchhoff and Mindlin plates, respectively. Time domain analysis, implementing high order finite differences (Chapra and Canale 1988) together with least squares curve fitting, was performed to calculate the parameters M , C and K of the equivalent SDOF systems for all stations after removing the last part of the time-deflection to reduce the effect of any possible problems occurring after the drop of the weight on the loading plate. Table 1 summarizes the results.

Table 1. Summary of back-calculated Parameters

Station	K (kN/m)	E _s (MPa)	h _p (mm)	h _a (mm)
A	27	54 (39)	74 (80)	50
B	43	64 (46)	109 (117)	100
C	60	79 (56)	132 (141)	150
D	65	82 (58)	140 (150)	200

Note: h_a = actual thickness; Kirchhoff (Mindlin)

The main observations include: the Kirchhoff plate predicts higher moduli and smaller equivalent asphalt thicknesses when compared to the values obtained from the Mindlin theory; and the back-calculated subgrade modulus appears to increase with pavement thickness. While similar observations have been made, when interpreting FWD data, the LWD back-calculated moduli are more sensitive to pavement thickness than those from FWD tests. This is attributed to the LWD sampling a smaller volume of material. Since the effect of the location of boundaries becomes more important for thicker pavements, this would have had an impact on the findings.

6 TEMPERATURE VARIATIONS

It is often assumed that average resilient modulus can be assumed for the asphalt concrete layer. On the other hand, it is well known that the temperature of the asphalt concrete (AC) layer is a key factor affecting the elastic modulus of asphalt concrete. The remainder of the paper addresses the impact of temperature on pavement response for FWD and LWD to find the answers to the following questions: Are the back-calculated subgrade modulus estimates sensitive to surface fluctuations in temperature? How does backcalculated layer stiffness vary as temperature changes in terms of what one measures using the various tests?

The measured temperature variations at 10 mm depth were fit to a sine function, which was used for subsequent analyses; i.e.

$$T(t) = 13\sin[6.02(t-12)] + 28 \quad [8]$$

where t is the time (hours), starting at 12:00 am. Figure 6 shows good agreement when comparing calibrated function and the field data.

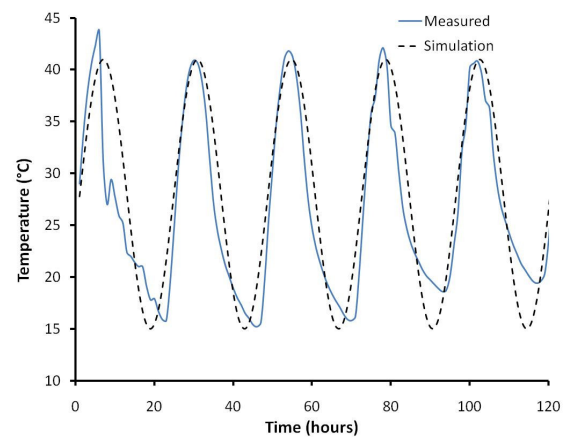


Figure 6. Simulated and measured temperature

Finite difference simulations were then completed to study the spatial and temporal temperature fluctuations over time assuming that the temperature stays constant at 600 mm depth (22.5°C) and specifying the temperature history at the 10 mm location. The objectives of the heat conduction simulations were to determine temperature profiles within the pavement-subgrade structure and to confirm the thermocouple temperature measurements, as well as to obtain a better understanding of the temperature dynamics. Two sets of simulations were performed, one with gravel as base layer and the other with clay to see how sensitive the results would be to differences in thermal properties.

Based on a review of the literature, the thermal diffusivity of asphalt concrete, gravel and clay are 0.4348, 0.4804 and 0.5324 mm²/s, respectively. For details the reader is referred to Nezhentseva (2009). The temperatures in the asphalt concrete were not very sensitive to the thermal properties of the supporting material. Figure 7 shows representative predictions and measured temperature variations with depth for different times of day of the 100 mm thick pavement. Both predictions and measurements provide similar trends. It was observed that the temperature changed quite rapidly in the upper part of the asphalt pavement during the day while it remained almost constant in the underlying layers, which is characteristic for summer conditions.

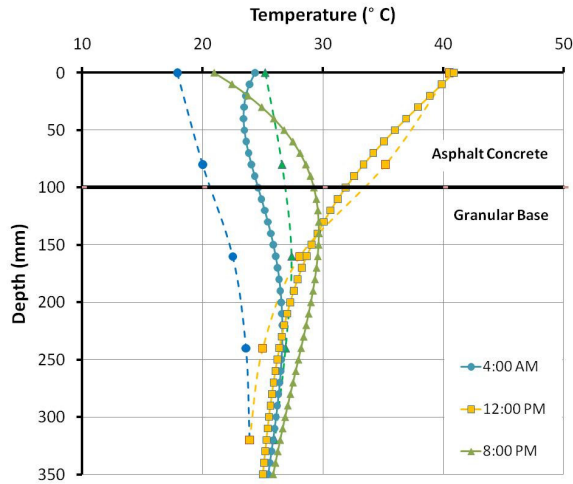


Figure 7. Distribution of temperature with depth.

Figure 8 shows simulated variations of elastic modulus over time and depth in the asphalt concrete layer (200 mm thick) using the Ali and Lopez (1996) relation

$$E_p = e^{9.372 - 0.0361 \cdot T} \quad [9]$$

where E_p is the asphalt concrete elastic modulus (MPa) and T is the pavement temperature ($^{\circ}\text{C}$). It had been found that the modulus variation in the upper 100 mm was insensitive to the thickness of the pavement. At the surface of pavement layer, the elastic modulus reaches its maximum value of 5500 MPa between 8 pm and 12 am and its minimum value of 2700 MPa at noon. Rapid changes of the elastic modulus during the day can be observed within approximately the top 0.07 m. Elastic modulus decreases with depth at night, while in the middle of the day when temperatures reach the highest value (and pavement becomes softer) the elastic modulus values increase with depth.

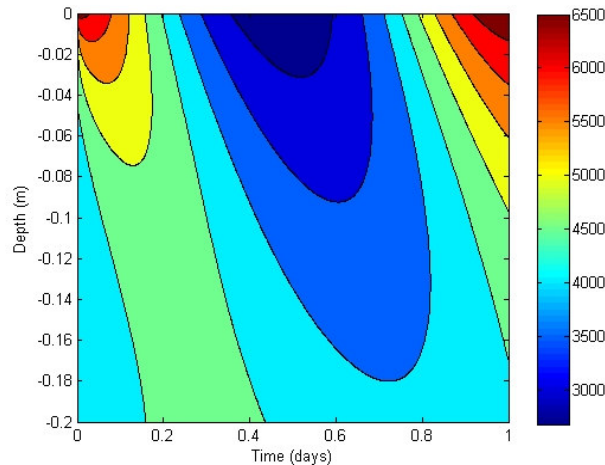


Figure 8. Daily elastic moduli variations in the AC layer for 200 mm asphalt layer

7 TEMPERATURE EFFECTS

Non-isothermal simulations, in which the elastic modulus of the asphalt concrete was non-uniform and depended on surface temperature of pavement layer during the day, were performed to study the effect of temperature changes in pavements on the response of the pavement-subgrade system to loading. The pavement layer was divided into sub-layers 50 mm thick to simplify the model with the representative modulus corresponding to the midpoint of the sub-layer. The variation of the resilient modulus of asphalt concrete layer with depth was calculated according to the Ali and Lopez (1996) relation, corresponding to reference times: 4 am (lowest), 12 pm (highest) and 8 pm (average pavement surface temperatures). The finite element details are presented in Nezhentseva (2009).

7.1 Elastostatic Data Interpretation

Backcalculation analysis was performed on the synthetic load-deflection histories corresponding to FWD and LWD simulations to understand how temperature affects the effective surface modulus predictions. The main goal was to identify how sensitive backcalculated E_s is to the temperature changes in asphalt concrete for FWD and LWD device loading.

The elastostatic backcalculation approach presented previously was implemented in a FORTRAN computer code developed at McMaster. Effective surface moduli were backcalculated at various offsets using Eq. 3 together with the maximum load and surface displacements. This was done corresponding to hypothetical measurements at various times of the day (4 am, 12pm and 8pm). Representative effective surface modulus profiles are shown in Figure 9 for the cases of 100 and 200 mm thick pavements supported by a 100 MPa subgrade.

The trends for both devices look similar. The figures clearly show that an increase in pavement thickness (h_p) affects the growth of the maximum value of effective surface moduli E_{SM} for both devices (at zero offset), as one might expect. Although not shown, a common observation for both devices is that the value of the effective surface modulus E_{SM} increases with an increase in the stiffness of a subgrade layer E_s . This is reasonable as the surface modulus at zero offset ($r=0$) represents the stiffness of the entire pavement-subgrade system. E_{SM} has a higher value at 4 am when the temperature in the AC layer reaches its minimum, while it decreases as the day temperature reaches its peak at noon. The value of effective surface modulus E_{SM} at zero offset corresponding to the LWD was found to be consistently higher than the value predicted by FWD. The LWD and FWD devices excite different volumes of the pavement-subgrade system such that the LWD mostly captures the

stiffness of the pavement-base layers, which is higher than that of the subgrade resulting in the higher E_{SM} at zero offset than that of the FWD. Variations of E_{SM} close to the load are significant during the day.

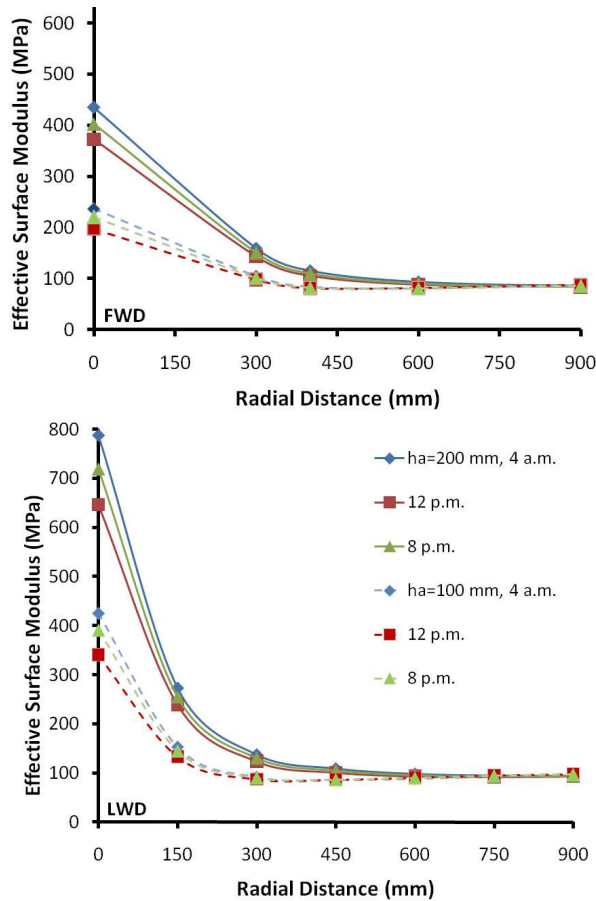


Figure 9 Effective surface modulus for FWD and LWD.

E_{SM} decreases with increase of the distance from a load centre as it begins to represent the stiffness of the lower layer (Jung, 1993). At some point, the E_{SM} changes become negligible forming a single relation. The radial distance (offset) where the effective surface modulus is constant is approximately 400-800 mm for the FWD and 300-600 mm for the LWD. The load plate radius is believed to have some impact on this characteristic distance. According to Stolle and Jung (1992) the distance where E_s reaches its minimum value is approximately equal to equivalent thickness h_e for two-layer approximation. At larger offsets, E_{SM} approaches the value of E_s . One observes that the insensitivity of E_{SM} at larger offsets, which indicates that the backcalculated subgrade modulus is not sensitive to temperature fluctuations in the asphalt layer, as one might expect. It is however important to remember that deflections for the LWD are usually not recorded at larger offsets.

Figure 10 shows the back-calculated elastic modulus of subgrade against input E_s . Although not perfect, there appears to be a reasonable one-to-one relation. The values corresponding to the LWD are, of course, hypothetical given that displacements are usually not recorded at large offsets.

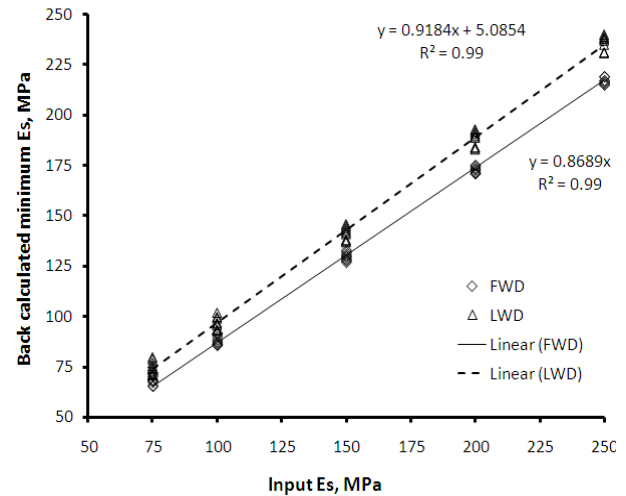


Figure 10. Minimum back-calculated versus input E_s

7.2 Elastodynamic Interpretation

Typical simulated load-deflection histories for FWD measurements are presented in Figure 11. These trends reflect those observed when testing flexible pavements during an average hot summer day when the surface of the asphalt concrete pavement layer attains its minimum at 4 am and maximum temperature at noon. In the present study, time domain analysis was used to evaluate M , C and K corresponding to a SDOF approximation for the FWD and LWD simulations, in which temperature within the asphalt layer varied with depth. M , C and K values were back-calculated using the synthetic load-displacement histories.

The main observations for the stiffness coefficient K were: K increases with an increase of the subgrade modulus E_s ; K increases with an increase of the pavement thickness h_a ; and K increases with a decrease of the pavement surface temperature.

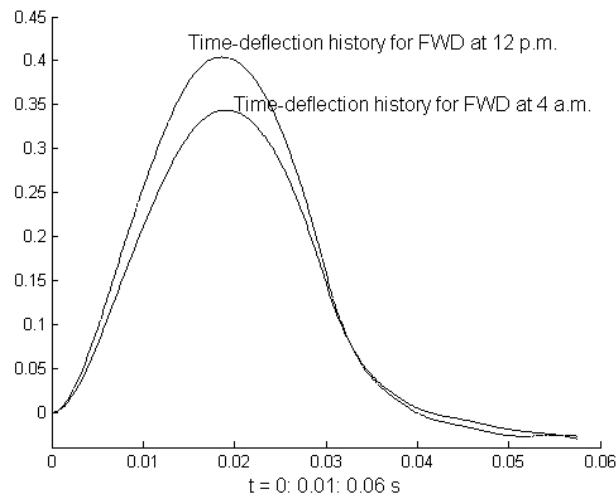


Figure 11. Deflection history for the various times

The main observations for the damping coefficient C were: for a pavement with the same thickness of asphalt concrete layer C decreases with an increase of the subgrade modulus E_s ; and C increases with an increase of the pavement thickness h_p . As the pavement surface temperature increases (which implies decreasing elastic modulus of asphalt concrete E_p) the damping coefficient C decreases. The back-calculation results of C and K for the LWD were more sensitive to temperature and subgrade stiffness changes than those back-calculated from the FWD data.

8 CONCLUDING REMARKS

The light weight deflectometer is a portable version of the falling weight deflectometer. The main disadvantage is that it has fewer sensors available to provide information that can be used to estimate the characteristics of a pavement. One way of extracting more information is to make use of the full displacement and load histories. Relations have been developed that allow one to make better use of the measurements.

A limitation of the LWD is the properties that it captures reflect the materials in a much smaller volume than what the FWD is capable of measuring. As a result, the LWD is more susceptible to temperature variation than the FWD. One could, however, take advantage of this characteristic such that both devices could complement one another in order to better identify the system properties.

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