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Nondestructive testing of asphalt pavements for structural condition evaluation: a state of the art

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Nondestructive testing provides ideal means to test pavement structure in a rapid and convenient manner. In last few years, significant development has taken place in this field. This paper, presents some of the major conventional as well as emerging nondestructive evaluation methods for *in situ* structural assessment of asphalt pavements. Discussion of methods is primarily directed towards the estimation of layer moduli and thickness values which are direct indicators of the structural strength of pavement. Estimation of density, moisture content, etc. has not been emphasised in this paper.

Keywords: Deflection; Layer moduli; Thickness estimation; Backcalculation; Spectral analysis

1. Introduction

Structural condition evaluation of existing asphalt pavement serves purposes such as, (i) estimation of its present sufficiency for the purpose served, (ii) estimation of remaining service-life and (iii) decision on the choice of rehabilitation measure. A desirable method is the one which neither causes disruption to traffic, nor causes any damage to the existing pavement structure. For quick acquisition of test data, a reliable and consistent test technique is needed. Nondestructive testing (NDT) methods provide all these features and are ideally suited for application to pavements. With NDT methods, a number of data sets can be acquired at the same point which provides statistical reliability to the experimentally acquired data.

In a structural evaluation process, the primary objective is to estimate the structural strength. For asphalt pavements, it may be expressed in terms of *in situ* layer elastic moduli, layer thickness, inter-layer bond conditions and anomaly characterisation. A simple and direct evaluation method for layer properties estimation is by visual examination after digging pits or taking cores from the pavement, which are destructive methods. Dynamic cone penetrometer test can give a qualitative (and to some extent quantitative) estimation of layer thickness, though it has some limitations (Loizos and Plati 2007, Roy 2007) in terms of test-speed, error and subjectivity in testing.

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This paper discusses some of the emerging as well as contemporary nondestructive evaluation (NDE) methods which have found applications for *in situ* structural evaluation of asphalt pavements. Underlying principles and test set-up of various techniques alongwith their scope and performance are discussed in detail. Discussion is confined to the NDE methods specific to the estimation of *in situ* thickness and moduli values of asphalt pavement layers. It does not specifically cover the issues related to estimation of density, moisture content, etc.

2. Basic principles of NDE techniques for asphalt pavements

A number of NDE methods have been developed for evaluation of asphalt pavements. For *in situ* NDT of pavements, only the top surface is available. Thus, methods requiring access to two different surfaces of the pavement structure cannot be employed in this case (X-ray method, etc.). Laboratory techniques such as resonance column method and ultrasonic testing have been used to determine the seismic moduli for complimenting the evaluation procedure (Nazarian *et al.* 1999, Celaya and Nazarian 2007). These techniques can also be used for calibration of field NDT methods (Hammons *et al.* 2006).

Many of the NDE methods involve implementation of forward approach for analysis and the inverse approach for interpretation. In forward approach, the system is idealised in the form of a model to study its physical response (say, deflection) due to some stimulus (say, force), when the properties of the pavement layers (say, layer elastic moduli) are assumed. A simple approach to forward modeling of pavement structure is to idealise it as a horizontally layered, linearly elastic, isotropic and homogenous media. More complex forward analysis schemes can be developed by dropping these idealisations, thereby achieving closer approximation to the actual response. In fact, the seismic/ loading response of pavement structure is quite complex, and its modeling is itself a difficult task (Brown 1997). Asphalt layer is strongly sensitive to nature of loading (magnitude, frequency and duration) as well as temperature (Krishnan and Rajagopal 2003); whereas the elastic modulus of granular layer and subgrade is sensitive to stress conditions (Lekarp *et al.* 2000).

The inverse approach involves deductive analysis for which an unconstrained optimisation scheme is applied to the acquired data (say, deflection) so as to estimate the desired properties (say, layer elastic moduli) of the pavement system. Thus, the inverse algorithm typically involves recursive calls to the forward analysis with some assumed values of the unknown parameters, and any pre-defined error parameter is minimised through subsequent iterations. The error parameter is generally defined as root mean square of the difference between the obtained and predicted responses. Since, inverse calculation requires computation of forward calculations at each iteration, use of more realistic forward modeling does enhance the accuracy but it significantly increases the computation time as well.

Based on the underlying principles, the NDE methods for asphalt pavements can be classified under two broad headings as follows.

- Deflection-basin methods

    - Static loading method
    - Steady state loading method
    - Dynamic impulse loading method

- Wave propagation methods

    - Stress/Elastic wave method
    - Electromagnetic wave method

These methods are discussed in the following sections.

3. Deflection-basin methods

In deflection-basin methods, vertical deflections at various points on the surface of the pavement test-section, due to an applied load, are measured. Type of the load applied may be static, steady state harmonic or transient impulse type (ASTM D2003). The measurement of deflection at the load point and at locations radially outwards from it, can be done with the help of a dial gauge/linear variable differential transformer, or by using velocity transducers (geophones) (Haas *et al.* 1994). Various types of deflection based NDE methods are discussed further in the following subsections.

3.1 Static loading method

In this method, a static load is applied to the test section and its deflection is measured. Benkelman beam deflection (BBD) test is a simple and relatively inexpensive method under this category.

Basic device in a BBD test consists of a lightweight frame to which a beam probe is attached alongwith a dial gauge (Zube and Forsyth 1966). BBD measures the deflection due to static loading of dual tyres of a loaded truck. Measurement process starts by placing the probe between the rear dual wheels of the test vehicle (figure 1). Initial dial gauge reading is recorded. The truck is made to move away by a specified distance and as the rebound rate of pavement surface falls below a specified level, intermediate reading is taken. Truck is again made to move further away and the final reading is taken. Such an approach is adopted as an attempt to reduce/nullify the influence of deflection bowl on the readings (Haas *et al.* 1994, Chakroborty and Das 2003). A characteristic BBD value, which is larger than allowable, warrants an overlay for the pavement.

The BBD device has been improved and made capable of multi-point probing with increased accuracy. Rolling weight deflectometer (RWD) is an example of such improved devices that employ similar basic principle (Bay and Stokoe 1998).

3.2 Steady state loading

In this method, a low frequency oscillatory loading is applied to the pavement and the deflection response is recorded (ASTM D4602 2003). The loading is generally applied

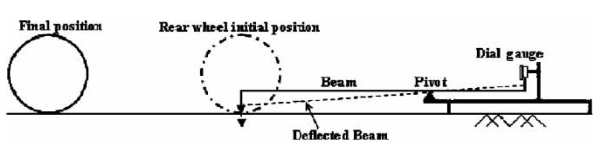

Figure 1. A typical Bankelman beam deflectometer setup.

by counter rotating mass or electro-hydraulic system. The two trailer wheels equally distribute the vertical dynamic load to the pavement. A geophone series measures the velocity of movement with respect to time, which on integration gives the deflection basin (Haas *et al.* 1994). Thus, the deflection measurement for these devices are free from influence of deflection bowl. However, test loading is harmonic and does not have any rest period. Hence, the loading condition is somewhat different (Fwa 2006) and the magnitude of load applied is also smaller (Ullidtz 1987) than the actual situation of in-service pavements. Also, a relatively large amount of pre-load is applied to the pavement which may adversely affect the accuracy of pavement response (Fwa 2006).

3.3 Transient impulse loading

This method uses an impulsive transient load application to the pavement test-section and the deflection response is recorded at a series of radial points (ASTM D4694 2003). This type of device is typically referred to as falling weight deflectometer (FWD) and is schematically shown in (figure 2). The amount of impact load, duration and the loading area is adjusted in such a way that it closely corresponds to the actual loading by a standard truck on in-service road (Sebaaly *et al.* 1991). The peak load is controlled by varying the falling mass, height of drop and the spring constant. The time of impulse load may vary between 0.025 and 0.30 s, and the applied load may vary between 4.45 and 156 kN (Bay and Stokoe 1998, Goktepe *et al.* 2006). Generally, the load pulses are of halfsine type and the loading area is a 30 cm diameter circular pad (Bay and Stokoe 1998). Velocity transducers Geophones record the deflection response. Depending on the type of FWD, 3 to 7 geophones are used to capture the deflection profile (Haas *et al.* 1994, Advisory Circular, No.150/5370-11A 2004). Load is measured by a load cell, and its variation may be used to evaluate the stress dependence of the layer modulus (Roesset 1998, Rahiom and George 2003, NCHRP Design Guide1-37A 2006). Since the stiffness of asphalt layer is affected by temperature, a suitable temperature correction factor needs to be applied to the observed deflection values to convert them to values at the standard temperature (Chen *et al.* 2000, Park *et al.* 2002).

Some lighter versions of FWDs, called as light falling weight deflectometer, are also available in which the impact load is relatively small (Loizos *et al.* 2003, Goktepe *et al.* 2006). LFWD is generally found to be suitable for base and subgrade evaluation (Loizos *et al.* 2003, Alshibli *et al.* 2005, Loizos and Boukovalas 2005). In heavy weight deflectometer (HWD) heavier load is used to simulate aircraft or other heavier vehicles (Turner *et al.* 2003).

Major advantage of this technique is that it closely simulates the actual in-service pavement loading. Another benefit is that the road need not be closed to traffic during the test

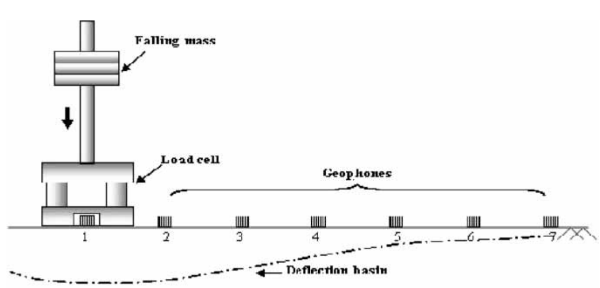

Figure 2. Schematics of a typical FWD test set up.

process. However, FWD method is relatively expensive and inverse analysis is complex. For taking FWD measurements one needs to stop every time for deflection measurement. This problem is taken care of by the rolling dynamic deflectometer (RDD) where the vehicle moves at slow speed (of the order of 2.4 km/hr) and the deflection is continuously measured (Bay and Stokoe 1998, Goktepe *et al.* 2006) by rolling sensors. RDD also measures the deflection from inertial frame of reference, hence the error associated with the geometry of the road does not come into picture (Bay and Stokoe 1998, Bay *et al.* 2000). Since, the deflections are measured continuously, the statistical confidence is also good (Lee *et al.* 2006).

Forward analysis approach which simulates the FWD loading can be divided into two broad categories, static analysis (Lytton 1989, Fwa 2006) and dynamic analysis (Roesset *et al.* 1994, Loizos *et al.* 2003, Loizos and Boukovalas 2005). The dynamic analysis is able to take into account the inertia effects, and loss of impact energy due to hysteresis and seismic wave radiation (Chang *et al.* 1992, Loizos *et al.* 2003). Researches (Roesset and Shao 1985, Chang *et al.* 1992, Uzan 1994) suggest that the use of static analysis may sometimes give erroneous results, especially when there exists a stiff bedrock layer below the pavement at a shallow depth.

The problem of backcalculation, i.e. inverse analysis is a complex one, and efforts are being made to evolve a generalised backcalculation approach (May and Von Quintus 1994) that can reach the solution quickly and accurately. Various approaches based on (i) closed formed solutions (Scrivner *et al.* 1973, Hall and Mohseni 1991, Advisory Circular, No.150/5370-11A 2004), (ii) database search (Anderson 1989, Chou and Lytton 1991), (iii) optimisation techniques (Bush and Alexander 1985, Sivaneswaran *et al.* 1991, Harichandran *et al.* 1993), (iv) regression equations (Ali and Khosla 1987, Mahoney *et al.* 1993, Roque *et al.* 1998), (v) evolutionary algorithms (Meier and Rix 1994a,b, Sharma and Das) and (vi) combination of these methods have been implemented for backcalculation of layer moduli. The difficulties faced by various backcalculation approaches may be summarised as follows (Sharma and Das):

- The closed formed solutions can handle only limited and idealistic situations.
- The regression methods are fast, but sometimes may give inaccurate results (Lenngren 1991).
- The methods based on conventional optimisation techniques take a long time to backcalculate the layer moduli (Harichandran *et al.* 1993). There are chances that the solution may converge to a local optimal point. Also, the convergence depends on the initial solution (seed values) chosen (Harichandran *et al.* 1993, May and Von Quintus 1994).
- The approach based on artificial neural network (ANN) takes a long time for training data-sets and requires fresh training every time the input parameters change. Computation time in genetic algorithm (GA) based program is also large (Fwa *et al.* 1997, Kameyana *et al.* 1997).

3.4 Closing remark

Table 1 compares various types of deflection based equipment according to the characteristics of the impulse force applied.

Table 1. Comparison of load and frequency range for various deflection based equipment.

Test equipment	*Type of force*	*Impulse force level (KN)*	*Frequency range (Hz)*	*References*
Benkelman beam	Static	≈23–45	0	Bay and Stokoe (1998), Fwa (2006)
RWD	Pseudostatic	76–89	<4	Bay and Stokoe (1998)
Steady-state loading	Dynamic	2.2–36 peak-to-peak	5–70	Bay and Stokoe (1998)
FWD	Broadband dynamic	6.7–120	≈0–60 Hz	Pittman (1996), Turner *et al.* (2003)
RDD	Dynamic	9–310 peak-to-peak	5–100	Turner *et al.* (2003)
HWD	Broadband dynamic	30–240	0	Pittman (1996), Turner *et al.* (2003)

4. Wave propagation methods

The wave propagation based NDE methods employ wave propagation principles (for stress/electromagnetic waves) in a multilayer pavement structure, and the response is acquired, analyzed and interpreted. These methods are discussed in the following sections. The frequency-ranges of different types of waves that are utilised in these methods are presented in (figure 3).

4.1 Stress/elastic wave method

A stress/elastic wave is a type of mechanical wave that propagates in elastic/viscoelastic materials due to a stress based disturbance. There are two types of stress waves, namely, body-waves and surface-waves. The salient characteristics and propagation principles of these waves in multilayered media are discussed elsewhere (Richart *et al.* 1970, Ryden 2004, Goel and Das 2006). Stress waves may be generated through different devices like drop-weight, strike hammer, or a transducer. The sensing element is also a transducer receiver (e.g. accelerometer). The data is recorded through a data acquisition system and subsequently analysed to get the required estimates. It is important to note that the estimated seismic moduli of pavement layers by these methods may typically show larger values as compared to those estimated by deflection based methods, since a low magnitude loading is applied at a high strain rate in this case. Therefore, suitable correction models need to be incorporated in the analysis scheme (Nazarian and Alvarado 2006). Based on the analysis

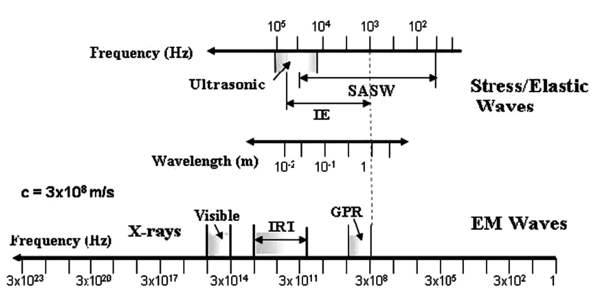

Figure 3. Frequency ranges for different wave-propagation based NDE methods.

technique used, the stress wave method can further be classified as:

- Impact echo (IE)/pulse echo method
- Pulse velocity method
- Spectral analysis of surface waves (SASW) method

The first two methods employ body waves, whereas the third method primarily employs surface wave response to evaluate the pavement structure. These are discussed in the following subsections.

4.1.1 Impact echo/pulse echo method. In the IE/pulse echo method, an impact is made at the test surface to generate stress waves and their reflections from any inhomogeneity in layer medium, interface, void, crack, etc. are recorded to extract information on its location (figure 4, Sansalone 1997). It is an established method for testing concrete structures (Sack and Olson 1995, Gucunski and Maher 2002 Nazarian *et al.* 2003, Cho and Linb 2005).

From the amplitude-time data of reflected body waves, the frequency peaks are identified using frequency domain analysis. Each of these peak indicates the resonant echo frequencies. Inverse of these frequencies gives the time required for the waves to get echoed from the pavement interface or the crack surface (Sack and Olson 1995, Sansalone 1997). For a single layer structure, the thickness of the layer (t) can be calculated as (Hill *et al.* 2000)

$$t = \frac{\beta V_{\mathrm{p}}}{f} \tag{1}$$

where, t is the thickness of the layer, V_{p} is the longitudinal stress wave (P-wave) velocity, f is the resonant frequency and β is a correction factor. Theoretical basis of this factor β has been explained in a recent study (Gibson and Popovics 2005). If the V_{p} through the media is known, the depth of crack or interface can be estimated. IE method can be combined with the SASW method for V_{p} and thickness determination (ASTM C1383 2000, Kim *et al.* 2006). The SASW method is discussed in a later section. Ultrasonic pulse echo has been used for estimation of thickness and depth of surface cracks for asphalt pavements with success (Underwood and Kim 2002, Celaya and Nazarian 2007).

4.1.2 Pulse velocity method. In the pulse velocity method, the P-wave velocity in the material medium is determined which is used to calculate its dynamic elastic modulus or the

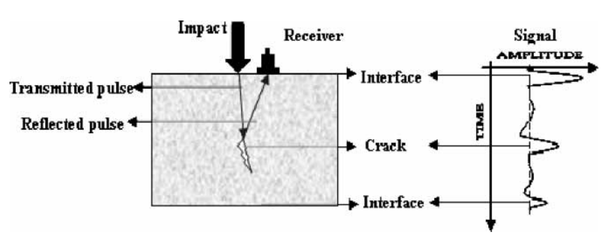

Figure 4. Conceptual description of IE test.

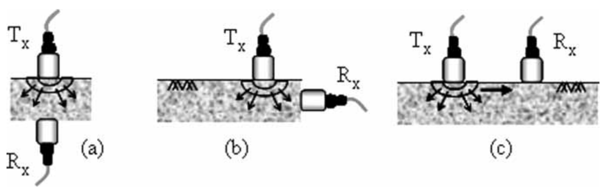

Figure 5. Transducer configuration in UPV testing, showing (a) direct, (b) semi-direct and (c) indirect transmission of waves.

Poisson's ratio using equations (2) and (3).

$$\frac{V_\mathrm{p}}{V_\mathrm{s}} = \sqrt{\frac{2(1-\nu)}{(1-2\nu)}} \tag{2}$$

$$E = 2\rho V_\mathrm{s}^2 (1+\nu) \tag{3}$$

where, V_p is the P-wave velocity, V_s is shear or S-wave velocity, ν is Poisson's ratio, E is stiffness modulus and ρ is the mass density of the material medium. Test set-up consists of a wave-pulse transmitter (T_x) and a transducer receiver (R_x) at a known distance apart (figure 5). Transit time for the wave-pulse is recorded by the data acquisition unit which then gives the stress wave velocity (ASTM C597-02 2002). Ultrasonic waves are commonly employed in this test. Laboratory studies suggest (Pellinen and Witczak 2002, Jiang *et al.* 2006, Kanta Rao *et al.* 2006) that the ultrasonic pulse velocity (UPV) method can estimate the dynamic modulus of asphalt with reasonable accuracy. For field testing on in-service pavements, indirect configuration of sensors needs to be used, as only top surface is accessible. P-wave velocity also varies with the homogeneity of the material medium and this property can be used to characterise it. In some recent studies, UPV has been used to monitor top–down cracking (Khazanovich *et al.* 2005), fatigue damage and crack healing in asphalt pavements (Abo-Qudais and Suleiman 2005).

4.1.3 Spectral analysis of surface waves method. The SASW method (Heisey *et al.* 1982, Nazarian *et al.* 1988, 1999) introduced a relatively simple and portable surface wave test technique for pavements. Its major feature is the use of spectral analysis technique for signal processing which makes it possible to analyze a range of frequencies with a convenient hammer-impact type surface wave source and two transducer receivers. The method operates at negligible strain level and provides estimates of the elastic moduli and thicknesses of layers.

The basic assumption used in this method is that only the fundamental mode of Rayleigh wave (R-wave) propagation is excited and recorded. Higher modes of R-wave propagation are neglected. Physical quantity that is being determined, is the S-wave velocity (V_s) which is a characteristic engineering property of a material (Richart *et al.* 1970). Commonly used three-step evaluation procedure for this method is (i) generation of experimental dispersion curve (EDC), (ii) forward modeling of R-wave dispersion and (iii) inversion to generate vertical stiffness profile of the test section (Goel and Das 2006).

EDC is a curve between V_s and the generated wavelengths for the test section. It is generated from data obtained through field testing which consists of a suitable impact source to generate surface waves of a desired frequency range that are sensed by two vertical, single-axis transducer receivers at known offsets (figure 6). Time domain data sets may be obtained

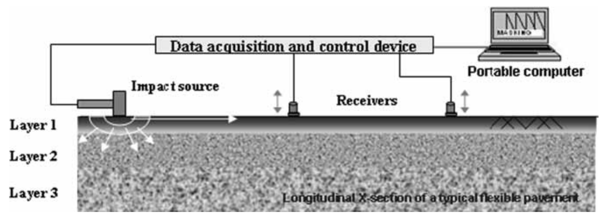

Figure 6. A typical SASW test set-up.

from both forward and reverse configurations of the source-receiver system. Common receiver midpoint or common source test geometries (Nazarian and Stokoe 1986, Hiltunen and Woods 1989) are generally used to collect a number of data sets and stacking technique is applied to increase the signal to noise ratio (SNR). To calculate the phase-difference between the signals from the two receivers, the data is transformed to frequency domain using fast Fourier transform algorithm. Phase-difference values at each frequency gives the corresponding R-wave phase velocities V_R (Nazarian and Desai 1993). Bergman (1948) proposed a relation between V_R and V_s as follows:

$$\frac{V_s}{V_R} = \frac{1 + \nu}{0.87 + 1.12\nu} \tag{4}$$

where; ν is the poisson's ratio of the material medium. This basic equation can be suitably used for V_s calculation (Sanchez-Salinero *et al.* 1987, Roesset *et al.* 1990, Nazarian *et al.* 1999).

Forward modeling consists of generating a theoretical dispersion curve (TDC) for chosen values of layer properties. A computationally adaptable formulation, known as Haskell–Thomson's propagator-matrix method (Thomson 1950, Haskell 1953, Jones 1955) was established in early 1950's, that related the R-wave dispersion property in a homogeneous multi-layered media (such as asphalt pavement) with its elastic properties. This formulation gives the surface wave dispersion function as a product of layer matrices that relates the displacement and stress components at any interface to those for the deepest layer and is a function of the frequency and wave-number. A determinant search technique computes the values of the wave-number for which the determinant is zero (called as eigen values). For asphalt pavement profiles, no real eigen values exist above a specific maximum frequency. This leads to complex-domain computation of eigen values.

Propagator-matrix method became the basis for most of the theoretical forward modeling approaches of R-wave dispersion, such as: stiffness-matrix method, linearised stiffness-matrix method, etc. (Kauseland and Roüsset 1981, Storme *et al.* 2005). Various tools utilising finite-difference (Hossian and Drnevich 1989), Green's functions (Kausel and Peek 1982, Hisada 1994), wavelet transforms (Kim and Park 2002), etc. have also been applied to solve for dispersion characteristics.

In the inversion process, a suitable analysis tool is employed to compare EDC with the TDC. This gives the final vertical V_s profile of the test site which is then used to determine the stiffness profile for the test-site using equation (3). It should be noted here that the estimated asphalt modulus is sensitive to the test temperature and hence proper correction needs to be applied (Mallick *et al.* 2005, Gucunski *et al.* 2006).

Besides the automated methods for EDC generation (Nazarian and Desai 1993), schemes employing manual masking of raw data are also used which demand experience in use. Analysis tools such as statistical/curve-fitting methods (Yuan and Nazarian 1993), simulation (Ganji *et al.* 1998), ANN (Williams and Gucunski 1995, Gucunski *et al.* 2000, Nazarian *et al.* 2004), GA (Al-Hunaidi 1998) and more recently simulated annealing (Beaty *et al.* 2002, Ryden and Park 2006) have been used for automation of the inversion process. A different signal processing methodology employing continuous wavelet transform of surface waves has recently been used for void detection in pavements (Shokouhi *et al.* 2003). Construction or testing information for the test site may also be incorporated in the inversion process for more efficient profiling.

A number of variants of the basic evaluation method, as described above, have been reported. Park *et al.* (1998) introduced the multi-channel analysis of surface waves (MASW) method which makes use of more than two receivers to detect the other higher modes present in the surface waves. This method attempts to generate a more accurate dispersion curve representation using pattern recognition techniques which also automate the procedure. Multichannel simulation using one receiver (MSOR) method for pavement characterisation (Ryden *et al.* 2001) is based on MASW technique, and utilises a single fixed receiver and moving source (or vice versa) to take a number of data sets, and then use them to simulate multichannel data. A convenient testing and fast analysis procedure gives more efficiency to this method. Ultrasonic surface waves have also been successfully employed to estimate the depth of surface cracks and elastic modulus of asphalt layer (Mallick *et al.* 2005, Mallick and Nazarian 2006).

4.2 Electromagnetic wave method

In electromagnetic wave method, the reflections of the transmitted waves caused by any changes in the material and/or layer properties, are recorded and analyzed to deduce the required information. Infrared thermography (IRT) and ground penetrating radar (GPR) can be identified as two types of NDE techniques that find application on asphalt pavement, and are discussed in the following.

4.2.1 Infrared thermography. IRT employs an infrared camera to capture the thermal image from top of the pavement. The subsurface defects affect the heat flow within the pavement, and this in turn affects the temperature distribution of the concerned area. The IRT can capture images of a larger area quickly. To some extent, this method can capture the location and extent of sub-surface distresses in the form of cracking, segregation, ageing and construction non-homogeneity, etc. (Gordon *et al.* 1998, Mahoney *et al.* 2000, Oloufa *et al.* 2004). The test results can also get affected by various field conditions such as time of testing, cloud cover, wind flow, pavement surface texture, solar radiation and sub-surface conditions (moisture, frost penetration, etc.).

4.2.2 Ground penetrating radar. GPR consists of a radar device that transmits electromagnetic pulse. The subsurface anomalies, voids, location of dowel bar (applicable to concrete pavement), void below pavement, depth of bedrock, ground water level, pavement moisture content and frost penetration depth, type of subgrade soil strata, etc.

can be estimated by this method (Al-Qadi 1992, Saarenketo 1997, Scullion and Saarenketo 1997, Morey 1998, Saarenketo and Scullion 2000, Al-Qadi and Lahouar 2005, Maser *et al.* 2006). Researchers have found that the estimation of surface-layer thickness of asphalt pavement by this technique is good (Maser 1999, Saarenketo and Scullion 2000, Maser *et al.* 2006). GPR facility also provides a continuous data record for the full length of the test section, thereby giving direct information of test section at each point.

Figure 7 schematically explains the working of a typical GPR system (Maser *et al.* 2006). Depending on the type of antenna used, a GPR system can be of two types; air coupled (or horn-antenna) and ground coupled system. The two are further discussed in the following.

- In air-coupled system, the antenna is typically placed 150–500 mm above the pavement surface (Al-Qadi and Lahouar 2005). The test set-up is mounted on a vehicle which operates on test pavement at specified speed. Operating speed depends on antenna type used and required frequency of data recording. The survey can be conducted at a speed of even 60 km/hr and the typical depth of penetration in the pavement is of the order of 0.5 to 0.9 m (Saarenketo and Scullion 2000). Accuracy of this method depends on the resolution of the time scan and the dielectric constant of the material (Maser *et al.* 2006). As the equipment moves on the pavement surface, sequence of wavefronts are generated, and from this, the layer-boundaries can be directly visualised from the pictorial output of the collected wavefront data (Maser *et al.* 2006).

The relative reflection amplitude at the nth interface can be expressed as (Al-Qadi and Lahouar 2005)

$$\frac{A_n}{A_{\text{inc}}} = \frac{\sqrt{\epsilon_{r,n}} - \sqrt{\epsilon_{r,n+1}}}{\sqrt{\epsilon_{r,n}} + \sqrt{\epsilon_{r,n+1}}} \left[\prod_{i=0}^{n-1} (1 - \gamma_i^2) \right] \mathrm{e}^{-\eta_o \sum_{i=0}^{n} \frac{\sigma_i d_i}{\sqrt{\epsilon_{r,i}}}} \tag{5}$$

for $n = 1, 2, \cdots N$, where, N is the number of pavement layers, A_n is the reflection amplitude at the nth layer interface, A_{inc} is the amplitude of the incident GPR signal, $\epsilon_{r,n}$ and σ_n are the dielectric constant and conductivity of the nth layer, respectively, γ_i is the refection coefficient at the ith interface, η_o is the wave impedance of free space. A_{inc}, can be estimated by collecting GPR data over a large and flat copper plate, placed on the test surface (Al-Qadi *et al.* 2001, Al-Qadi and Lahouar 2005). The reflection from plate's surface is assumed to be equal and opposite to the incident GPR signal. The dielectric constant of the first layer can

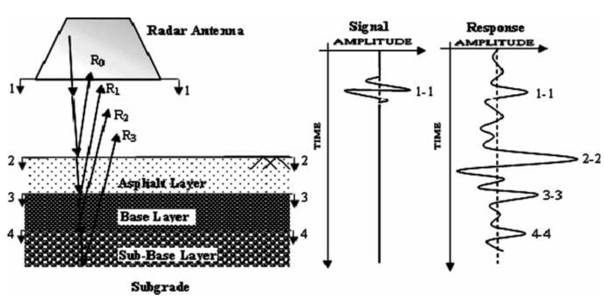

Figure 7. Working of a typical GPR system.

be obtained from the equation (5) by putting $n = 0$, thus,

$$\epsilon_{r,1} = \left(\frac{1 - \frac{A_o}{A_{\text{inc}}}}{1 + \frac{A_o}{A_{\text{inc}}}} \right)^2 \tag{6}$$

Similarly, the expression for di-electric constant for the second layer ($\epsilon_{r,2}$) can be obtained by putting $n = 2$ in equation (5) and using the value of $\epsilon_{r,1}$ obtained form equation (6) and so on. The depth of the ith layer of a pavement can be obtained as (ASTM D4748 2003, Al-Qadi and Lahouar 2005, Loizos and Plati 2007):

$$d_i = \frac{ct_i}{2\sqrt{\epsilon_{r,i}}} \tag{7}$$

where, d_i is the thickness of the ith layer, t_i is the two way travel time of the electromagnetic wave through the ith layer, c is the speed of light in free space (approximately equal to 1×10^8 m/s) and $\epsilon_{r,i}$ is the dielectric constant of the ith layer. Its important to note here that these simple equations assume the asphalt layer to be homogeneous and isotropic with uniform dielectric constant. However, this assumption may not be always true for old or faulty pavement layers, thereby decreasing the accuracy evaluation (Saarenketo and Scullion 2000, Lahouar *et al.* 2002, Loizos and Plati 2007).

- In ground coupled system, the antenna remains in full contact with the ground. Loss due to reflection at the pavement surface being less, the depth of penetration is more. But, this system has a limitation on the vehicle speed (Al-Qadi and Lahouar 2005).

Generally, the common mid-point (CMP) technique for ground coupled system is employed to increase the SNR (signal to noise ratio) (Lahouar *et al.* 2002). In this technique, two antennas are used, one is transmitter and the other is receiver. Initially, the two antennas are kept adjacent to each other and then they are moved equal distances step-wise from the mid-point and the data on travel time (t_i) is collected. The following relationship can be used (Maser *et al.* 2006)

$$t_i = \frac{2}{V}\sqrt{(x(i)^2 + d^2)} \tag{8}$$

where, i is the scan number, V is the velocity of wave in pavement layer, d is the thickness of the pavement layer, $x(i)$ is the distance of any antenna from the CMP in ith scan and t_i is the travel time of the pulse wave from transmitter to receiver. The resolution of this test largely depends on waveform sampling and the ability of distinction between the reflected wave from the direct wave (Maser *et al.* 2006).

Maser *et al.* (2006) found that the ground coupled GPR system can reliably identify a pavement thickness which is larger than 100 mm, whereas air-coupled GPR can measure asphalt layer thickness of the order of 50 mm. It is argued that amplitude based characterisation through GPR may not be very appropriate for evaluation of pavements, because amplitude is also affected by various factors (for example, height of the antenna) other than the reflection from the interfaces (Gordon *et al.* 1998). To take care of this problem, Mesher *et al.* (1995) developed multi-antenna bi-static array GPR system specific to evaluation of pavement thickness.

The GPR technology is developing at a rapid pace, in both towards the equipment as well as analysis algorithm. Some of the improvements towards the equipment include stepped frequency, synthetic aperture radar, switched antenna array, etc. (Maser 1996, Gordon *et al.* 1998, Loulizi *et al.* 2003); whereas those towards the analysis algorithm include application of wavelet analysis, deconvolution and data migration, ANN, fuzzy logic, optimisation, etc. (Maser 1994, Gordon *et al.* 1998, Loulizi *et al.* 2003, Al-Qadi and Lahouar 2005).

Various researchers have found different levels of error in estimation of pavement thickness by GPR technology. Calibration of GPR using field core data is expected to enhance the accuracy achieved (Willett *et al.* 2006, Loizos and Plati 2007). In general, it is seen that the thickness estimation is more erroneous for thin layers (Gordon *et al.* 1998, Al-Qadi and Lahouar 2005, Loizos and Plati 2007). For newly built pavements, a greater accuracy has been reported (Al-Qadi *et al.* 2003, Maser *et al.* 2006). A listing of accuracy levels observed by different researches in estimating the thickness is presented in (table 2).

Some of the present limitations of the GPR technique, as identified by various researchers, are summarised as follows:

- Interpretation of GPR data requires experience and is subjective in nature (Loulizi 2001).
- An inability to clearly differentiate between layers of similar materials is present due to a low contrast in their dielectric properties (Al-Qadi *et al.* 2001, Loulizi 2001, Al-Qadi and Lahouar 2005).
- An increased attenuation of electromagnetic signal may be caused in the pavement layers due to presence of salt, water or high iron content in slag aggregates. As such, the analysis results may be misleading sometimes, specially for concrete pavements (Mesher *et al.* 1995, Gordon *et al.* 1998, Al-Qadi *et al.* 2001, ASTM D4748 2003, Maser *et al.* 2006).
- With high frequency antenna, resolution of recorded data gets enhanced, but the depth of penetration reduces (Gordon *et al.* 1998, Al-Qadi and Lahouar 2005, Loizos and Plati 2007).

4.3 Closing remarks

Various wave propagation techniques, as applied for evaluating asphalt pavements, have been discussed in the previous sections. (Table 3) summarises some of the salient characteristics of these techniques. The ultrasonic waves travel much slower (about 100,000 times) than electromagnetic waves providing means to display information in real-time

Table 2. Accuracy in pavement thickness estimation using GPR.

Layer (thickness)	*Order of error† (± %)*	*References*
Asphalt (<50 mm)	0.4–10.3	Willett *et al.* (2006)
Asphalt (51–150 mm)	3.6–9.6	Maser (1996), Fauchard *et al.* (2003), Loulizi *et al.* (2003), Loizos and Plati (2007)
Asphalt (150–250 mm)	5.9–9.6	Maser (1996), Loizos and Plati (2007)
Asphalt (250–380 mm)	6.8–7.5	Maser (1996), Lahouar *et al.* (2002)
Asphalt (380–500 mm)	7.5	Maser (1996)
Granular base (150–330 mm)	12	Maser (1996)

† As compared to average core-thickness values.

Table 3. Operating characteristics of different wave propagation techniques for pavement evaluation.

NDE technique	*Depth reachable*	*Wave type*	*Frequency range*	*References*
IE	Up to 0.2 m	Stress	1–80 KHz	Nazarian *et al.* (1993), ASTMC 1383 (2000)
SASW	Up to 2 m	Stress	50 Hz–50 KHz	Nazarian *et al.* (1988, 1993)
UPV	–	Stress	50–100 KHz	Nazarian *et al.* (1993), ASTM C597-02 (2002), Jiang *et al.* (2006)
GPR ground-coupled	Up to 3 m	Electromagnetic	400 MHz–1.5 GHz	Saarenketo and Scullion (2000), Hugenschmidt (2002)
GPR air-coupled	0.5–0.9 m	Electromagnetic	250 MHz–3.0 GHz	Saarenketo and Scullion (2000), Loizos and Plati (2007)
IRT	≈ 0.5 m	Electromagnetic	$\approx 10^4$ GHz	Oloufa *et al.* (2004)

(David *et al.* 2002, Shull 2002). However, due to their high frequency and low energy response, they have been primarily used for evaluation of top asphaltic layer. User experience is required to choose a suitable NDT technique specific to the test objective and field conditions. For example, specialised techniques such as IRT can be successfully employed for evaluation of airport pavements (Moropoulou *et al.* 2001) as they make use of large temperature variations occurring there in.

5. Closure

It is argued that the stress wave related equipment (say, IE) work well for concrete pavements, because the signal impedance contrast between concrete and the base material is sharper as compared to asphalt pavements (Maser 2003, Maser *et al.* 2006). On the other hand, for GPR methods, the dielectric contrast between asphalt and the base material is sharper than between concrete and the base material (Maser *et al.* 2006). Since GPR testing is continuous, future automated versions of GPR technology has a strong potential of being developed as a routine pavement evaluation equipment (Saarenketo and Scullion 2000).

The accuracy achieved by employing individual NDE techniques can be improved by combining data from two or more methods in the final analysis (Abdallah *et al.* 2001). Similarly, FWD (or RDD) and GPR have been found as a good combination of technologies for measuring asphalt layer thickness as well as stiffness (Saarenketo and Scullion 2000, Hanna 2002, Lee *et al.* 2006). IRT can also be used to decide GPR survey locations thereby enhancing its effectiveness in evaluation (Weil 1993). The SASW and FWD methods are generally more suitable for evaluation of top and bottom layers respectively. This is due to a difference in the basic evaluation method and the loading which generates different magnitude wavelengths that give information on different depths of the pavement. So, if data from these two methods are combined, a more accurate and complete pavement profile may be obtained. Equipment such as seismic pavement analyzer, effectively employ this principle (Abdallah *et al.* 2006).

The present paper has discussed about various methods available for NDE of an asphalt pavement structure. A gradual shift can be noticed towards the usage of automatic equipment and adoption of quick evaluation techniques as compared to the conventional ones. A carefully chosen combination of different NDE methods can readily enhance the accuracy and reliability in the evaluation of pavements.

6. List of symbols

t = thickness of the layer
f = frequency
β = correction factor
E = Elastic modulus
ν = Poisson's ratio
V_p = Velocity of P-wave
V_s = Velocity of S-wave
ρ = Density

N = number of pavement layers
A_n = reflection amplitude at the nth layer interface
A_{inc} = amplitude of the incident GPR signal
$\epsilon_{r,n}$ = dielectric constant of the nth layer
σ_n = electrical conductivity of the nth layer
γ_i = refection coefficient at the ith interface
η_o = wave impedance of free space
d_i = thickness of the ith layer
t_i = two way travel time of the electromagnetic way through the ith layer
c = speed of light in free space (approximately equal to 3×10^8 m/s)
$\epsilon_{r,i}$ = dielectric constant of the ith layer
i = scan number
V = velocity of wave in pavement layer
d = thickness of the pavement layer
$x(i)$ = distance of any antenna from the CMP in ith scan
t_i = travel time of the pulse wave from the transmitter to receiver

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