

Pavement Structural Evaluation

Production Level FWD Back-Analysis Using the Full Time History

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Keywords: Falling Weight Deflectometer, back-analysis, pavement structural evaluation

ABSTRACT:

When pavement structural evaluation is carried out using the Falling Weight Deflectometer, the full displacement-time history for each geophone is measured in the field, but for normal production only the peak deflection data are stored for subsequent processing. Velocity and acceleration information can however be readily captured. A preliminary study has been made to examine whether useful time-dependent parameters could be extracted from the full time history and used by practitioners. Software has been developed for capturing wave speeds and correlating with approximate moduli of the surface layer, to complement conventional back-analyses. Surface wave speeds appear promising indicators to allow rapid distinction of near surface material types and structural condition, at no additional cost or time.

1 INTRODUCTION

The Falling Weight Deflectometer (FWD) generates and records a pavement deflection bowl similar to that produced by a heavy vehicle travelling at highway speeds. Back-analyses of the deflections, using elastic theory are commonly used for structural evaluation for asset management and/or design of remedial treatments.

The field programs used to capture FWD results can be set to display at the time of test, the full time history of all deflection sensors. Normally only the peak deflections of the FWD are recorded on disk and the bulk of the data is discarded, because it is not used in the standard pseudo-static back-analysis. There has been considerable research into the mathematics of a full dynamic analysis (Ullidtz & Coetzee, 1995) but the methods require substantial computing capacity as well as incorporation of additional dynamic parameters (viscous and visco-elastic properties). For these reasons, there appears to be no general use of dynamic pavement back-analysis by practitioners. A simplistic approach has therefore been taken in this research, to establish a procedure for capturing deflection wave speeds, and determine whether any significant information can be extracted from the full time history plot. Simple parameters that can be used empirically to enhance mechanistic analyses have been sought. The principal objective is to distinguish between pavement types (namely composition of the surface layer – whether structural asphalt, friction course, cement stabilised or unbound granular) and therefore greatly reduce the frequency of destructive testing.

2 STRUCTURAL EVALUATION

Structural evaluations of FWD deflection bowls utilise back-analyses that tend to compute the overall stiffness of each layer, i.e. a function of modulus (E), thickness (h) and Poisson's ratio (μ):

$$\text{Stiffness} = h^3 E / (1 - \mu^2) \quad (1)$$

Back-analysis can determine E correctly, only if h is known. Therefore a separate determination of either E or h is required. Normally the layer thickness (h) is determined from test pitting or GPR, but an alternative means of estimating the nature and modulus of the surface layer would be very advantageous because the use of destructive testing or other techniques could be minimised.

2.1 Case Studies

Full time histories of FWD deflections were recorded on pavements of known composition, namely

- Cement stabilised basecourse with thin chip-seal surfacing
- Sound, new unbound granular basecourse with thin chip-seal surfacing and firm subgrade
- Distressed unbound granular basecourse with thin chip-seal surfacing and weak subgrade
- Thin (30 mm) asphaltic concrete on unbound granular basecourse
- Structural asphalt (80 mm) on unbound granular basecourse

Figures 1 and 2 show typical results from the structural asphaltic pavement and the distressed unbound basecourse, respectively. Figure 1 (the stiffest pavement in the set) shows that the peak deflection at each sensor is reached at about the same time in the first 4 geophones (spaced at 0, 300, 450 and 600 mm). Figure 2 however (the weakest pavement studied) shows a much more pronounced time delay between the peaks of successive sensors.

It is evident that the lateral propagation of the deflection bowl is much more rapid in the asphaltic concrete, suggesting that the surface wave speed may provide some indication of the nature of the upper layer of the pavement. The records were processed to determine the time of the peak displacement at each sensor. The distance between sensors was then used to calculate the surface peak wave velocity.

It is interesting to note from these graphs that at the time of peak deflection at the central geophone (indicated with a vertical line on Figures 1& 2), the outermost geophones have not begun to deflect. Also for the case of the unbound granular pavement, Figure 2, the outer geophone peak deflection occurs only after the loading pulse has almost completely attenuated. This confirms, as might be expected, that the FWD transient loading (designed to simulate a heavy vehicle wheel travelling at highway speeds) causes a compression wave to radiate from the plate. When determining layer moduli, the assumption used in most back analyses is that the load and all deflections occur at the same time. The result will be to substantially overestimate the subgrade modulus in any pavement where the peak deflections occur at significantly different times. Actual strains occurring at the top of the subgrade will be much higher than those calculated from conventional elastic theory for static loading. This finding was supported by a recent study using strain gauges at the top of the subgrade of an instrumented pavement (Arnold et al, 2001). Although that study required corrections for a nominal "rigid base" at a depth of 1.5 m, measured strains were often 50% greater than those predicted by conventional back analyses. The fact that back-calculated strains may not correspond closely with actual strains does not affect the mechanistic-empirical design procedure. That is provided of course the empirical component, ie the strain (or stress) criterion used for design, has been established using a back-calculation method which has a similar basis to that used for forward design.

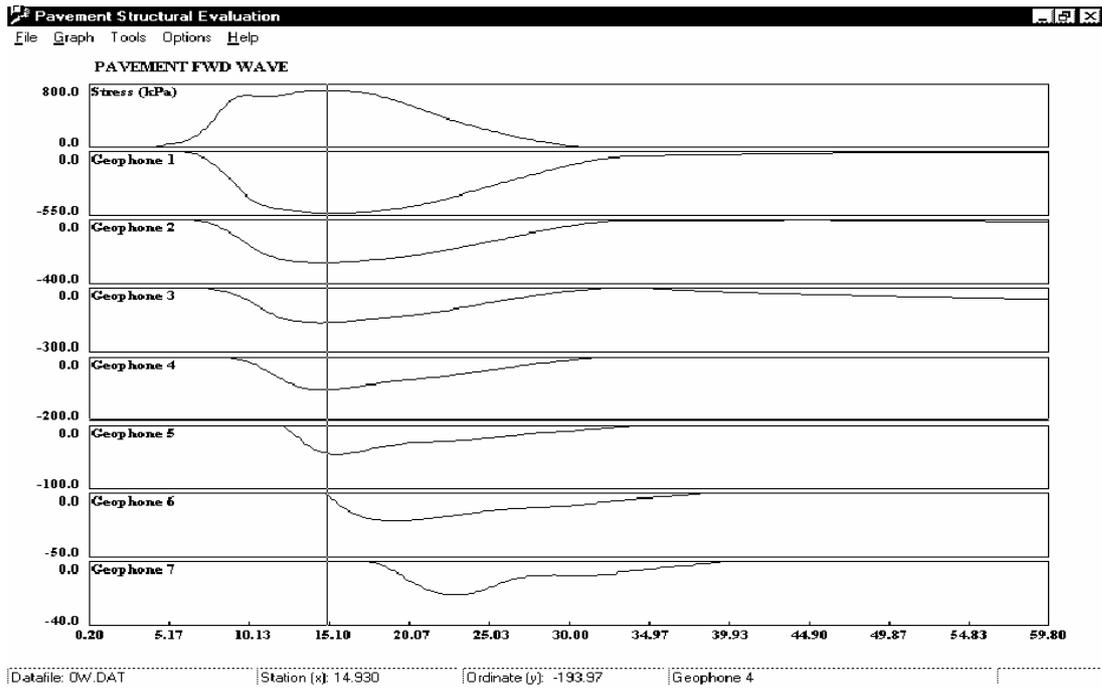


Figure 1: FWD full time history record for an asphaltic concrete pavement

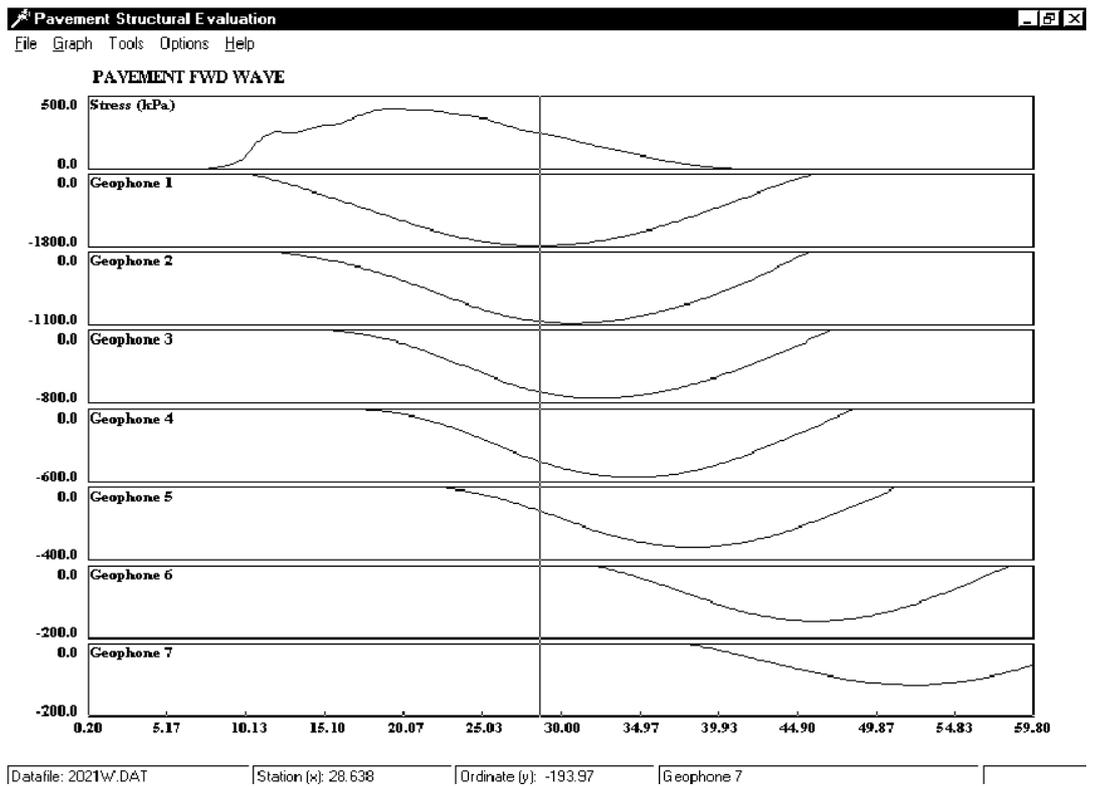


Figure 2: FWD full time history record for an unbound granular pavement

Figures 1 and 2 indicate that the wave speeds tend to be high close to the FWD loading plate but become asymptotic to residual values at the larger offsets. Various parameters that could be determined from the full time history of movement were considered. Differentials were used to calculate vertical velocities and accelerations both on loading and rebound, but lateral wave speeds were the only parameters which, from this preliminary study, appeared to present any useful correlation with surface layer type. As surface irregularities under the loading plate could affect the propagation of the surface wave, the second sensor rather than the central sensor located within the loading plate, was adopted as the origin for time measurements. The peak deflection wave velocity (v_{pd}) was then calculated, taking simply the time for the maximum deflection to reach sensors at successive offsets. A similar procedure was used for determining the velocity (v_{fb}) corresponding to the first break at each sensor, in the same manner as used to pick p wave arrivals from seismic records.

2.2 Capture of Wave Speed.

Software was developed to record in the field, the relevant wave speed information at the time of each FWD test. The velocity of propagation of the peak deflection wave, can be readily determined because only one peak exists for each geophone.

The velocity of the first break detected at each geophone is more difficult to define uniquely. For geophones close to the plate, the first break is a downward deflection, whereas for those further away there tends to be an initial small upward deflection prior to the main downward deflection.

If each set of full time histories is plotted, a sensible average first break velocity may be found manually from the slope of the line on a time-distance plot between successive upward breaks, and comparing this with the plot of successive downward breaks separately.

When establishing an algorithm for picking first breaks, a threshold “noise” level needs to be set because a noise level of about 1-2 microns is often present on full time history plots. It is important to record the sign of the break whether upward or downward. Software can then be confined to determine first break velocities only between relevant geophone records which have the same first break direction. Because the time delay between adjacent geophones is often not large compared with the time sampling interval, it is necessary to rely more on speeds calculated between geophones which are distant rather than adjacent. A weighting factor proportional to the distance between geophones is useful when determining the overall average first break velocity from software-generated combinations (15 inter-geophone velocity samples from a 7 geophone machine or 28 from a 9 geophone machine).

3 ANALYSIS

The peak deflection velocity can be measured with good precision from the FWD test, however it does not correspond directly with any of the conventional measures for wave speed. Filters and the close proximity to the impact source are likely to invalidate direct estimates.

The Rayleigh wave velocity (close to the shear wave velocity, v_s) can be used to determine the surface layer elastic modulus (E_s) from:

$$E_s = 2 \rho v_s^2 (1 + \mu) \quad (2)$$

where:

μ is Poisson's Ratio (typically 0.35 for unbound materials and 0.2 for cement bound)

ρ is the mass density of the material (typically about 2.0 t/m³).

The seismic p wave velocity is difficult to measure accurately with the FWD (because of the rubber buffers which soften the first break, as well as the factors noted above). But the first break in the FWD recording may approximate to a lower bound for the p wave velocity (v_p), in which case the elastic modulus (E_p) would be determined from:

$$E_p = \rho v_p^2(1+\mu)(1-2\mu)/(1-\mu) \quad (3)$$

Both the above equations indicate that moduli measured from surface waves are likely to be proportional to the square of the wave velocity (assuming the other factors have small influence). Because the modulus of the surface layer rather than its wave speed is of interest for pavement evaluation, further studies may allow approximate determination of elastic moduli in the surface layer by deriving empirical constants related to the square of either the first break wave velocity or the peak deflection velocity. Meanwhile, Figure 3 gives a plot of the surface wave speeds for the various types of pavement included in this study. Initial results are not perfect, but are encouraging, as each material type does group to some extent.

More importantly, for any given road which is expected to have a single pavement profile, the plot gives some appreciation of the relative quality of the near surface layer(s).

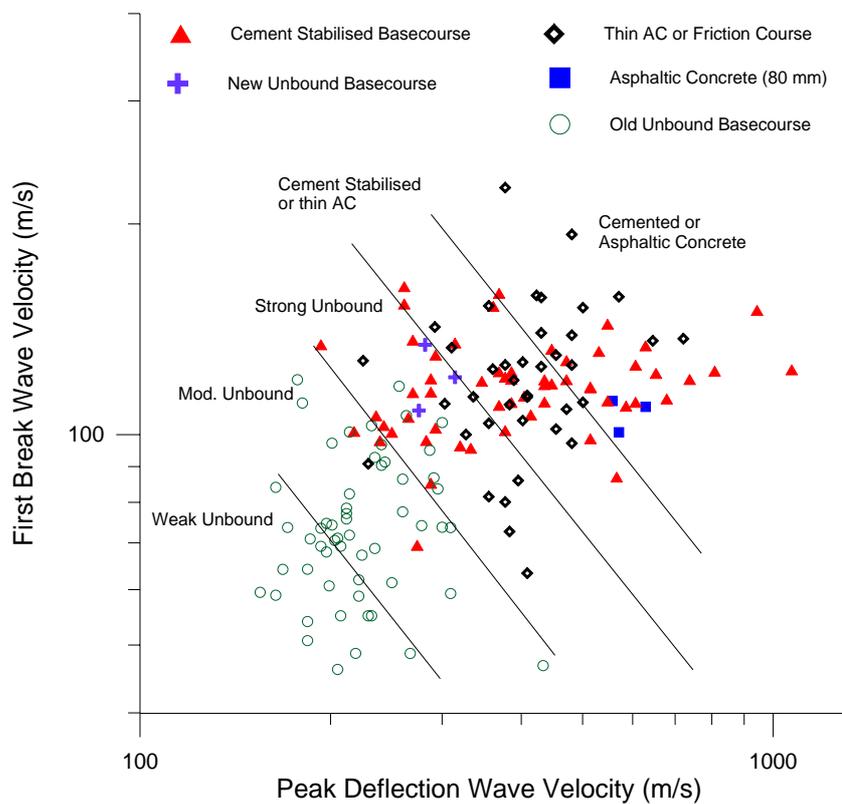


Figure 3: Estimation of surface layer type from surface wave velocities.

4 CONCLUSIONS

Currently, most practitioners discard about 99% of the pavement response data from an FWD test because only the peak deflections are recorded. Wave speeds through the surface layer can be recorded at no additional time or cost requirement, and used to supplement conventional analyses. Software for capturing wave speed information without needing to store the full time history has been developed. A preliminary study using a small number of pavements has been carried out. More test data are required to determine the reliability with which the various material types can be distinguished. Also it will be necessary to fully test the data capture methods for all situations, including thick structural asphaltic pavements. However, for unbound granular pavements with thin seals, the surface wave speeds (or approximate moduli determined from them) are sufficiently sensitive to provide the practitioner with a useful tool for the following:

- to adjust assumed thicknesses of the surface layer (Layer 1), by comparing conventional back-calculated moduli with the wave speed (calibrated as data becomes available for the local materials and construction conditions),
- to provide sub-sectioning into uniform intervals for modelling and deciding the stationing limits for applying information from individual cores or test pits
- to allow differentiation between unbound and cement stabilised basecourses. The latter distinction is critical for realistic modelling using mechanistic-empirical analysis. It is also required when using the AASHTO Guide or HDM models, for deciding which condition is applicable for determination of structural number
- to identify locations which will provide the most appropriate sites for test pits in order that destructive testing can be minimised by focusing on specific targets, particularly where the surface condition is masked from effective visual inspection because of a recent seal coat

5 REFERENCES

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