

Use of FWD Data for Prediction of Bonding Between Pavement Layers

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A new back-analysis method is suggested to assess the bonding condition between bituminous layers in addition to the pavement layer stiffnesses, from Falling Weight Deflectometer (FWD) test results. The shear bond stiffness at the interface is considered as a variable affecting the FWD deflections and therefore back-calculated in a similar manner to a layer stiffness. A sensitivity analysis carried out on theoretical pavements showed that, assuming full bonding between bituminous layers instead of the actual values will change the FWD back-analysed bituminous and sub-base stiffnesses by up to 50% and will not affect the sub-grade stiffness.

Case studies are presented describing the evaluation of several flexible pavements using the new back-analysis process. Poor bonding was suspected between the bituminous layers due to the following reasons; i) the conventional back-analysis method predicted low effective stiffness for the bituminous layers, ii) separation of the layers during coring was noticed, iii) acceptable laboratory measured bituminous stiffnesses were found.

Re-testing the above pavements after six months indicated an improvement of bonding condition between bituminous layers after trafficking and at higher testing temperatures.

Keywords: Pavement Evaluation, FWD, Back-analysis, Bonding Condition

INTRODUCTION

The Falling Weight Deflectometer (FWD) is widely used for flexible pavement evaluation and assessment of residual life. The FWD applies an impact load to the pavement surface, to simulate the action of traffic, and the pavement response in terms of deflections is measured at several radial locations from the load centre. The pavement is commonly modelled as a multi-layered linear elastic system, where each layer is described by its stiffness and Poisson's ratio. The

deflection bowls obtained from the FWD testing are used to back-calculate the stiffness of the pavement layers, by matching the measured deflections to computed values. The thickness of pavement layers, which can be determined by a coring or ground radar survey, is required for the analysis process. The back-calculated stiffnesses are utilised to determine critical pavement strains under a standard axle load, and therefore the pavement life employing the appropriate failure criteria. The residual life is then compared with the future traffic flows to decide the

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pavement strengthening requirements. Additional material testing is usually performed both on samples cored from the pavement, and in-situ to assist in characterisation of material behaviour. Indirect Tensile Stiffness Modulus (ITSM) in the Nottingham Asphalt Tester (NAT) (Cooper and Brown 1989) for the bituminous materials, and in-situ Dynamic Cone Penetrometer (DCP) tests (Kleyn, et al 1982) for the unbound materials, are commonly used.

This paper proposes and illustrates a new back-calculation process for assessment of the bonding condition between bituminous layers, in addition to the layer stiffnesses, from FWD test results.

BONDING CONDITION BETWEEN BITUMINOUS LAYERS

Weak bonding between bituminous pavement layers may develop during construction due to poor quality control, lack of tack coat, contamination of the lower layer or laying in cold weather (Lepert, et al 1992). Furthermore a poorly compacted sub-base may cause bituminous layer slippage, since sufficient support is not provided during construction.

At present many testing methods have been used to determine the bonding condition between layers such as, seismic testing analysis (Lepert, et al 1992), torque bond test, pull off test, wedge splitting test (Tschegg, et al 1995) and shear box test (Uzan, et al 1978).

The seismic pavement analyser is an in-situ non-destructive testing technique whereas the torque bond and pull off tests are destructive and more suitable for measuring the bond strength of thin overlays. The wedge splitting and shear box tests enable fundamental measurements to be made with precise control of normal and shearing forces, but their use are normally restricted to the laboratory.

Bituminous layer de-bonding will reduce the back-calculated stiffness from FWD test results (DOT 1994), which is the effective stiffness that reflects the pavement behaviour and not unique bituminous material properties.

A numerical solution of a multi-layer pavement system requires knowledge of the boundary condition

between layers, in order to calculate the structural response to surface loading.

Assuming the interface between two pavement layers as a thin material with a shear modulus G and a thickness t , therefore the shear stress (τ) at the interface due to pavement surface loading can be expressed as;

$$\tau = G \cdot \gamma$$

where,

γ is the shear strain of the thin interface material ($\Delta u / t$),

Δu is the relative horizontal displacement on both sides of the interface.

Hence

$$\tau = (G/t) \Delta u$$

or

$$\tau = K_s \Delta u$$

where, K_s is the shear or bonding stiffness at the interface (MN/m^3).

This equation represents Goodman's constitutive law to describe the interface behaviour (Goodman et al 1968), and has been used in many multi-layer analysis programs such as BISAR.

Back-calculating the bond stiffness (K_s) from FWD deflection bowls in a similar manner to layer stiffnesses has several advantages over some of the above methods, such as:

- i) FWD testing is routinely performed for pavement layer evaluation.
- ii) FWD is non-destructive testing.
- iii) The in-situ bond stiffness is calculated as a fundamental engineering value, rather than a qualitative prediction, for use in pavement analysis and overlay design.

However, this method of bonding assessment is suggested to supplement and not replace any of the above testing techniques. For example the FWD can be used over a large area to map out the variations in bond stiffness, before carrying out destructive methods such as the torque test. The later can be used to measure the bond strength in a limited number of characteristic locations.

TABLE I Hypothetical Pavement Layer Stiffnesses

Pavement	Layer Stiffness (MN/m ²)		
	Bituminous	Sub-base	Subgrade
A	4000	100	50
B	6000	200	100
C	8000	300	150
D	10000	400	200

SENSITIVITY ANALYSIS

This section illustrates the theoretical influence of bonding stiffness on surface deflections and back-calculated layer stiffnesses in a multi-layer pavement structure.

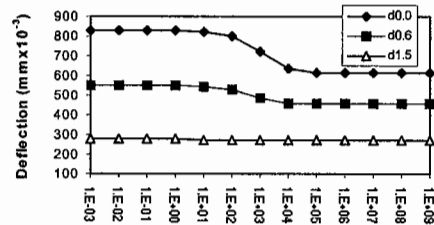
Four structures were investigated in this study to represent a wide range of flexible pavements in term of layer stiffness (E), as shown in Table I. A bituminous layer thickness of 300 mm, over 400 mm sub-base, over a subgrade of infinite depth was assumed.

For each pavement, the interface bonding stiffness (K_{s1}) at a depth of 100mm from the pavement surface was changed gradually from 10^{-3} MN/m³ to 10^9 MN/m³ and the surface deflections were calculated. Figure 1 presents the deflections at the load centre ($d_{0,0}$), and also at 600mm ($d_{0,6}$) and 1500mm ($d_{1,5}$) radial distances, for each pavement.

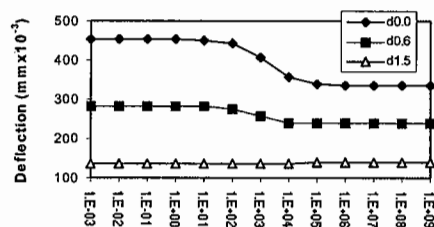
Figure 1 shows that:

- i) Values of K_{s1} less than 10 MN/m³ can be regarded as complete de-bonding whilst values above 10^5 MN/m³ can be assumed to give full bonding. Between these two limits there is a significant change in pavement response in terms of deflections near to the load.
- ii) K_{s1} has a larger influence on deflections close to the load centre but little influence on deflections away from the load.
- iii) The stronger the overall pavement structure the lower the influence of the bonding stiffness on surface deflections.

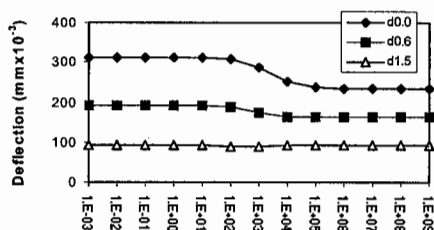
The hypothetical deflection bowls developed for each pavement by including various bonding stiffnesses, were considered as those measured by FWD



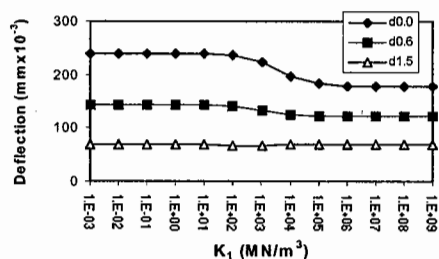
Pavement A



Pavement B



Pavement C



Pavement D

FIGURE 1 Influence of Bonding Condition on Surface Deflections

tests. They were input into the PADAL program (Brown, et al 1987) to back-calculate the three layer stiffnesses, assuming full bond existed between the bituminous layers. These back-calculated stiffnesses should carry some error to compensate for modelling the bonding stiffness as full adhesion rather than the actual values.

Figure 2 illustrates the ratio of back-calculated to actual layer stiffnesses versus the ratio of actual to assumed bonding stiffness between bituminous layers, for each layer.

Figure 2 shows that the stiffness ratio of the bituminous and sub-base layers has changed by up to 50% due to bonding stiffness modelling errors but hardly any influence of bituminous layer de-bonding on the subgrade stiffness was noticed. It also shows that, if bond is not modelled, the bituminous layer stiffnesses will be always less than the actual values. However, sometimes the analysis program tries to compensate by increasing sub-base stiffnesses to give values greater than the actual ones.

THE NEW BACK-CALCULATION PROCESS

A description of the new back-calculation process is presented in this paper; however more details can be found elsewhere (Al Hakim, et al 1997). A simple static analysis for the pavement as a linear elastic system is adopted for the back-calculation method. The stiffnesses of the bituminous, sub-base and subgrade layers, in addition to the bonding stiffness between the bituminous layers, are back-calculated from the deflection bowls. The surface deflection bowls under FWD loading, pavement material types, layer thicknesses and Poisson's ratios have to be known for the analysed pavement. Figure 3 presents the flow diagram for the two-stage back-calculation process, which involves development of a deflection database.

In the first stage the deflection database is built-up for a combination of layer stiffnesses (E) assuming a constant bonding stiffness between bituminous layers (K_{s1}). A search is then carried out using multiple regression analysis for the best equations for the pavement stiffnesses as dependent variables, with the deflections as independent variables. The models with the best correlation are employed to calculate the layer stiffnesses from measured deflections.

The second stage involves developing another deflection database, with upper layer stiffnesses slightly tuned from the value found from the first stage, and the bonding stiffness between bituminous

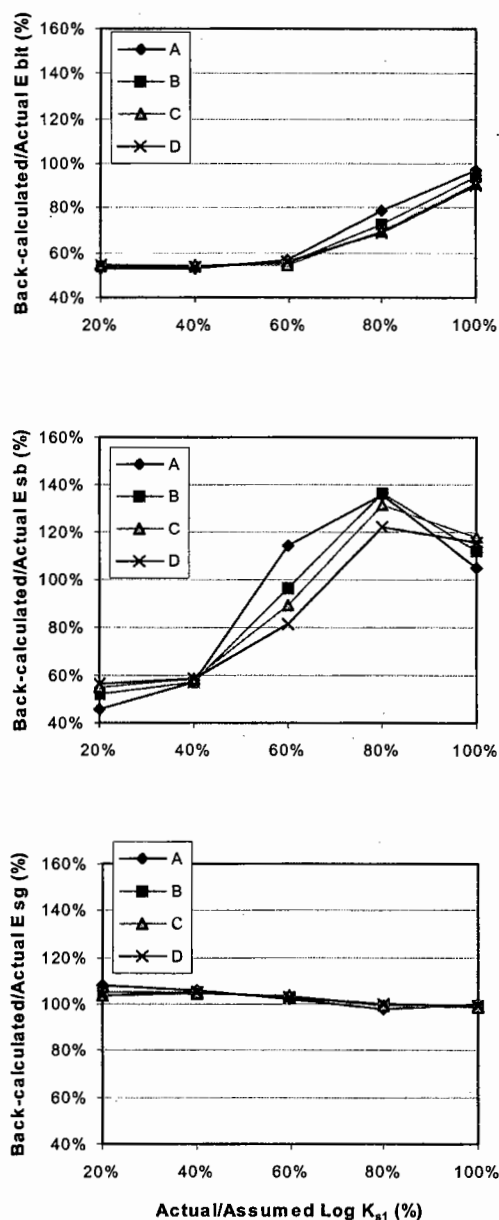


FIGURE 2 Influence of Bonding Condition on Back-calculated Layer Stiffnesses

layers (K_{s1}) varied from complete de-bonding to full adhesion ($10 \text{ MN/m}^3 - 10^5 \text{ MN/m}^3$). The back-calculated stiffnesses of the lower layers are fixed, as found from the first stage, since these are not influenced by

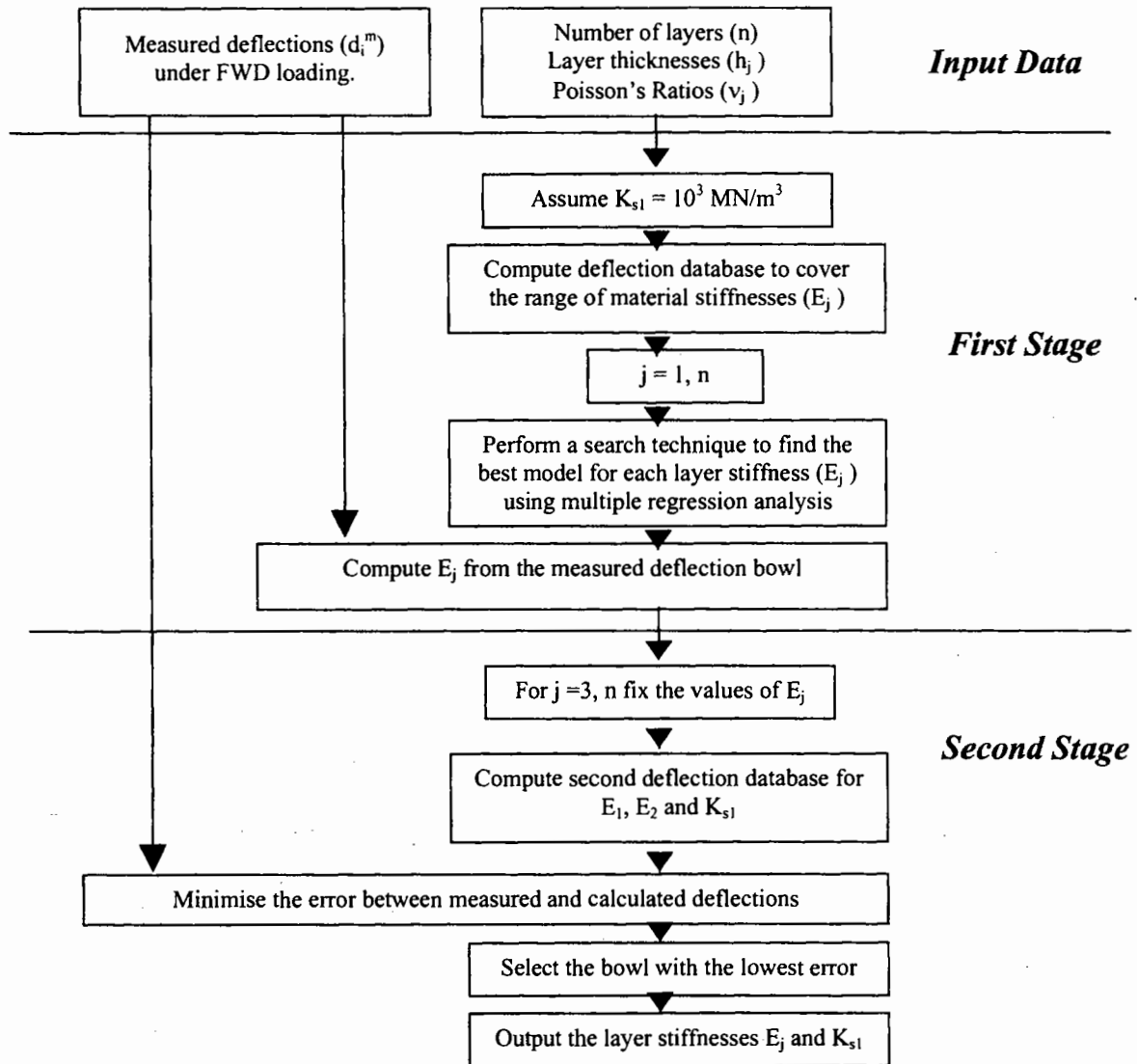


FIGURE 3 Flow Chart for the New Two-stage Back-calculation Process

bond changes between the bituminous layers, as shown earlier. The shape of the deflection bowl, the pavement strength and layer thicknesses are considered in the search technique and layer stiffness fine tuning. The calculated deflection bowl that has the lowest errors compared with measured values is considered the best for stiffness prediction.

The stiffnesses of the lower layers found from the first stage, together with the stiffness of the upper layers and the bonding stiffness found from the second stage, are taken as the back-calculated values for each deflection bowl.

TABLE II Summary of the Tested Sections, Cores and NAT Stiffness

Section	Lane	Pavement temperature during FWD testing (°C)		Core No. Within the section	Average NAT stiffness @ 20°C for each section (MN/m ²)
		1 st Visit	2 nd Visit		
A	1	5	15	1 ^a , 2 ^a , 3	11800
B	1	5	16	—	—
C	2	5	16	4 ^a , 5 ^a , 6 ^a	9900
D	2	5	17	—	—
E	1	6	16	7 ^a	—
F	1	6	16	8	4000
G	1	6	16	9 ^a	4200
H	1	6	17	—	—
I	1	8	11	—	—
J	1	8	11	10 ^a	11000
K	1	8	10	11 ^a	5400
L	2	8	10	—	—
M	2	8	11	12 ^a , 13 ^a	7300
N	2	8	11	—	—

a. Core de-bonded under the torque force of the cutter.

CASE STUDIES

Several newly constructed pavements were investigated using FWD test results for quality control purposes. Additionally, indirect tensile stiffness tests were performed in the NAT on cores extracted from the pavement.

The pavements were sub-divided into several sections according to construction details and deflection profiles as shown in Table II. Two layers of bituminous materials, with average thickness of 100mm and 140mm, over 500mm sub-base over a subgrade layer were used in the analysis. However a thicker lower bituminous layer of 210mm was constructed in Sections G, H, I, K, L and N.

Pavement temperatures were recorded during FWD testing and these are presented in Table II. Average NAT stiffnesses @ 20°C for the bituminous material were also presented in Table II. Table III presents the 50 percentile deflections for each section, normalised to an FWD pressure of 700 kPa.

TABLE III Summary of the 50 Percentile FWD Deflections (First Visit)

Section	Deflection (mm × 10 ⁻³)						
	<i>d</i> _{0.0}	<i>d</i> _{0.3}	<i>d</i> _{0.6}	<i>d</i> _{0.9}	<i>d</i> _{1.2}	<i>d</i> _{1.5}	<i>d</i> _{2.1}
A	298	244	175	122	84	59	38
B	191	167	135	103	79	58	34
C	338	277	194	134	93	65	40
D	199	175	143	113	87	66	39
E	361	274	174	110	72	52	36
F	151	132	105	82	64	48	30
G	193	162	126	98	74	56	36
H	145	126	101	81	69	53	37
I	151	128	99	81	66	50	32
J	244	190	126	85	60	45	29
K	193	159	115	82	57	42	28
L	126	111	92	76	61	47	30
M	213	172	121	86	63	46	30
N	137	119	93	75	61	49	32

Note: *d*_{0.0}, *d*_{0.3} denote measured deflections at the FWD load centre and 0.3m from the load centre respectively.

TABLE IV Summary of the 50 Percentile Back-calculated Pavement Properties Using PADAL and the New Methods (First Visit)

Section	PADAL			New Method			
	E_{Bit} (MN/m ²)	E_{SB} (MN/m ²)	E_{SG} (MN/m ²)	K_{st} (MN/m ³)	E_{Bit} (MN/m ²)	E_{SB} (MN/m ²)	E_{SG} (MN/m ²)
A	3900	80	140	20	12100	85	145
B	12000	160	130	450	19400	60	125
C	4500	70	110	10	18900	75	110
D	13000	160	160	2400	20000	50	160
E	2600	70	200	10	8700	130	210
F	12000	90	180	200	18000	60	180
G	4600	80	110	10	11000	90	110
H	9700	130	160	2000	18000	150	165
I	9000	70	130	300	19100	140	135
J	4200	70	200	50	13700	110	225
K	5100	50	250	10	18600	180	250
L	14000	140	160	700	20000	95	165
M	5500	85	210	140	16800	100	210
N	11000	90	140	10	20000	120	140

Note: $Bond\ Condition$
 Good K_{st} (MN/m³) $>10^5$
 Intermediate $<10^5, >10$
 Poor <10

The PADAL program was initially used to back-calculate the deflection bowls, assuming full bond existed between the bituminous layers, as commonly the case in the traditional analysis methods. The 50 percentile stiffness values are presented in Table IV for each section.

Poor bonding was suspected between bituminous layers in many sections due to the following reasons:

- i) The back-calculated bituminous stiffness values are lower than expected for such new material tested at low temperatures (5 to 8°C).
- ii) Layer de-bonding was noticed during coring.
- iii) The visual inspection of the cores did not suggest any voids or poor materials.
- iv) The NAT results tested at 20°C show acceptable materials with stiffnesses ranging from 4000 to 11800 MN/m² (high values are for High Modulus Base material HMB15).

The low stiffness values calculated from the FWD test can be explained by the errors in modelling the pavement structure (i.e. full bond was assumed). It is believed that bituminous layer de-bonding was due to

laying stiff materials in cold weather and/or on a weak foundation support.

Using the new method the deflection bowls were then used to back-calculate the bonding stiffness as a pavement parameter in addition to the layer stiffness. The 50 percentile pavement back-calculated parameters are also shown in Table IV.

Table IV shows various bonding stiffnesses between the bituminous layers for different sections, ranging from poor to intermediate condition. The bituminous material stiffnesses measured in the NAT, and calculated from FWD data using both PADAL and the new method, are presented in Figure 4. Some researchers have suggested (Hossain and Scofield 1992, Thom, et al 1997 and Chan and Armitage 1997) that the FWD stiffness can be 1.5 to 2 times higher than those tested in the laboratory at the same temperature, due to several reasons, including the shorter loading time during the FWD testing and restraint within the pavement. Figure 4 shows some PADAL stiffness values (at 5 to 8°C) lower than the NAT results (at 20°C). The results from the new method of calculation are more reasonable.

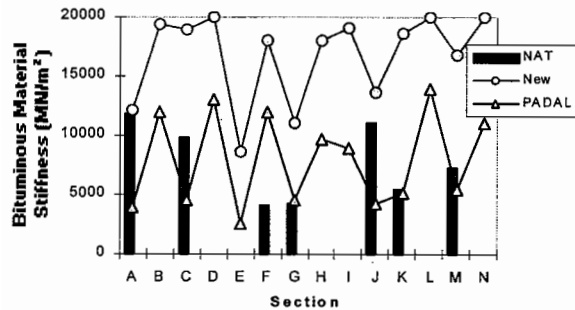


FIGURE 4 Comparison Between Average Section Stiffness Using NAT at 20°C and Both PADAL and the New Back-calculation Methods at Site Temperature

Table IV shows the back-calculated sub-base stiffnesses are often lower than subgrade stiffnesses, which is probably caused by the high moisture and the poorly compacted sub-base. Poor drainage system was observed during the site investigation.

Different sub-base stiffness values were back-calculated from the two methods of analysis in several sections. This may be due to different deflection matching techniques or bond modelling effects. Figure 2 shows that sub-base stiffness is likely to be under predicted when bonding condition approaches full slippage and over predicted when approaching full bond. This is broadly consistent with the results presented in Table IV. Similar subgrade stiffnesses were found by the two methods as expected from the results shown in Figure 2.

PADAL analysis results showed that the subgrade materials are linear, where the stiffnesses do not increase significantly with depth.

Effect of Traffic, Temperature and Age on Bonding Stiffness

To investigate the development of bonding stiffness with age and traffic, the same pavement was re-tested after six months of trafficking. Table V presents the 50 percentile FWD deflections and Table VI shows the calculated results using both PADAL and the new method.

TABLE V Summary of the 50 Percentile FWD Normalised Deflections (Second Visit)

Section	Deflection ($mm \times 10^{-3}$)						
	$d_{0.0}$	$d_{0.3}$	$d_{0.6}$	$d_{0.9}$	$d_{1.2}$	$d_{1.5}$	$d_{2.1}$
A	295	242	177	117	80	59	39
B	221	191	157	117	85	67	41
C	332	276	197	128	85	63	41
D	231	200	163	121	87	66	40
E	323	253	163	98	63	50	36
F	173	147	119	87	65	49	31
G	201	165	134	97	73	56	35
H	162	134	114	89	71	56	38
I	137	119	105	81	62	50	32
J	245	200	142	94	64	47	30
K	188	157	117	81	56	41	28
L	187	153	110	75	53	41	28
M	238	197	149	102	70	53	35
N	159	133	103	79	59	47	31

Note: $d_{0.0}$, $d_{0.3}$ denote measured deflections at the FWD load centre and 0.3m from the load centre respectively.

TABLE VI Summary of the 50 Percentile Back-calculated Pavement Properties Using PADAL and the New Methods (Second Visit)

Section	PADAL			New Method			
	E_{Bit} (MN/m^2)	E_{SB} (MN/m^2)	E_{SG} (MN/m^2)	K_{SI} (MN/m^2)	E_{Bit} (MN/m^2)	E_{SB} (MN/m^2)	E_{SG} (MN/m^2)
A	3900	80	140	190	11300	100	135
B	9400	130	100	100000	13500	60	100
C	3500	70	120	50	12100	100	115
D	8700	120	110	6000	16700	60	115
E	3100	80	220	10	13500	200	210
F	11000	70	230	10000	12900	100	230

Section	PADAL			New Method			
	E_{Bit} (MN/m ²)	E_{SB} (MN/m ²)	E_{SG} (MN/m ²)	K_{SI} (MN/m ²)	E_{Bit} (MN/m ²)	E_{SB} (MN/m ²)	E_{SG} (MN/m ²)
G	4800	60	110	190	12700	65	110
H	8400	100	110	100000	8700	160	115
I	13000	120	130	10000	11300	145	135
J	4500	60	200	160	14100	80	210
K	5900	50	270	10	14500	60	265
L	6500	70	240	10	13500	105	245
M	4800	70	170	280	14300	85	165
N	8800	80	170	10	19200	60	170

Higher pavement temperatures were recorded during the second visit (tested in June), as shown in Table II, and therefore lower bituminous stiffnesses are calculated compared with the first visit (see Table VI).

The sub-base stiffnesses varied compared with the first visit in some sections. This is probably due to the improvement in the drainage system, which was carried out during this period and/or compaction of the materials under traffic loading.

Since lane two is hardly subjected to commercial vehicle loading, a comparison between trafficked and untrafficked pavement was possible. Table VII categorises different section in terms of parameter variation between the two visits.

Figure 5 shows the back-calculated bonding stiffness of each section for both visits. With exception of section L, the bond between bituminous layers has improved under environment condition and traffic loading. Figure 5 and Table VII show that:

i) A combination of traffic and higher temperature will improve the bond stiffness.

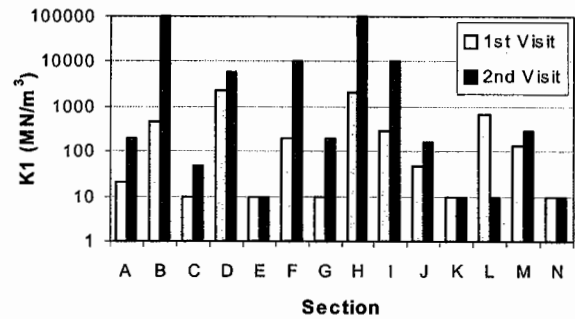


FIGURE 5 Bonding Stiffness Comparison Between First and Second Visits

- ii) Higher testing temperature may produce better bonding (Sections C and D).
- iii) Traffic has improved the bonding between bituminous layers (Sections I and J).
- iv) Age alone does not seem to improve the poorly bonded layers (Sections L, M and N).
- v) Sections with very poor bond do not appear to improve with time and traffic (Sections E and K).

TABLE VII Comparison Between First and Second Visits Section External Parameter Variations

Section	Lane	Traffic (msa) ^a	Temperature variations	Age	Parameter
A, B	1	0.5	5 to 16	6 months	Traffic, Temperature and Age
C, D	2	0	5 to 16	6 months	Temperature and Age
E, F, G, H	1	0.5	6 to 16	6 months	Traffic, Temperature and Age
I, J, K	1	0.5	8 to 10	6 months	Traffic and Age
L, M, N	2	0	8 to 10	6 months	Age

a. Million of standard 80 kN axle

It should be noted that the above results will have some errors since they have been back-calculated from real FWD test results. Real data will have errors due to deflection measurements, layer thickness variations, material non-linearly, static analysis of dynamic FWD loading and the deflection matching process.

From a practical point of view, the median stiffness values for a section rather than individual FWD test results are considered to better describe the pavement condition. Additional testing to support the pavement evaluation findings is usually required, such as NAT stiffness, visual survey, core condition and bonding strength measurement tests. Therefore the engineer will correlate the above results together with some engineering judgement to describe the pavement condition and its residual life.

In order to investigate the development bond stiffness with time / temperature / trafficking, controlled trial sections with known bonding stiffness should be constructed. FWD tests should be carried out on the trial sections, under different traffic and environmental conditions. The results from other bonding testing devices should be correlated with the FWD back-analysed bonding stiffness.

EFFECT OF POOR BONDING BETWEEN BITUMINOUS LAYERS ON PAVEMENT FAILURE MECHANISMS

The classical design approach addresses two forms of pavement failure in flexible pavements; fatigue cracking in the bituminous material and overstressing of the subgrade. The fatigue cracking is assumed to initiate, under traffic loading at the bottom of the bituminous layer and propagates toward the surface. However recent works (Nunn 1997 and Gerritsen *et al* 1987) suggest different deterioration mechanisms for thicker pavements, namely surface initiated cracking and rutting within the bituminous materials. Both methods assume that the bituminous materials are well bonded and will act as one coherent layer during the pavement's life.

For the above investigated case studies, a lower pavement design life could be predicted if PADAL back-analysed bituminous stiffnesses were used in the assessment. This is due to the low calculated stiffnesses which compensate for the poor bonding between bituminous layers.

Assuming the classical pavement failure modes apply, tensile strains at the bottom of each bituminous layers need to be calculated if the effect of poor bonding is to be taken into account (since the layers will act independently). Therefore for two poorly bonded layers, two cracks may initiate and propagate within the bituminous material under traffic loading, causing faster pavement deterioration.

On the other hand if the crack started from the surface and propagated towards a de-bonded interface, it might extend horizontally causing further slippage but not extend into the lower layers.

For pavement with thin bound layers and poor bonding, the lower overall load spreading ability could lead to greater traffic induced strains in the subgrade, and a lower life.

Therefore the bonding stiffness should be considered when assessing the pavement failure mechanisms and evaluating residual life. This can be quantified initially by end product pavement testing, using the FWD and other devices. However, further research into the development of bonding using controlled sections should be carried out, in order to define the appropriate models and "acceptable" bond levels for a given temperature, pavement type and traffic ranges.

CONCLUSIONS

Conventional pavement evaluation using FWD data and back-calculation of layer stiffnesses may have some errors if poor bonding exists between bituminous layers. Theoretical studies have showed that errors up to 50% may result in bituminous and sub-base stiffnesses if full bonding is assumed rather than the actual values. However negligible influence was found on subgrade stiffness due to the above modelling error.

A new method for back-calculation of bond and layer stiffnesses, using a two-stage database approach, has been used to evaluate several newly constructed pavements with suspected de-bonding. The pavements were re-tested after six months to investigate the influence of traffic and temperature on bonding development. The FWD test results generally indicated an improvement of bonding compared with the first visit.

The effect of bonding between bituminous layers on flexible pavement failure mechanisms and long term pavement behaviour is not well understood. Therefore an investigation should be performed, on trial pavement sections with controlled bonding condition, using the FWD and other testing techniques in order to understand bonding development. This could lead to a more comprehensive model for pavement design and evaluation.

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