Fatigue design criteria for low noise surfaces on New Zealand roads

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D. Alabaster and A. Fussell
Transit New Zealand, Christchurch

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PO Box 2840, Waterloo Quay, Wellington, New Zealand
Telephone 64-4 931 8700; Facsimile 64-4 931 8701
Email: *research@landtransport.govt.nz*
Website: *www.landtransport.govt.nz*

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Transit New Zealand, PO Box 1479, Christchurch

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Introduction

The ever increasing number of motor vehicles and their daily use is developing into a formidable threat to the quality of urban social and economic life. Pollutant emissions pose direct threats to human health, animals, vegetation and building materials, and the noise produced by road transport is by far the most pervasive of all the transport modes. Noise-mitigating porous asphalt road surfaces are becoming an increasingly important tool to reduce traffic noise in urban areas.

Internationally porous asphalts are typically laid on top of structural asphalt layers. In New Zealand structural asphalt is generally prohibitively expensive and porous asphalt is used directly on chipseal-surfaced unbound granular pavements in these pavements. In the late 1980s, rule-of-thumb guidelines were developed to try to prevent fatigue cracking. Recent pavement failures on major projects have highlighted that the current rule-of-thumb design procedures are inadequate. Instead more rigorous design procedures that employ current modern mechanistic pavement design practices (e.g. CIRCLY) or falling weight deflectometer (FWD) data are required.

This research carried out in 2004-05 aimed to achieve the following objectives:

- develop a horizontal tensile strain versus fatigue life curve that can be applied to the base of an Open Graded Porous Asphalt (OGPA) layer;
- establish a relationship between basecourse surface curvature and OGPA fatigue life;
- evaluate the extension to fatigue life of using enhanced binders in OGPA;

with the aim of improving the fatigue design of surfaces that will reduce road noise.

Pavement tests

Two accelerated pavement tests were undertaken at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF). Pavement Test 1 was to develop the horizontal tensile strain versus fatigue life curve and establish a relationship between basecourse surface curvature and fatigue life. The results of this test were surprising and, with the agreement of the project's Steering Group, Pavement Test 2 concentrated on examining extension of fatigue life by initial trafficking rather than using enhanced binders.

Conclusions

The objectives were based on the traditional view of thin asphalt behaviour applied in the Austroads Pavement Design and Rehabilitation Guides and existing research into asphalt behaviour. The first objective was not achieved as it was not possible to find instrumentation of a scale that would not interfere with the performance of the thin surfaces. In the second objective deformation was found to lead to surface failure before fatigue occurs. As a result of the second objective findings the third objective was modified in agreement with the Steering Group to examine this problem in more depth and attempt to extend surface lives by initial trafficking.

Therefore three of the Sections (A, B and C) of the first test showed that deformation leads to surface failure before fatigue occurs when high basecourse curvatures are measured. Section E in the first test showed that, if pavements are constructed well, then applying low noise surfaces immediately after construction is possible.

The laboratory fatigue results contradict current wisdom that OGPA is more tolerant of deflection than asphaltic concrete. However, without being able to generate fatigue at CAPTIF, this finding could not be validated.

The second test showed that the pavements could tolerate more deflection if initial trafficking was undertaken. The current deflection criteria are conservative even if factors such as temperature and ageing are considered.

Recommendations

1. Predicting surface failure

The outcomes of the project suggest that the Austroads Rehabilitation Design Guide is very conservative in predicting fatigue which in reality occurs much later. Practically speaking, deformation leads to failure of the surface before fatigue of the surface occurs.

2. Timing the application of low noise surfaces

The first test (Pavement Test 1) showed that, if pavements are constructed well, then applying low noise surfaces immediately after construction is possible. From analysis of the first test FWD readings, a conservative approach would consider that all deflections having curvatures over 250 μm are unacceptable (i.e. on pavements with design loadings over 100,000 ESA) and that such results would require additional analysis.

In this additional analysis, pavements with basecourse of known good rut resisting performance and a degree of saturation below 60% could be identified, and surfaced immediately.

3. Using initial trafficking to reduce deformation

The second test (Pavement Test 2) suggests that surfaces failing the performance and saturation criteria would be acceptable after an initial trafficking of 100,000 ESA. This trafficking would reduce initial deformation and thus lead to acceptable surface life.

In this case, if RMA (Resource Management Act) noise requirements need to be met immediately, the speed limit could be lowered while the chipseal surface is trafficked.

If lowering the speed limit is not practical, the basecourse could be modified to increase its resistance to deformation, and then the OGPA applied.

Further analysis of field performance should be performed to assist in confirming the new criteria for sealing with low noise surfaces.

4. Further research

Further research into the prediction of deformation in unbound granular pavements and methods to prevent post-compaction rutting would give additional confidence when sealing with low noise surfaces on new road construction.

Abstract

Internationally low noise porous asphalts are typically laid on top of structural asphalt layers. In New Zealand structural asphalt is generally prohibitively expensive and porous asphalt is used directly on chipseal-surfaced unbound granular pavements.

Two accelerated pavement tests were undertaken at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) in 2004–2005. The first test was to develop a horizontal tensile strain versus fatigue life curve and establish a relationship between basecourse surface curvature and fatigue life. The second test evaluated the extension of fatigue life by short trafficking before surfacing rather than using enhanced binders in porous asphalt.

The outcomes of the project suggest that the Austroads Rehabilitation Design Guide is very conservative in predicting fatigue and that deformation leads to surface failure before fatigue of the pavement occurs. Pavements to be sealed with low noise surfaces could tolerate more deflection if initial trafficking was undertaken.

1. Introduction

1.1 Background

The ever increasing number of motor vehicles and their daily use is developing into a formidable threat to the quality of urban social and economic life. Pollutant emissions pose direct threats to human health, animals, vegetation and building materials, and the noise produced by road transport is by far the most pervasive of all the transport modes (Boulter et al. 1999). Noise-mitigating porous asphalt road surfaces are becoming an increasingly important tool to reduce traffic noise in urban areas.

Originally porous asphalt was developed to reduce the risk of aquaplaning but has since been used for its other safety benefits, which include improved wet weather skid resistance, a very significant reduction in water spray thrown up from vehicle tyres in wet conditions, improved visibility of road markings in wet conditions, and reduced vehicle vibrations and thus increased user comfort.

Early environmental studies on a range of porous and non-porous surfaces showed that, in the dry, the noise level generated by vehicles running on a porous surface was, on average, approximately 3 to 4 dB(A) lower than the noise from a non-porous surface. To achieve this degree of noise reduction by changing the traffic flow or altering the route alignment, would require the traffic flow to be more than halved, or the distance of residential buildings from the traffic stream to be more than doubled (Nicholls 1997).

Open graded porous asphalt (OGPA) systems, such as TwinLay developed by Heijmans in Holland, have reportedly reduced road noise by 5 dB (decibels) when compared with traditional dense mix asphaltic concrete. In New Zealand Fulton Hogan have developed a similar system known as WhispA, which is being trialled (Vercoe & Hegley 2002) for its noise-reducing properties in Auckland and Dunedin. Traditional porous asphalt systems based on Transit New Zealand's specifications reduce noise in the order of 3 dB.

A literature survey on the topic of porous asphalt showed that internationally the design of porous asphalt surfaces is restricted to material and construction specifications. The porous asphalt is designed to ensure that various desirable properties, such as skid resistance, porosity and noise reduction, are achieved. These porous asphalt surfaces are not considered to contribute to the structural strength of the pavement and are thus excluded when designing the thickness of the pavement layers. In the UK pavements are typically constructed with an asphalt base of at least 100 mm in thickness. Thus porous asphalt surfaces placed on top of the asphalt base have little chance to fail by fatigue cracking as the porous asphalt surface bonds with the asphalt base and the tensile strains occur at the base of the asphalt rather than in the surface. As a consequence no design guidelines are available to ensure that porous asphalt surfaces do not fail by fatigue (Austroads 2004a, b).

In New Zealand, structural asphalt layers are generally prohibitively expensive and porous asphalt is used directly on chipseal-surfaced unbound granular pavements. In the late

1980s, rule-of-thumb guidelines were developed (Sheppard 1989) to try to prevent fatigue cracking in such pavements. Recent failures on major projects, such as Route PJK in Tauranga, have highlighted that the current rule-of-thumb design procedures are inadequate. More rigorous design procedures that employ current modern mechanistic pavement design practices (e.g. CIRCLY) or falling weight deflectometer (FWD) data are required.

This research carried out in 2004-05 aimed to achieve the following objectives:

- develop a horizontal tensile strain versus fatigue life curve that can be applied to the base of an OGPA layer;
- establish a relationship between basecourse surface curvature and OGPA fatigue life;
- evaluate the extension to fatigue life of using enhanced binders in OGPA;

with the aim of improving the fatigue design of surfaces that will reduce road noise.

1.2 Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF)

CAPTIF is located in Christchurch, New Zealand. It consists of a circular track, 58 m long (on the centreline) contained within a 1.5 m deep x 4 m wide concrete tank so that the moisture content of the pavement materials can be controlled and the boundary conditions are known. A centre platform carries the machinery and electronics needed to drive the system. Mounted on this platform is a sliding frame that can move horizontally by 1 m. This radial movement enables the wheel paths to be varied laterally and can be used to have the two 'vehicles' operating in independent wheel paths. An elevation view is shown in Figure 1.1.

At the ends of this frame, two radial arms connect to the Simulated Loading and Vehicle Emulator (SLAVE) units that are shown in Figure 1.2. These arms are hinged in the vertical plane so that the SLAVEs can be removed from the track during pavement construction, profile measurement, etc., and in the horizontal plane to allow for vehicle bounce.

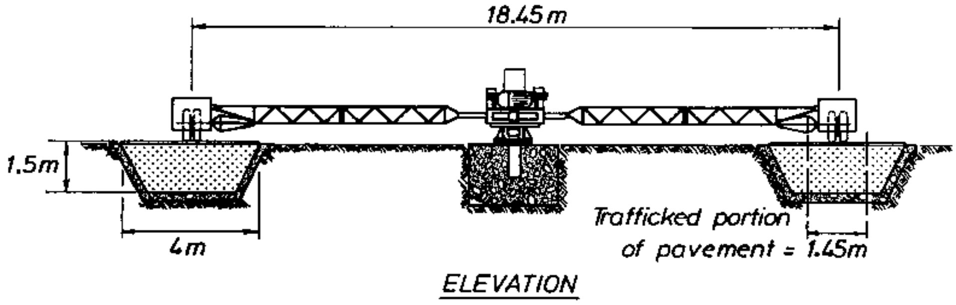


Figure 1.1 Elevation view of CAPTIF.

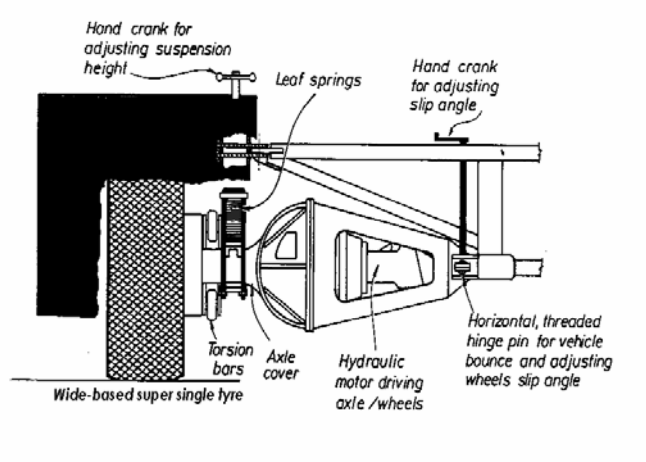


Figure 1.2 The CAPTIF SLAVE unit.

CAPTIF is unique among accelerated pavement test facilities in that it was specifically designed to generate realistic dynamic wheel forces. A more detailed description of CAPTIF is detailed in other Transfund Research Reports on mass limits (Arnold et al. 2005) and by Pidwerbesky (1995).

1.3 Proposed methodology

To meet the original objectives two accelerated pavement tests were proposed. They were to test OGPA-surfaced unbound granular pavements built at CAPTIF, located in Christchurch.

1. The *first pavement test* (Pavement Test 1) would establish a relationship between various design parameters (acting on the base of the OGPA) and fatigue life.

2. The *second pavement test* (Pavement Test 2) would compare the fatigue performance of three sections of enhanced OGPA with a standard section of OGPA pavement.

1.3.1 Pavement Test 1

The first pavement test would establish a relationship between the horizontal tensile strain (on the base of the OGPA) and fatigue life. It was on standard OGPA and the test track was divided into five sections (A to E). The pavement sections had the same thickness of porous asphalt but the basecourse was successively deeper at each section. This arrangement was to progressively reduce the strain on the base of the porous asphalt, allowing a strain versus fatigue curve to be derived.

Also a proposal was to take two approaches to characterise the fatigue life:

- The first approach was to develop a traditional horizontal tensile strain versus fatigue life curve using actual strain measurements taken at the base of the OGPA. This approach would provide the design information for traditional greenfield pavement designs.

- The second approach was to develop a curvature (of the basecourse before applying the surface) versus fatigue life curve. This approach is adopted by Austroads for the design of thin and structural asphalt overlays because of the difficulties of back-calculation of pavement parameters from deflection testing. It is in effect exactly the same as the first approach, except that it removes the need for calculating strains on the base of the OGPA layer. Air and pavement temperatures were monitored during testing to ensure that appropriate corrections can be made and applied to other environmental conditions.

Research in the United Kingdom (UK) found that progressive binder hardening governs the life of porous asphalt placed over structural asphalt (i.e. an environment with no horizontal tensile strains). Opus International Consultants is looking at better methods to assess the long-term durability of porous asphalt in environments that have no horizontal tensile strain, in their Porous Asphalt Durability project. Experience on highways in the UK (Nicholls 1997) suggests that durability benefits can be gained by using enhanced binders.

The basic premise is that enhanced binders generally allow higher binder contents to be used. Such high binder contents improve the durability of porous asphalt by providing a thicker binder film, which effectively slows the hardening process.

Binder content is generally limited by the tendency of excess binder to drain from the mixture, which can result in areas that are either binder-rich, or lean and lacking in fines. Binder-rich areas will lack hydraulic conductivity and binder-lean areas will be prone to fretting.

The UK research showed that polymer modified binders (PMB) modified with Ethylene vinyl acetate (EVA) actually tended to behave as harder binders with a consequential reduction in longevity, while those modified by SBS (Stryene-butadiene-styrene) produced a performance comparable with the control yet using less bitumen. The presence of hydrated lime tended to lower the hardening rate, hence increasing the durability. Adding fibres to the porous asphalt allowed higher binder contents to be achieved and improved durability. Traditional dense asphaltic concrete design suggests that such durability property improvements should also flow on to the fatigue performance of these materials.

1.3.2 Pavement Test 2

The second pavement test was to compare the fatigue performance of three sections of enhanced OGPA with a standard section of OGPA pavement.

Based on the UK experience the three experimental sections were to have contained:

- one section having binder modified with 5% SBS;
- one section with a fibre-enhanced binder, and
- the third section with a two-layer system applied that was significantly thicker than the control. However traditional linear elastic design theories suggest that its fatigue performance may be poorer than the thinner control section.

Laboratory testing was undertaken to characterise the performance of each surface. The Indirect Tensile Fatigue Test, which is soon to be adopted as a European standard, was used in preference to Austroads' flexural fatigue of asphalt beam test. The Indirect Tensile Fatigue Test was adopted as it is simpler, thus cheaper, and more likely to be adopted by users. The test has been shown to correlate well with fatigue beam testing. As OGPA is primarily used as a wearing course overseas, little research has been done on fatigue testing it. And, as with dense asphaltic concrete testing, the expectation is that, at the very least, a shift factor will need to be applied to the results before they can be used in the field.

To allow for possible future increases in heavy vehicle mass limits, the SLAVE vehicles (Figure 1.2) travelled separate paths, Vehicle A was loaded to 40 kN and Vehicle B to 50 kN. This will simulate the current 8.2-tonne legal axle load and a 10-tonne single-axle dual-tyre arrangement.

The results of the first accelerated pavement test were surprising given the Austroads deflection and curvature approaches to asphalt overlay design. The results suggested that sufficient curvature could not be developed to fatigue a thin surface before the surface failed by rutting. This result suggested that improving the durability of porous asphalts with polymers would obviously not improve the fatigue life of the surface. However it did suggest that trafficking unbound pavements before applying a porous asphalt may improve the life of the surface. With the agreement of the project's Steering Group the second test was altered to examine this option which, at the same time, would also test the hypothesis that basecourse deformation was in fact the cause of cracking in thin asphalt layers.

2. Pavement Test 1

2.1 Pavement design

Previous pavements at CAPTIF were reviewed to develop designs with sufficient curvature that would theoretically cause the surface to fail by fatigue, for the experiment. The following conclusions were drawn from the data about the subgrade materials that have been used at CAPTIF.

- Previous projects had found Tod clay and Waikari silt to be practical subgrades to use at CAPTIF. The Tod clay was particularly useful as its strength changed relatively slowly with increasing moisture content.
- Both materials have similar elastic moduli values measured by a Loadman device (~75 MPa) when compacted close (within 2%) to their Optimum Moisture Contents (OMC). When the Tod clay was constructed at OMC+10% (30%), the moduli halved to 38 MPa.
- The actual compactive effort for the subgrade (using sheepsfoot trench rollers used at CAPTIF) appears to sit about one quarter to one third of the way between the standard and heavy compaction standard. The New Zealand standard compaction data would therefore be appropriate to use for determining the compaction targets and moisture contents.

Therefore two options are available for a strong subgrade (Tod and Waikari at OMC), and one option for a weak subgrade (Tod at OMC+10%).

Based on the above information, the following design assumptions were made:

- The Waikari and Tod materials have the same stiffness characteristics at OMC.
- The stiffness of the Tod material at OMC+10% is 50% of the OMC value.

The following matrix was analysed: three subgrades (Waikari @ OMC, Tod @ OMC and Tod @ OMC+10%) by three aggregate thicknesses (150, 300 and 450 mm).

Loading was approximated with a single circular load rather than two circular loads with an area that corresponds to the measured dual-tyre footprint (11R22.5 tyres at 800 kPa) for 40 and 60 kN loads. For a 40 kN load, the radius is 171 mm and the pressure is 435 kPa. For a 60 kN load, the radius is 189 mm and the pressure is 535 kPa. However, to avoid the HS OGPA failing by deformation it was decided to limit the maximum load to 50 kN.

Note that the inflation pressure is not equal to the average contact pressure of the tyre as normally assumed in design. This approach was chosen as the tyre footprint and total load are fixed and therefore the average contact pressure over the footprint should be used.

The materials were analysed using CIRCLY 4.1 in an iterative non-linear method with an anisotropy factor of 2.

Table 2.1 CIRCLY analysis results from 2 loadings, 2 subgrades, and 2 aggregate thicknesses.

Central Deflections

Aggregate thickness (mm)	40 kN Load		60 kN Load	
	Tod OMC+10%	Waikari/Tod OMC	Tod OMC+10%	Waikari/Tod OMC
150	2.305	1.338	3.001	1.727
300	1.629	1.074	2.085	1.350
450	1.366	1.004	1.691	1.218
PR3-0032				
325	1.750	0.900		

Subgrade Strains

Aggregate thickness (mm)	40 kN Load		60 kN Load	
	Tod OMC+10%	Waikari/Tod OMC	Tod OMC+10%	Waikari/Tod OMC
150	5197	3047	6458	3767
300	2885	1784	3692	2287
450	1865	1159	2409	1511

The calculated deflections correspond well with the deflections from the PR3-0032 subgrade project (Table 2.1), giving confidence to the analysis methodology and assumptions.

The final design comprised five pavement test sections:

- Section A. Tod OMC and 150 mm aggregate cover, 30 mm Mix10 Asphalt as a control section (medium deflection 1.3 mm).
- Section B. Tod OMC and 150 mm aggregate cover, 30 mm OGPA (medium deflection 1.3 mm).
- Section C. Tod OMC+10% and 150 mm aggregate cover, 30 mm OGPA (high deflection 2.3 mm).
- Section D. Tod OMC+10% and 300 mm aggregate cover, 30 mm OGPA (medium deflection 1.6 mm).
- Section E. Tod OMC and 300 mm aggregate cover, 30 mm OGPA (low deflection 1.0 mm).

Insufficient Tod clay was available on site for the design so the design was re-analysed to check if Waikari silt could be used as a subgrade below the Tod subgrade. The results suggested the change would have a consistent and only minor effect on the design deflections. Thus the subgrade was split into two layers: a lower subgrade of Waikari silt consisting of five 150-mm layers at OMC, and an upper subgrade of three or four layers of Tod clay at OMC and OMC+10% depending on the basecourse thickness.

As part of the pavement design process, an initial test was undertaken at CAPTIF to check standard and High Strength (HS) OGPAs under the proposed higher loadings. The post-mortem trenches from the previous project PR3-0032 were repaired and 80/100 binder Standard and HS OGPA were laid and tested. The standard OGPA deformed under the higher loading but, as the HS OGPA performed well, HS OGPA was used for Pavement Test 1, and the maximum load was limited to 50 kN.

The proposed control dense asphalt and HS OGPA were tested using the Indirect Tensile Fatigue Test and the results are shown in Figure 2.1. The results suggest that noticeable differences in performance should be seen, with the OGPA lasting approximately half as long as the asphalt at the same horizontal tensile strain.

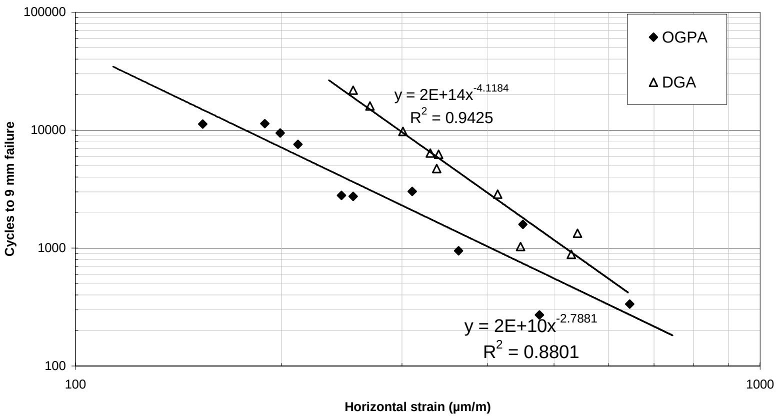


Figure 2.1 Indirect Tensile Fatigue test results for HS OGPA and DGA (dense graded asphalt).

2.2 Pavement instrumentation

The original methodology called for both asphalt strain gauges (to measure actual strain) and FWD readings (to measure curvature) to be related to cycles to failure. A review of available asphalt strain gauges found that they were all too large and were likely influence the outcome of the tests (Table 2.2).

Table 2.2 Dimensions of different asphalt strain gauges from different research organisations reviewed for Pavement Test 1.

Organisations	Designation	Total Length (mm)	Anchor Width (mm)	Thickness (mm)
CAPTIF	ARRB	100	50	8
CAPTIF	Dynatest	135	75	8
BAST	BAST + HBM	133	80	16
CEDEX	MM EA- 06-10CBE-120	50 approx	20 approx	5 approx
ETHZ/LCPC/ LAVOC	KYOWA	120	15	4.5
TU DELFT	KM 100HAS	155	85	Not given
LAVAL	Fibre optic	Uncertain		

The εmu strain measuring system was installed in stations 7, 14, 37 and 51 to assist in understanding the pavement's performance (Figure 2.2). The system measures actual strains in the pavement by measuring the displacement of inductive coils as vehicles pass over them.

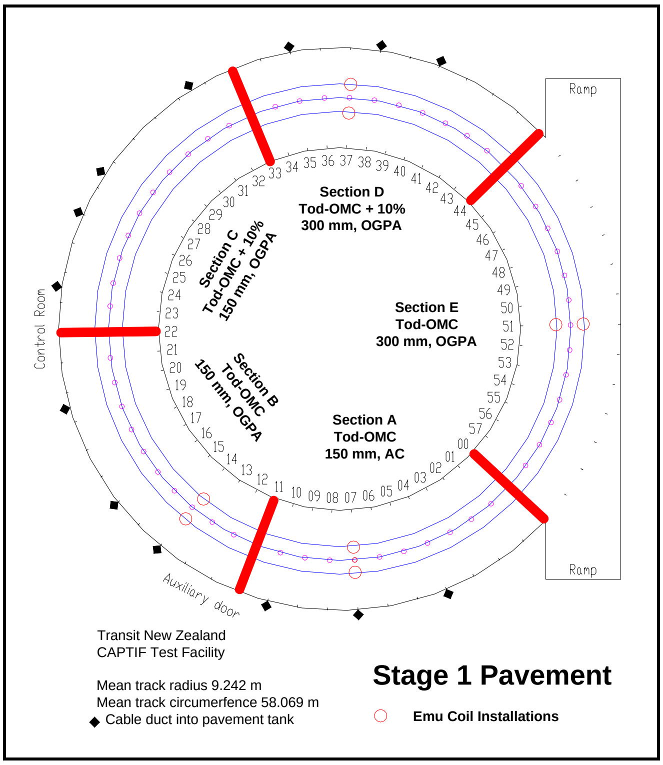


Figure 2.2 Pavement sections and instrument locations on the CAPTIF test facility for Pavement Test 1.

2.3 Pavement construction

The first pavement was constructed between February and July 2004. The entire tank was excavated, and reconstructing the subgrade and basecourse required 10 lifts of material. There was considerable difficulty in accurately constructing uniform sections having a CBR of 2. However the construction results and performance data suggest that these were built uniformly.

2.3.1 Subgrade density and CBR testing

The target density of the lower subgrade was set from previous projects at 1850 kg/m³ with a moisture content of 12%. The density and moisture content targets recorded in Table 2.3 for the upper subgrade were set from MDD/OMC test results and the weak Tod clay was set at OMC+8% because OMC+10% was found to be impractical.

Table 2.3 Upper subgrade density targets for Sections A–E.

Section	Moisture content (%)	Target density (kg/m^3)	Aggregate depth (mm)
A, B	22	1610	150
C	30	1450	150
D	30	1450	300
E	22	1610	300

Table 2.4 Average densities (DD) and moisture contents (MC) of Section subgrade layers (and standard deviations (SD)).

Section	Subgrade layer	Avg DD (kg/m^3)	SD of DD (kg/m^3)	Avg MC (%)	SD of MC (%)
A	SG2	1861	39	12	1
	SG3	1836	32	10	1
	SG4	1874	23	12	1
	SG5	1833	49	13	1
	SG6	1527	18	23	1
	SG7	1523	23	23	0
	SG8	1540	24	23	1
	SG9	1559	19	24	1
B	SG2	1869	15	12	0
	SG3	1831	31	10	1
	SG4	1848	23	12	0
	SG5	1840	29	12	1
	SG6	1510	19	22	2
	SG7	1500	18	22	1
	SG8	1552	18	23	1
	SG9	1520	41	24	1
C	SG2	1848	26	12	1
	SG3	1846	32	10	1
	SG4	1857	17	13	1
	SG5	1828	21	13	2
	SG6	1475	20	25	2
	SG7	1322	50	34	2
	SG8	1408	31	31	1
	SG9	1464	25	29	1
D	SG2	1861	36	11	1
	SG3	1826	37	9	1
	SG4	1832	15	11	1
	SG5	1810	24	14	1
	SG6	1466	36	27	2
	SG7	1330	43	34	2
	SG8	1457	32	28	2
E	SG2	1877	31	11	1
	SG3	1824	54	10	1
	SG4	1850	19	13	0
	SG5	1803	29	14	1
	SG6	1523	32	23	2
	SG7	1505	23	23	2
	SG8	1547	17	23	1

Densities were measured for each layer with a Nuclear Density Meter, which was calibrated with sand replacement tests. Table 2.4 suggests that the density and moisture targets were generally met for the lower subgrade layers (SG 2–SG 5). The correct moisture content for the Tod clay (at OMC) in Sections A, B and E was achieved, although the densities were harder to achieve and the results were typically 50–100 kg/m³ lower than targeted. Working with the Tod clay at OMC+8% was difficult and, even though each layer was relatively consistent, there was significant variation between the layers and any moisture was impossible to remove once it had been added to the material.

Three Scala penetrometer tests were undertaken in each section to test the in situ strength of the constructed subgrade. The in situ CBR was estimated from the 'RG Brickell' relationship and the readings given in Table 2.5 are the average estimated CBRs over the upper 300 mm of the subgrade.

The results show that the average strength targeted for each section had been achieved, with Sections C and D significantly weaker than Sections A, B and E. It also shows that the construction was uniform.

Table 2.5 Subgrade CBRs for Sections A–E.

Section	Surface Material	Subgrade Material	Average CBR	Min CBR	Max CBR
A	AC	Tod OMC	7	7	8
B	OGPA	Tod OMC	9	9	10
C	OGPA	Tod OMC+10%	2	2	2
D	OGPA	Tod OMC+10%	3	2	4
E	OGPA	Tod OMC	8	8	9

2.3.2 Subgrade FWD modulus values

The final subgrade surface was also tested with an FWD. Tests were undertaken at each station and in both wheel paths where possible. The test set-up used a 300-mm diameter loading plate and a nominal plate pressure of 400 kPa. The results were variable with calculated elastic moduli values ranging from 38 to 168 MPa for the dry material, and from 23 to 95 MPa for the wet material (Figure 2.3). For both materials, only a limited number of high values were obtained and, if these are excluded from the analysis, the average moduli values for the dry (Tod @ OMC) and wet (Tod @ OMC+10%) materials are 71 MPa and 39 MPa respectively.

One reason for the high variability in the moduli values may be related to the uneven surface condition. The loading plate may not have been in full contact with the surface resulting in an uneven load distribution and/or rocking or bounce of the plate. The FWD results from the wet material are consistent, probably because the material had a smoother surface. Another reason for the inconsistency in the dry material could be the presence of shrinkage cracks in the dry material.

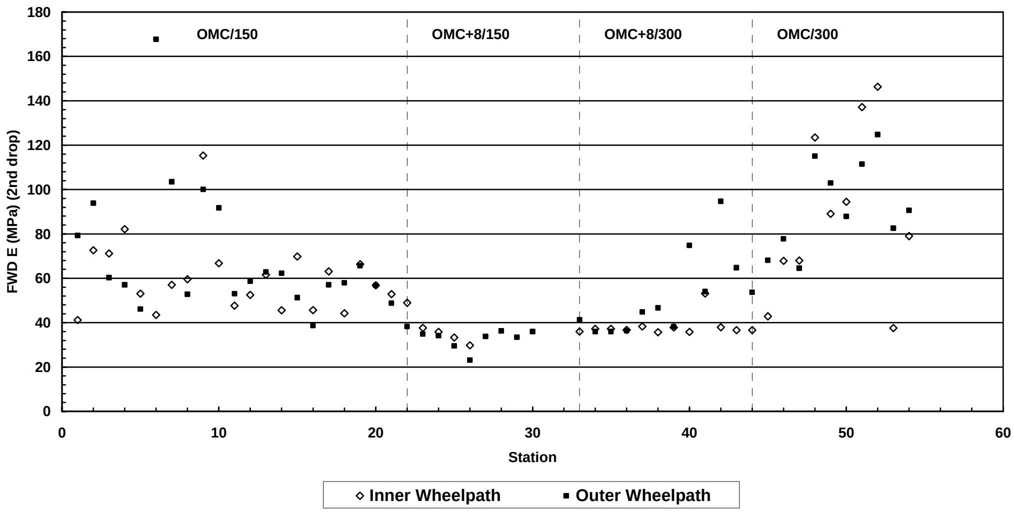


Figure 2.3 FWD modulus values obtained for subgrades under inner and outer wheel paths at each Station.

2.3.3 Basecourse construction

The basecourse used for the test was a TNZ M/4-compliant AP40 basecourse from Fulton Hogan's Pound Road Quarry, Christchurch. Material from this quarry has been used in previous projects and has performed very well.

Table 2.6 provides details of final dry densities (DD), Moisture Contents (MC), Percentage of Maximum Dry Density (%MDD) and Percentage of Saturation (%SAT) achieved in each Section. While these density values are a little less than those used in TNZ B/2:2005, they are typical of what is achieved at CAPTIF and are indicative of the difficulties of compacting material on very weak subgrades.

Table 2.6 Results of dry densities (DD), Moisture contents (MC), % Maximum dry densities (%MDD), and % Saturation (%SAT) (and Standard deviations SD) obtained for basecourses (BC) of Sections A–E.

Section	Avg DD (kg/m^3)	SD DD (kg/m^3)	Avg MC (%)	SD MC (%)	Avg %MDD	SD %MDD	Avg %SAT	SD %SAT
A	2233	42	3.1	0.4	96.3	1.8	41	6
B	2238	24	3.6	0.4	96.5	1.1	49	6
C	2216	37	3.2	0.5	95.5	1.6	41	8
D	2228	43	3.2	0.5	96.0	1.8	43	8
E	2242	36	3.6	0.5	96.7	1.5	50	9

2.3.4 Surface construction

The basecourse surface, once dry, was swept by power-broom and primed with emulsion using a hand lance. The contractor successfully calibrated the CAPTIF spray bar to industry standards, and calibration was carried out on a simulated test track in the contractor's yard. The first-coat chipseal applied before the asphalt and OGPA surfaces was a success. Chip was spread with a standard spreader truck, the gates on the roller feed were adjusted for the radius of the track, and a steel/rubber-combo roller pressed the chip in.

The final construction sequence was as follows and reflects that the seal was applied late in the sealing season:

- Emulsion Prime Coat, handsprayed at approx 0.3 ℓ/m^2 to leave approx 0.15 ℓ/m^2 residual binder;

- First Coat Seal (Single Coat Seal), 180/200 pen grade, 1 pph Kerosene, 0.8 pph Megamine Adhesion Agent, Grade 5 SC10 chip, Target Application Rate 1.40 ℓ/m^2 hot.

The actual application rate was 1.47 ℓ/m^2 in the first part that was sealed (Stations 2–44) and 1.2 ℓ/m^2 for the second part.

The chipseal surface was heavily tack-coated and an asphaltic concrete paving machine placed the TNZ M/10:2005 mix10 asphaltic concrete and the TNZ P/11:2003 HS (high strength) OGPA. The sealing crew used a footpath roller behind the paving machine. Once the paving machine had completed the circle and left the building, the entire surface was rolled with a 3.5-tonne steel-drum roller.

2.3.5 Layer thicknesses

Layer thicknesses have been measured with a 4-m beam and ruler at each permanent station mark during construction. The final asphalt (AC) and basecourse (BC) layer thicknesses in each section are given in Table 2.7, which shows that the design thicknesses have been realised. The additional variation in the basecourse thickness for Section D is the result of the soft subgrade, and the result for Section C is surprisingly good for a very soft clay material.

Table 2.7 Thicknesses (and standard deviations SD) of asphalt surface (AC) and basecourse (BC) layers after construction.

Section	Avg AC thickness (mm)	SD AC thickness	Avg BC thickness (mm)	SD BC thickness
A	34	4	144	12
B	40	8	150	11
C	36	3	151	8
D	32	3	310	14
E	37	3	300	8

2.4 Pavement characterisation

The Austroads Pavement Rehabilitation Guide (Austroads 2004a) provides procedures for determining the design life of thin asphaltic concrete overlays from deflection readings. Table 2.8 provides a summary of the initial FWD readings taken in the two wheel paths (with centrelines at 135 cm (at 50 kN) and 245 cm (at 40 kN)) in each Section.

Table 2.8 Initial FWD deflections at given stresses (kPa), central deflections (d0), bowl curvatures (d0–d200), and Coefficient of Variation (CV) for wheel paths at 135 cm and 245 cm from centreline.

Section	Wheel path	Avg Stress (kPa)	SD Stress (kPa)	Avg d0 (µm)	SD d0 (µm)	Avg CC (d0-d200) (µm)	SD CC (d0-d200) (µm)	CV
A	135	622	4	650	76	288	52	0.12
	245	622	6	608	52	260	31	0.09
B	135	620	6	697	96	299	62	0.14
	245	622	5	636	87	283	51	0.14
C	135	597	18	1512	521	640	215	0.34
	245	612	14	1237	450	568	221	0.36
D	135	620	6	696	67	288	35	0.10
	245	622	5	635	82	280	39	0.13
E	135	621	5	564	54	243	41	0.10
	245	623	5	531	48	230	29	0.09

The design procedure estimates the number of equivalent standard axles (ESA) to reach unacceptable permanent deformation from the central deflection (d0) of the FWD, and the number of equivalent standard axles (ESA) to reach asphalt fatigue and obtained from curvature of the FWD deflection bowl (d0-d200). The curvature function is defined as the deflection measured directly under the load (d0) minus the deflection measured 200 mm from the load (d200). FWD testing was undertaken after the OGPA and asphalt surfaces had been applied. This results in a slightly less conservative result (15% lower for deflection and 25% less for curvature, see Section 6.2.6 of the Austroads Guide 2004a).

The Austroads Guide suggests that long lengths of pavement requiring overlays should be split up into homogeneous subsections to prevent constructing excessively deep overlays where they are not required. Subsections are considered homogeneous if their Coefficient of Variation (CV, or standard deviation divided by the mean) is less than 0.25. This test has been applied to the CAPTIF test sections and the results (Table 2.8) show that all but Section C was homogeneous. A review of the construction results did not find any obvious reason for this. The results also suggest that the variation seen in the subgrade FWD testing was indeed a result of the surface condition rather than the pavement itself, as Section C had the most consistent results during subgrade testing.

The Austroads design procedures apply a number of corrections to normalise the pavement deflections and curvatures before the deformation and fatigue lives are estimated from the data’s characteristic values. The deflections are corrected to a stress of 566 kPa using the Austroads assumption that the deflections are linearly related to stress.

A seasonal correction was not applied to the data as the pavement was indoors and in its weakest state. When testing on asphalt, a correction is applied for the difference in pavement temperature during FWD testing and the expected mean temperature of the pavement during loading. In this case no correction was applied as the testing temperature was 13°C and the average loading temperature was 16.5°C, and this results in a negligible change (Figure 6.2, Austroads 2004a). A correction of 1.14 was applied to the deflections to normalise them for use with the deformation charts (Figure 6.3 in Austroads 2004a). As well, an allowance was made for the fact that the readings were on the asphalt surface (15% increase for deflection and 25% increase for curvature, according to Section 6.2.6 of the Austroads Guide).

Table 2.9 Characteristic deflection (CD) and characteristic curvature (CC) values at three values for factor *f*, for two wheel paths at 135 cm and 245cm distances, for Sections A–E.

Section	Wheel path	Normalised Values			
		CD for various f values			CC
		2	1.65	1.30	0
A	135	958	926	894	327
	245	849	828	806	296
B	135	1064	1024	984	341
	245	967	931	894	323
C	135	3174	2947	2720	758
	245	2591	2400	2209	656
D	135	995	967	939	329
	245	953	919	885	318
E	135	803	780	758	277
	245	748	728	708	262

The characteristic curvatures (CC) (given in Table 2.9) are the means of the curvature values. The characteristic deflection (CD) for a homogeneous subsection is defined as equal to the average deflection $\propto$, plus a factor *f* times the standard deviation *s* (Equation 2.1). Thus:

$$CD = \mu + fs \quad \text{Equation 2.1}$$

Factor *f* is selected by the designer to provide a suitable level of probability that the characteristic value is not exceeded by an individual value.

The following values for *f* are suggested:

- If more that 30 readings are taken, an *f* of 2 provides a 97.5% probability that all deflections will be covered. This factor is suggested to be applied to freeways/ arterials and highways with lane AADT>2000.
- An *f* of 1.65 provides a 95% probability, and is suggested for arterials and highways with lane AADT <2000.
- An *f* of 1.30 provides a 90% probability and is suggested for other roads.
- Where less than 30 readings are taken, the maximum deflection may be a better characteristic deflection.

Deformation and fatigue lives have been estimated from Figures 2.4 and 2.5 that are in the Austroads Pavement Rehabilitation Guide (2004a). The Weighted Mean Average Annual Pavement Temperature (WMAPT) used was the average surface temperature (which had been recorded every 10 minutes) during loading, and it was 16.5 °C.

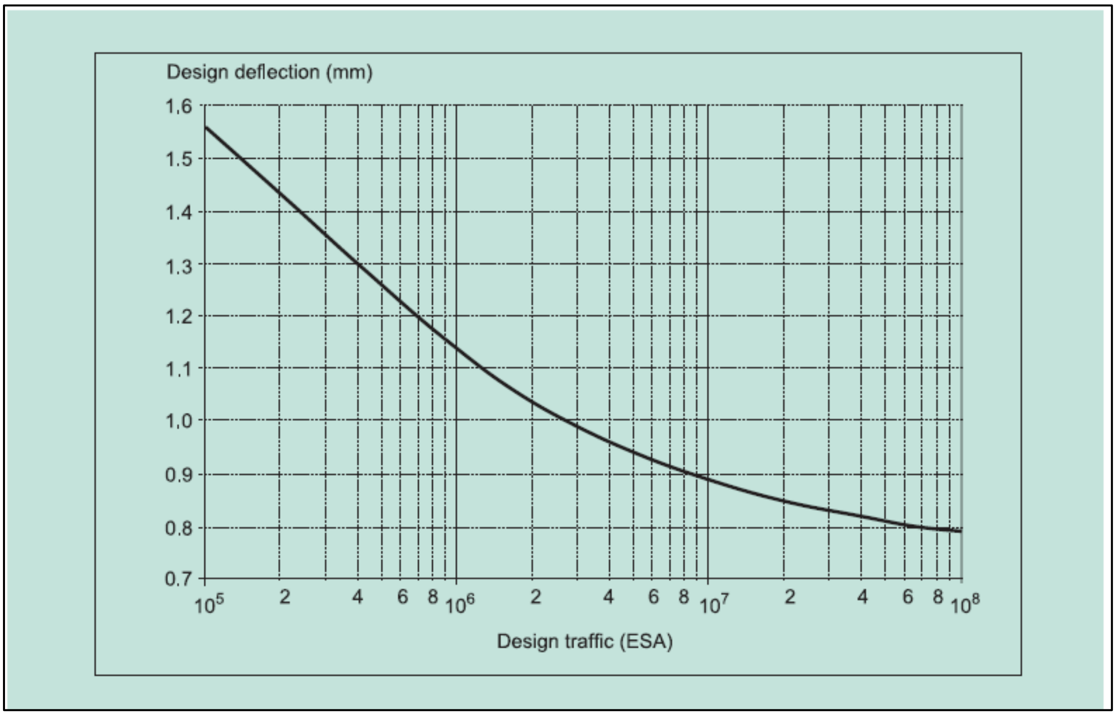


Figure 2.4 Design deflections to be used to limit permanent deformation (Figure 6.5 in Austroads 2004a).

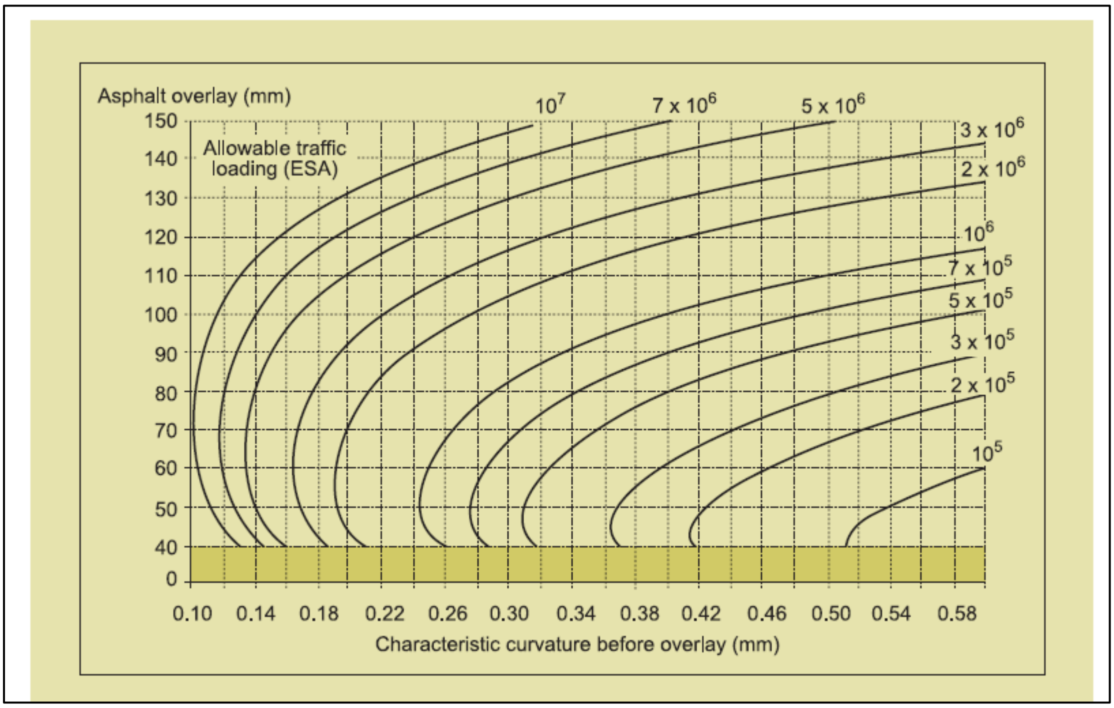


Figure 2.5 Asphalt overlay fatigue lives (calculated from overlay thicknesses and Characteristic curvatures (CC) before overlay) at WMAPT of 15-20°C (Figure A6.2.1 in Austroads 2004a).

Estimates of the design lives are provided in Table 2.10. As expected the actual design life in terms of permanent deformation exceeds the fatigue life by an order of magnitude. This combined with the laboratory fatigue testing suggested that the OGPA should fail in fatigue at approximately half the values provided in the table. The loading plan called for 1,000,000 ESAs to be applied by 1,000,000 passes of a 40 kN vehicle in the outer 245 cm wheel path and 2,400,000 ESAs to be applied by 1,000,000 passes of a 50 kN vehicle in the inner 135 cm wheel path.

Table 2.10 Predicted lives obtained from Austroads Pavement Rehabilitation Guide (2004a) using three values of factor *f*, for two wheel paths at 135 cm and 245 cm distances, for Sections A–E.

Section	Wheel path	Life (ESA x10^6)			
		Permanent deformation for *f* values			Fatigue
		2	1.65	1.30	0
A	135	4	6	8	0.4
	245	20	30	60	0.6
B	135	1.6	2	3	0.4
	245	4	6	8	0.4
C	135	<0.1	<0.1	<0.1	<0.1
	245	<0.1	<0.1	<0.1	<0.1
D	135	2.8	4	6	0.4
	245	4	6	8	0.4
E	135	60	100	100	0.7
	245	100	100	100	1

While the average strengths of Sections A, B and D showed little difference after construction, a significant difference could be seen after 115,000 loading cycles.

Tables 2.11 and 2.12 respectively provide first the raw results, and then the differences between the two sets of readings. The effects of the repairs, which occurred before the 115,000 cycle FWD readings, can be seen in Section C.

The increases in central deflection and curvature are most significant in the heavily loaded wheel path. The largest increases occurring in Sections A and B (which are structurally the same), then less of an increase in Section D, and the strongest section, Section E, had the lowest increases in central deflections and bowl curvature.

2.5 Pavement performance

Between September 2004 and February 2005, 943,000 loading cycles were applied to the test pavement. Repeated and largely unexplained suspension failures in the SLAVEs at the end of the project resulted in a slightly shorter number of loading cycles. The inside 135 cm wheel path had been loaded to 50 kN and the outside 245 cm wheel path was loaded to 40 kN. Figure 2.6 indicates where and when failures were observed in the test pavement.

Table 2.11 FWD readings as raw results for stress (kPa), central deflections (d0), and bowl curvatures (d0-d200) (and standard deviations SD) after 115,000 cycles of loading on two wheel paths at 135 cm and 245 cm distances for Sections A–E.

Section	Wheel path	Avg Stress (kPa)	SD Stress (kPa)	Avg d0 (μm)	SD d0 (μm)	Avg d0-d200 (μm)	SD d0-d200 (μm)
A	135	609	8	1339	250	755	197
	245	622	6	1003	114	510	87
B	135	609	10	1288	354	677	241
	245	613	6	982	223	427	100
C	135	629	10	796	163	269	84
	245	626	8	792	254	282	167
D	135	616	7	1075	156	454	72
	245	613	5	822	149	327	76
E	135	615	9	682	77	285	44
	245	615	4	599	50	215	36

Table 2.12 Differences in FWD readings between 0 and 115,000 load cycles for stress (kPa), central deflections (d0), and bowl curvatures (d0-d200) (and standard deviations SD) after 115,000 cycles of loading on two wheel paths at 135 cm and 245 cm distances for Sections A–E.

Section	Wheel path	Avg Stress (kPa)	SD Stress (kPa)	Avg d0 (μm)	SD d0 (μm)	Avg d0-d200 (μm)	SD d0-d200 (μm)
A	135	-13	4	689	174	467	145
	245	0	1	395	62	249	56
B	135	-11	4	591	258	378	179
	245	-9	2	346	136	144	49
C	135	31	-8	-716	-359	-371	-131
	245	14	-7	-445	-196	-286	-54
D	135	-3	1	378	88	166	38
	245	-9	0	187	67	47	37
E	135	-6	4	118	23	42	3
	245	-8	-1	67	2	-15	8

At first glance the location of the failures is reasonably consistent with the Austroads predictions, but on closer inspection of the failures (Figures 2.7-2.9), they are all deformation-related failures rather than the fatigue failures predicted by the Austroads Guide. Inspections at the regular testing intervals failed to find fatigue cracking before the deformation failures occurred. In this case a failure was defined as the point at which the vehicles could no longer traffic the pavement.

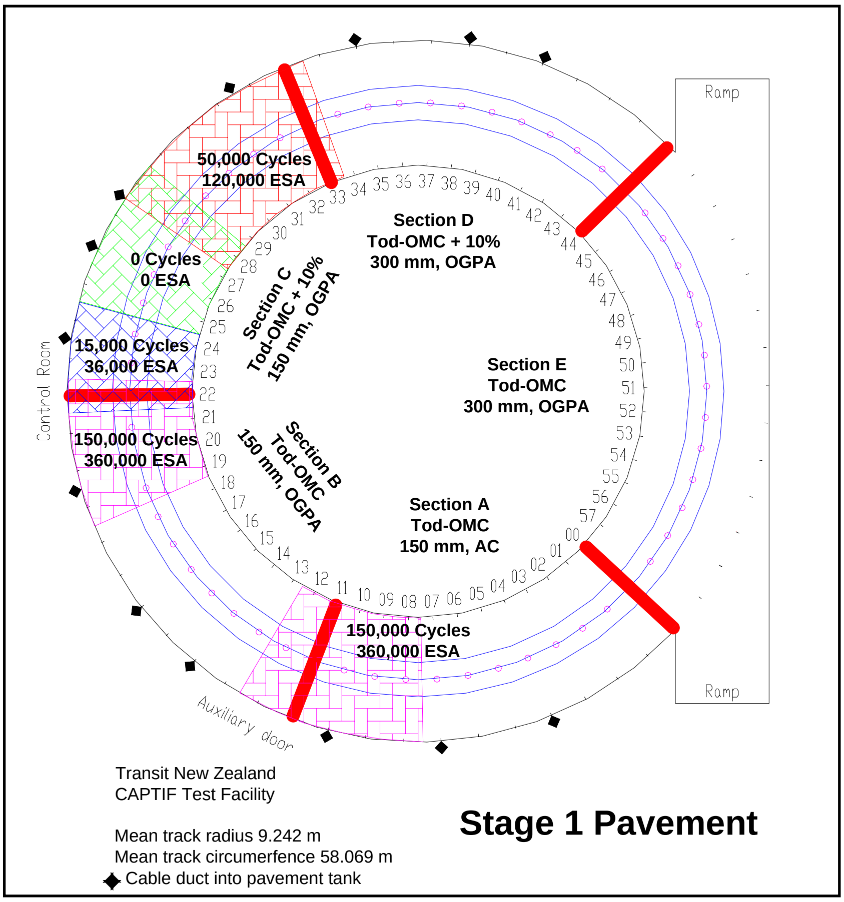


Figure 2.6 Positions of pavement failures on Sections A–E.

Figure 2.7 Failure between Stations 25–27 at 0 load cycles.

Figure 2.8 Failure at Stations 22–24 at 36,000 ESA.

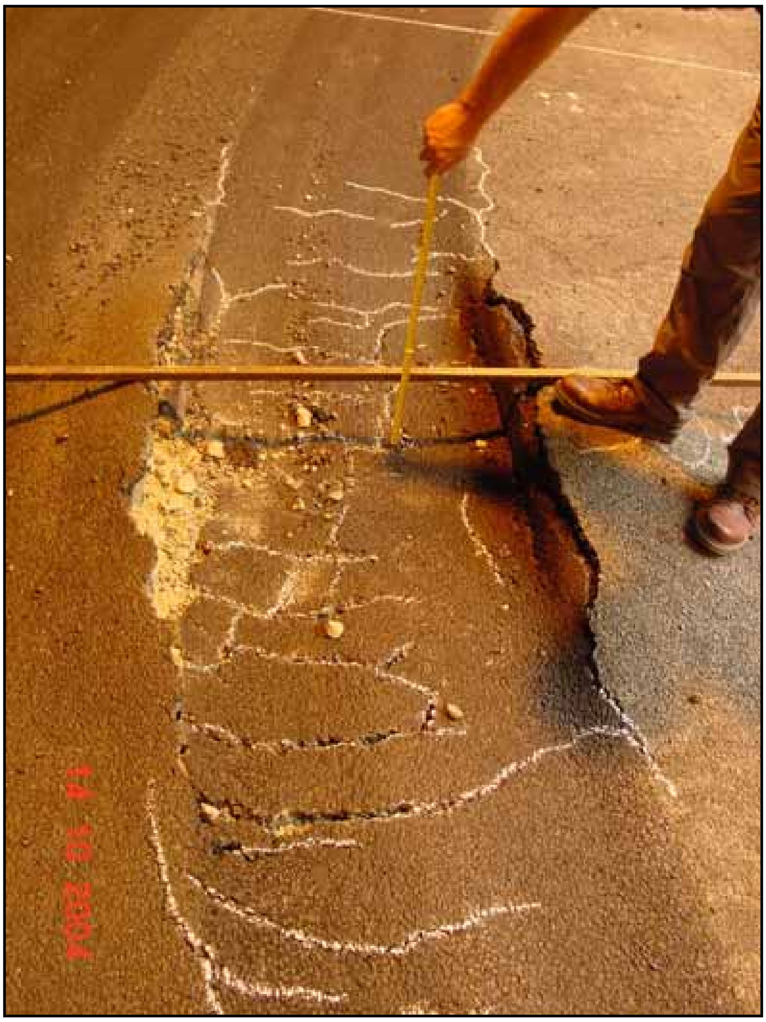


Figure 2.9 Failure between Stations 29–33 at 120,000 ESA.

Figure 2.10 shows that, while a relationship exists between curvature and load cycles to failure, it is not a strong relationship and central deflection provides a virtually identical relationship.

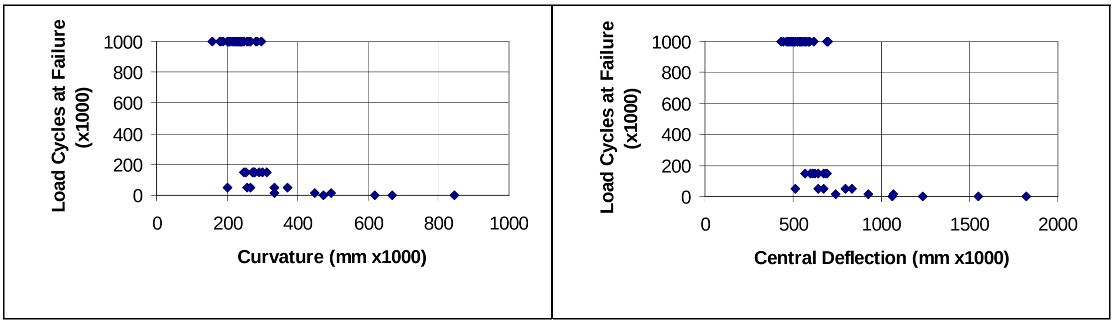


Figure 2.10 Curvature and Central Deflection v load cycles to failure.

Figure 2.11 presents the Vertical Surface Deformation (VSD) results for the sections (shown in Figures 2.7-2.9) that were repaired. VSD is the change in the pavement's transverse profile from the as-constructed profile. Note that in each repair, damaged areas either side of the peak deformation were also repaired, so the peak value is used for analysing the repairs. It appears that the pavements were failing when the peak value for a section reached approximately 15-20 mm VSD.

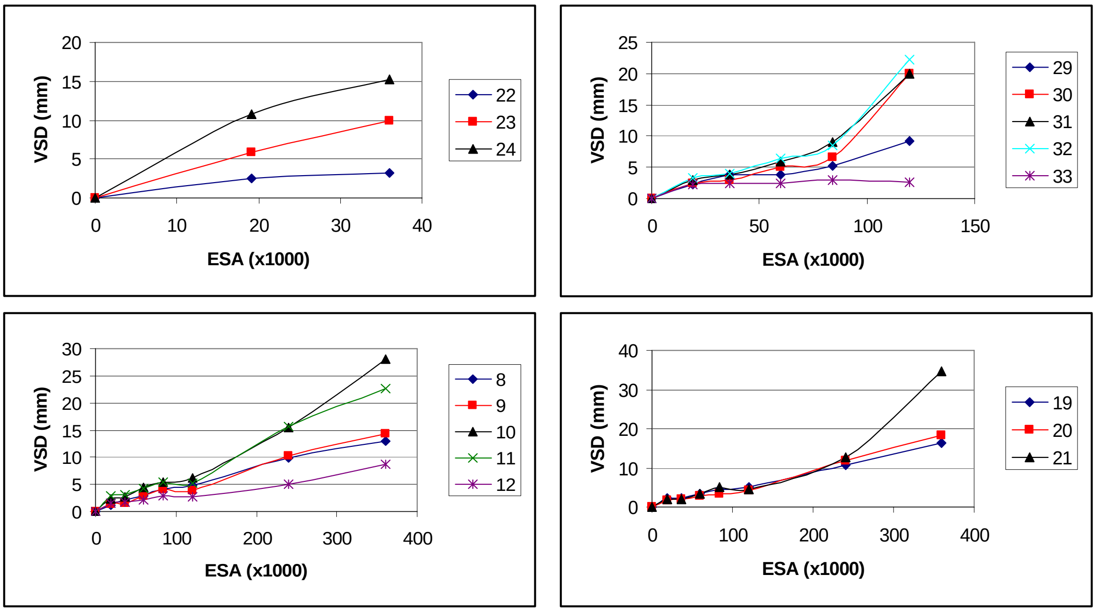


Figure 2.11 VSD values for stations at which pavement failure occurred.

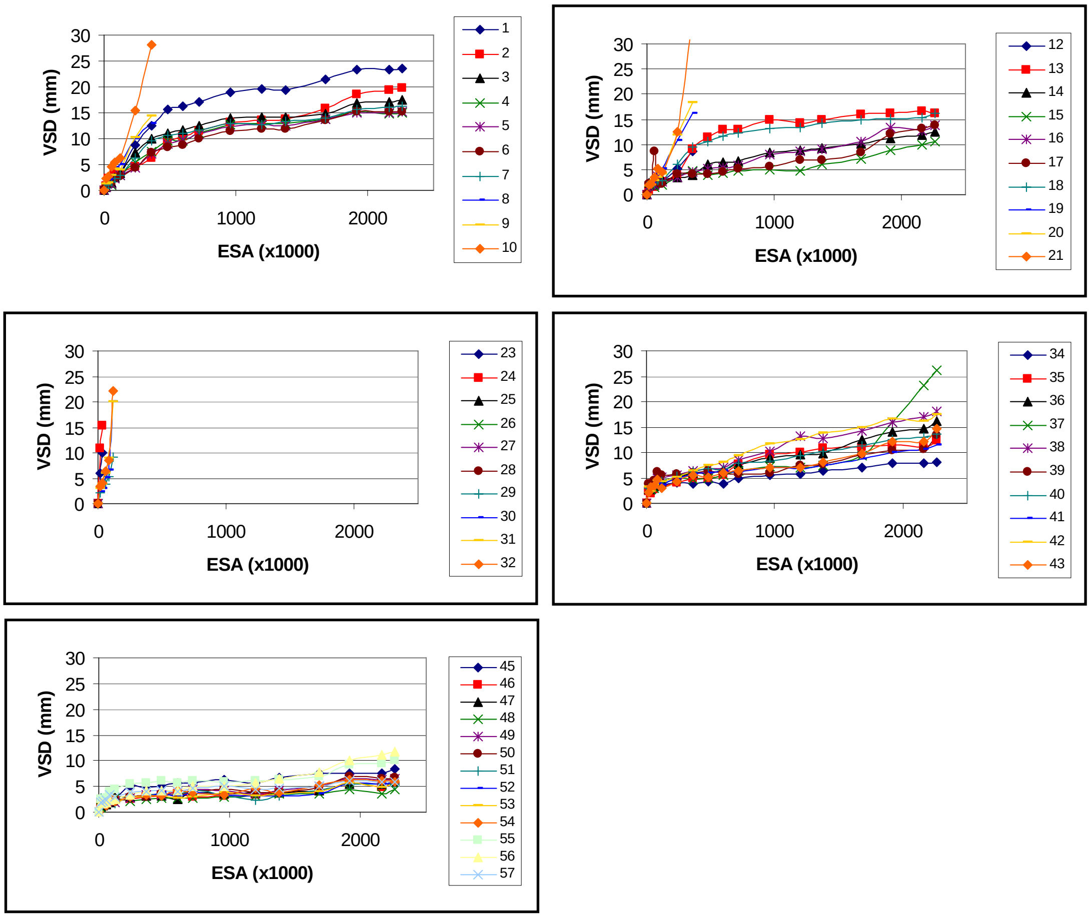


Figure 2.12 Final VSD values under 50 kN wheel path.

The VSD profiles for each section of the track are presented in Figures 2.12 and 2.13. The profiles suggest that the VSD limit of 15–20 mm deformation is reasonable. Only stations 1 and 2 in Section A exceed this limit, but they had been surfaced with the dense asphaltic concrete which laboratory fatigue testing would suggest is more tolerant to deformation. The profiles also suggest that, despite the relatively sharp-looking failures in Figures 2.7–2.9 suggesting a shear failure in the basecourse, the basecourse used had sufficient shear strength when placed on sufficiently strong subgrades.

Figure 2.14 shows the vertical resilient strains measured by εmu coils at different depths in each section during the test. The εmu strain measurements shed additional light on the results, and show that Sections A and B perform in a similar manner with similar strains, while Section E with smaller strains on the same subgrade performs better. However Section D's performance calls into question the current Austroads Pavement Design subgrade strain criterion, because it had incredibly high strains recorded in the weaker subgrade (at 380-460 mm) yet it performed as well as Sections A and B. However the current Austroads Pavement Rehabilitation Guide (2004a) would estimate that this Section D would have failed well before Sections A and B.

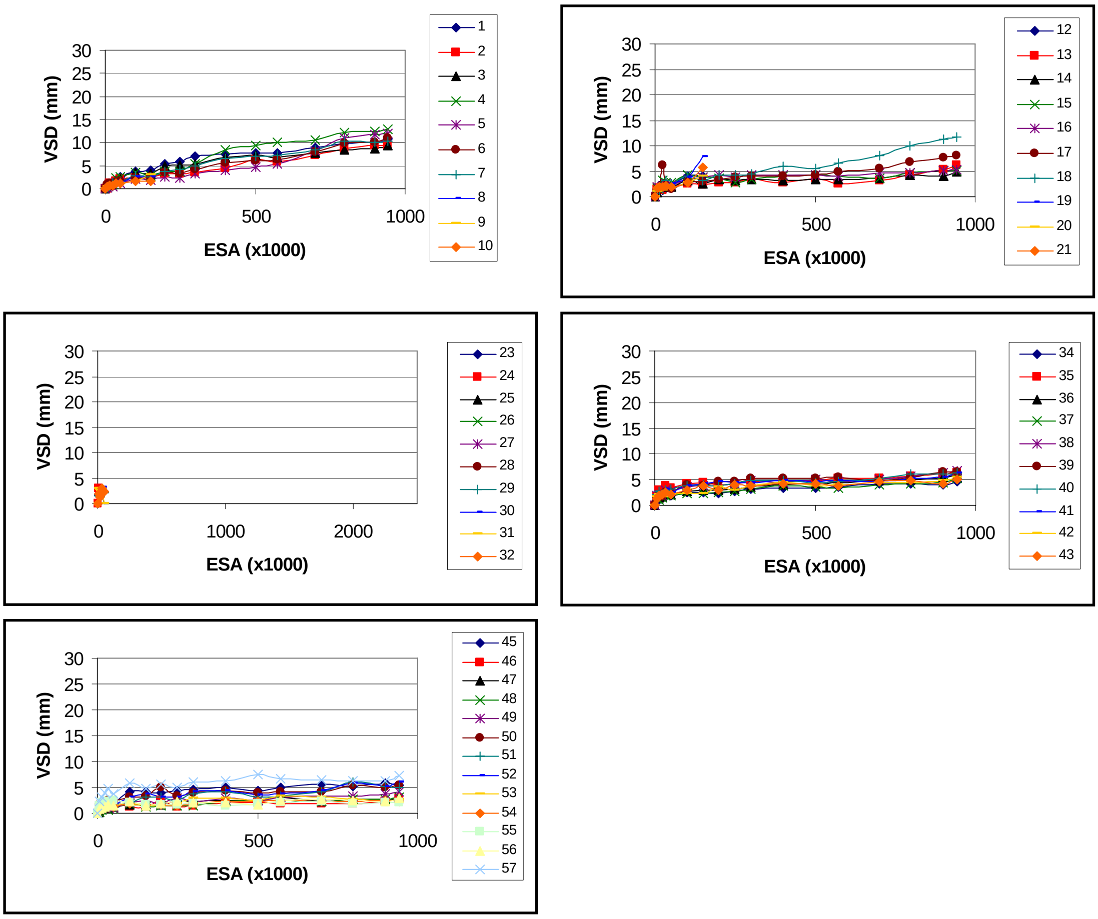


Figure 2.13 Final VSD values under 40 kN wheel path.

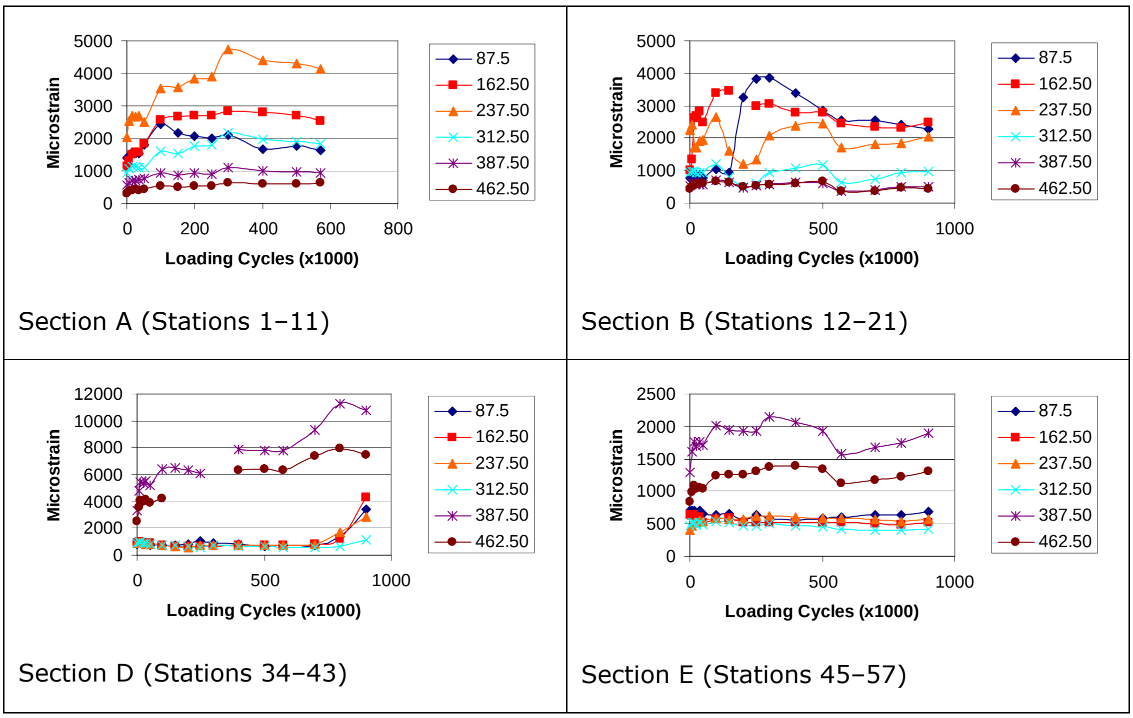


Figure 2.14 εmu strains for 135 cm wheel path at 50 kN, for Sections A, B, D, E.

2.6 Pavement Test 1 conclusions

As observed in Chapter 1, the results of the first accelerated pavement test were surprising given the Austroads deflection and curvature approaches to asphalt overlay design. The results suggested that sufficient curvature could not be developed to fatigue a thin surface before failing the surface by rutting. This result suggested that improving the durability of porous asphalts with polymers would obviously not improve the fatigue life of the surface. However it did suggest that trafficking unbound pavements before applying a porous asphalt may improve the life of the surface.

Therefore the project's Steering Group was asked to re-examine options for Pavement Test 2 and for testing a new hypothesis. Discussions are outlined in Chapter 3.

The construction data presented in Section 2.2 of this report shows that the pavement sections were well constructed, and the FWD data and analysis presented in Section 2.3 show that, according to the Austroads Pavement Rehabilitation Guide, the pavement sections should have failed in fatigue well before failing in deformation. Section C was predicted to fail in deformation and fatigue well under 10,000 ESA and with the exception of the stations that failed statically (after a vehicle was parked on it overnight) at zero cycles, first failed in deformation after 36,000 ESA. Sections A and B had been given predicted fatigue lives of 400,000 ESA and deformation lives of 4,000,000 and 1,600,000 ESA respectively, yet they failed in deformation at 360,000 ESA. Review of the VSD data indicates that pavements are failing when the peak VSD value for a failing area reaches approximately 15-20 mm VSD.

3. Steering Group discussion

The results of Pavement Test 1 were presented to the project's Steering Group which comprised Dr Bryan Pidwerbesky from Fulton Hogan, John Patrick from Opus's Central Laboratories and Dr Greg Arnold from Transit New Zealand. They discussed if Pavement Test 2 could be modified to re-examine the option of trafficking unbound pavements before applying a porous asphalt, and for testing the new hypothesis that basecourse deformation was in fact the cause of cracking in thin asphalt layers.

3.1 Mode of failure

Given that the mode of failure was deformation rather than fatigue, the Steering Group and researchers agreed that the implications of the deformation mode should be further examined rather than testing different binders. Of particular interest were pavements which had already been trafficked, as this would be the case in a typical rehabilitation project. The proposal was that the pavement be rebuilt with the cross-sections used for Sections B and D, trafficked initially without a porous surfacing, and then the surfacing applied. The benefits of this would be two-fold: first it would confirm whether the mode of failure was deformation or fatigue; and second it would provide an indication of whether pre-trafficking a pavement before laying the surface layers could be a way of improving the life of porous surfacings laid over new pavements.

3.2 Alternative pavement materials

The Steering Group asked the researchers to confirm that the curvatures were reasonable, and whether the curvatures of the deflection bowl could be increased by either using sand with rubber buffings or a volcanic subgrade from the central North Island.

Sand with rubber buffings had been used as an alternative subgrade at CAPTIF in the 1980s (Finnegan & Seddon 1981). However only central deflection data could be obtained, and these values were similar to those being currently used (around 700-800 μm). Curvature results on an earlier pavement with a 12-mm layer of straight rubber buffings had resulted in 200-300 μm curvatures which had been considered unnaturally high in their associated report. This result suggested that sand and rubber are likely to have lower curvatures. Discussions with local consultant Mike Smith of MWH (Montgomery Watson Harza, Christchurch) suggested that, if the sand was not well compacted, there would also be issues with rutting and that considerable amounts of water would be required for the compaction. The idea was discounted as not feasible.

Another alternative of using volcanic material from the central North Island was discussed with Ken Hudson of Duffill Watts and Tse. He noted that the sensitive structure of volcanic subgrades, which leads to their high strength and high deflections, would be lost on re-compaction of any volcanic material imported to CAPTIF. So that idea was also abandoned.

The appropriateness of the curvature achieved in Pavement Test 1 was checked by comparing it to data from the RAMM database. It was compared to FWD data from four Transit regions corrected to 566 kPa.

Table 3.1 shows that the average initial values of curvature obtained at CAPTIF (see Tables 2.8 and 2.11) exceed the average from each region and are close to the 90%tile values recorded in each region. When compared to the FWD readings at 115,000 load cycles, Sections A and B had average readings that were comparable to the 95%tile readings and Section D had readings comparable to the 90%tile readings.

Table 3.1 FWD results from the RAMM database for four Transit regions.

RAMM Region	Average d0–d200 (µm)	90%tile d0-d200 (µm)	95%tile d0-d200 (µm)	90%tile d0 (µm)	95%tile d0 (µm)
Canterbury	190	350	450	1200	1600
Napier	195	350	400	1100	1300
Hamilton	195	350	450	1500	1900
Wanganui	225	400	500	1900	2100+

The Steering Group also suggested examining data recorded from the Taupo Transit region, which is acknowledged to be a high deflection area. However, a search of the RAMM database for the Taupo area could not find any curvature values above 450 µm, despite some central deflections in excess of 2000 µm. A site on SH1 at RS[1] 613 at Taupo is typical of the area, with a 95%tile curvature of 275 µm and a 95%tile d0 of 1400 µm. Another site, RS 664 well south of Taupo, provided the most extreme data found in the area, with a 95%tile curvature of 425 µm for a 95%tile d0 of 1600 µm, and maxima of 700 µm and 2200 µm respectively. This indicates that the curvature values that were obtained in the first pavement should be sufficient to validate the figures given in the Austroads Rehabilitation Guide for known high deflection problem areas such as Taupo.

3.3 Design for Pavement Test 2

The pavement design adopted for Pavement Test 2 was to use the structures from Sections B and D. The Steering Group also suggested using two surfaces: the standard HS OGPA used in the first test, and an OGPA with a harder binder that would be more fatigue-prone.

Finite Element Method analysis of Pavement Test 1 combined with analysis using the Shell Nomographs in the Austroads Pavement Design Guide (2004b) suggested that increasing the stiffness of the binder by using a 60/70 penetration grade bitumen and using a slightly thicker 50 mm layer, should reduce the fatigue life by half that of Pavement Test 1. Given the performance of the OGPA material in the laboratory tests this should result in fatigue at a quarter of the life predicted by Austroads for dense graded asphalt. The design details for Pavement Test 2 based on these decisions are given in Chapter 4.1.

[1] RS – Route Station.

4. Pavement Test 2

4.1 Pavement design

As a result of the Steering Group discussions (Chapter 3), the design of Sections A and B for Pavement Test 2 was 150 mm of basecourse on a Tod subgrade at OMC, which should give a curvature of 450-500 µm after initial trafficking. Section A was surfaced with 50 mm of the standard OGPA and Section B was surfaced with the harder binder OGPA.

Sections C and D consisted of 300 mm of basecourse on a Tod subgrade at OMC+8%, which should give a curvature of 300 µm after initial trafficking. Section D was surfaced with the OGPA with standard binder, and Section C was surfaced with the OGPA with harder binder.

Trafficking was in one wheel path only, with a wheel load of 40 kN. The basecourse initially was surfaced with a chipseal, and 134,000 load applications were applied to condition the pavement and to allow the initial rutting to occur. Limited measurements were taken during this stage. The chipseal was overlaid with either of the two OGPA surfacings and trafficked to 1,000,000 load applications.

Figure 4.1 is a graphical representation of the pavement structure. The OGPA with harder binder was laid from stations 09 to 38 and the standard OGPA was applied from stations 38 to 09.

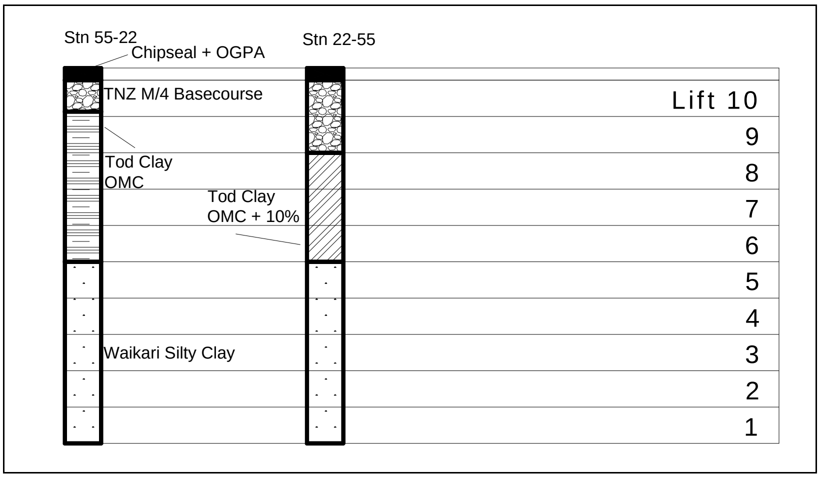


Figure 4.1 Pavement structures used for Sections A, B (left) and Sections C, D (right) for Pavement test 2.

4.2 Pavement instrumentation

For this second stage of the project the εmu system was again installed. Sections A and B of Pavement Test 1 were of the same pavement structure but appeared to have relatively significant variations in strain (Figure 2.14). This time the coils were spread over a number of stations both under the wheels and between the wheels to examine this variation, as well as to give further insight into pavement behaviour.

4.3 Pavement construction

4.3.1 Subgrade density and CBR testing

Densities were measured for each subgrade layer with a Nuclear Density Meter, which was calibrated with sand replacement tests. Table 4.1 suggests that the correct moisture content for the Tod clay (at OMC) in Sections A and B was achieved, although the densities were harder to achieve and the results were typically 100 kg/m³ lower than targeted. Working with the Tod clay at OMC+8% was again difficult, and the density was 50 kg/m³ higher than targeted and the moisture content was 1-2% lower.

Table 4.1 Subgrade densities (DD) and moisture contents (MC) for subgrades (SG) of Sections A to D.

Section	Avg DD (kg/m^3)	SD DD (kg/m^3)	Avg MC (%)	SD MC (%)
A	1505	50	22	2
B	1504	33	22	1
C	1493	28	26	1
D	1487	33	27	1

Three Scala penetrometer tests were undertaken in each Section to test the in-situ strength of the constructed subgrade. The in situ CBR was estimated from the RG Brickell relationship, and the readings given in Table 4.2 are the average estimated CBRs for the upper 300 mm of the subgrade. It shows the construction is uniform but that Sections A and B are weaker than the subgrade used in Pavement Test 1.

Table 4.2 Average estimated CBRs in the upper 300 mm of the two subgrades under the two OGPA surfaces.

Section	Surface material	Subgrade material	Avg CBR %	Min of CBR%	Max of CBR%
A	OGPA	Tod OMC	6	6	6
B	Hard OGPA	Tod OMC	5	4	6
C	Hard OGPA	Tod OMC+10%	2	2	3
D	OGPA	Tod OMC+10%	2	2	3

4.3.2 Basecourse construction

The basecourse used for the second test was TNZ M/4-compliant AP40 basecourse from Fulton Hogan's Pound Road Quarry, and the basecourse used in the overlay itself was also TNZ M/4-compliant AP20 material from the same source, and overlaid using a basecourse paver.

Tables 4.3 and 4.4 provide details of final Densities (DD), Moisture Content (MC), Percentage of Maximum Dry Density (%MDD) and Percentage of Saturation (%SAT) achieved in each section from the original pavement without overlay, and the pavement with overlay. While these values are a little less than those used to comply with TNZ B/2, they are typical of what is achieved at CAPTIF and are again indicative of the difficulties of compacting material on very weak pavements.

Table 4.3 Densities (DD) and moisture contents (MC) of basecourses (BC) of the original pavement for Pavement Test 1 with no overlay.

Section	Avg DD (kg/m^3)	SD DD (kg/m^3)	Avg MC (%)	SD MC (%)	Avg % MDD	SD % MDD	Avg %SAT	SD %SAT
A	2208	25	2	0	95	1	28	3
B	2206	23	2	0	95	1	30	3
C	2244	22	3	0	97	1	39	7
D	2250	32	3	0	97	1	40	6

Table 4.4 Densities (DD) and moisture contents (MC) of basecourses of the pavement for Pavement Test 2 with overlay (BC Overlay).

Section	Avg DD (kg/m^3)	SD DD (kg/m^3)	Avg MC (%)	SD MC (%)	Avg %MDD	SD %MDD	Avg %SAT	SD %SAT
A	2164	38	3	0	96	2	34	6
B	2175	20	4	0	97	1	42	6
C	2178	65	3	1	97	3	38	12
D	2147	35	3	1	95	2	35	7

4.3.3 Surface construction

The pre-overlay surface is not described for brevity's sake because the overlay was laid on it without removing or scarifying it. The surface of the basecourse component of the overlay, once dry, was swept by power-broom and primed with emulsion using a hand lance. The contractor successfully calibrated the CAPTIF spray bar to BCA T/1:1999 Modified, with calibration carried out on a simulated test track in the contractor's yard. The two-coat chipseal applied before laying the asphalt and OGPA surfacings was a success. Chip was spread with a standard spreader truck, the gates on the roller feed were adjusted for the radius of the track, and a steel/rubber-combo roller pressed in the chip.

The chipseal construction details are as follows:

Emulsion Prime Coat, handsprayed at approx, 0.3 ℓ/m^2 to leave approx. 0.15 ℓ/m^2 residual binder, followed by

First Coat Two Coat Seal, 180/200 pen grade, 1 pph Kerosene, 0.8 pph Megamine Adhesion Agent, Grade 3 & 5 chip, Target Application Rate 1.80 ℓ/m^2 hot.

After the initial trafficking, the chipseal surface was heavily tack-coated and the TNZ P/11 compliant HS [high strength] OGPAs with 60/70 and 80/100 binders were placed by an asphaltic concrete paving machine. The sealing crew used a footpath roller behind the paving machine. Once the paving machine had completed the track and left the building, the entire surface was rolled with a 3.5-tonne steel-drum roller.

4.3.4 Layer thicknesses

Layer thicknesses have been measured with a 4-m beam and ruler at each permanent station mark during construction. The final layer thickness in each section is given in Table 4.5 and it shows that the design thicknesses were realised in the original pavement. The variation in the basecourse depth in Sections C and D is surprisingly low considering the very soft (Tod+10%) subgrade.

Table 4.5 Thicknesses of the original pavement layers in Sections A–D.

Section	Average basecourse thickness	
	Depth (mm)	SD (mm)
A	152	10
B	145	7
C	334	11
D	335	8

The total basecourse, overlay and OGPA thicknesses in each section achieved after the overlay was laid are given in Table 4.6. It shows that the overlay thicknesses are reasonably consistent in Sections B, C and D, but that the paving machine took some time to establish the correct thickness in Section A. The targeted thickness of 50 mm for the OGPA was achieved.

Table 4.6 Layer thicknesses (for OGPA, basecourse and overlay) achieved after overlay was put down.

Section	Average thickness OGPA		Average thickness Basecourse		Thickness of Overlay
	(mm)	SD (mm)	(mm)	SD (mm)	(mm)
A	50	5	243	10	91
B	47	3	221	8	76
C	45	4	405	15	71
D	52	5	400	9	66

4.4 Pavement characterisation

Tables 4.7 to 4.9 provide a summary of the initial FWD readings in each section. FWD testing was undertaken directly after construction (Table 4.7), directly after the construction of the overlay (Table 4.8), after initial trafficking of 67,000 cycles (134,000 ESA) and before the OGPA surfaces were applied (Table 4.9).

Table 4.7 Initial FWD deflection of basic pavement directly after construction, before laying the basecourse overlay, for the 190 mm wheel path.

Section	Av. Stress (kPa)	SD Stress (kPa)	Av. d0 (µm)	SD d0 (µm)	Av. d0-d200 (µm)	SD d0-d200 (µm)	CV
A	551	17	1418	258	679	159	0.18
B	533	8	1769	141	856	82	0.08
C	517	14	1529	378	633	162	0.25
D	523	8	1486	206	619	121	0.14

d0 – central deflection (µm)
d0-d200 – bowl curvature (µm)
CV – Coefficient of Variation

Table 4.8 Initial FWD deflection after construction of the overlay, and before initial trafficking, for the 190 mm wheel path.

Section	Av. Stress (kPa)	SD Stress (kPa)	Av. d0 (µm)	SD d0(µm)	Av. d0-d200 (µm)	SD d0-d200 (µm)	CV
A	614	8	1032	185	470	144	0.18
B	615	9	1235	239	570	123	0.19
C	622	19	1257	284	479	151	0.23
D	626	14	1314	155	490	86	0.12

Table 4.9 Initial FWD deflection after construction of overlay and after the initial trafficking of 67,000 cycles (and before OGPA surfacings were applied), for the 190 mm wheel path.

Section	Av. Stress (kPa)	SD Stress (kPa)	Av. d0 (µm)	SD d0 (µm)	Av. d0-d200 (µm)	SD d0-d200 (µm)	Av. Stress (kPa)
A	619	5	888	113	407	62	0.13
B	619	5	1020	103	459	50	0.10
C	617	9	1179	200	410	85	0.17
D	619	11	1176	209	395	97	0.18

The deflections were corrected to a stress of 566 kPa using the Austroads assumption that the deflections are linearly related to stress. A seasonal correction was not applied to the data as the pavement is in its weakest state being indoors. A correction of 1.10 was applied to the deflections to normalise them for use with the deformation charts (Figure 6.3 in Austroads 2004a).

Table 4.10 Characteristic deflection (CD) and curvature values (CC) after initial trafficking, for three values of factor *f*, for the 190 mm wheel path.

Section	Normalised Values			
	CD for three *f* values			CC
	2	1.65	1.3	0
A	1121	1081	1041	372
B	1232	1196	1160	419
C	1592	1521	1451	376
D	1603	1530	1456	362

For explanation of factor *f*, see Pavement Test 1.

Deformation and fatigue lives were estimated from Figures 2.4 and 2.5 (in Austroads 2004a) and are provided in Table 4.11. The Weighted Mean Average Annual Pavement Temperature (WMAPT) used was the average surface temperature during loading which was 14.7°C.

Table 4.11 Predicted pavement lives obtained from Austroads Rehabilitation Manual (2004a) for three factor *f* values in the 190 mm wheel path, at WMAPT of 14.7°C.

Section	Life (ESA x10^6)			
	Permanent Deformation (mm) for three *f* values			Fatigue (mm)
	2	1.65	1.3	0
A	1.1	1.2	1.8	0.18
B	0.6	0.7	0.8	0.21
C	<0.1	0.11	0.18	0.18
D	<0.1	0.11	0.18	0.18

Note that initial laboratory testing in Pavement Test 1 suggested that OGPA should fail in half the predicted time, and using the harder binder on Sections B and C should therefore mean those sections should fail in one quarter of the time predicted by the Austroads calculations.

4.5 Pavement performance

Loading was applied to the test pavement (Figure 4.2) between August and December 2005. The initial pavement developed significant ruts (Figure 4.3) around the track and a shear failure between stations 20–24 (Figure 4.4) after 10,000 ESA[2]. The decision was made to apply a nominal 70-mm granular overlay to the track and start again as the level of rutting was far too high when compared to the level of loading applied. After applying the overlay, 134,000 ESA were applied on the 190-cm wheel path with both vehicles loaded to 40 kN. A number of minor surfacing failures occurred that were then patched (Figures 4.5–4.6). After the OGPA surfaces had been laid a further 1,000,000 ESA were

[2] 2 ESA = 1 cycle.

applied with the only incident being a debonding failure at stations 25–27 after 200,000 ESA (Figure 4.7). Figure 4.2 indicates where and when the failures were observed in the test pavement.

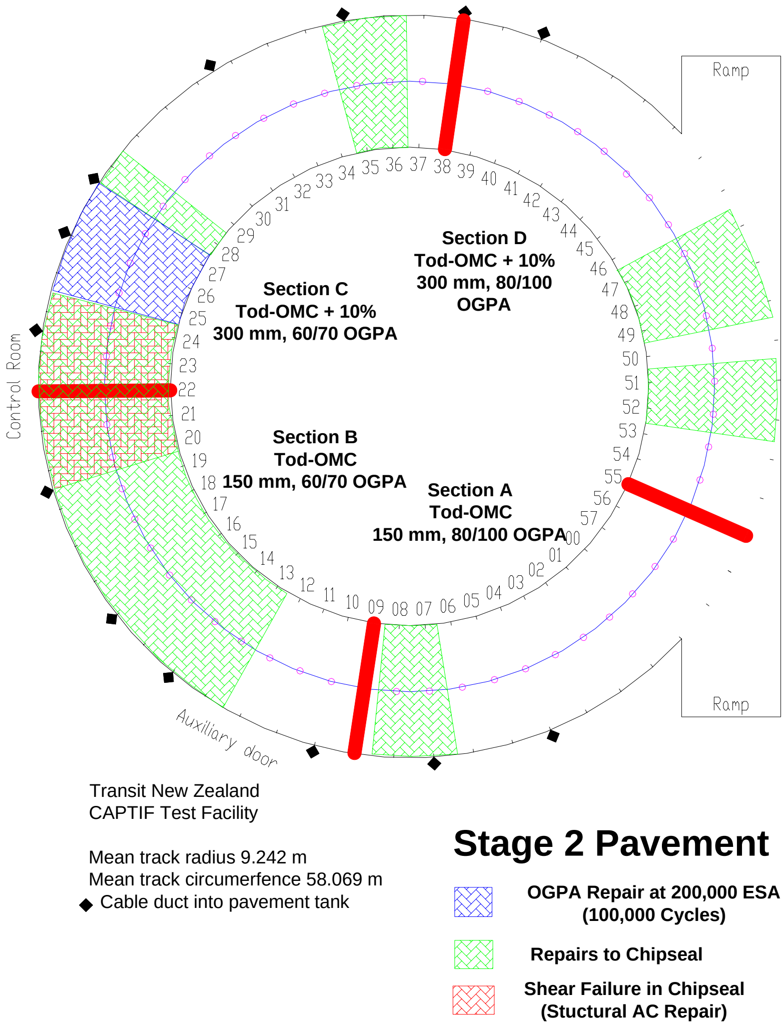


Figure 4.2 Pavement Test 2: positions of repair locations and when failures occurred.

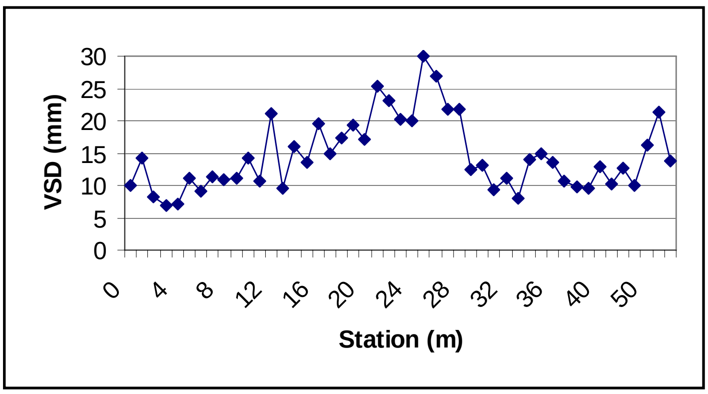


Figure 4.3 Initial Pavement VSD recorded after 10,000 ESA.

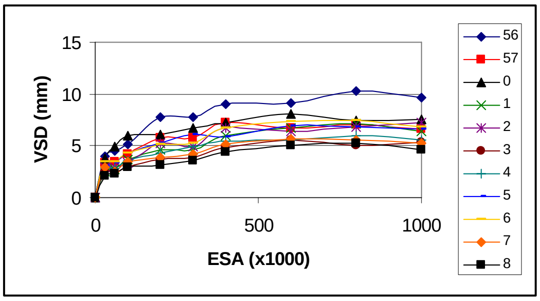
Figure 4.4 Initial construction shear failure between Stations 20–24.

Figure 4.5 Typical surface failures that occurred at different stations during Pavement Test 2.

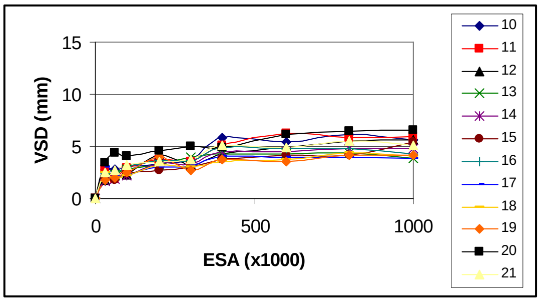
Figure 4.6 Typical surface repair of a failed area.

Figure 4.7 OGPA debonding failure at 200,000 ESA between Stations 25–27.

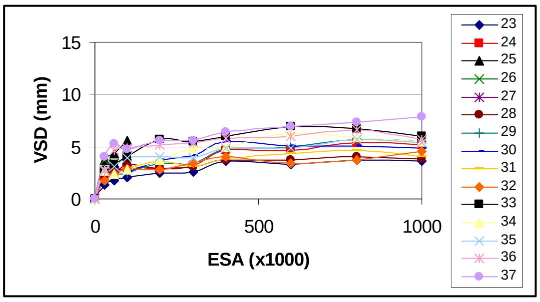
As noted above, the pavement performed very well. Figure 4.8 provides the VSD progression for each station, and shows that, with pre-trafficking and using the 40 KN load, none of the sections came close to the 15 mm VSD limit that was causing problems with deformation-related failures in Pavement Test 1. No cracking was observed despite the fact that a harder binder was used and failure had been predicted after trafficking as low as 50,000 ESA.



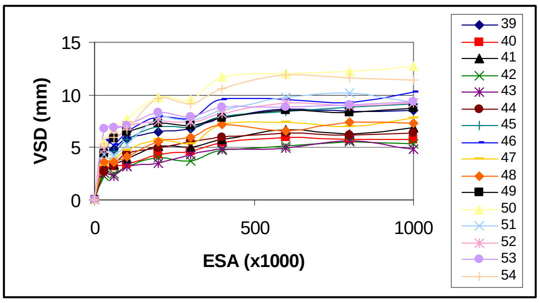
Section A



Section B

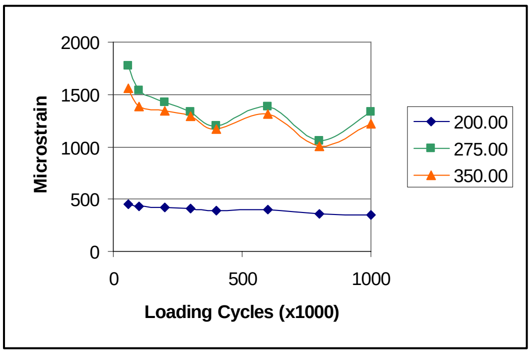


Section C

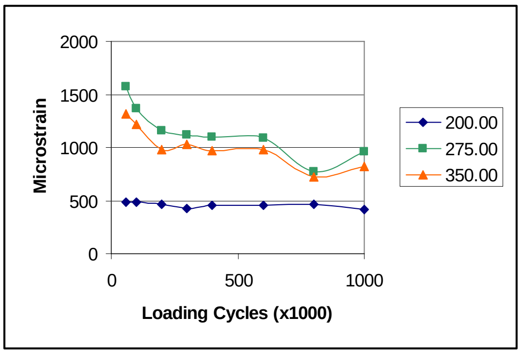


Section D

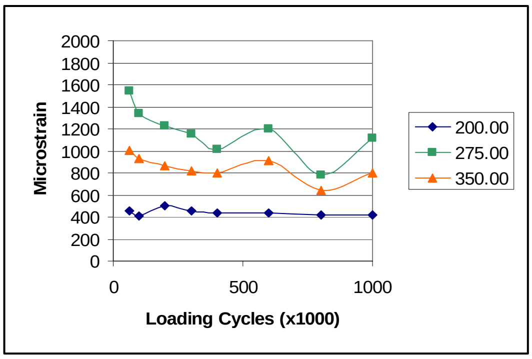
Figure 4.8 VSD progression in Sections A and D with standard OGPA, and Sections B and C with hard OGPA.



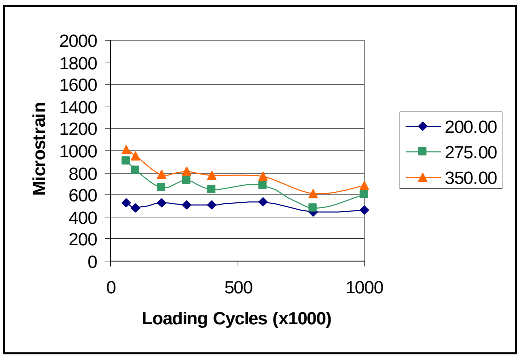
Station 0



Station 2

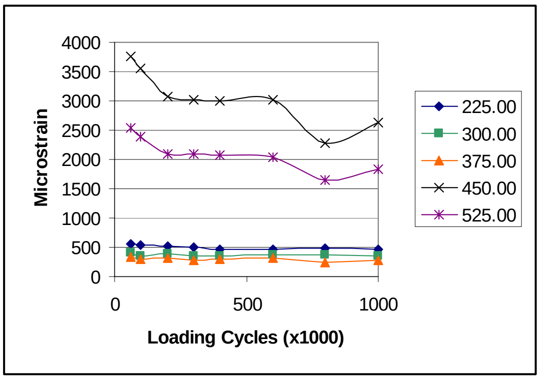


Station 4

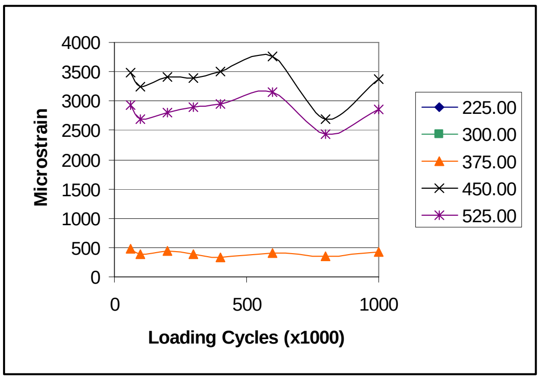


Station 6

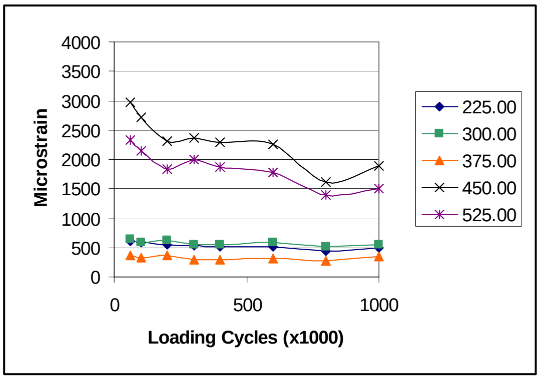
Figure 4.9 εmu strains between wheels at Stations 0, 2, 4, 6.



Station 38

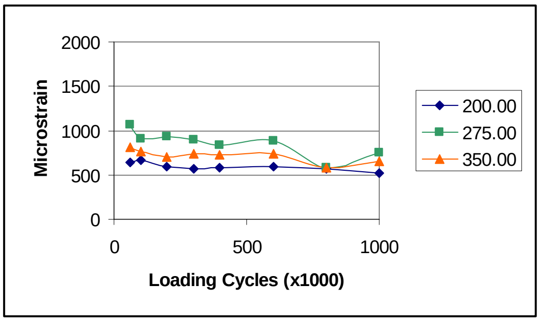


Station 40

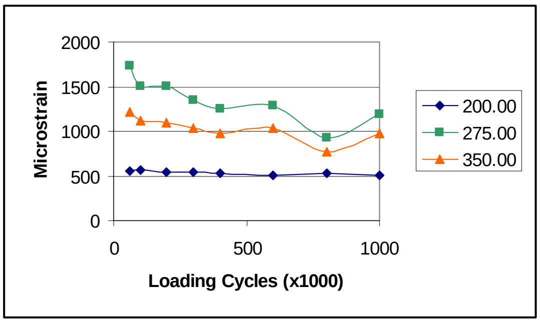


Station 42

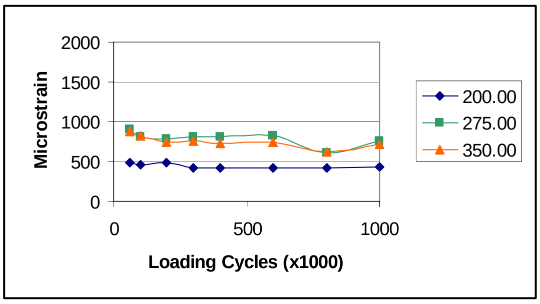
Figure 4.10 εmu strains between wheels at Stations 38, 40, 42.



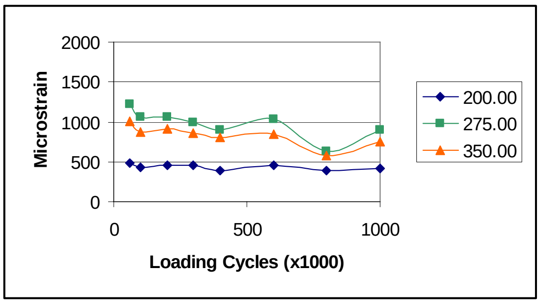
Station 1



Station 3

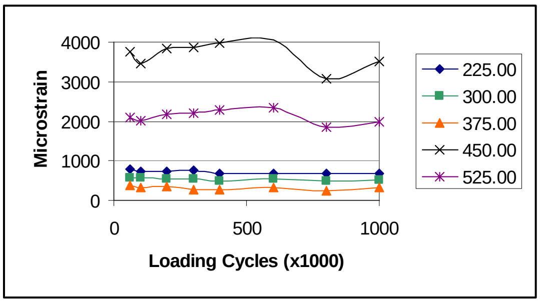


Station 5

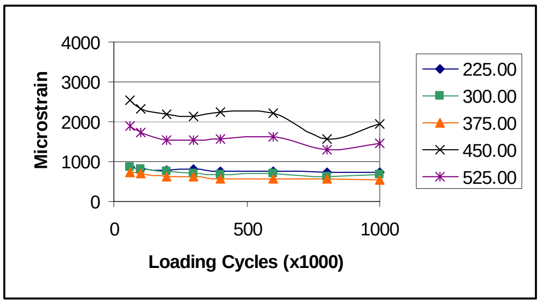


Station 7

Figure 4.11 εmu strains under wheels at Stations 1, 3, 5, 7.



Station 39



Station 41

Figure 4.12 εmu strains under wheels at Stations 39 and 41.

Figures 4.9–4.12 provide insight into why the second test had performed so well, as they show the progression of vertical strain at various depths within the pavement during the test. While there is significant variation within the measurements in each section, the downward trend in strains as the test progressed is the same at all the stations. This suggests that the pavements are strengthening in both the subgrade and the basecourse.

The reason for the variation between stations was investigated. Standard coil pairs had been installed and, as part of the routine εmu installation, were fixed 75 mm apart using a specially prepared rod. If the standard pair is measuring a constant value, then the electronics of the εmu system are operating correctly. Figure 4.13 shows that the εmu system was operating correctly throughout the project, and it suggests that the variation was unlikely to be an error in the measurements obtained by the coils.

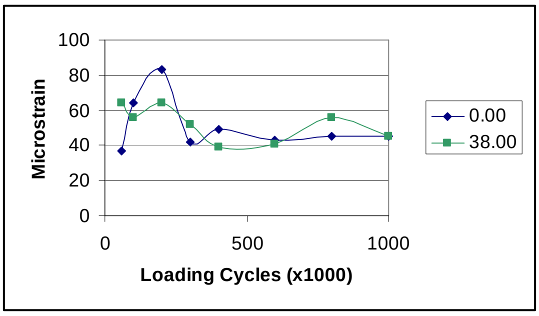
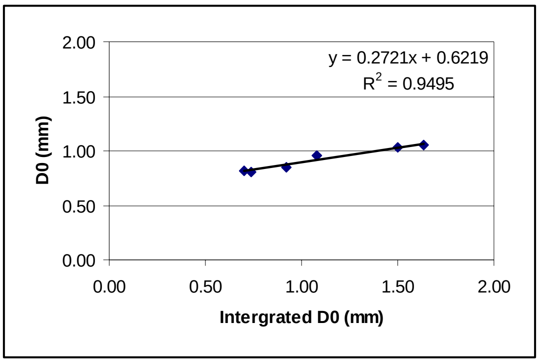
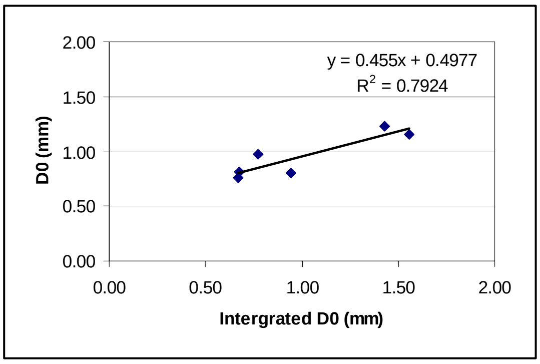


Figure 4.13 Traces from εmu standard pairs to check that the εmu system is operating correctly.

To examine if the variation in εmu readings is related to the materials used in the pavement, the measured strains over the depth of the pavement could be integrated, and thence an integrated central deflection that relates to the Central Deflection of the FWD could be derived (Figure 4.14).

The integration is very basic as only a few data points were available. Between the wheel it assumes that the strain was zero at the surface and at the bottom of the tank. Under the wheel it assumes the strain is constant from the top pair of εmu coils to the wheel, and drops to zero at the base of the tank. The relationship is not one to one as the number of gauges is limited, the loading arrangement is not the same, and integration assumptions are simplistic. Given that the relationships are reasonable between the integrated central deflection and the central deflection of the FWD, the variation of the strain readings does appear to be related to variation in the pavement materials.





Under wheel **Between wheel**

Figure 4.14 FWD d0 v Integrated εmu d0 under and between wheels.

4.6 Pavement Test 2 conclusions

Pavement Test 2 was designed and built with deflections and curvatures which, according to the Austroads Pavement Rehabilitation Guide (2004a), should have failed in fatigue at approximately one fifth of the 1,000,000 ESA applied. The laboratory testing from Pavement Test 1 suggested that, for the standard binder OGPA, fatigue should have occurred at half the life predicted by Austroads, and the Shell Nomographs (used in Austroads 2004b) suggest that the harder binder OGPA should have failed at quarter of the loads predicted by Austroads.

Both rutting (VSD) and strain data suggested that the pavement was performing well when the test was stopped at 1,000,000 ESA. The strain data suggested that both the subgrade and basecourse were hardening with time. Linear extrapolation of VSD data suggests that all the sections would have survived another 2,000,000 to 3,000,000 ESA before deformation became a problem.

Pavement Test 2 confirms the findings of the first test, that the Austroads Rehabilitation Guide design requirements for deflection and curvature are very conservative.

Table 4.12 indicates that, while the second pavement had significantly lower predicted deformation and fatigue lives, its rutting performance was comparable to the pavement in Pavement Test 1 at the end of the initial compaction phase (approximately 100,000 ESA), Table 4.12 also indicates that the initial trafficking has reduced the VSD progression and would help prevent the failures recorded in the first test.

Table 4.12 Effect of initial trafficking on the development of VSD.

Section	Predicted Life x10^6 (ESA)		VSD at 100,000 ESA (mm)	
	Test 1	Test 2	Test 1	Test 2
A	60	1.8	3.2	3.9
B	8	0.8	3.4	2.7
C		0.18		3.5
D	8	0.18	3.2	5.3

5. Conclusions

The research aimed to achieve the following objectives:

- develop a horizontal tensile strain versus fatigue life curve that can be applied to the base of an Open Graded Porous Asphalt (OGPA) layer;
- establish a relationship between basecourse surface curvature and OGPA fatigue life;
- evaluate the extension to fatigue life of using enhanced binders in OGPA,

with the aim of improving the fatigue design of surfaces that will reduce road noise.

These objectives were based on the traditional view of thin asphalt behaviour applied in the Austroads Pavement Design and Rehabilitation Guides (2004a, b) and existing research into asphalt behaviour. The first objective was not achieved as it was not possible to find instrumentation of a scale that would not interfere with the performance of the thin surfaces. The second objective found that deformation leads to surfacing failure before fatigue occurs and, as a result, the third objective was modified by agreement with the Steering Group to examine this problem in more depth.

Sections A, B and C of Pavement Test 1 showed that deformation leads to surface failure before fatigue occurs in pavements that have high basecourse curvatures. Section E in Pavement Test 1 showed that, if pavements are constructed well, then applying surfaces that reduce low noise immediately after construction is possible.

The laboratory fatigue results contradict current wisdom that OGPA is more tolerant of deflection than asphaltic concrete. However without being able to generate fatigue, the validity of this cannot be determined.

Pavement Test 2 showed that on existing pavements the deflection criteria could be safely increased if trafficking was undertaken first. The current deflection criteria are therefore conservative even if factors such as temperature and ageing are considered.

6. Recommendations

1. Predicting surface failure

The outcomes of the project suggest that the Austroads Rehabilitation Design Guide is very conservative in predicting fatigue which in reality occurs much later. Practically speaking, deformation leads to failure of the surface before fatigue of the surface occurs.

2. Timing the application of low noise surfaces

The first test (Pavement Test 1) showed that, if pavements are constructed well, then applying low noise surfaces immediately after construction is possible. From analysis of the first test FWD readings, a conservative approach would consider all deflections having curvatures over 250 μm to be unacceptable (on pavements with design loadings over 100,000 ESA) and that such results would require additional analysis.

In this additional analysis, pavements with basecourse of known good rut resistance performance and a degree of saturation below 60% could be identified, and surfaced immediately.

3. Using initial trafficking to reduce deformation

The second test (Pavement Test 2) suggests that surfaces failing the performance and saturation criteria would be acceptable after an initial trafficking of 100,000 ESA. This trafficking would reduce initial deformation and thus lead to acceptable surface life.

In this case, if RMA (Resource Management Act) noise requirements need to be met immediately, the speed limit on the pavement could be lowered while the chipseal surface is trafficked.

If lowering the speed limit is not practical, the basecourse could be modified to increase its resistance to deformation, and then the OGPA is applied.

Further analysis of field performance should be performed to assist in confirming the new criteria for sealing with low noise surfaces.

4. Further research

Further research into the prediction of deformation in unbound granular pavements and methods to prevent post-compaction rutting would give additional confidence when sealing with low noise surfaces on new road construction.

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