The design of stabilised pavements in New Zealand

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Abbreviations and acronyms

CAPTIF	Canterbury Accelerated Pavement Testing Indoor Facility
CBR	Californian Bearing Ratio
CSIR	South African Council for Scientific and Industrial Research
CV	coefficient of variation
DD	dry density
D0	central deflection
DOS	degree of saturation
ESA	equivalent standard axles
FWD	Falling Weight Deflectometer
HMA	hot-mix asphalt
ITMR	indirect tensile resilient modulus
ITS	indirect tensile strength
MC	moisture content
MDD	maximum dry density
MESA	millions of equivalent standard axles
OWC	optimum water contents
RLT	repeated load triaxial
SAR	standard axle repetitions
SARc	standard axle repetitions for estimating damage to cement-bound materials
TSR	tensile strength retained
UCS	unconfined compressive strength
VSD	vertical surface deformations
WIM	weigh-in-motion

Contents

Executive summary

Areas of New Zealand are running out of the premium aggregates that meet the demanding specifications required for unbound granular road construction. Stabilising aggregate provides a viable alternative to the use of premium aggregates. Stabilisation is defined as a process where the intrinsic properties of an aggregate are altered by the addition of a stabilisation binder, in order to meet the performance expectations in its operating environment.

This research project validates the benefits of stabilisation and provides new design tools to allow recycling of existing roads and the use of modified marginal aggregates where additional aggregate is required on new roads.

Auckland alone is predicted to require 12 million tonnes of aggregate in 2020, a doubling of what was required in 1991. Because aggregates are a finite resource and there are restrictions on the development of new quarries, the problem Auckland is facing now will also become apparent in other areas of New Zealand. In addition, the funding of improvements to the roading infrastructure in areas such as the east coast of the North Island is becoming increasingly difficult, as the low traffic volumes and the large transport distances required to supply quality aggregates mean the required economic criteria are often not met.

The objective of this project was to improve the sustainability of New Zealand roads by undertaking the following tasks:

1. Determine the benefits of using cement- and/or lime-modified aggregates in terms of increased performance (rutting resistance) and incorporate this in a design methodology, filling a gap identified by Austroads.
2. Validate the benefits of foamed bitumen/cement-stabilised aggregates in terms of increased performance (rutting and fatigue resistance) and incorporate this in a design methodology, building on research by the University of Canterbury and/or the South African Interim Technical Guidelines.
3. Understand the continuum from unbound (no binder), modified (small amounts of binder) to bound (high amounts of binder) behaviour.
4. Review the appropriateness of the Austroads tensile strain criterion for bound aggregates, which is considered by many New Zealand designers to be overly conservative.

The methodology to complete the objectives involved a combination of accelerated pavement tests at the Canterbury Accelerate Pavement Testing Indoor Facility (CAPTIF) in 2007/2008 and 2008/2009, and a limited field review of the performance of bound stabilised pavements.

Key findings and recommendations for the project objectives:

- Objective 1: It was shown that modifying the tested aggregates with 1% cement could reduce rutting and improve the rutting life of the pavement by 200–300% compared with the unbound pavement. However, stiffness loss occurred during this testing. Section 6.4 of this report presents a design methodology that can be used with initial laboratory data to estimate the initial improvement in rutting performance. Further research is needed into the implications of this stiffness loss in cement-only materials. Figure 6.6 in this report presents a wider approach that includes other recent NZ Transport Agency (NZTA) research to address the stiffness loss.
- Objective 2: It was shown that modifying aggregates with foamed bitumen and cement reduced rutting and created a 500% improvement in life compared with the unbound pavement, without any loss of stiffness (fatigue) during the project. The design methodology presented in section 6.4 can

also be used for foamed bitumen. Figure 6.6 presents a wider approach, including other recent NZTA research, which explains why stiffness loss was not observed.

- Objective 3: The project did not result in a better understanding of the continuum from modified to bound behaviour; however, it provided some useful findings. The research suggests that bound behaviour clearly occurs occurs at 3–4% cement contents. The CAPTIF testing showed at 4% cement contents there was very little rutting, but significant stiffness loss. At 3% cement contents in the field, fatigue failures were observed. At CAPTIF, materials with 4% cement showed significant losses of stiffness and the stiffness tended to a value of 1% cement.

  It would be a prudent limit for design to start considering bound behaviour at 2% cement contents, which from the CAPTIF test would be a vibratory-hammer-prepared soaked ITS over a limit of 600KPa limited (when mixed and tested in the lab). This would form an upper limit for using the procedures in figure 6.6. Above this limit there is a risk of cracking, leading to water entering the pavement, and potentially rapid failure and difficult repairs. Below this limit there is a risk of the stiffness reducing and the performance not being as good as estimated (this is considered in the figure 6.6 procedures). These values are considerably higher than those proposed by Austroads but are, in part, a function of the way New Zealand engineers ask for the sample to be prepared.

- Objective 4: The CAPTIF test and field study suggests the Austroads tensile strain criterion appears to produce inappropriate results for New Zealand conditions, and the South African approach appears to produce more appropriate results. Materials with 4% cement, tested at CAPTIF, led to a 1000% increase in rutting life compared with the unbound pavement; however there was significant stiffness loss. 2012 NZTA research produced a laboratory-based approach to the design of bound materials, which is shown in this report.

  The current Austroads Design procedures did not accurately predict the improvement in the modified materials (as already noted by Austroads; however, it was conservative.

  The benefits of modified aggregates can be included in current Austroads design approach with the empirical procedures laid out in section 6.4 of the report. These preliminary recommendations have been made for rehabilitation design and new design. The procedures can used to estimate design life from laboratory-mix design data on any proposed material.

Findings and recommendations regarding foamed bitumen

- The addition of foamed bitumen significantly improved the performance of materials with 1% cement studied in this research.
- The current design methods for foamed-bitumen pavements are over-conservative. However the blank use of an unsublayered 800Mpa modulus appears not to be appropriate as a model, and an alternative approach is suggested in section 5 of this report.
- The pavement behaved in a ductile manner even when water was deliberately introduced.
- Foamed-bitumen contents that maximise indirect tensile strength (ITS) in pavements (while retaining ductility) should be adopted where there is a potential risk of water introduction into the pavement layers. Note that this research only tested foamed bitumen with 1% cement content.
- The repeated load triaxial (RLT) test using the current stress levels in the draft T/15 specification was not able to detect the effect of foamed bitumen in materials with cement, and complementary tests should be conducted to assess the properties of foamed-bitumen mixes.
- The stiffness loss that would be expected from laboratory fatigue beam testing was not observed at CAPTIF.

- Given that there appears to be no stiffness loss with loading the empirical design approach given in section 5, this can be safely used (as long as ductile behaviour is maintained), but further validation and consideration given to the full procedures in section 6.4 is suggested.

Findings and recommendations regarding cement/lime

- The RLT test using the current stress levels in the draft T/15 specification was not able to predict the change in performance of the materials as cement contents rose over 1.5%.
- The testing found a strong correlation with the unconfined compressive strength (UCS), ITS and Indirect Tensile (IT) modulus results. There was a poor correlation between the IT modulus and the RLT modulus.
- During construction there was a good correlation between the laboratory-mixed and field-mixed UCS values. However the field results were approximately 80% of the laboratory values.
- During construction there was a good correlation between the laboratory-mixed and field-mixed ITS values. However the field results were approximately 70% of the laboratory values.
- Precracking the basecourse reduced the average basecourse modulus after construction by 40%.
- The precracked cement section did not fully heal. The 4% cement, once precracked, behaved in a similar manner to 1% cement uncracked.
- The cement-modified pavements behaved in a brittle manner. When water was accidentally introduced to one section it quickly reverted to unbound stiffnesses. However wet cement-modified pavements did perform better than wet unbound pavements.
- Post-mortem testing confirmed that the cement-stabilised test sections were, as expected, damaged by the loading.
- The basecourse modulus from initial falling weight deflectometer (FWD) testing showed a good relationship with the load-carrying capability of the pavements.
- ITS testing on field-obtained samples showed a good relationship with improvements in the load-carrying capability of the pavements.
- The stiffness loss that would be expected from laboratory fatigue-beam testing was observed at CAPTIF. However that stiffness loss occurred across all cemented sections in a relatively uniform number of loads. That was not expected from the laboratory tests. This may be due to the brittle nature of the material.
- Given that there appeared to be stiffness loss with loading, the empirical design approach given in section 5 of this report should be further validated before wide use. However the 2012 NZTA research report *Development of tensile fatigue criteria for bound materials* by Arnold, Morke and van der Weshuizen suggests limits that should prevent stiffness from degrading significantly (see figure 6.6).

The results of this research, when validated, will increase the utilisation of locally available materials and promote the recycling of existing materials. This will in turn reduce the cost of pavement construction, rehabilitation and maintenance, without compromising performance.

This research will also reduce the travel time delays associated with rehabilitation work in terms of frequency and duration. Modified materials have the potential to last longer before needing rehabilitation and the construction techniques associated with modified materials, such as in-situ modification, are very fast when compared to traditional overlays, taking days rather than months to compete projects. In addition, these stabilised pavements will better resist the impacts of any changes in the mass limits of heavy vehicles and increasing traffic volumes, and will provide improved performance in wet conditions.

Abstract

Areas of New Zealand are running out of premium aggregates that meet the demanding specifications used in unbound granular road construction. Stabilising aggregate provides a viable alternative to using premium aggregates.

The objective of this project was to improve the sustainability of New Zealand roads via a combination of Accelerated Pavement Tests at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) in 2007/2008 and 2008/2009, and a limited field review of the performance of bound stabilised pavements.

A number of recommendations have resulted from this research concerning the design, testing and construction of modified and bound pavement layers.

1 Introduction

1.1 Background

Areas of New Zealand are running out of the premium aggregates that meet the demanding specifications required for unbound granular road construction. Stabilising aggregate provides a viable alternative to the use of premium aggregates. Stabilisation is defined as a process where the intrinsic properties of an aggregate are altered by the addition of a stabilisation binder, in order to meet the performance expectations in its operating environment.

This research project validates the benefits of stabilisation and provides new design tools to allow recycling of existing roads and the use of modified marginal aggregates where additional aggregate is required on new roads.

Auckland alone is predicted to require 12 million tonnes of aggregate in 2020, a doubling of what was required in 1991. Because aggregates are a finite resource and there are restrictions on the development of new quarries, the problem Auckland is facing now will also become apparent in other areas of New Zealand. In addition, the funding of improvements to the roading infrastructure in areas such as the east coast of the North Island is becoming increasingly difficult, as the low traffic volumes and the large transport distances required to supply quality aggregates mean that the required economic criteria are often not met.

Pavement designers in New Zealand, who have been trying to use alternative materials in new construction and recycle existing materials in rehabilitation, have been severely constrained by a lack of data on the performance of a range of stabilised materials. This project sought to fill the gaps in this knowledge by presenting two CAPTIF (Canterbury Accelerated Pavement Testing Indoor Facility) trials and a field study. The objective was to improve the sustainability of New Zealand roads by undertaking the following tasks:

1. Determine the benefits of using cement- and/or lime-modified aggregates in terms of increased performance (rutting resistance) and incorporate this in a design methodology, filling a gap identified by Austroads.
2. Validate the benefits of foamed bitumen/cement-stabilised aggregates in terms of increased performance (rutting and fatigue resistance) and incorporate this in a design methodology, building on research by the University of Canterbury and/or the South African Interim Technical Guidelines.
3. Understand the continuum from unbound (no binder), modified (small amounts of binder) to bound (high amounts of binder) behaviour.
4. Review the appropriateness of the Austroads tensile strain criterion for bound aggregates, which is considered by many New Zealand designers to be overly conservative.

The results of this research will increase the utilisation of locally available materials and promote the recycling of existing materials. This will, in turn, reduce the cost of pavement construction, rehabilitation and maintenance, without compromising performance. In addition, these stabilised pavements will better resist the impacts of any changes in the mass limits of heavy vehicles and increasing traffic volumes, and will provide improved performance in wet conditions.

1.2 Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF)

CAPTIF is located in Christchurch, New Zealand. It consists of a circular track, 58m long (on the centreline) contained within a 1.5m deep x 4m 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 1m. 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 vehicle units shown in figure 1.2. These arms are hinged in the vertical plane so that the vehicles 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

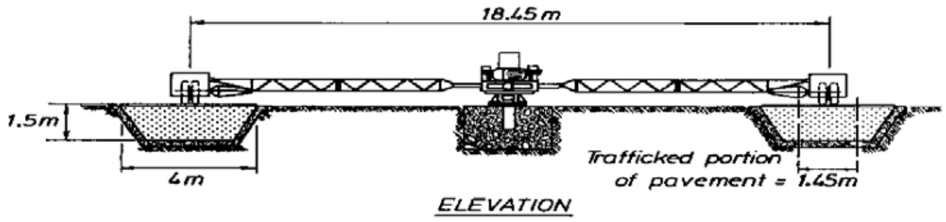
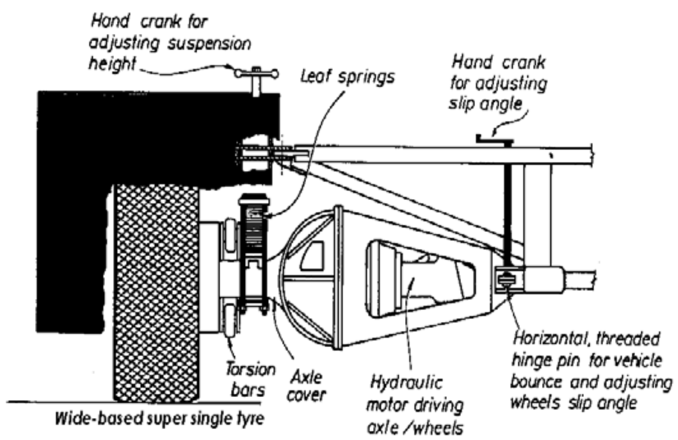


Figure 1.2 The CAPTIF vehicle unit



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 provided by Pidwerbesky (1995).

1.3 Description of the methodology

The research methodology involved a combination of accelerated pavement tests (discussed in sections 2 and 3 of this report) and a field review of the performance of stabilised pavements as discussed in section 4.

The CAPTIF tests were undertaken to investigate a range of cement-modified and foamed-bitumen-stabilised aggregates. These tests covered objectives 1, 2 and 3. The project steering group assisted in the selection of suitable marginal materials.

In addition to standard tests, additional laboratory experiments that characterised the properties of the treated materials were carried out. For the cement-modified materials, resilient modulus and permanent strain testing were conducted in the repeated load triaxial test (RLT), utilising the methodology developed by Dr Arnold (2004); unconfined compressive strength (UCS) and indirect tensile strength (ITS) were also measured. The purpose of these laboratory experiments was to link the material properties with the expected field performance.

The experimental laboratory work for the foamed stabilisation research was a continuation of Dr Salah's foamed stabilisation research funded by Transfund (Saleh 2004b), where laboratory experiments that characterised the properties of the foamed-bitumen-treated materials were carried out. Resilient modulus, UCS and ITS were measured. Again, the purpose of these laboratory experiments was to link the material properties with the expected field performance. The results of these tests also formed part of Dr Gonzalez' PhD thesis titled *An experimental study of the deformational and performance characteristics of foamed bitumen stabilised pavements* (Gonzalez 2009).

The field review sought to meet objectives 3 and 4, in particular reviewing the stabilised aggregates used on Mangatu Road, Gisborne District (Turner 1998). The aim was to assess, from a CIRCLY analysis, the amount of pavement damage relative to pavement cross-section, stabilised aggregates characteristics, traffic loading, and strains at critical locations.

Through consultation and analysis of the RAMM database, a number of sites were selected throughout the country for the field study. These sites included pavements that had performed well for more than 20 years to pavements that had failed in four years. In addition to testing at each site, existing records were examined and data obtained on pavement structure, falling weight deflectometer (FWD), subgrade Californian Bearing Ratio (CBR), layer thickness and historical trends in rutting, roughness, maintenance activities and costs, and traffic loading. In addition, 150mm diameter cores of the modified/stabilised layers were obtained and tested for resilient modulus and ITS.

The analysis of the field data consisted of modelling the pavements through the Austroads and South African pavement design guide methodologies. The analysis highlighted the significant factors that were considered to affect the performance of modified granular materials and incorporated the findings of the Opus Central Laboratories' FRST[1] project into the characterisation of modified materials.

1The Foundation for Research Science and Technology (FRST) project 'High Performance Roads' included an objective called 'Pavement Strengthening'. This project investigated the use of Spectral Analysis of Surface Waves (SASW) as a method to determine the in-situ modulus of pavement materials. This technique was used in this project on the trial sites.

2 Test 1 – Foamed bitumen and cement

2.1 Test 1 objectives

The first test pavement was constructed with six different pavement sections, to study objectives 1 and 2 of the project. This test was to study both existing structural pavement design methodologies and mix design assumptions, and provided an opportunity to test the benefits of cement modification side-by-side with foamed-bitumen modification.

As the objective of the experiment was to study the effect of adding foamed bitumen and cement to an untreated granular material, four sections were stabilised using 1% cement at different bitumen contents. One section was retained as a control with unbound material only, and another section had foamed bitumen only, to separate the effects of the foamed bitumen from those of the cement. The test sections were named B12C10, B14C10, B28C10, B00C10, B00C00 and B22C00, where the first two digits (after B) indicated the bitumen content and the last two (after C) indicated the cement content. For instance, section B14C10 was built adding 1.4% of foamed bitumen and 1.0% cement. Two short transition zones were incorporated in order to achieve a better contrast between the sections stabilised with foamed bitumen plus cement and the other three sections.

The sections were labelled A–F, and the design parameters in each section are given in table 2.1, together with the finally constructed parameters. As can be seen from the table there were some difficulties in constructing the foamed-bitumen sections.

Table 2.1 Design and constructed parameters

Section	Design parameter	Constructed parameter	Designation
A	1.5% foamed bitumen, 1% cement	1.2% foamed bitumen, 1% cement	B12C10
B	2.7% foamed bitumen, 1% cement	1.4% foamed bitumen, 1% cement	B14C10
C	4% foamed bitumen, 1% cement	2.8% foamed bitumen, 1% cement	B28C10
D	1% cement	1% cement	B00C10
E	Unbound (no binder)	Unbound (no binder)	B00C00
F	2.7% foamed bitumen	2.2% foamed bitumen	B22C00

2.2 Laboratory mix design

2.2.1 Bitumen characterisation

Prior to the pavement construction there was an extensive laboratory mix design programme undertaken. Laboratory samples of foam were produced with a Wirtgen WLB 10 laboratory-based foaming nozzle that was available at the University of Canterbury. The bitumen used for this study was provided by the hot-mix asphalt (HMA) plant operated by Fulton Hogan in Christchurch. The penetration range was 80/100, which is one of the most common grades used in New Zealand. As not all bitumen can be foamed, the method proposed by the South African Interim Guidelines (Asphalt Academy 2002) was followed to characterise the foam. This method is based on the 'foam index' (FI), which is an indicator of the entropy of the foam – the higher the FI, the higher the quality of the foam.

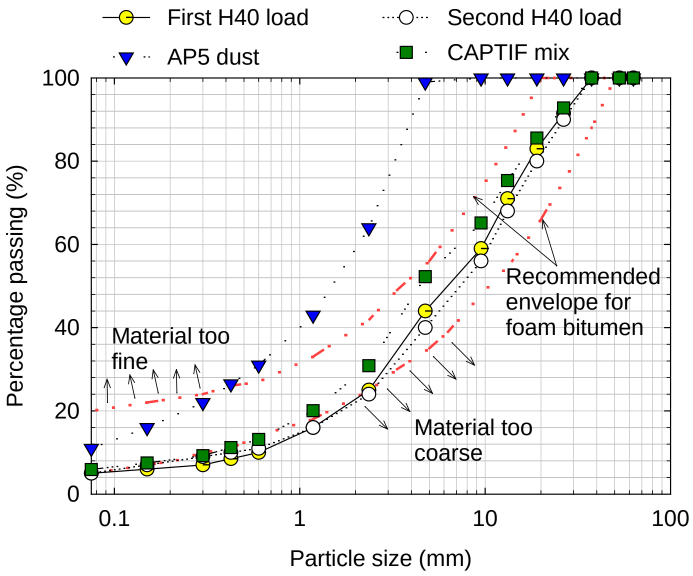
To undertake the foam characterisation, the bitumen was tested at three temperatures: 160°C, 170°C and 180°C. The bitumen was foamed using three water contents: 1.5%, 2.5% and 3.5%.

The results showed that foaming properties at 160°C and 170°C were quite similar and the optimum foaming water content was 2.5%. However, 170°C was preferred because it was the foaming temperature usually used in the field.

2.2.2 Aggregate characterisation

The industry steering group suggested using a GAP65 aggregate from Auckland. The GAP65 aggregate, designated as H40 for the project, was too coarse for foamed-bitumen stabilisation without adjustment. The material was mixed with a local crushed dust (AP5) available from Fulton Hogan's Pound Road quarry in Christchurch. A blend of 85% AP65 and 15% of AP5 brings the AP65 grading curve into the recommended foamed-bitumen grading envelopes (figure 2.1).

Figure 2.1 Material grading curves



The optimum water content (OWC) was determined following the New Zealand vibrating hammer compaction test. Results indicate that the OWC for the mix was close to 6.0%.

2.2.3 Foamed-bitumen mixes

A set of specimens was prepared using vibratory compaction in 150mm diameter moulds. Three bitumen contents were tested: 2.0%, 3.0% and 4.0%.

The densities achieved were around 2200kg/m^3, which was close to the maximum dry density determined from the compaction test of the AP65/AP5 material. ITS testing suggested that the optimum bitumen content was 2.7%, which was considered a reasonable value given the materials grading. Both unsoaked and soaked ITS values were determined and the tensile strength retained (TSR) was calculated for these specimens, which yielded values of 64%, 68% and 61% respectively

Further details of the AP65 can be found in Gonzalez (2009).

2.3 Pavement design

The only pavement design methods available in New Zealand before this project was undertaken were the South African Council for Scientific and Industrial Research (CSIR) guidelines (Theyse and Muthen 2001), and the New Zealand Supplement to the Austroads pavement design guide (Transit NZ 2005). The South

African guidelines classify the foamed-bitumen mixes into four categories according to UCS and ITS values, and provide pavement designs methods for materials FB2 and FB3. From the mix design used for this research, the pavement material could be classified as FB3, but was borderline FB2. The FB3 material described in the CSIR guidelines correspond to a granular material stabilised using 3.0% of bitumen and 1.0% cement.

The pavement thickness was designed for the optimum foamed-bitumen content from the laboratory study – 2.7% bitumen and 1.0% cement. The recommended values for the pavement design are shown in table 2.2.

Table 2.2 FB3 pavement design parameters

Material classification	FB3	
	Range	Value used
Cement:bitumen ratio		0.33
Stiffness (MPa)	680–1625	1100
Poisson ratio		0.35
Strain-at-break ($\mu\varepsilon$)	290–590	490
Cohesion (kPa)	110–210	120
Friction angle (degrees)	34–53	45

The CSIR guidelines suggest that the treated foamed-bitumen pavement behaves in phases. The first phase is when the layer is in an intact, undamaged condition and provides some fatigue resistance. This phase ends when the layer has reached an equivalent granular state, with the time to reach this state defined as the effective fatigue life. The term 'equivalent granular state' is used to describe the loss in resilient modulus (stiffness) of the material and is comparable to granular materials only in the stiffness and not in the physical composition of the materials.

The CSIR guidelines recommend the use of multilayer linear elastic software such as CIRCLY, mePADS or ELSYM5 for the mechanistic analysis of the pavement layers. The New Zealand Supplement to the Austroads pavement design guide (Transit NZ 2005) provides a slightly different approach based mainly on the contracting industry's experience, and states that it is unclear whether the first phase of the CSIR approach actually occurs and it may not be appropriate for foamed bitumen. The New Zealand Supplement suggests the following recommendations for modelling:

- elastic modulus of the order of 800MPa
- Poisson ratio = 0.30
- an anisotropic layer
- no sublayering.

2.3.1 Pavement design using the CSIR equations

A structural analysis using the CSIR approach was performed. The pavement was modelled in two phases and the initial thickness was set to 200mm. The stiffness during phase I was calculated as the average of FB2 and FB3 materials (1350MPa). The stiffness during phase II was assumed to 800MPa. Pavement designs using different subgrade support were performed. Subgrade stiffnesses of 40, 60, 80, 100 and 120MPa were utilised. All layers were treated as isotropic. Two 20kN wheel loads were applied with a contact pressure of 750kPa and a separation between wheels of 330mm.

A 95% of reliability was chosen and the terminal rut was set to 20mm for the failure condition. For the phase I equations the 'strain at break' parameter was set to eb=331$\mu\varepsilon$ (the average between the FB2 and FB3 materials). For the phase 2 equations, the cohesion was set to 267kPa, Fterm to 3.0, a relative density of 0.85, and the cement/bitumen ratio was set to 0.36. The results are presented in table 2.3 in millions of equivalent standard axles (MESA).

Table 2.3 CSIR pavement design

Subgrade (CBR %)	Phase I (MESA)	Phase 2 (MESA)	Total (MESA)
4%	0.35	0.03	0.38
6%	0.43	0.04	0.47
8%	0.50	0.05	0.55
10%	0.56	0.06	0.62
12%	0.61	0.07	0.68

2.3.2 Pavement design using the New Zealand Supplement method

A similar analysis was performed using the New Zealand Supplement method. In this case the stabilised layer was considered isotropic with no sublayering, but the Austroads 2004 sublayering was adopted for the subgrade. The critical parameter used was the maximum vertical strain on the top of the subgrade and the allowable standard axles were calculated using the Austroads subgrade strain criteria. The results are presented in table 2.4.

Table 2.4 New Zealand pavement design

Subgrade (CBR%)	Foamed-bitumen layer stiffness (MPa)	Maximum vertical subgrade strain ($\mu\varepsilon$)	Allowable standard axles (MESA)
4%	800	1862	0.08
6%	800	1506	0.35
8%	800	1281	1.1
12%	800	1122	2.8

2.3.3 Final design

It can be seen from the previous sections that both design methods provide quite different results. Experience from previous CAPTIF pavements suggested that the 200mm of basecourse would be suitable for the experiment.

2.4 CAPTIF pavement construction

2.4.1 Subgrade

The top 525mm of the subgrade at CAPTIF was replaced for this project and remaining Waikari silty clay subgrade from previous CAPTIF projects was left undisturbed in the concrete tank. The top 525mm of the subgrade was Tod clay placed in lifts of 225, 150 and 150mm and compacted using CAPTIF's pivot steer trench roller. Quality control was maintained with layer-by-layer density and moisture-content testing. Once the construction of the subgrade was finished, FWD deflections of the subgrade surface were measured by applying a pressure of 470kPa (total load of 33kN) to verify the layer homogeneity. The peak deflections showed an averaged 1.510mm with a standard deviation of 0.177mm (coefficient of variation – CV – 11.7%), which is considered fairly homogenous for this type of subgrade material. A simple back-calculation analysis yielded a subgrade stiffness of approximately 60MPa, which was in agreement with the target stiffness from the structural design of pavements.

In-situ CBR testing was conducted with a Scala penetrometer. The in-situ CBR was estimated from the RG Brickell relationship provided in the *Road Research Unit TR1 technical recommendation* (Brickell 1985), and the reading given in table 2.5 is the average estimated CBR over the top 300mm of subgrade. The results show that the average strength across the sections was uniform.

Table 2.5 Subgrade CBR

Section	Material	Average	Min	Max
A	Low foamed bitumen	8	7	9
B	Optimum foamed bitumen	7	6	8
C	High foamed bitumen	7	6	8
D	Cement-modifed	8	7	9
E	Unmodifed Auckland GAP40	9	8	10
F	No-cement foamed bitumen	9	8	9

2.4.2 Basecourse construction

Two hundred tonnes of aggregate was transported 800km from Auckland to Christchurch. The material was delivered in two loads shortly before construction. Approximately 30 tonnes of AP5 crusher dust material was imported from a local quarry in Christchurch.

The CAPTIF building could not accommodate the very large road construction machinery used in stabilisation work and therefore it was not feasible to directly stabilise the materials in place. Therefore, the aggregate was blended with bitumen and/or cement outside the CAPTIF building using a Wirtgen 2500 SR recycling machine and was transported into the building by loader, and a basecourse paver laid the materials in two lifts.

For the stabilisation process, trenches 26m long by 5m wide were excavated outside the CAPTIF building for each material. The trenches had 5m run-in and run-out zones to allow the machine to reach equilibrium before material was used in the track. A total thickness of 340mm of GAP65 material was laid in two lifts and compacted to 95% of maximum dry density (2.11t/m^3) at its OWC (4.0%). Later, a 70mm thick layer of AP5 crusher dust was spread and compacted to 95% maximum dry density (1.82t/m^3) at OWC (9.0%). The target stabilisation depth was 400mm and the thickness ratio of 70:330 approximately yielded the target mass ratio of 15:85 obtained in the laboratory mix design.

Once the untreated material was ready, the trenches were stabilised with the recycling machine, keeping a constant speed and adding the stabilising agents. The water content was adjusted according to the bitumen content to account for the contribution of the binder as a compaction fluid. The stabilised material was transported into the CAPTIF building (located about 50m from the trenches) by loaders. During this process, material samples were taken for laboratory testing, described later in this report. A paver was used to place the basecourse material in two layers of 100mm each and a steel roller was used for compaction. The roller used was lighter than a roller used in normal field construction of stabilised layers. Therefore, the basecourse was compacted in two lifts to account for the lower compaction energy applied. Nevertheless, the same compaction effort was applied to all stabilised pavements. The time between trench stabilisation and final compaction of the stabilised basecourse layer for one pavement section was between two and three hours.

The construction of the control unbound granular section (B00C00) was slightly different. Instead of using a mix of aggregate and crusher dust, only the plain GAP65 aggregate was used and it was laid in a single layer of 200mm. The particle size distribution in this section was not exactly the same as the other sections, but the incorporation of the AP5 crusher dust was considered to be part of the stabilisation process.

Before laying the final surface layer the sections were cured at ambient temperature for 30 days. During this period, a light falling weight deflectometer (PRIMA) was used to measure the reduction in the pavement deflections caused by moisture loss. Results showed an important reduction in deflections during the first week after construction, followed by a plateau afterwards where little variation was observed.

Before sealing, layer profiles, density and moisture measurements were taken on the basecourse. A correction factor was applied to the nuclear density gauge to account for the bitumen content in the sections stabilised with foamed bitumen. Dry densities and moisture contents are provided in table 2.6.

Table 2.6 Dry densities (DD) and moisture contents (MC)

Section	Layer	Avg DD (kg/m^3)	St dev DD (kg/m^3)	%MDD[a]	Avg MC (%)	St dev MC (%)
A	BC1	2143	64	99	6.1	0.6
A	BC2	2220	0	103	3.8	0.3
B	BC1	2117	21	98	7.0	0.7
B	BC2	2170	17	100	5.4	0.6
C	BC1	2090	60	97	4.7	0.7
C	BC2	2140	36	99	3.7	0.4
D	BC1	2083	31	96	6.1	0.4
D	BC2	2147	15	99	5.7	0.5
E	BC2	2255	21	102	5.4	0.4
F	BC1	2117	15	98	5.4	0.2
F	BC2	2153	23	100	4.8	0.6

a) Maximum dry density.

2.4.3 Surface layers

The surfacing was constructed 30 days after construction of the basecourse layers. All sections were sealed with a single-coat chipseal. After a week to allow the chipseal to set up, all sections were surfaced with a skim coat of AC10 hot mix covering the top of the chipseal, to allow accurate measurement of the surface profiles. The approximate thickness of this surface was 20mm (see table 2.7). During the execution of the experiment and after the application of 200,000 load cycles, the original thin surface started to show significant wear. A thin, 30mm layer of hot-mix asphalt (HMA) was laid over the original surface, leading to a total surface thickness of approximately 50mm for the rest of the project (see table 2.8).

Table 2.7 Layer thickness prior to overlay

Section	Avg AC[a] (mm)	St dev AC (mm)	Avg BC[b] (mm)	St dev BC (mm)
A	16	2	212	9
B	15	2	205	7
C	16	1	206	6
D	16	2	208	11
E	17	1	195	6
F	17	1	201	6

a) Asphalt.

b) Basecourse.

Table 2.8 Layer thickness after overlay

Section	Avg AC (mm)	St dev AC (mm)	Avg BC (mm)	St dev BC (mm)
A	39	3	212	9
B	43	3	205	7
C	44	4	206	6
D	43	3	208	11
E	48	3	195	6
F	47	4	201	6

2.4.4 In-pavement instrumentation

The pavement instrumentation at CAPTIF includes 3D Emu soil-strain transducers for measurement of vertical, transverse and longitudinal strains in the pavement. The soil-strain measuring system determines strains with good resolution (±50μm/m). The sensors use the principle of inductance coupling between two free-floating, flat, circular wire-wound induction coils coated in epoxy, with a diameter of 50mm. Details of the system can be found in Steven (2005). The strain coils were installed during the formation of the subgrade and the basecourse layers, to minimise the disturbance to the materials.

The Emu strain coils were located coaxially at a spacing of 75mm, directly under the inner wheel. The reported depth of the vertical strains corresponded to the midpoint between two coils, while the reported depth of the longitudinal and transversal strain corresponded to the coil depth. The 3D Emu stacks were located at stations 2, 11, 25, 31, 40 and 52.

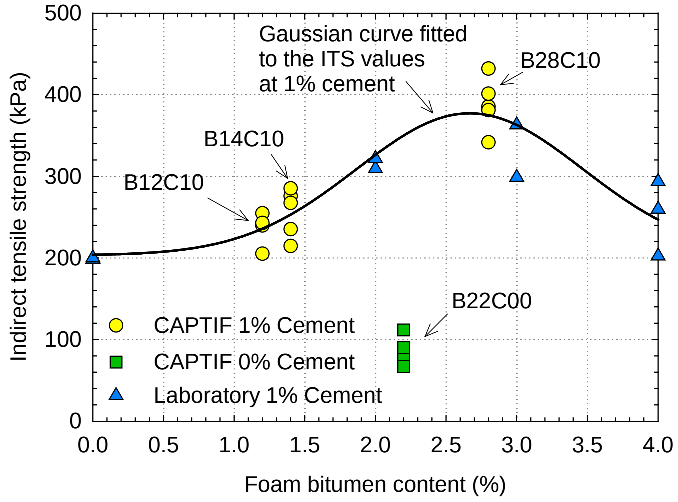
2.4.5 Laboratory testing

During construction of the basecourse layers, samples were taken for ITS and RLT tests. The ITS samples with dimensions of 150mm in diameter and 100mm in height were prepared using vibratory compaction. The samples were cured for two weeks and tested in a constant displacement loading machine at a rate of 50.8mm/min and at room temperature (20°C). Due to time and equipment restraints, only samples with foamed bitumen were prepared for ITS testing. The field samples were not soaked after curing.

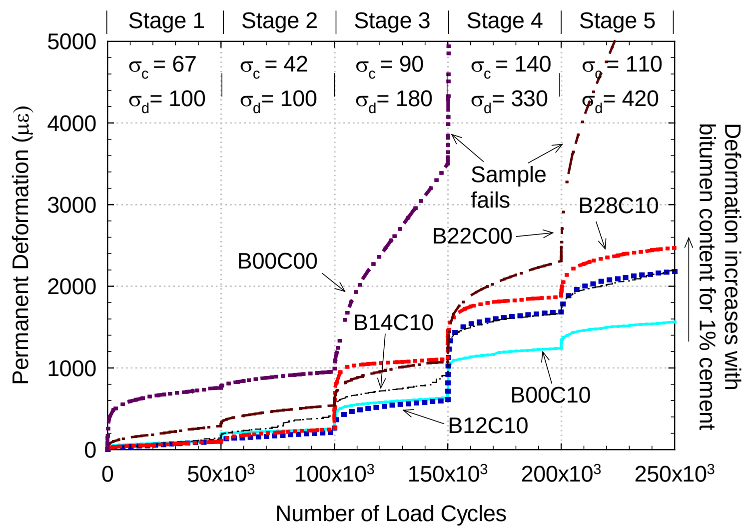
Large triaxial samples (150mm in diameter and approximately 300mm in height) were also prepared using vibratory compaction. The samples were cured for 28 days in double sealed plastic bags and were not soaked after curing. The stress sequence proposed by the New Zealand standard for triaxial testing followed the draft TNZ T/15 specification (2007). The standard applies six stress stages, where the first ('stage 0') is a preconditioning stress stage. Each stress stage consists of 50,000 Haversine load cycles of 250 milliseconds. The confining (σ_c) and deviator (σ_d) stress applied were included in the test results (see figure 2.2). Only one specimen per foamed-bitumen content was prepared and tested at room temperature (20°C). The resilient moduli were recorded during these tests at each stage, but only the average modulus is reported below in this report.

Figure 2.2 Laboratory tests

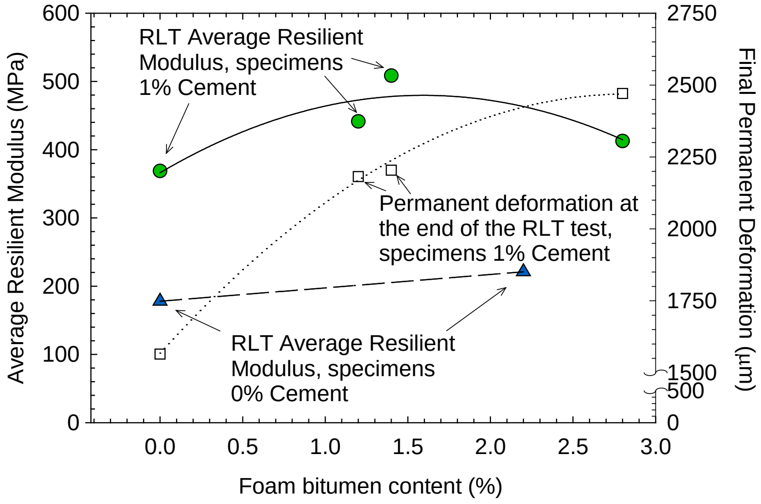
(a) ITS



(b) RLT permanent deformation



(c) RLT resilient modulus and permanent deformation versus bitumen content



2.5 Accelerated pavement testing

2.5.1 Loading sequence and speed

The CAPTIF loading vehicles are half of a single-axle dual-tyre with a tyre separation of 350mm, inflated to 700kPa. The original project contemplated a constant loading of 40kN for each vehicle. However, since little rutting was measured during the early stages of the project, at 150,000 load cycles the load was increased to 50kN. At 502,000 load cycles the load was increased again to 60kN to induce a failure in the pavement sections.

The speed of the vehicles was kept constant at 40kph during most of the project, but when the units were loaded with 60kN the speed was reduced to 30kph to avoid excessive bouncing of the vehicles. The load was applied on one wheel path, with a lateral wander of 100mm.

By the end of the test little difference was found in the rutting measurements of sections stabilised with foamed bitumen and cement (sections B12C10, B14C10 and B28C10). To accelerate the surface deformation in these sections, it was decided to cut part of the pavement sections from the HMA surface to the top of the basecourse (50mm deep cuts). The separation between the cuts was 200mm, and the surface was sawed from stations 3 to 6 in section B12C10, 14 to 17 in section B14C10, and 20 to 23 in section B28C10. Once the cutting job was finished, water was uniformly applied over the surface using sprinklers available in CAPTIF. With a constant water flow, additional 60kN load cycles were applied.

2.5.2 Data collection

At prescribed intervals in the project, strain and stress data was collected at 10kph and 40kph. During these measurements the lateral movements of the wheels was restrained. Vertical surface deformation (VSD) readings were taken at each station at the same intervals. For the rutting calculations, the rutting measured after the construction of the HMA overlay was added to the initial rutting measurements.

The FWD testing was carried out before the trafficking (0 load cycles) and after the application of one hundred thousand, and one million, load cycles. The initial testing was conducted at a standard 40kN load in the wheel path at each station and either side of the three primary stations in each section.

CAPTIF Geo Beam tests were conducted during the initial phase of the project (0–35,000 load cycles) and during the last stage (502,000–1,326,000 load cycles), applying a load of 40 and 60kN respectively. The speed of the wheels during the beam test was about 6kph.

2.5.3 Post-mortem analysis

Trenches were excavated in each section and the levels of each pavement layer were recorded to identify the plastic deformation in each section. The location of some basecourse strain coils was recorded to estimate the potential plastic deformation in the basecourse layers. Material samples were taken for binder extraction to the BSI (2005) 'Soluble binder extraction test', moisture content and visual assessment.

2.5.4 FWD results

Initial FWD readings taken after construction indicated that the subgrade (see table 2.9) was uniformly constructed; however, the stiffness had improved from the earlier testing directly on the subgrade and the different pavement sections provided significant differences in modulus and predicted life, using the Austroads subgrade strain criteria (see table 2.9). This increase in stiffness suggested strong non-linear effects in the subgrade – which was likely, given the high plasticity of the clay. However, it was difficult to determine whether these effects were wholly real or whether they were in part inaccuracies of the modelling. According to the FWD results, the optimum mix design for foamed bitumen was in Section C. Section F FWD testing would suggest the addition of cement to foamed bitumen is probably not required if a long curing period (such as in greenfields construction) can be provided. And section D suggests that cement on its own does a good job.

Table 2.9 Elmod FWD results

Section name	Average basecourse thickness (mm)	Average basecourse modulus (MPa)	Average subgrade modulus (MPa)	10 percentile life estimate from FWD testing (MESA)
A	228	303	109	0.2
B	220	461	104	0.4
C	222	786	131	3.3
D	224	350	112	0.2
E	212	216	97	0.0
F	217	404	128	0.5

2.5.5 Rutting performance

The averaged VSD measurements for each pavement section are presented in figure 2.3a. Both VSDs and rut depths were calculated from the data. VSD is defined as the maximum change in height between a transverse profile taken before loading and the current transverse profile, whilst the rut depth is calculated from a theoretical straight line between the high points either side of the wheel path to lowest point of transverse profile in the wheel path. Thus the VSD value only presents the downward vertical movement of the pavement in the wheel path, while rutting is a combination of downward movement in the middle of the wheel path and upward movement on the edges.

The curves presented show the typical behaviour of pavements with a bedding phase during the initial vehicle loading followed by a plateau phase. When the load was increased to 50kN, another increase in the rutting rate was observed. From 300,000 load cycles (after the asphalt overlay), the rutting increased approximately linearly up to 1,000,000 load cycles for all sections.

After 1,000,000 load cycles, sections B00C10, B00C00 and B22C00 started to show large amounts of heaving and rutting. Sections B12C10, B14C10 and B28C10 were still performing well at 1,326,000 cycles but were also showing signs of accelerated distress. Even at 1,326,000 cycles there was little difference between sections B12C10, B14C10 and B28C10.

2.5.6 Deflection tests

CAPTIF beam test and FWD readings are presented in figure 2.3b. The results presented correspond to deflections measured before the trafficking of the sections (0 load cycles), applying a load of 40kN in both deflection tests. The beam test and FWD provided similar results, illustrated by parallel curves fitted to the results. In general, FWD readings were lower than beam deflections. This could have been caused by the shorter load pulse applied by the FWD. The unbound section (B00C00) showed considerably higher deflections than the other sections.

Both cement and bitumen had an important effect in reducing the deflections of the pavements studied. The lowest deflection was measured in the section with the highest bitumen content and 1% cement (B28C10). The trends observed at this stage were kept relatively constant throughout the CAPTIF test, as shown in figure 2.3c (which has been corrected for the benefits of the overlay).

2.5.7 Strain measurements

Only two sets of vertical strain measurements are presented in this report, taken after the application of 502,000 and 1,326,000 load cycles. The wheel load applied in this part of the test was 60kN and the vehicle speed for the results presented here was 10kph (see loads in figure 2.3a). The trends observed at other stages of the test remained similar to the later loading, indicating that the modulus of the stabilised pavements remained relatively constant during the experiment. This contradicted other full-scale experiments on foamed-bitumen pavements (Long et al 2002, Asphalt Academy 2002), where an important reduction in the modulus was reported. However, in those experiments the cement content (2.0%) was higher than the foamed-bitumen content (1.8%) and therefore the behaviour of those pavements is not representative of the materials studied in CAPTIF test.

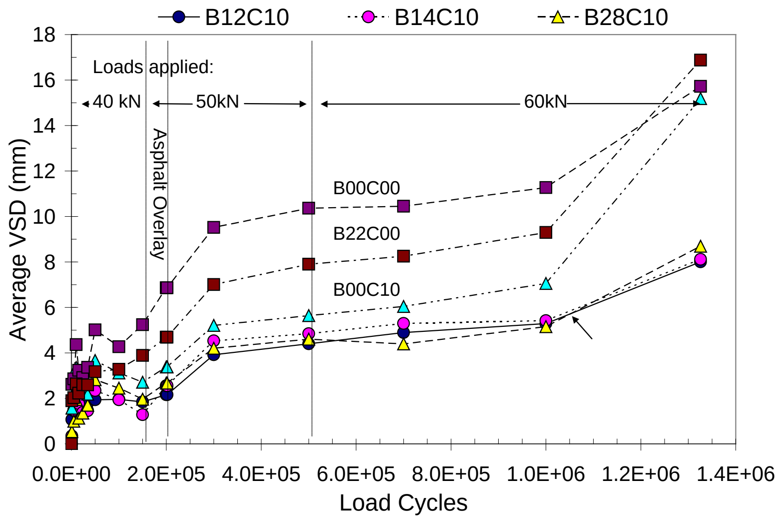
To better examine the effect of foamed bitumen, the measured strains were plotted against the bitumen contents. Figure 2.4 shows:

- the compressive vertical strains measured in the basecourse (depth=112.5mm)
- the tensile longitudinal strains measured close to the bottom of the basecourse layer (depth=150mm)
- the vertical compressive strain close to the top of the subgrade (depth=262.5mm).

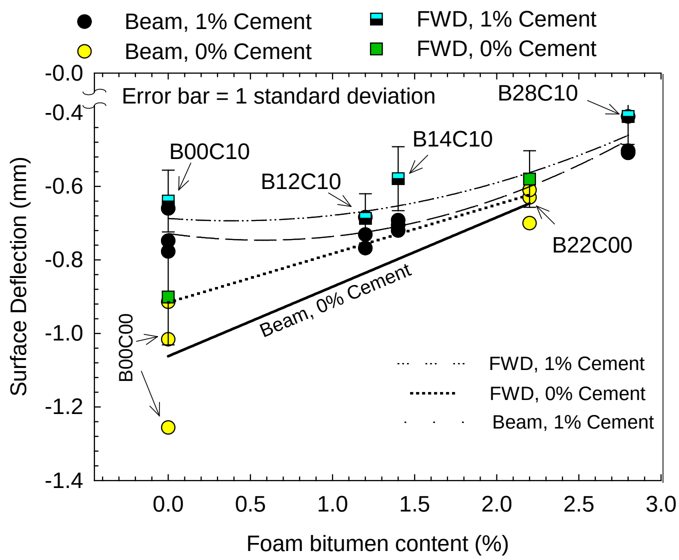
The location of these strains within the pavement structure has been recognised as critical points by the current design methods for foamed-bitumen pavements (Asphalt Academy 2002, Austroads 2004).

Figure 2.3 Pavement response

(a) Vertical surface deformation (VSD)



(b) CAPTIF beam and FWD deflections at 40kN before trafficking (0 load cycles)



(c) Change in central deflection (corrected for overlay)

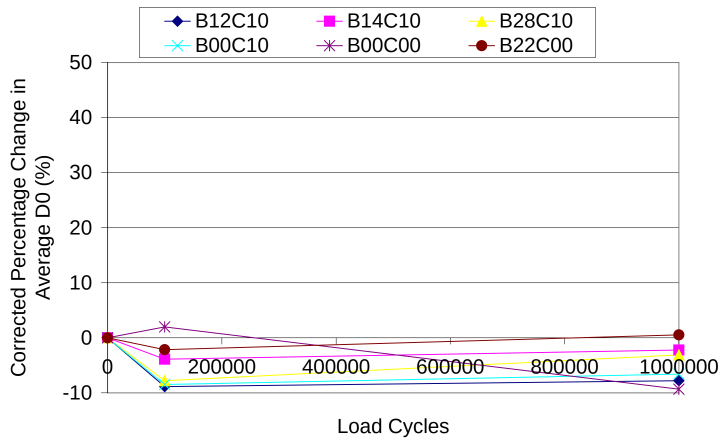
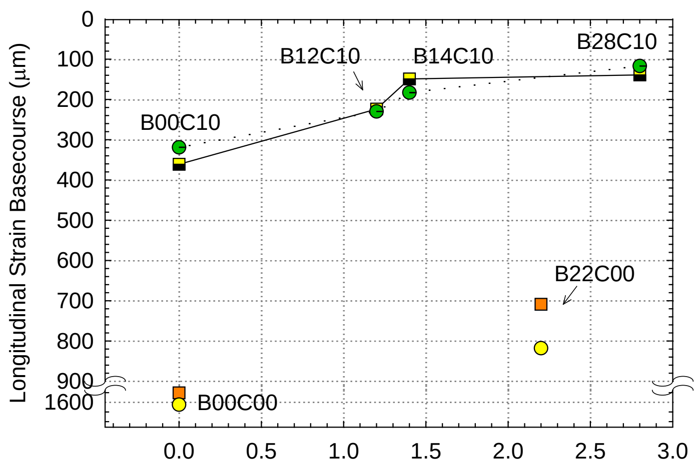
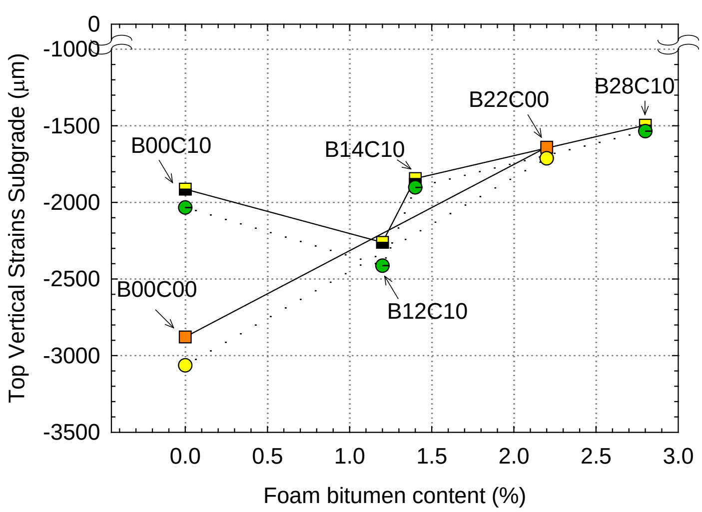


Figure 2.4 Strain measurements versus bitumen content at 502,000 and 1,326,000 load cycles

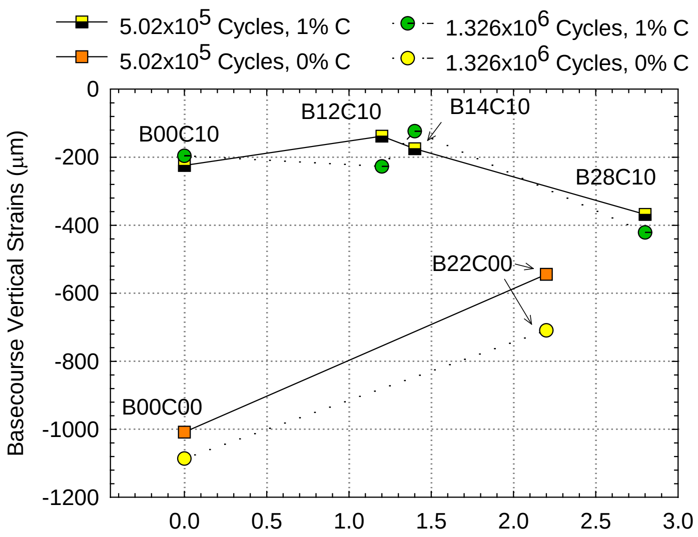
(a) Longitudinal tensile strains in the basecourse



(b) Vertical compressive strains at the top of the subgrade



(c) Vertical compressive strains in the basecourse



The vertical strains (figure 2.4c) measured at sections with 1% cement were considerably lower than the other two sections (B00C00 and B22C00). Measurements also indicated that foamed bitumen had a small effect in the compressive vertical strains measured in sections with 1% cement.

Figure 2.4a illustrates that cement had a large effect in the reduction of the longitudinal strains. Also, these strains were reduced by 50% when foamed bitumen was added.

The compressive vertical strains measured at the top of the subgrade (figure 2.4b) of the control section (B00C00) were considerably higher than the other sections. In the other sections these strains were lower at higher bitumen contents.

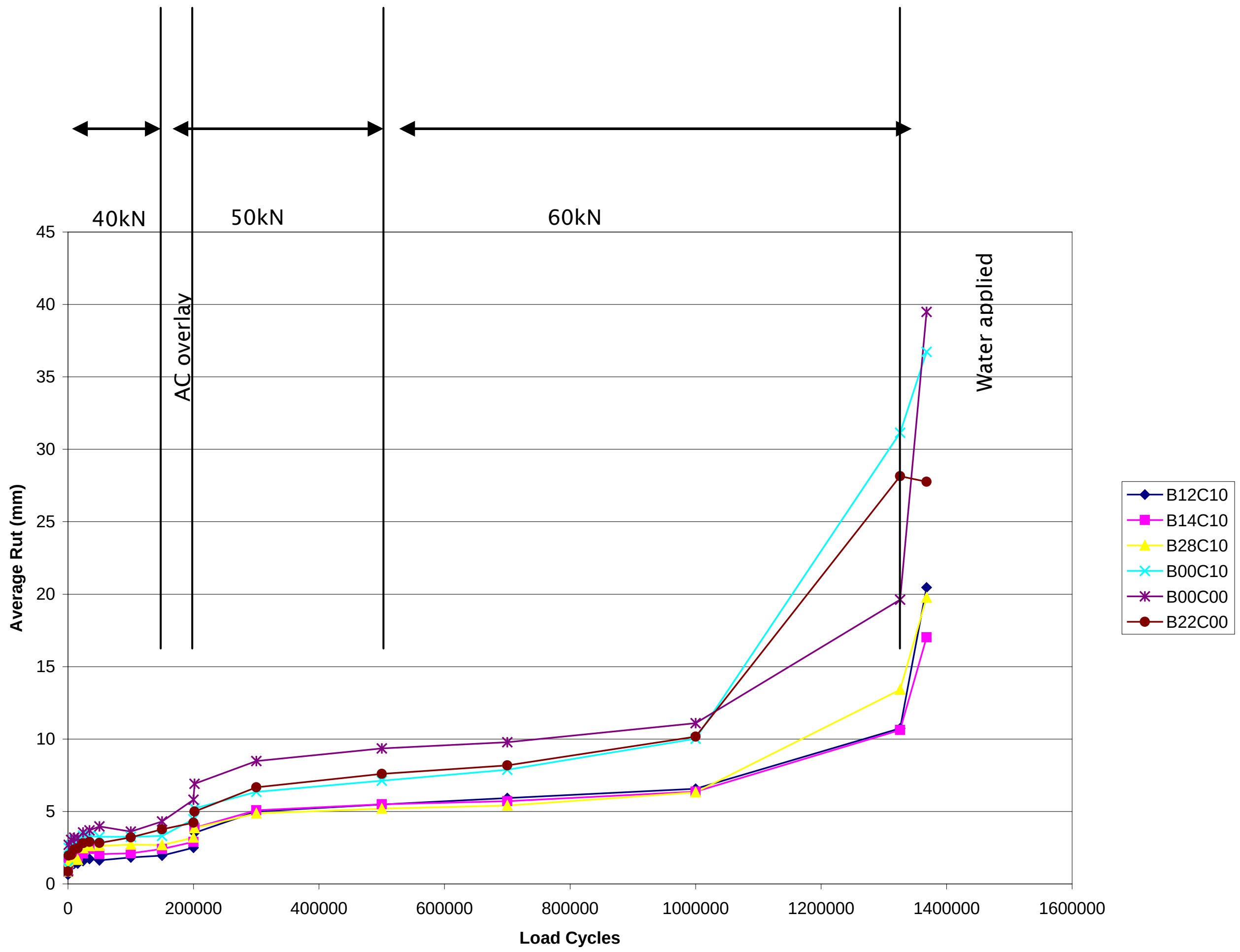
2.5.8 Final wet testing

The introduction of water and accelerated load to the sections stabilised with foamed bitumen and cement induced further deformation and surface cracking in the pavement sections. Nevertheless, the water penetrated into the subgrade, causing swelling of this layer and modifying the final levels of the pavement structure.

The water sprinkled over sections B12C10, B14C10 and B28C10 drifted to the other sections because of the circular movement of the wheels, penetrating into the pavement structure through the cracks already present prior to the final wet testing in these sections. Water pooling in the wheel path was particularly noticeable in sections B12C10, B14C10 and B28C10 due to the combination of initial rutting and the direct water application. Water was not directly applied to the other sections and water pooling in the wheel path was not noticeable. This water caused the same, but less severe, swelling effect in the subgrade.

However, the focus of this last part of the study was to identify a difference in rutting among the sections stabilised with foamed bitumen and cement. After the application of additional 42,000 load cycles under wet conditions, section B12C10 started to show extensive cracking and a small amount of further surface deformation, in comparison with the other two sections (see figure 2.5), with evidence of some loss of fines. No cracking was observed in sections B14C10 and B28C10. Also of interest in figure 2.5 is that sections B00C10 and B22C00 had, prior to the water application, been rutting poorly in comparison with the B00C00, but they performed well when water was applied.

Figure 2.5 Wet testing of foamed-bitumen sections



2.5.9 Post-mortem analysis

During the post-mortem, trenches were excavated to observe the layer performances. Photos are included in appendix A and layer profiles are shown in appendix B. The layer profiles indicate the swelling in the subgrade due to water application.

During the excavation of the pavements, the vertical position of the basecourse coils was recorded again to identify any possible permanent deformation within the basecourse layers. The original vertical distance between the strain coils during the construction of the pavement sections was 75mm. The position was measured using a simple ruler and transverse beam lined up across the pavement station marks, with an estimated precision of ±1mm. In stations 2 (B12C10) and 52 (B22C00), it was not possible to recover the coils. The measured separation of the coils in stations 11 (B14C10) and 25 (B28C10) was 75mm, suggesting that most of the surface deformation in these sections was accumulated in the subgrade layer. Conversely, the sensors recovered from stations 31 (B00C10) and 40 (B00C00) were displaced 4 and 5mm respectively, suggesting that permanent deformation occurred in the basecourse.

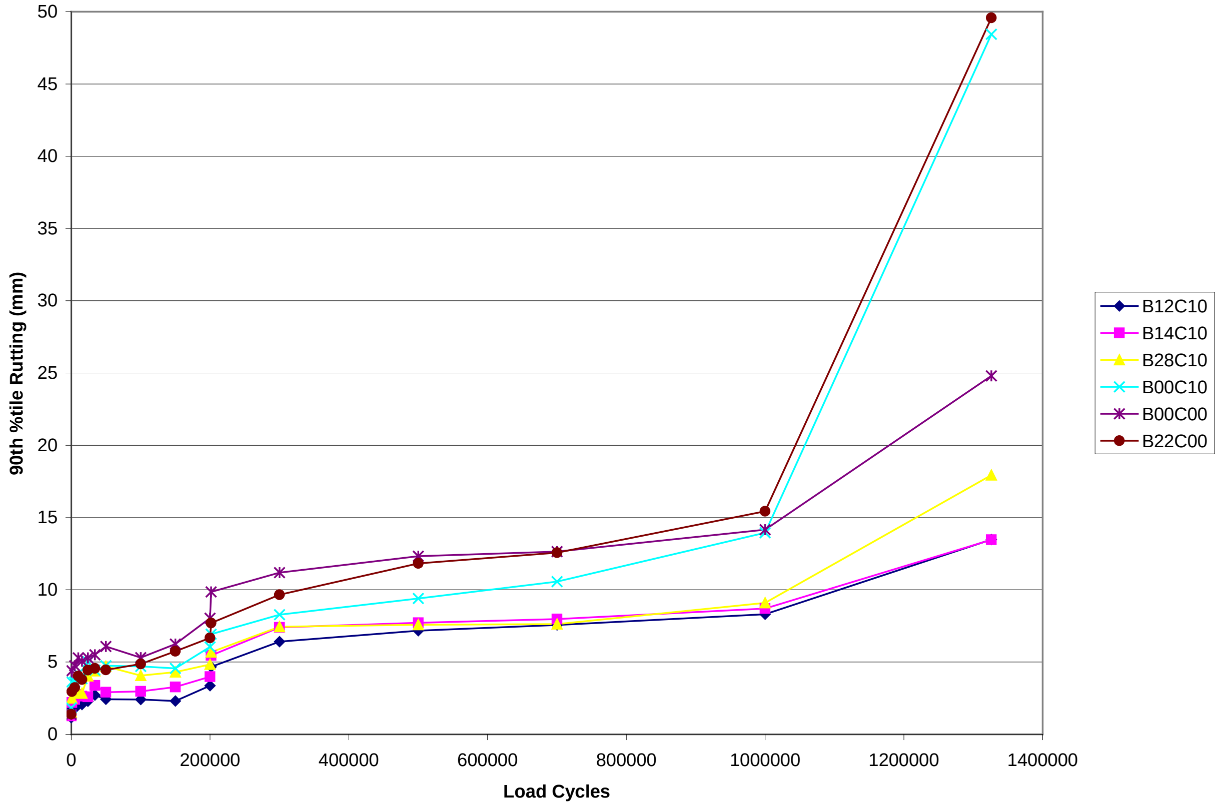
Samples for moisture content determination from the upper and lower basecourse, as well as the subgrade, were obtained during the post-mortem analysis. These indicated that water penetrated into the sections with foamed bitumen and cement, as well as the other three sections. Bitumen extraction tests confirmed the binder contents added in each section.

2.6 Analysis for pavement design

The data from the FWD testing was used in a CIRCLY analysis to make a prediction of performance for comparison with the observed rutting performance.

The CIRCLY analysis assumes that pavement reaches a terminal condition in terms of unacceptable deformation. This is generally considered to be where 10% of the pavement reaches a 20mm rut. This has been taken to be the 90th percentile rut, which is the mean rutting value plus 1.28 times the standard deviation. The statistical validity of doing this is difficult to justify but it serves as a useful approximation of a failure condition. So the data from the transverse profiles was used to determine the end of life of each of the pavement sections. Figure 2.6 following shows the 90th percentile rutting for the test and it can be seen that there was a need to extrapolate some of the data.

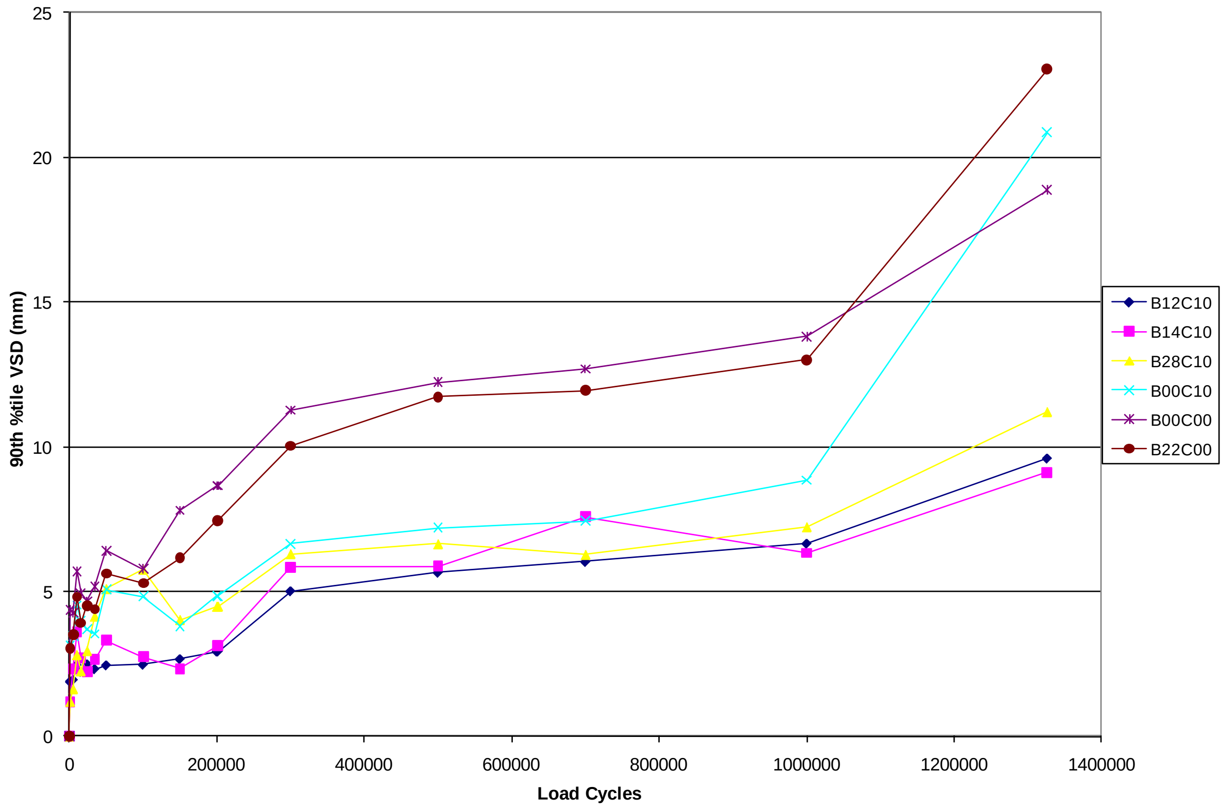
Figure 2.6 90th percentile rutting in each test section



From a practical point of view there was a need to extend the model beyond the data obtained and it was decided to use a model extended from the more stable average VSD data. The first stage was to convert the 20mm 90th percentile rut to a 90th percentile VSD (shown in figure 2.7 following). It is interesting to note that both sections B00C10 and B22C00 appeared to perform very poorly in rutting when compared with VSD later in their lives – both generating 90th percentile ruts of nearly 50mm.

Figure 2.5 suggests that this poor performance in rutting at the terminal condition can also be seen in the average data for B00C10 and B22C00.

Figure 2.7 90th percentile VSD in each test section



Before we discuss the link between average VSD and 90th percentile rut, we note that figures 2.8 and 2.9 confirm that both the average VSD and 90th percentile VSD were related to the average and 90th percentile ruts.

Figure 2.8 Relationships between average VSD and average rutting for each section

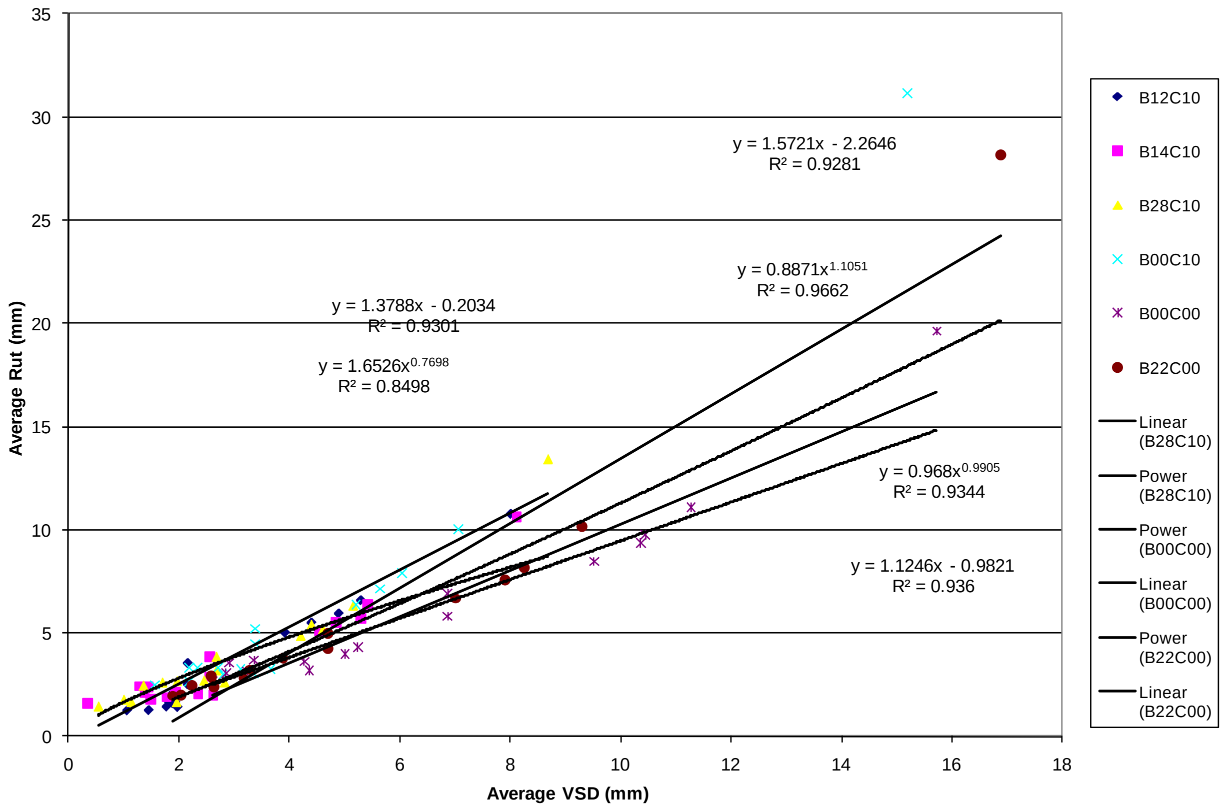
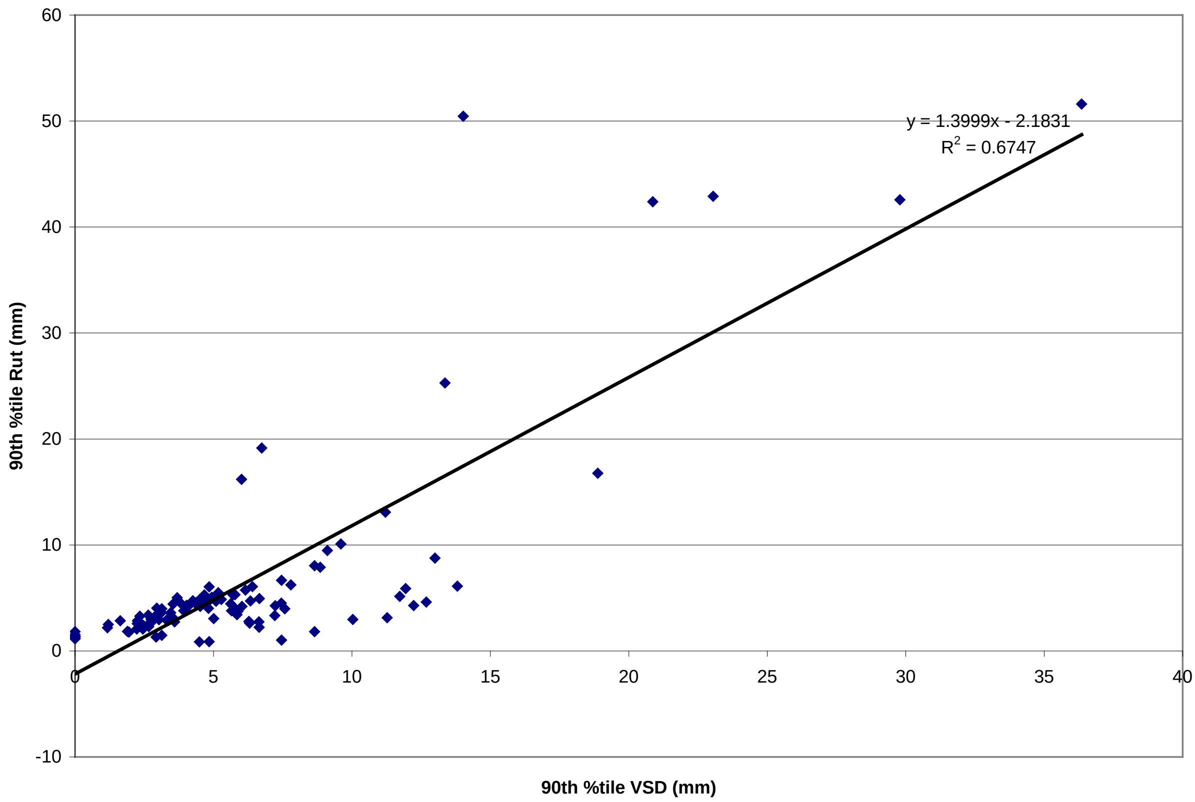
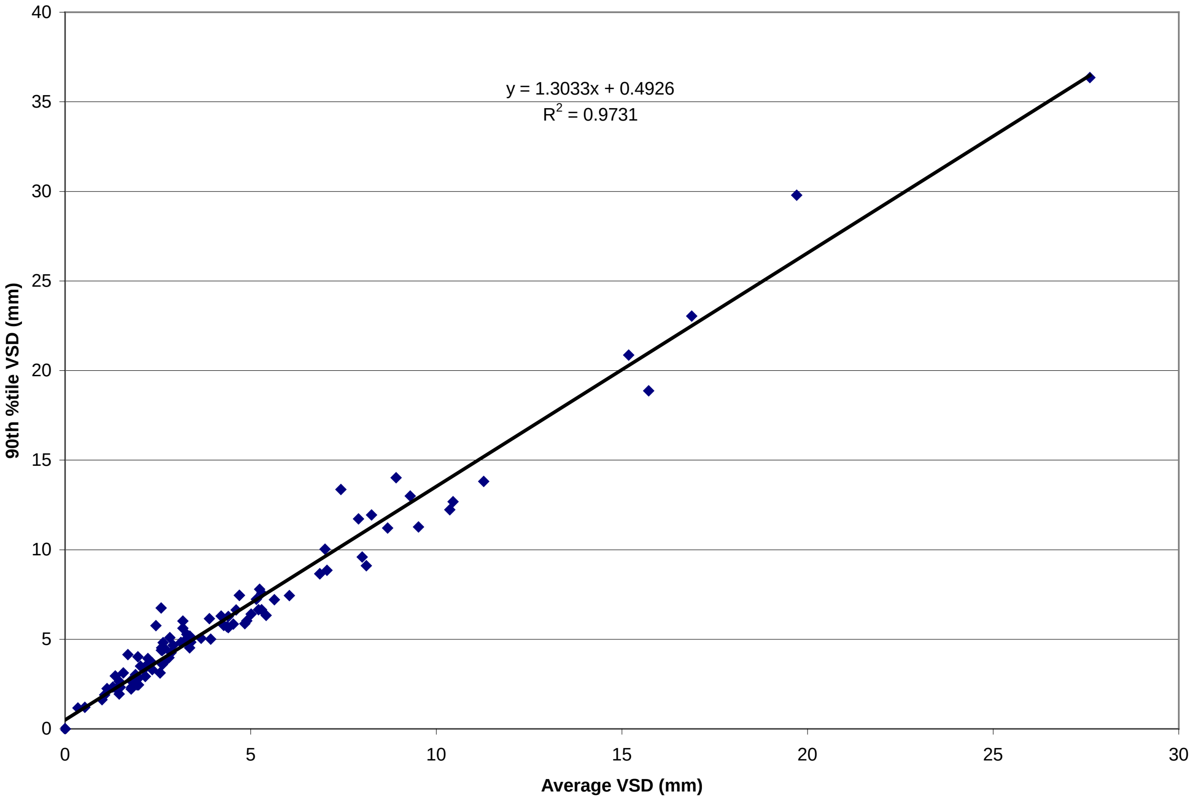


Figure 2.9 Relationship between 90th percentile VSD and 90th percentile rutting for all sections



The first stage to extend the data is shown in figure 2.9, which suggests that a 90th percentile VSD of 16mm equates to a 90th percentile rut of approximately 20mm.

Figure 2.10 Relationship between average VSD and 90th percentile VSD for all sections



The second stage is provided in figure 2.10, which links an average VSD of 12mm to a 90th percentile VSD of 16mm and thus the 20mm terminal condition. So the terminal condition is an average VSD of 12mm.

Figure 2.11 Basic extension of data to failure, using all data

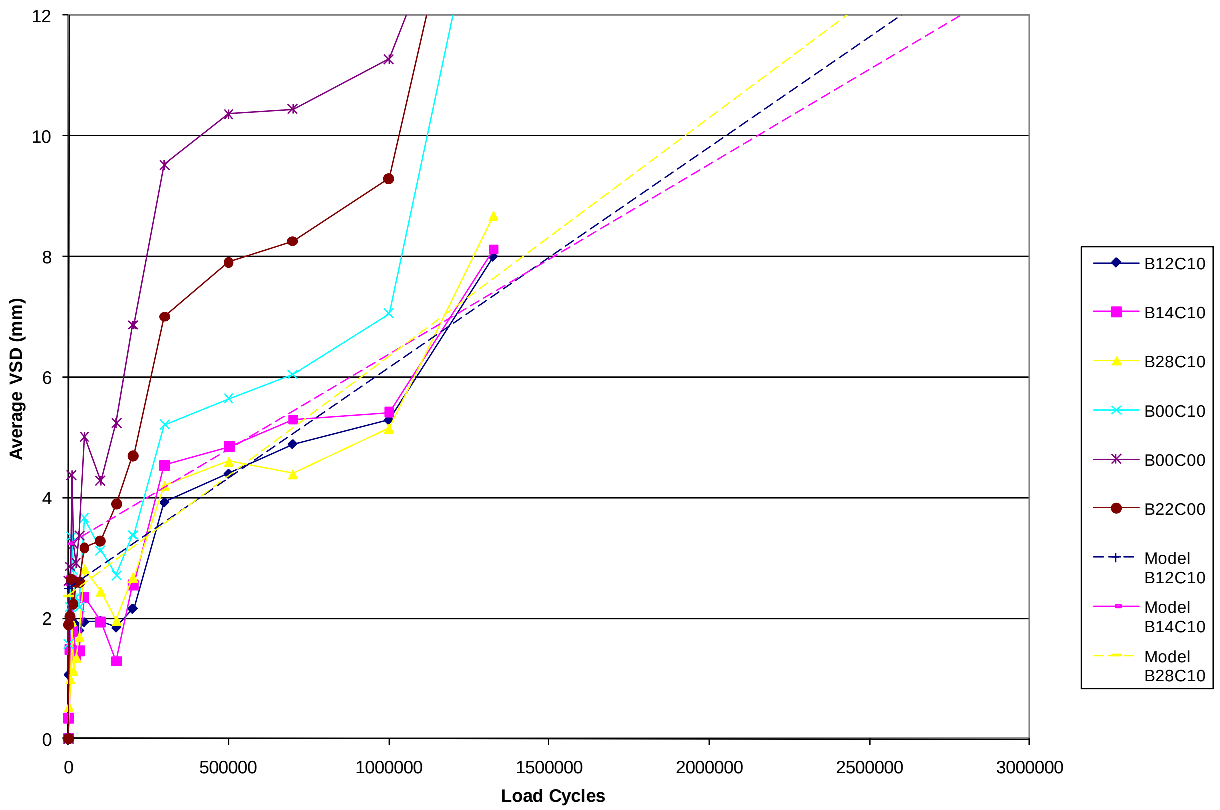


Figure 2.11 suggests that including all the data only requires the foamed bitumen results to be extrapolated to failure. However the dramatic increase in deformation on all the sections beyond 1 million cycles suggests that perhaps the subgrade had weakened due to the sustained rapid loading, perhaps in a manner similar to liquefaction in silts and fine sands under limited load cycles. Idriss and Boulanger (2008) discussed liquefaction and also noted cyclic softening in clays and plastic silts. They noted that clays can lose over half their shear strength under cyclic loading in as little as 1000 load cycles with loading frequencies of 1Hz, which is similar to CAPTIF's loading frequency of 0.25-0.5Hz.

An alternative analysis is performed in figure 2.12, which only uses the data to 1 million load cycles.

Figure 2.12 Alternative extension of data to failure, using data to 1 million cycles

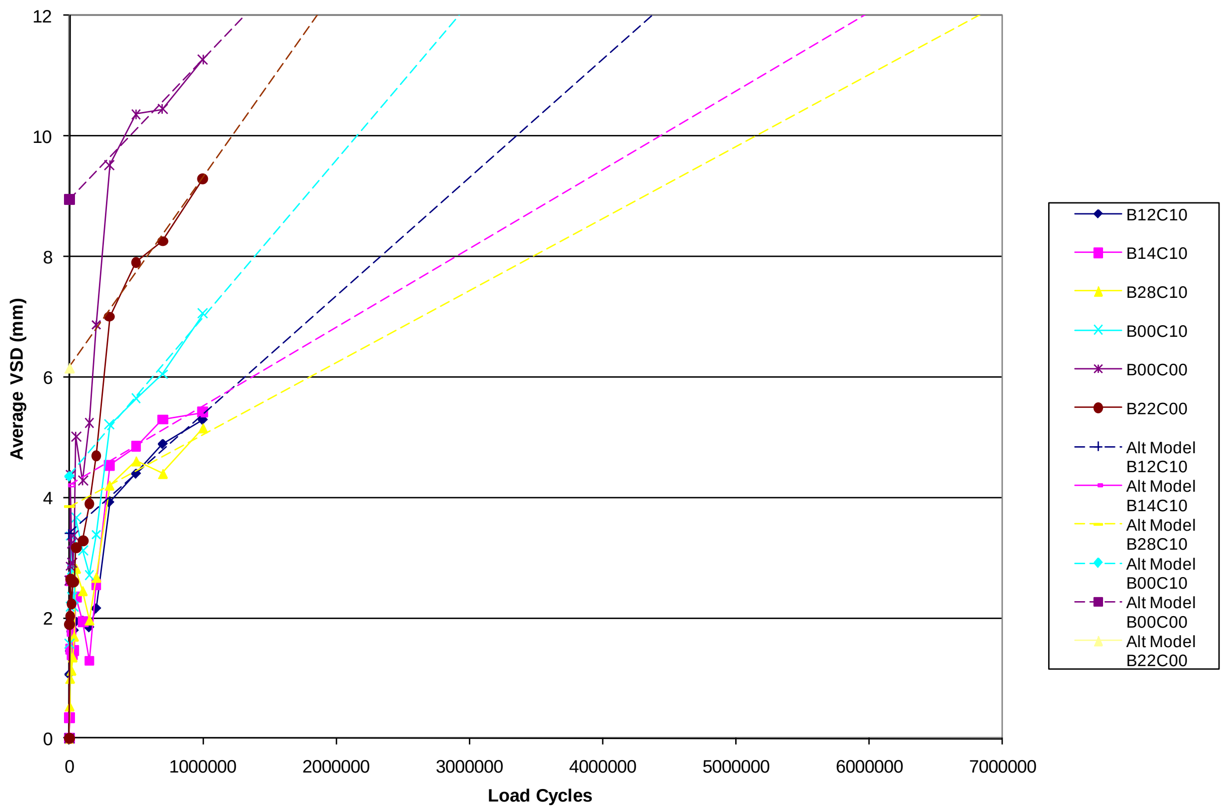


Table 2.10 below provides the number of 60KN load repetitions to the terminal 20mm condition for both the basic and alternative extensions of the data. It also provides the percentage improvement in load-carrying capability from the unbound case.

Table 2.10 Load repetitions to terminal condition

Section	Material	Basic model		Alternative model	
		60KN load cycles	Percent improvement	60KN load cycles	Percent improvement
A	1.2% foamed bitumen, 1% cement	2.6E+06	246%	4.4E+06	333%
B	1.4% foamed bitumen, 1% cement	2.8E+06	264%	6.0E+06	454%
C	2.8% foamed bitumen, 1% cement	2.4E+06	230%	6.8E+06	519%
D	1% cement	1.2E+06	114%	2.9E+06	223%
E	Unbound (no binder)	1.1E+06	100%	1.3E+06	100%
F	2.2% foamed bitumen	1.1E+06	106%	1.9E+06	142%

The data from the FWD testing after construction (table 2.9) was used in a series of CIRCLY analyses to make predictions of performance for comparison with the rutting performance. Table 2.11 below presents the parameters adopted for the CIRCLY model. An assumption of a constant CAPTIF-wide subgrade modulus was used, as the subgrade FWD testing suggested that the subgrade construction was more consistent than the 'after construction' FWD testing analysis predicts. Note that any change from this assumption would be minor, with section E, the unbound layer, having the lowest average modulus of 97MPa, and the stiffer section C, with 2.8% foamed bitumen, having the highest subgrade modulus of 131Mpa. Note that the 1992 approach to sublayering the basecourse materials was used because the current approach basically resulted in the same performance predicted for most of the sections.

Table 2.11 Parameters for the CIRCLY models

Section	Material	Depths (mm)		Moduli (MPa) – 1992 sublayering		
		AC	BC	AC	BC	SG
A	1.2% foamed bitumen, 1% cement	40	210	3000	300	110
B	1.4% foamed bitumen, 1% cement	40	210	3000	450	110
C	2.8% foamed bitumen, 1% cement	40	210	3000	800	110
D	1% cement	40	210	3000	350	110
E	Unbound (no binder)	50	190	3000	210	110
F	2.2% foamed bitumen	50	200	3000	400	110

The results of the CIRCLY model and the CAPTIF results are presented in figures 2.13 and 2.14 following. It can be seen that alternative extension of the results allows a poor, but better, match with the CIRCLY estimate – perhaps confirming that something unusual was happening beyond 1 million load cycles.

Figure 2.13 CIRCLY v basic actual life for each test section

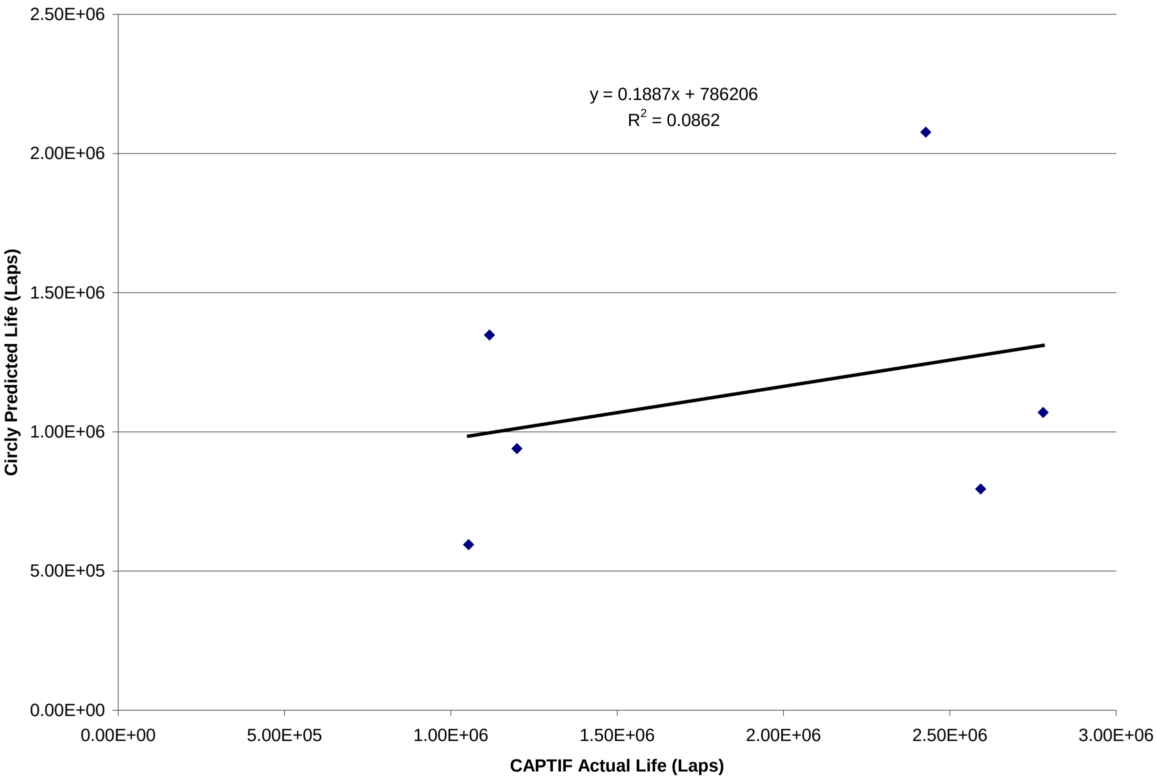
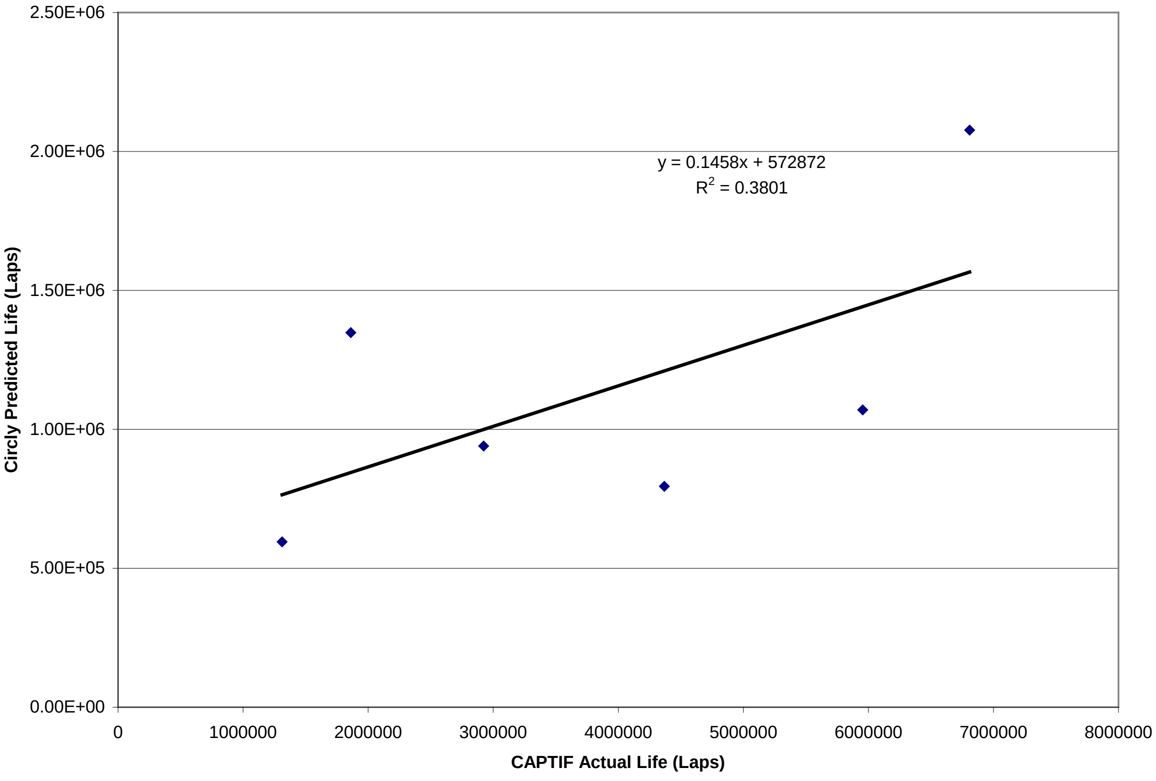
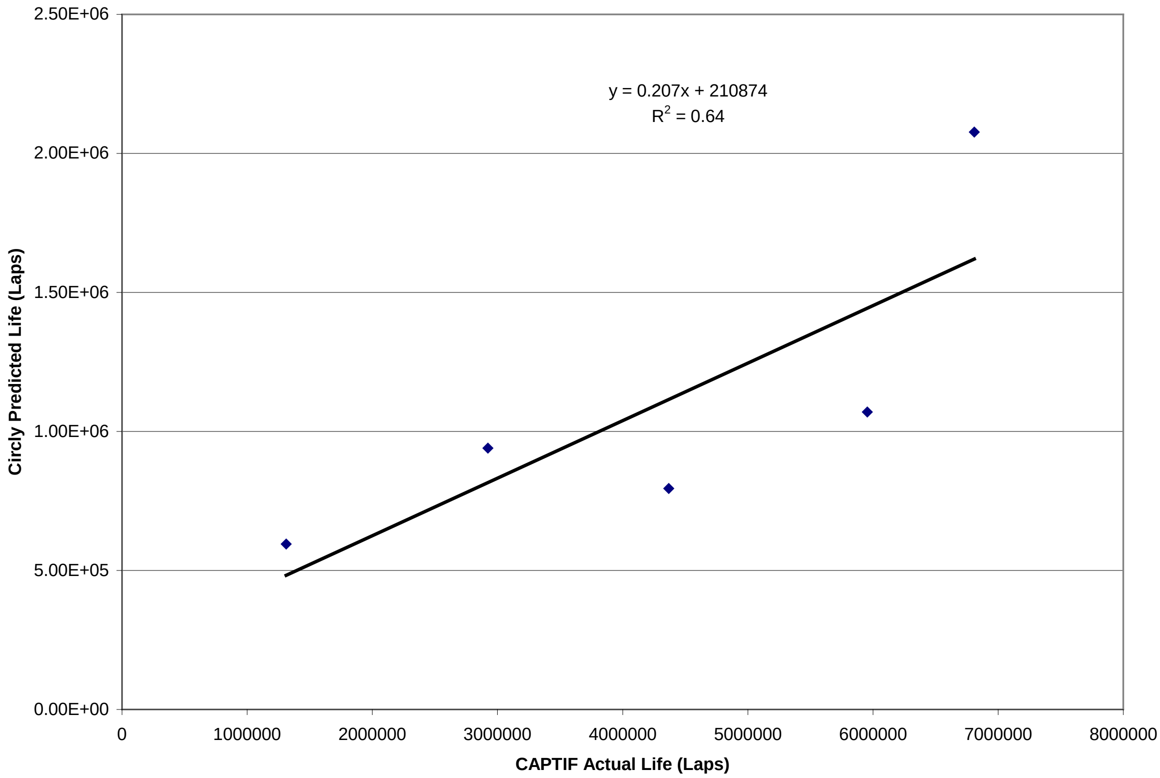


Figure 2.14 CIRCLY v alternative actual life for each test section



Removal of section F, which only contained foamed bitumen, further improves the results (figure 2.15). However the only obvious reason to remove the foamed-bitumen section would be because the RLT results suggest that it would alter the stiffness of the material, but it provides no improvement to the rutting resistance of the material.

Figure 2.15 CIRCLY v alternative actual life (section F – no-cement foamed bitumen removed)



2.7 Interpretation and discussion of results

The pavement test results presented indicate that stabilisation using foamed bitumen and cement improved the performance of the pavements. The rutting of sections B12C10, B14C10 and B28C10 was consistently lower than the other three sections.

The laboratory tests could identify the comparatively poor behaviour of materials without cement, indicating that active fillers are important in the early strength of the materials studied. However, the performance of sections B00C00 and B22C00 was fairly good in comparison with the laboratory testing, where the materials without cement showed a poor behaviour. This could be explained by the low moisture contents obtained in these sections after construction, confirmed during the post-mortem analysis.

The laboratory tests showed contradicting trends for materials at 1% cement. The ITS results at 1% cement indicated that at higher bitumen contents, the strength of the materials increased (figure 2.2a). This is the normal behaviour of foamed-bitumen mixes – there is an optimum bitumen content, after which the ITS value drops (Ruckel et al 1983, Kim and Lee 2006). Similar trends are observed from indirect tensile resilient modulus (ITMr) tests (Nataatmadja 2001, Saleh 2004a). Conversely, permanent deformation RLT specimens at 1% cement showed an opposite trend to those of the ITS results, where the higher the bitumen content, the higher the final permanent deformation (figure 2.2b). In addition, the average resilient modulus measured during the RLT tests (figure 2.2c) showed a peak in the specimen at 1.4% bitumen and 1% cement, and decreases in the specimen at 2.8% bitumen and 1% cement. This peak value was slightly higher than the other values measured on specimens at 1% cement, indicating that foamed bitumen does not have a signficant effect on the resilient modulus measured under RLT stress conditions.

The drawback of the RLT testing presented is that only one specimen was tested after 28 days of curing. However, comparable RLT trends (permanent deformation and resilient modulus) were also observed in several RLT specimens during the preliminary laboratory study before the CAPTIF tests, which is not presented in this report but can be found in Gonzalez (2009). In addition, similar results have been reported in the literature by Long et al (2002) and Long and Ventura (2004). In addition, the minor effect of foamed bitumen in the compressive strength of materials with cement has also been observed in UCS test results by Long et al (2002), Long and Ventura (2004), Frobel (2008) and Browne (2008).

The laboratory results seem to be somewhat related to the strains measurements in the basecourse. The vertical strain measured in the basecourse (figure 2.4c) had a similar trend to those of the resilient modulus in figure 2.2c. The longitudinal strain measured close to the bottom of the basecourse (figure 2.4a) followed the trend of the ITS tests (figure 2.2a). As was mentioned earlier, ITS and ITMr tests followed comparable trends measured during a preliminary CAPTIF laboratory study. These results indicated that both indirect tensile and triaxial resilient modulus tests show a relationship with the elastic deformation of the pavements under real loading.

Because the basecourse was placed over a fairly weak subgrade, the tensile behaviour (related to indirect tensile tests) had a predominant effect on the actual pavement behaviour. A simple multilayered linear elastic model of the pavements studied showed that only the upper 25% of the basecourse was working in a stress condition comparable to the RLT permanent deformation stress conditions (vertical and horizontal compressive stresses). The middle of the basecourse (where the basecourse strain coils were located) was a combination of low compressive and tensile horizontal stresses, while the bottom of the basecourse layer was working at compressive vertical stresses and horizontal tensile stresses.

The vertical strains measured at the top of the subgrade (figure 2.4b) followed a similar trend to those of the deflections (figure 2.3b), indicating the large effect of the subgrade on the elastic response of the pavements. The lowest deflection was measured in section B28C10, caused by the reduction of the horizontal strains at higher bitumen content.

Significantly, the FWD testing showed that none of the test sections appeared to lose stiffness during the test.

The wet testing at the end of CAPTIF experiment indicated that 1.4% and 2.8% foamed-bitumen contents considerably reduced the moisture susceptibility of the stabilised materials. However, only a reduced number of additional load cycles were applied under these conditions and little difference was observed between section B14C10 and B28C10 in terms of rutting or surface cracking.

Even though they were not part of the actual wet test, it was also interesting to note the poor performance of the unbound section during the wet testing and the better performance of the cement and no-cement foamed-bitumen sections.

The curing period of the pavements studied was fairly short – the trafficking of the sections finished only about 10 months after construction. This indicates that foamed bitumen contributed to the early strength of the pavement materials.

Using the FWD readings after construction in a CIRCLY model allowed a reasonable prediction of improvement in performance to be made. However, to make the design work, the 1992 Austroads procedures for sublayering the baselayers were required and the predictions made were still quite conservative. The prediction was improved by removing the obviously outlier material of foamed bitumen without cement.

2.8 Test 1 conclusions and recommendations

A full-scale experiment on foamed-bitumen pavements with different binder contents has been presented here. The following conclusions and recommendations are drawn from the results of this test:

- After the application of 1,326,000 load cycles (5,710,000ESAs), the rutting measured in sections B12C10, B14C10 and B28C10 was the lowest, showing that the addition of foamed bitumen significantly improved the performance of materials with 1% cement that were studied in this research. Sections B00C10 and B22C00, and the control untreated section (B00C00), showed significant rutting and heaving by the end of the test. Little difference was observed within the sections stabilised with foamed bitumen and 1% cement. Based on these results, it is recommended that 1% cement should be considered in the construction of foamed-bitumen mixes.
- Pavement section B28C10 was designed with two design methods to carry 1,000,000ESAs of 80kN. However, after the application of 5,710,000ESAs little rutting was observed, indicating that current design methods for foamed-bitumen pavements are over-conservative.
- To differentiate the rutting performance of the sections stabilised with foamed bitumen and cement, water was introduced through surface cuts. After the application of additional accelerated traffic load section B12C10 started to show surface cracking, while sections B14C10 and B28C10 performed well, indicating that foamed-bitumen contents close to the optimum ITS reduce the pavements' susceptibility to moisture. Based on these results, it is recommended that foamed-bitumen contents that maximise ITS in pavements should be adopted where there is a potential risk of water getting into the pavement layers.
- Though not part of the wet test, the unbound section performed poorly during the testing, and the cement and purely foamed-bitumen sections performed considerably better when wet than the unbound section.
- The deflections at section B28C10 were lower than the other sections, while the untreated section (B00C00) showed the largest values (figure 2.3b).
- The deflection testing during loading suggested that the pavements tested were not being damaged by the loading. The central deflections (D0) decreased slightly with loading (figure 2.3c).
- The ITS values at section B28C10 were double those of sections B12C10 and B14C10. The ITS values from section B22C00 were the lowest. The results indicate that ITS was a reasonably good predictor of the general rutting performance of the pavements studied, and therefore it is recommended as a test to measure the properties of foamed-bitumen mixes.
- The triaxial testing showed that the addition of 1% cement significantly enhances the quality of the foamed-bitumen mixes. However, in the laboratory tests the results obtained from the specimens with 1% cement, an increase in foamed-bitumen content, increases the permanent deformation. The resilient modulus values measured during the triaxial testing could not detect the important improvement in stiffness of section B28C10. This suggests that triaxial testing does not necessarily detect the effect of foamed bitumen in materials with cement, and complementary tests should be conducted to assess the properties of foamed-bitumen mixes.

- The basecourse strain measurements followed the trends observed in most of the laboratory results relatively well. The ITS showed a relationship with the longitudinal elastic strains, while the compressive vertical strains were related to the triaxial resilient modulus. Little difference was observed in compressive vertical strains measurements in sections with 1% cement and different foamed-bitumen contents, indicating that foamed bitumen has little effect in compression. The vertical compressive elastic strains measured at the top of the subgrade followed the surface deflection tests, indicating that surface deflection was controlled by the subgrade elastic response.
- The current design system, using a CIRCLY model, allowed a reasonable prediction of improvement in the performance of the foamed-bitumen sections. However the absolute values were conservative and some changes to the modelling methodology were required.

3 Test 2 – Cement and lime

3.1 Test 2 objectives

The second test pavement was constructed with six different pavement sections, to study objectives 1 and 3 of the project. The aim of this test was to study both existing structural pavement design methodologies and mix design assumptions, and it provided an opportunity to test the benefits of cement modification side-by-side with lime modification. The sections were labelled A–F, and the design parameter in each section is given in table 3.1. The first numeral stands for the accelerated test number – in this case B, C and L stand for bitumen, cement and lime respectively, and as for test 1, the two numerals after the binder designation stand for the binder content, 20 being 2.0%. The designator P stands for precracked.

Table 3.1 Design parameters

Section	Design parameter	Designation
A	Unbound	2B00C00
B	1% cement	2B00C10
C	2% cement	2B00C20
D	2% lime	2B00L20
E	4% cement	2B00C40
F	4% cement – precracked	2B00C40P

3.2 Laboratory mix design

A number of alternative materials were considered for the project, with the aim of improving a material that did not comply with the current NZTA M/4 specification for unbound basecourse. The parent material settled on is an alluvial Springston formation Canterbury greywacke (obtained from a local pit) that had been blended with clay to provide an easy-to-use granular fill material. The sand equivalent was 24 and the clay used in the blend had a plasticity index (PI) of 6. The blend had a broken-face content of 58. Figure 3.1 below provides the particle size distribution of the blended material and the TNZ M/4 grading envelope for comparison.

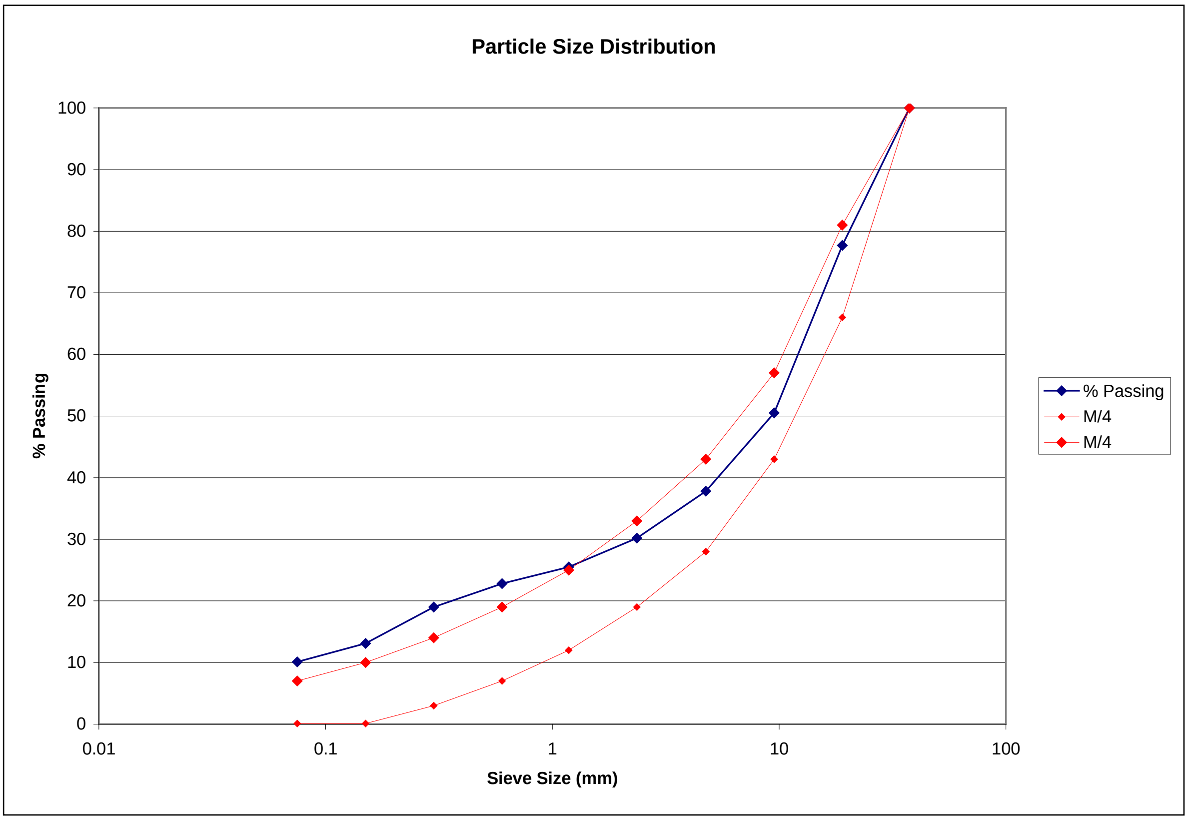
Laboratory-mix design and testing was undertaken at Fulton Hogan's Waikato Laboratory. Initial testing was completed on a wide range of cement and lime contents to determine OWC and maximum dry densities under a vibrating hammer (NZS 4402 1986: test 4.1.3)

3.2.1 ITS testing

ITS testing was conducted over the range of cement contents and one lime content.

The ITS testing followed the Fulton Hogan (FH) Waikato laboratory work instruction 063. This allowed use of the New Zealand Vibrating Hammer (NZS 4402 1986: test 4.1.3) for setting compaction targets. The samples were compacted at OWC and were cured for six days in a sealed plastic bag at room temperature. They were then soaked for 24 hours at room temperature prior to testing.

Figure 3.1 Particle size distribution of the untreated aggregate



The results in table 3.2 show that the basecourse selected responded extremely well to the cement and moderately to the lime. The densities obtained were remarkably high – typical maximum dry densities on unbound NZTA M/4 Canterbury greywacke used previously at CAPTIF were around 2.26t/m^3.

Table 3.2 ITS results

Reactivity agent	Agent (%)	DD (kg/m^3)	Soaked ITS (KPa)	MC before	MC after	Compaction
Cement	1%	2333	423	3.80%	3.80%	98.00%
Cement	1.50%	2355	557	3.80%	3.80%	98.80%
Cement	2%	2364	761	3.70%	3.90%	99.00%
Cement	3%	2351	1139	4.10%	4.00%	98.30%
Cement	4%	2345	1263	4.40%	4.20%	97.80%
Cement	6%	2366	1582	4.50%	3.90%	98.10%
Lime	2%	2333	75	4.50%	4.50%	98.70%

3.2.2 UCS testing

UCS testing was also conducted on the selected cement and lime contents.

The UCS testing followed FH Waikato laboratory's work instruction 062. The samples were compacted at OMC and cured for seven days at room temperature in a sealed plastic bag, then soaked for four hours at room temperature prior to testing.

Again the testing showed (table 3.3) that the basecourse responded remarkably well to cement – under the Austroads Pavement Design Guide (2004) definition, even the lime-stabilised material would have been considered fully bound (7-day UCS > 0.8MPa). However it must be remembered that these samples were compacted to a vibrating hammer standard, rather than using standard compaction.

Table 3.3 UCS results

Reactivity agent	Agent (%)	DD (kg/m^3)	Soaked UCS (MPa)	MC before	MC after	Compact %
Cement	1.0%	2332	7.17	3.8%	3.8%	98.0%
Cement	1.5%	2349	9.79	3.8%	3.8%	98.5%
Cement	2.0%	2339	11.83	3.7%	3.6%	98.0%
Cement	3.0%	2354	14.41	4.1%	3.9%	98.4%
Cement	4.0%	2363	18.97	4.4%	4.0%	98.5%
Cement	6.0%	2360	25.47	4.5%	4.1%	97.9%
Lime	2.0%	2315	2.61	4.5%	4.4%	98.1%

3.2.3 Indirect tensile resilient modulus (ITMr) testing

Indirect tensile resilient modulus (ITMr) tests were conducted. No set methodology was available for this test. Samples were screened on the 26.5mm BS sieve and compacted using the vibrating hammer at OMC. Again, six-day curing and 24-hour soaking was used. The test was at room temperature (20°C approx). The load rise time (10% to 90% load) was 40ms. The pulse repetition period was 3 seconds. The recovered horizontal strain target was 50 ± 5 microstrain.

The results in table 3.4 again show that the basecourse material responded very well to the cement. The ITMr results started at 6095MPa with only 1% cement. The result for the lime was not as encouraging, at only 173MPa.

Table 3.4 Indirect tensile resilient modulus (ITMr)

Reactivity agent	Agent (%)	Diam (mm)	Height (mm)	Moist cont %		DD (kg/m^3)	Comp (%)	ITMr (MPa)	Poisson ratio
				@ comp	@ test				
Cement	1.0%	152	74	4.1	4	2373	99.7	6095	0.18
Cement	1.5%	152	73.4	4.17	4.1	2376	99.7	8459	0.1
Cement	2.0%	152	73.4	4.21	3.9	2365	99.1	15089	0.1
Cement	3.0%	152	73.4	4.5	4.1	2359	98.6	17202	0.1
Cement	4.0%	152	73.9	4.5	4	2369	98.7	19592	0.1
Cement	6.0%	152	74	4.5	3.7	2384	98.9	22246	0.1
Lime	2.0%	152	73.9	4.8	5	2339	99.1	173	0.43

3.2.4 Repeat load triaxial RLT testing

Repeat load triaxial (RLT) testing was undertaken to the Draft TNZ T/15 specification (2007). The stress conditions, loading speed and number of load cycles are provided in table 3.5, and figure 3.2 presents the stress path. The samples were compacted at OMC using a vibrating hammer. Natural samples were tested after 24 hours of soaking, and unsoaked. Cement and lime samples were tested after six days of curing and 24 hours of soaking.

Table 3.5 Six-stage RLT test

RLT testing stress stage	1	2	3	4	5	6
Deviator stress – qmax (kPa) (cyclic vertical stress)	90.0	100.0	100.0	180.0	330.0	420.0
Mean stress – pmax (kPa)	150.0	100.0	75.0	150.0	250.0	250.0
Cell pressure, s3max (kPa)	120.0	66.7	41.7	90.0	140.0	110.0
Major principal vertical stress, s1max (kPa)	210.0	166.7	141.7	270.0	470.0	530.0
Cyclic vertical loading speed	Sinusoidal/Haversine at 4 or 5Hz					
Number of loads (N)	50,000 for each test stage.					

Figure 3.2 RLT stress path

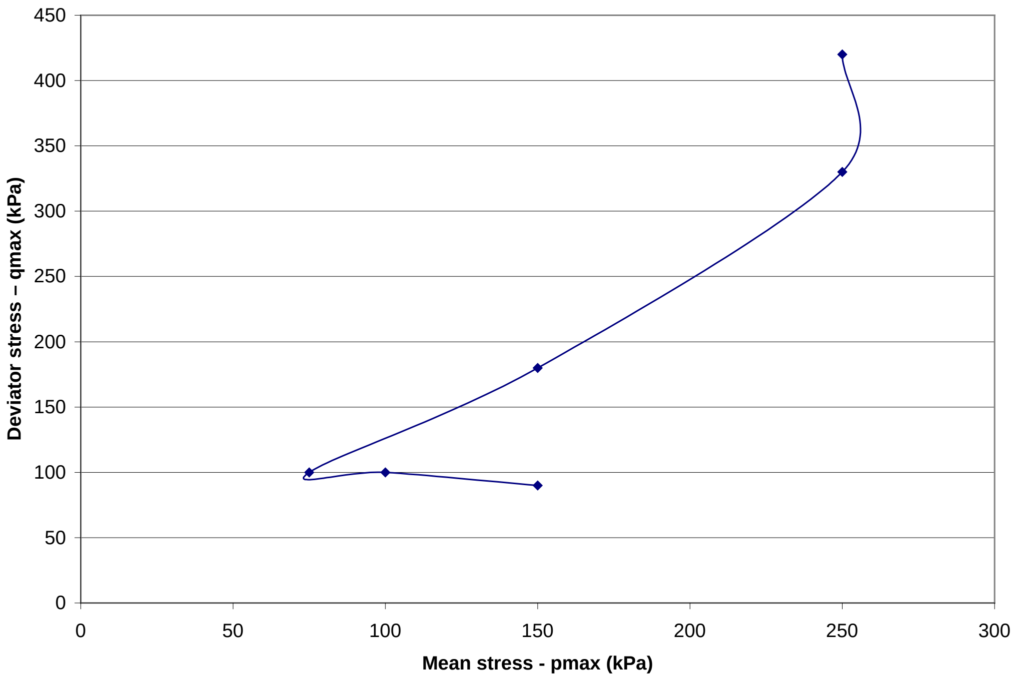


Table 3.6 summarises the RLT test sample preparation.

Table 3.6 RLT test sample preparation

Material description	DD t/m^3	MC after test %	% MDD	% OMC
Natural, soaked	2.302	4.2%	95%	110%
Natural, unsoaked	2.305	3.7%	95%	98%
1% cement, soaked	2.293	4.0%	95%	96%
1.5% cement, soaked	2.292	4.3%	95%	104%
2% cement, soaked	2.297	4.2%	96%	100%
3% cement, soaked	2.295	4.6%	95%	103%
4% cement, soaked	2.285	4.4%	95%	97%
6% cement, soaked	2.299	4.4%	95%	97%
2% lime, soaked	2.255	4.9%	95%	101%

Table 3.7 summarises the resilient modulus observed in each stage. Table 3.8 summaries the permanent strain rate observed in each stress stage and the draft T/15 traffic limit. It can be seen from the results that the addition of cement greatly improved the soaked resilient modulus of the material. Lime also improved the soaked resilient modulus.

Table 3.7 RLT resilient modulus results

Material description	Average resilient modulus (MPa)					
	Stage 1	Stage 2	Stage 3	Stage 4	Stage 5	Stage 6
Natural, soaked	201	179	169	243	303	
Natural, unsoaked	267	226	201	283	331	318
1% cement, soaked	537	582	594	745	918	995
1.5% cement, soaked	675	660	641	806	977	993
2% cement, soaked	1008	1009	969	1094	1123	1158
3% cement, soaked	531	538	540	704	956	1026
4% cement, soaked	682	665	652	828	1076	1172
6% cement, soaked	691	746	752	839	933	990
2% lime, soaked	272	270	265	314	348	365

The addition of cement took the material from unusable in its natural state to being allowed over 10 million ESA under the draft T/15 requirements with only 1% cement. The addition of lime took it to 3 million ESA.

Table 3.8 RLT permanent strain results

Material description	Permanent strain rate (microstrain*1000/cycles)						T/ 15 traffic limit
	Stage 1	Stage 2	Stage 3	Stage 4	Stage 5	Stage 6	
Natural, S[a]	2.790	3.010	2.780	6.530	47.100	Failed	Unable to be used
Natural, U[b]	1.340	1.430	1.140	2.970	15.800	0.051	Unable to be used
1% cement, S	0.396	0.944	-0.008	0.244	0.780	0.051	>10 million ESA
1.5% cement, S	0.508	0.396	-0.196	0.716	0.628	0.051	>10 million ESA
2% cement, S	0.272	0.160	-0.056	0.132	0.476	0.051	>10 million ESA
3% cement, S	1.240	0.524	0.068	0.184	0.700	0.051	>10 million ESA
4% cement, S	-0.208	-0.436	-0.096	0.460	1.370	0.051	>10 million ESA
6% cement, S	0.196	-0.040	-0.060	0.120	0.544	0.051	>10 million ESA
2% lime, S	1.510	1.690	0.952	2.910	7.340	0.051	3 million ESA

a) S = soaked testing.

b) U = unsoaked testing.

3.2.5 Summary of initial lab testing

Figure 3.3 presents a graphical summary of the UCS, ITS and ITMr results. The results are much higher than those from similar overseas research. A side study commissioned to investigate this suggested that these results were due to the differences in compaction techniques used. Typically overseas researchers use 'standard' drop hammers for compaction, while the New Zealand industry uses vibrating hammers.

Figure 3.3 UCS, ITS, RL ITS v binder content

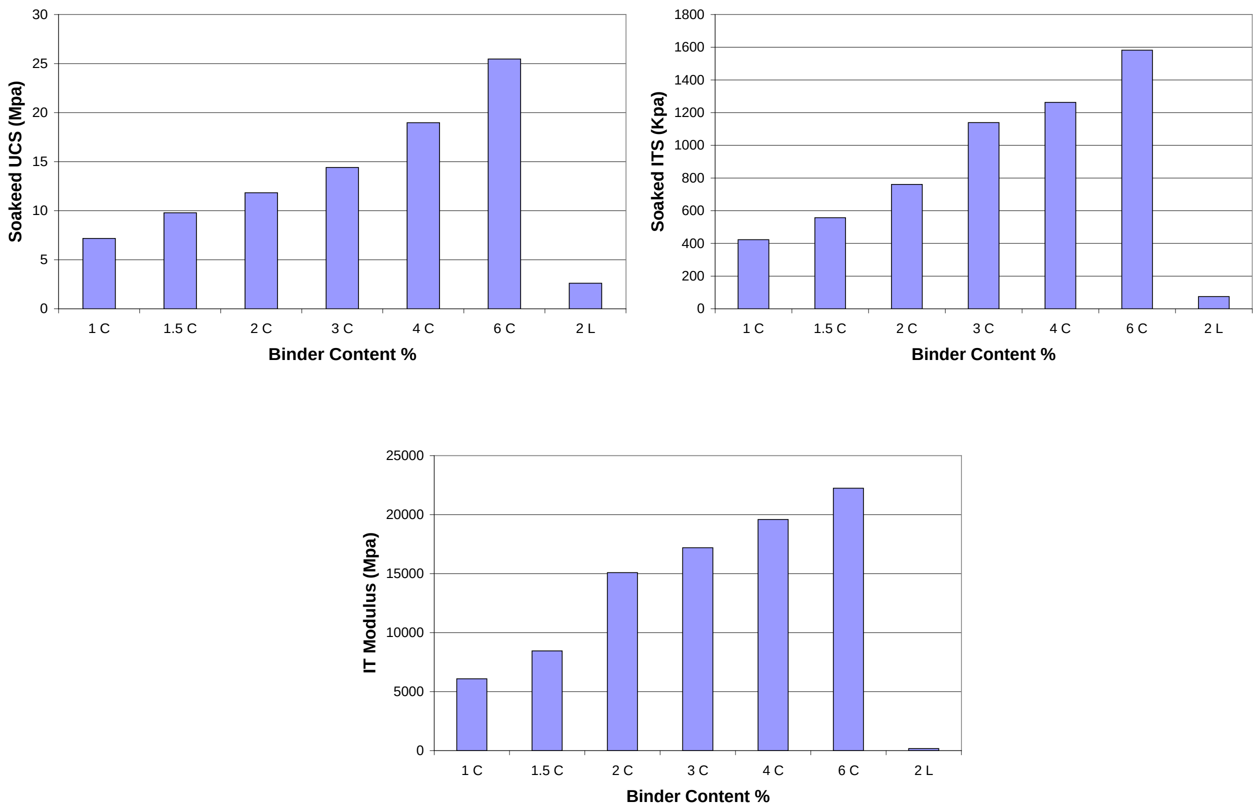


Figure 3.4 summarises the change in resilient modulus and permanent strain rate with change in binder content and type. It shows the average values across the six stress stages and the sixth and harshest stage results. It can be seen that effect of the cement is dramatic; however, the effect of increasing the cement content is not obvious.

Figure 3.4 Permanent strain rates and modulus v binder content

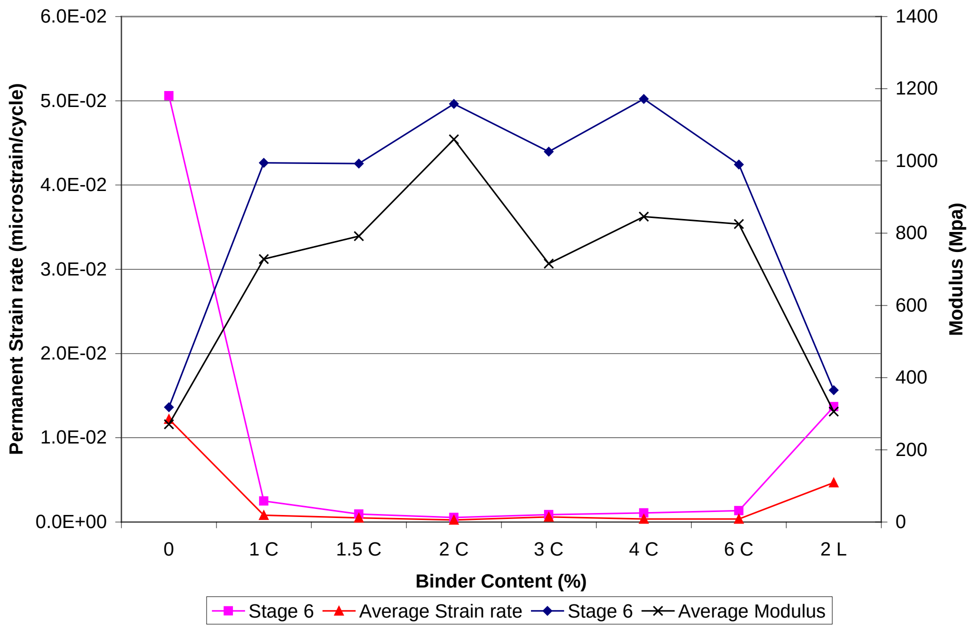


Figure 3.5 provides a more detailed view of the effects of adding cement and shows that beyond 1% cement, the RLT could not distinguish improvements from increasing the binder content.

Figure 3.5 Permanent strain rates and modulus v cement binder content

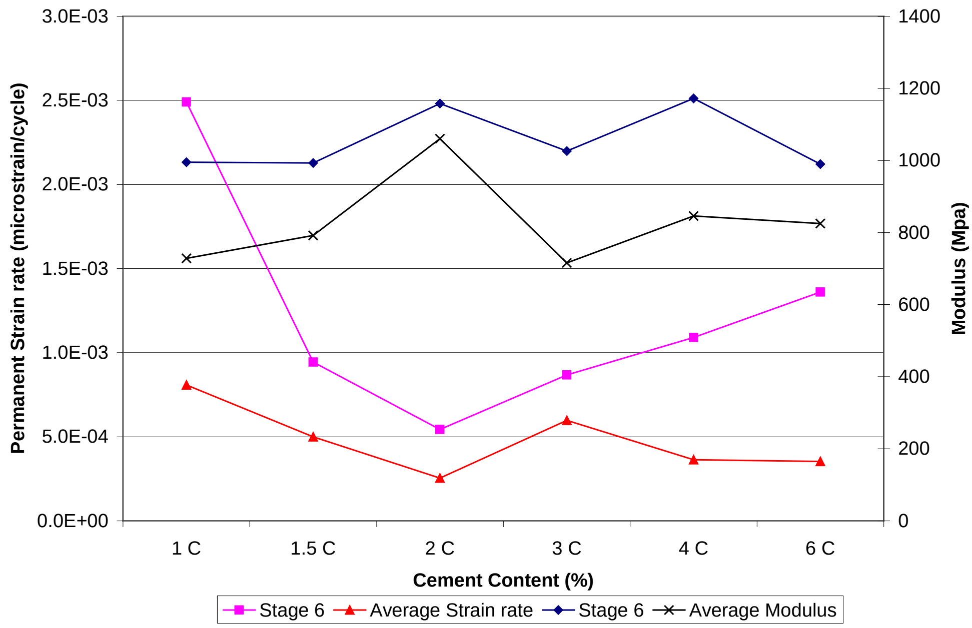


Figure 3.6 plots the average modulus in each stress stage. It was hoped that a change from stress-dependent to a non-stress-dependent modulus would be observed with increasing binder content. The flat line between stages 1 and 3 was because the stress states were fairly similar. However, overall no change in behaviour was observed at the higher stress stages. This suggests that the RLT is relatively insensitive to the changing amount of binder. In hindsight this is not terribly surprising, as at high cement contents the RLT test can be thought of as a repetitive low-stress UCS compressive test. As the UCS strength increases with cement content, the RLT test becomes a non-destructive modulus test, as the compressive strength of the material is well beyond the compressive stresses being applied.

Figure 3.6 Resilient modulus with stress stage

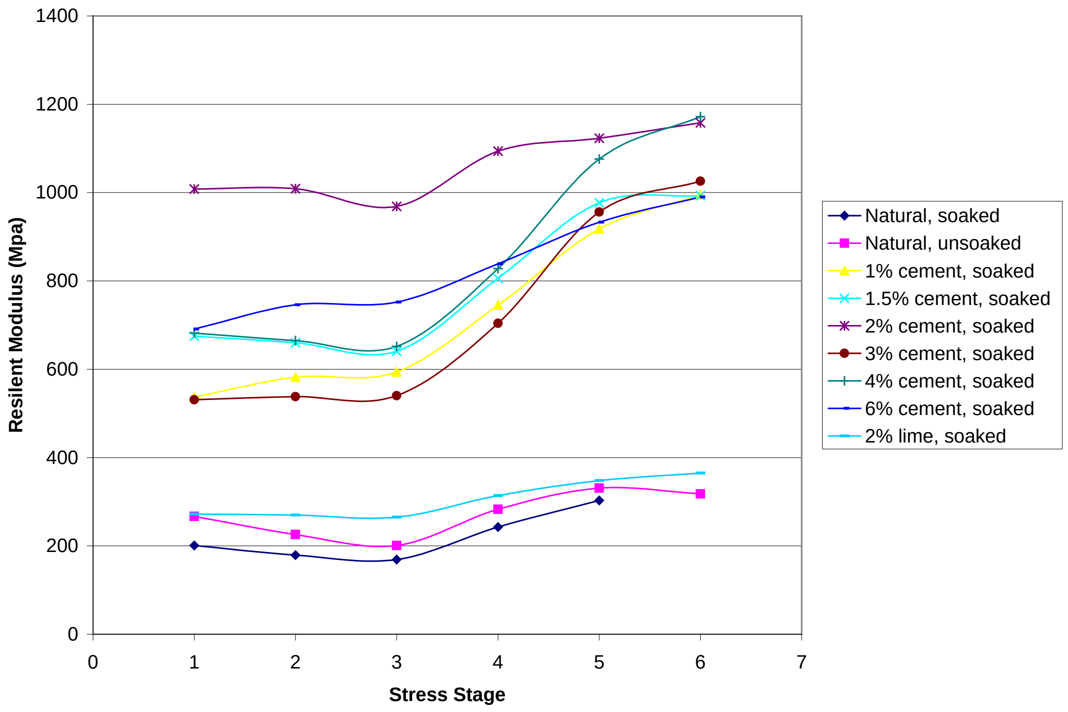
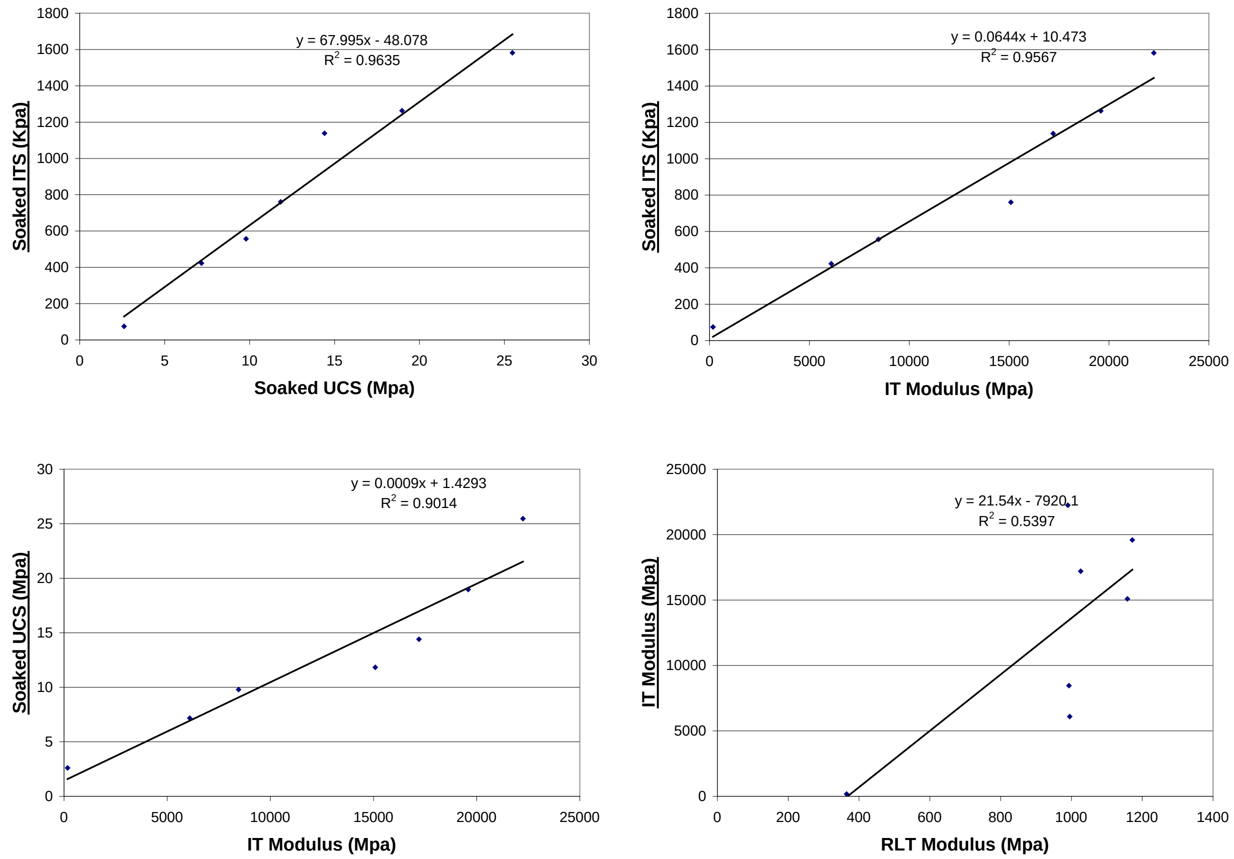


Figure 3.7 illustrates the relationships between the various laboratory tests conducted. There was a good correlation between ITS and UCS values, with an r^2 of 0.96. ITS and ITMr also shared a strong relationship, with an r^2 of 0.96. UCS and ITMr had a fairly strong correlation, with an r^2 of 0.90. However, as would be expected from the last section, the RLT modulus and ITMr only had a weak relationship, with an r^2 of 0.54. This was because the RLT had been effectively just testing the modulus of high cement content materials at low stresses in comparison to their compressive strength, while the ITS test was at modulus value at a tensile failure stress. Adding in the lime data, where there was a low ITS and the material was actually appropriately tested in the RLT, made for a particularly poor relationship.

Figure 3.7 Relationships between lab tests



3.3 Pavement construction

3.3.1 Pavement design

The pavement design for these test sections followed the design of the first test, and 200mm deep basecourse test sections were used.

3.3.2 Subgrade density and CBR testing

In-situ CBR testing was conducted with a Scala penetrometer at three stations in each section. The in-situ CBR was estimated from the RG Brickell relationship, and the reading given in table 3.9 is the average estimated CBR over the top 300mm of subgrade. Surprisingly, the results were lower than test 1.

Table 3.9 Subgrade CBR

Section	Material	Average	Min	Max
A	Unbound	5	3	10
B	1% cement	3	3	4
C	2% cement	4	3	4
D	2% lime	4	3	6
E	4% cement	5	4	5
F	4% cement precracked	3	3	4

3.3.3 Subgrade FWD modulus values

FWD tests were undertaken at each station. The test set-up used a 300mm diameter loading plate and a nominal plate pressure of 240kPa. The results were back-analysed in ELMOD and the modulus of the subgrade is presented in table 3.10. The results were consistent with the in-situ CBR test results from the Scala penetrometer testing.

Table 3.10 Isotropic subgrade modulus from Elmod analysis

Section	Material	Average	Min	Max
A	Unbound	24	18	27
B	1% cement	29	23	33
C	2% cement	34	25	48
D	2% lime	25	21	32
E	4% cement	26	14	35
F	4% cement precracked	29	15	48

3.3.4 Basecourse construction

For the stabilisation process, trenches were again excavated outside the CAPTIF building. A layer of aggregate was laid in the trench and compacted to 95% of maximum dry density at its OMC. The stabilising agents were spread from bags, compaction water was added with a water cart, and it was mixed with a recycling machine, keeping a constant speed. The stabilised material was transported into CAPTIF building (located about 50m from the trenches) by loaders. A paver was used to place the basecourse material in a single layer of 200mm and a steel roller was used for compaction in addition to CAPTIF's 750kg plate compactor. The time between trench stabilisation and final compaction of the stabilised basecourse layer for one pavement section was between two and three hours. During this time the Emu coils, which were protected and paved over, were also moved into their final positions in the pavement.

Before the sealing, density and moisture measurements were taken using a nuclear gauge. Sand replacement tests were also undertaken to provide a correction factor for moisture content. Table 3.11 shows that the density achieved was relatively low compared with the compaction targets, averaging around 90–93% of the maximum dry density. However as noted earlier, the compaction target was very high when compared with 2.26t/m^3, which was previously used for similar NZTA M/4 Canterbury greywacke basecourses. Later FWD testing and the pavement's actual performance suggested that these results, while low relative to the high compaction target, were in fact acceptable.

Table 3.11 NDM results

Section	Material	Avg DD	St dev DD	Avg MC	St dev MC	Avg %MDD	St dev %MDD	No. readings
A	Unbound	2237	28	2.8	0.18	92.4	1.1	14
B	1% cement	2181	14	3.0	0.34	90.9	0.6	15
C	2% cement	2221	34	3.4	0.22	90.3	1.4	14
D	2% lime	2221	31	4.0	0.33	93.3	1.3	14
E	4% cement	2193	26	3.4	0.30	91.0	1.1	15
F	4% cement precracked	2191	27	3.9	0.31	90.9	1.1	14

During construction, samples were also taken for UCS and ITS testing directly after hoeing, and transported to a laboratory 15 minutes away, where the samples were compacted. Three samples for each test were prepared from each site and the results in tables 3.12 and 3.13 show that the binders were appropriately measured and mixed by the process. But figure 3.8 shows that the results from the field tests, while still high, were considerably lower (by 20–30%) than those obtained in the initial laboratory testing, possibly due to the lower densities (98% v 95%) obtained because of the longer time between mixing the binders and compaction (approximately 1 hour).

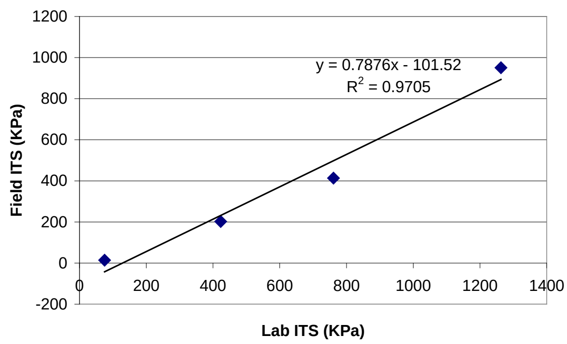
Table 3.12 ITS results

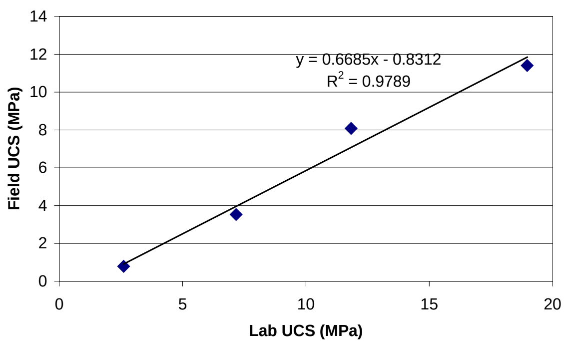
Reactivity agent	Agent (%)	DD (kg/m^3)	Soaked ITS (KPa)	MC before	MC after	Compaction
Cement	1%	2303	202	4.4	4.4	96
Cement	2%	2324	413	4.8	4.8	95
Cement	4%	2311	951	5.7	5.7	96
Lime	2%	2297	14	5.8	5.8	97

Table 3.13 UCS results

Reactivity agent	Agent (%)	DD (kg/m^3)	Soaked ITS (KPa)	MC before	MC after	Compaction
Cement	1.0%	2297	4	4.4	4.4	96
Cement	2.0%	2324	8	4.8	4.8	94
Cement	4.0%	2298	11	5.7	5.7	95
Lime	2.0%	2286	1	5.8	5.8	96

Figure 3.8 Lab v field ITS and UCS values





3.3.5 Surface construction

The pavement was surfaced with TNZ M/10 mix 10HMA. The basecourses were uniformly primed and the HMA was paver laid to a nominal 30mm thickness.

3.3.6 Layer thicknesses

The final average thickness and standard deviation of each layer is recorded in table 3.14 below. The HMA averages ranged from 28–42mm and the basecourses from 20–222mm.

Table 3.14 Layer thickness values

Section	Material	Avg HMA (mm)	St dev HMA (mm)	Avg BC (mm)	St dev BC (mm)	No. of readings
A	Unbound	42	7	206	9	1434
B	1% cement	36	3	212	5	1673
C	2% cement	35	3	216	6	1433
D	2% lime	40	6	207	8	1434
E	4% cement	31	3	221	6	1193
F	4% cement precracked	28	3	222	4	478

3.3.7 Final construction FWD testing

A month after construction of the basecourse layers and prior to loading, a final set of FWD tests were conducted and the results are provide in table 3.15 (below). The resilient modului of the basecourse layers (E1) clearly show the influence of adding the cement as a binder and also using precracking (section F). Interestingly, the change from 1–2 % cement is not as clear. The average modulus of the subgrade (E2) appears to have changed to approximately 110MPa.

Table 3.15 FWD basecourse and subgrade isotropic moduli from Elmod

Section	Material	Avg E1[a] (MPa)	Min E1 (MPa)	Max E1 (MPa)	Avg E2[b] (MPa)	Min E2 (MPa)	Max E2 (MPa)
A	Unbound	750	457	893	127	100	154
B	1% cement	2240	981	3048	98	62	140
C	2% cement	1992	1363	3461	129	95	146
D	2% lime	773	705	854	91	79	109
E	4% cement	4543	1878	6306	75	38	155
F	4% cement precracked	2771	1328	5161	123	81	152

a) E1 = the basecourse moduli.

b) E2 = the subgrade moduli.

3.4 Pavement testing

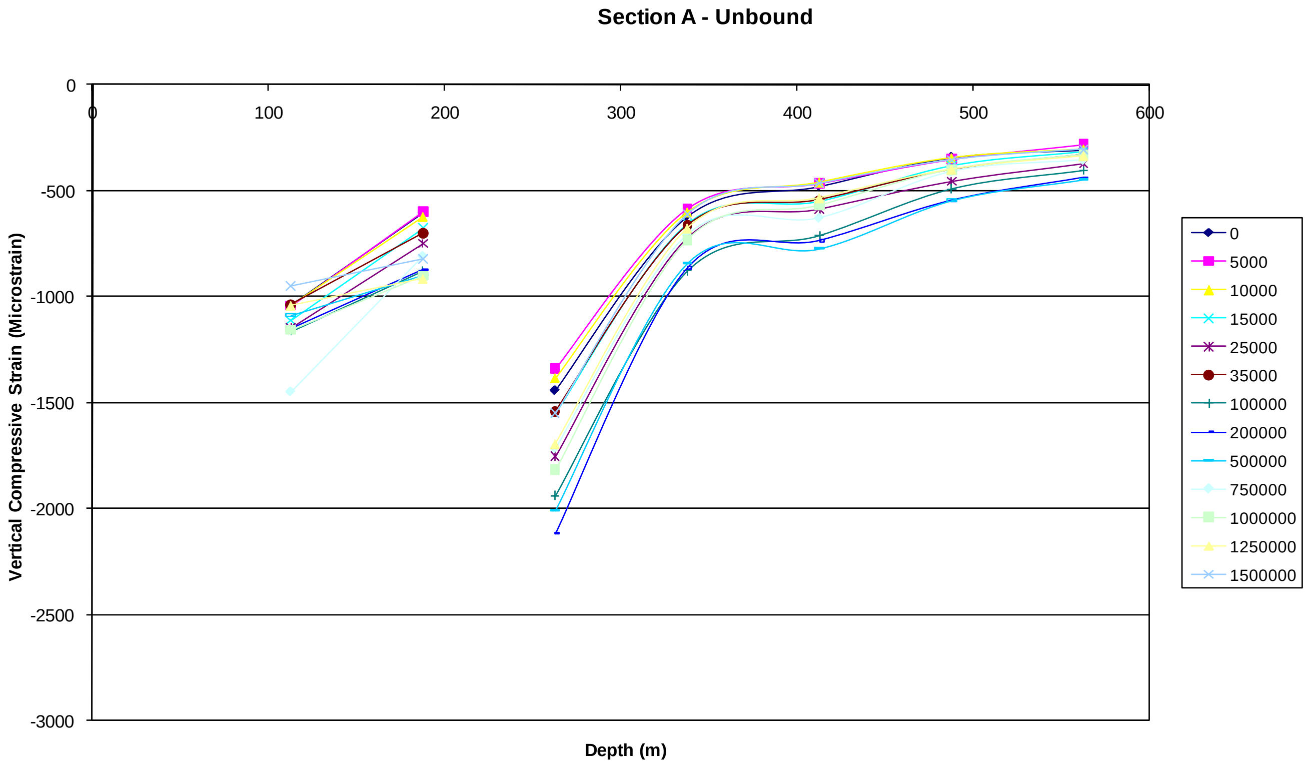
3.4.1 Vehicle loading

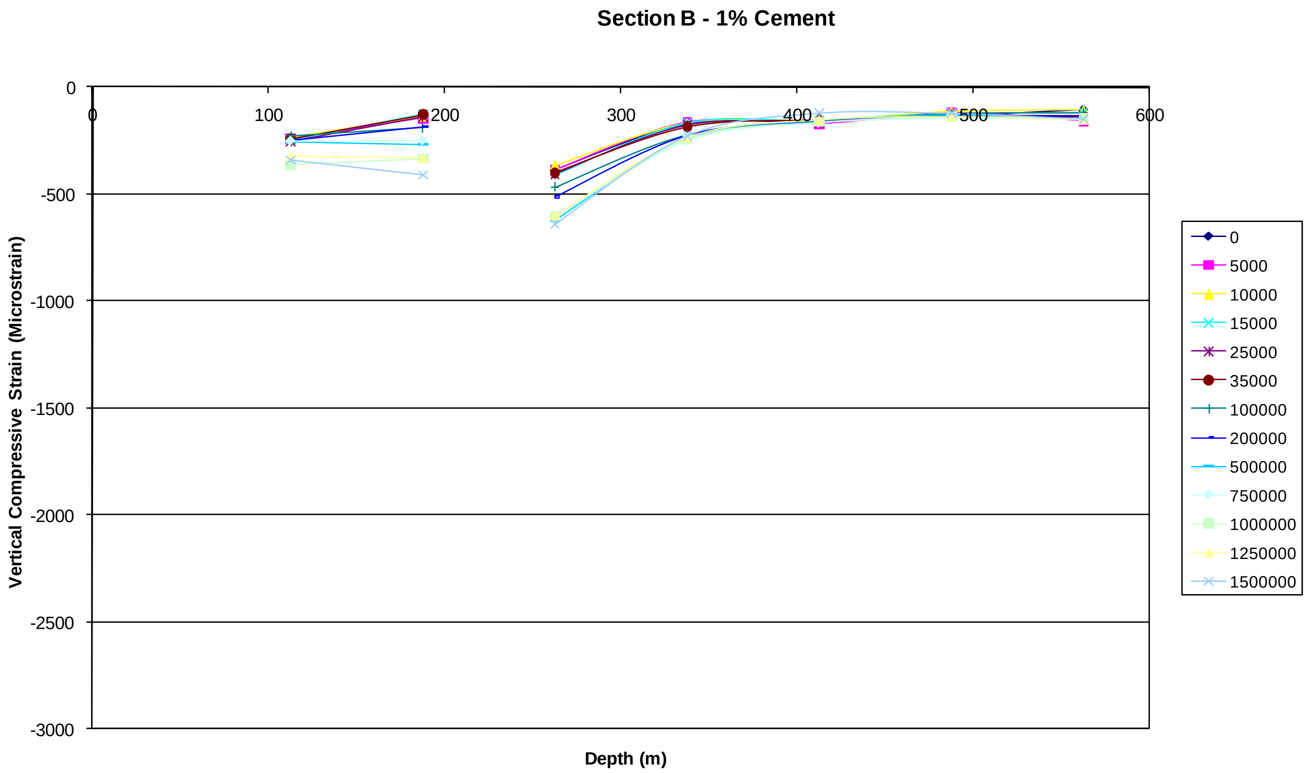
The CAPTIF vehicles were loaded to 60KN to simulate a 12-tonne single-axle dual tyre, and the tyre pressures adjusted to 700kPa. The suspension was checked with the EU bump test and loading was applied at 40km/hr. Loading was stopped at regular intervals to allow testing. Testing included Emu strain measurements, dynamic loading measurements, deflectometer readings and transverse profiles.

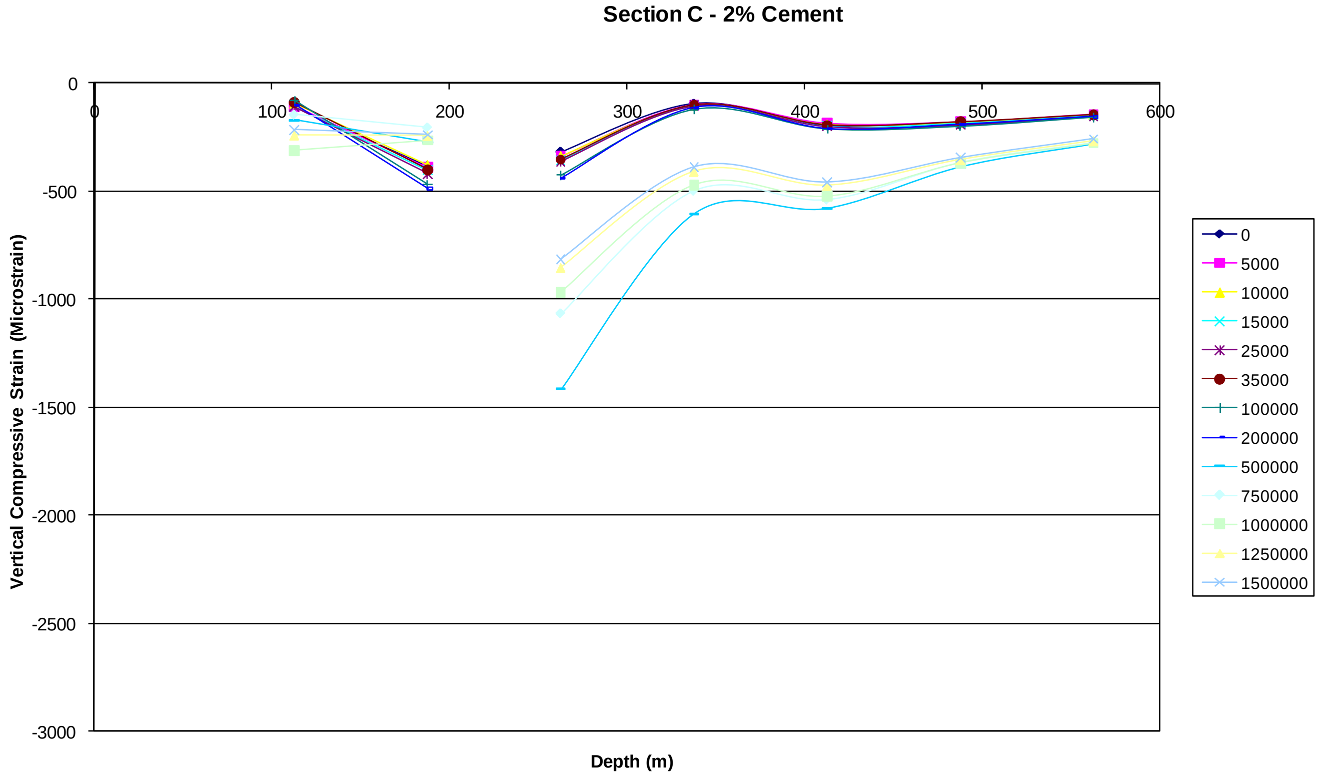
3.4.2 Emu testing

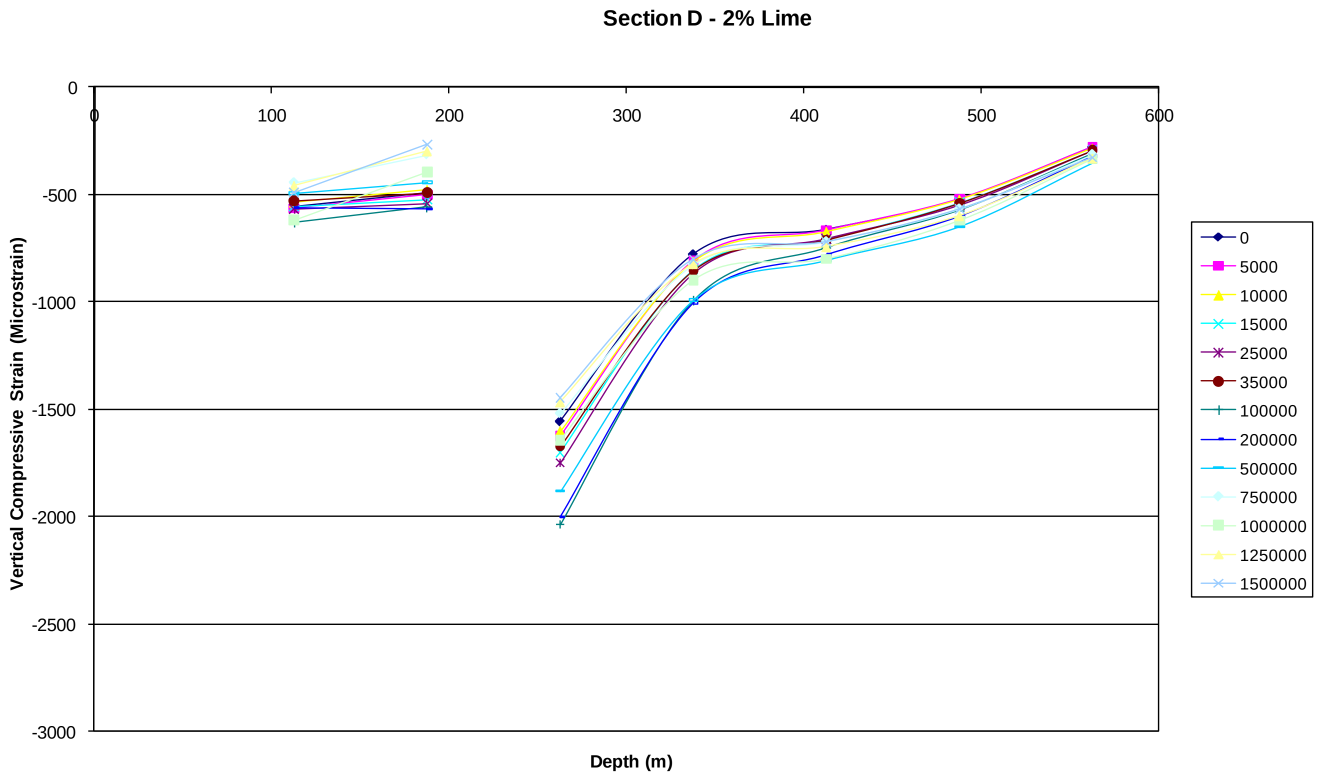
Figure 3.9 presents the vertical peak compressive strain readings in each section. The effect of the cement stabilisation can be seen in the readings. The sections with cement had significantly lower strains in the subgrade and lower strains in the basecourse. The series presented in the figures are the loading intervals. It can be seen that some changes seemed to be occurring with time.

Figure 3.9 Vertical compressive strain

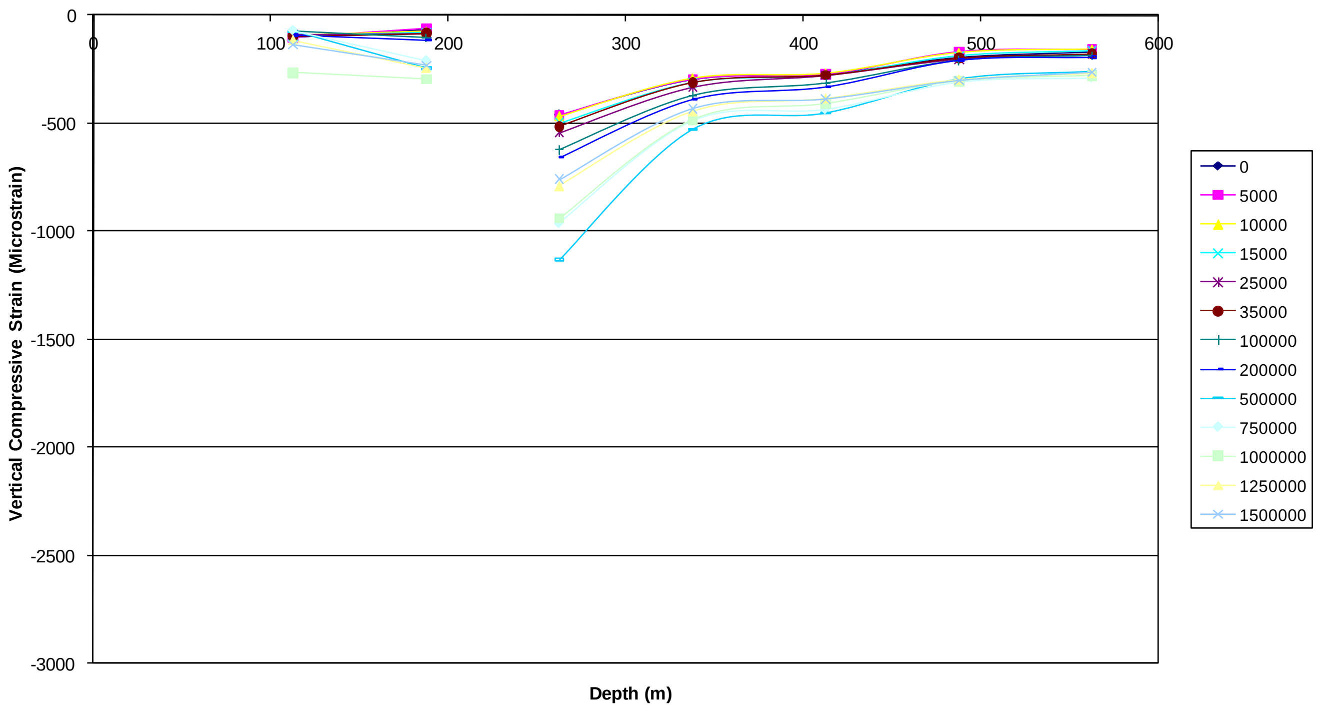





Section C - 2% Cement
Vertical Compressive Strain (Microstrain)
Depth (m)
0
-500
-1000
-1500
-2000
-2500
-3000
0
100
200
300
400
500
600
0
5000
10000
15000
25000
35000
100000
200000
500000
750000
1000000
1250000
1500000


Section D - 2% Lime
Vertical Compressive Strain (Microstrain)
Depth (m)
0
-500
-1000
-1500
-2000
-2500
-3000
0
100
200
300
400
500
600
0
5000
10000
15000
25000
35000
100000
200000
500000
750000
1000000
1250000
1500000

Section E - 4% Cement



Section F - 4% Cement Pre-Cracked

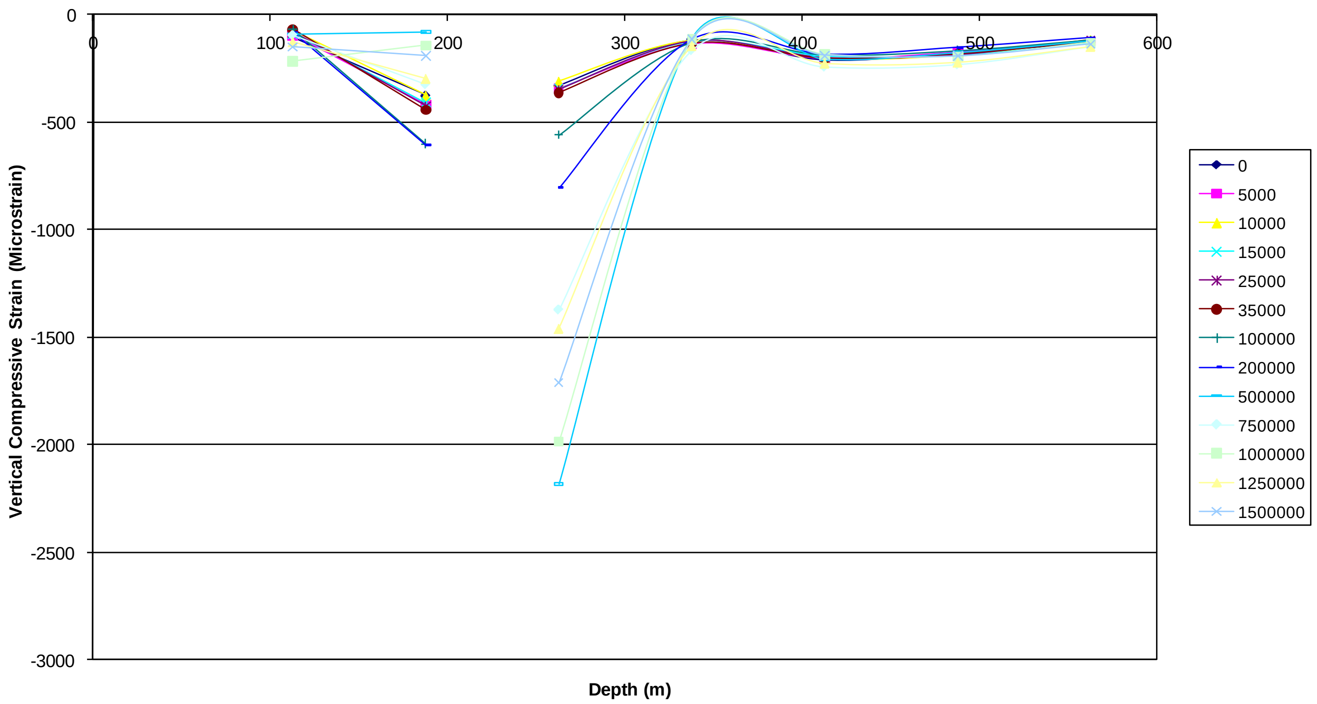
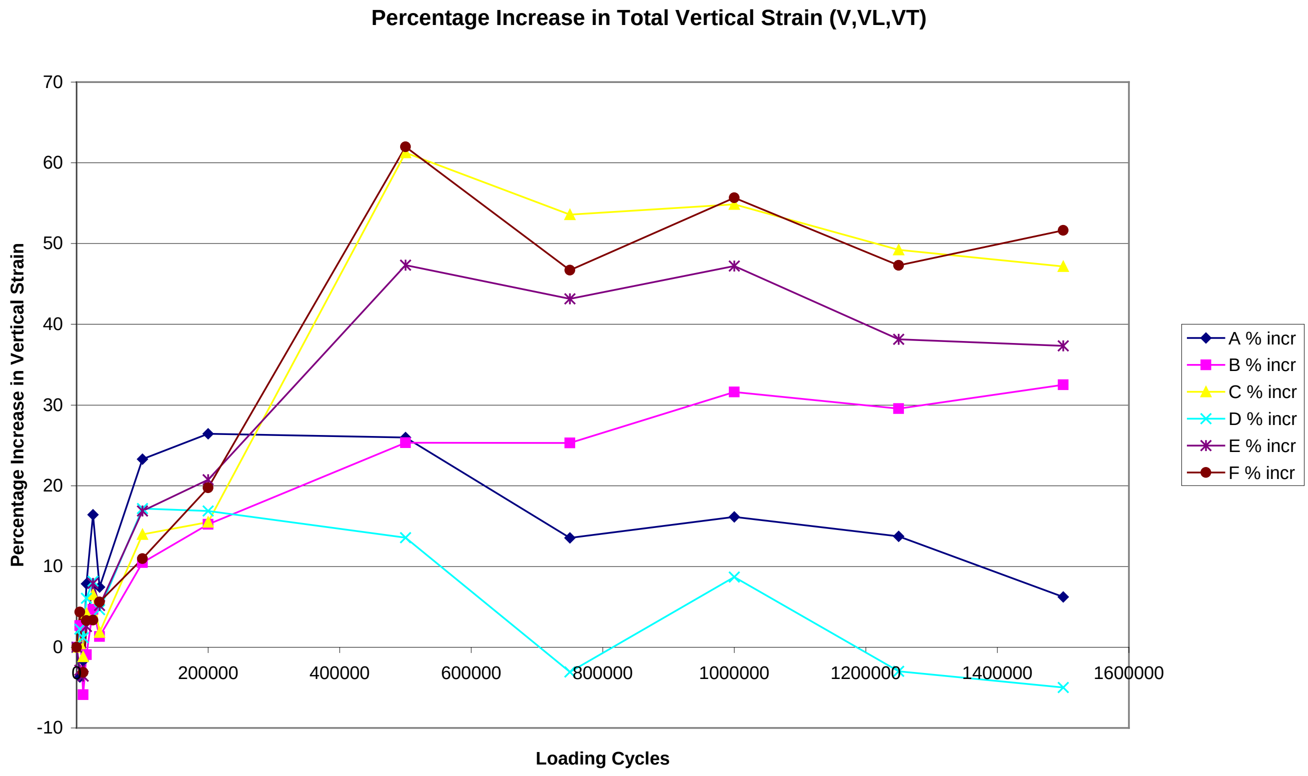


Figure 3.10 presents the percentage change in total vertical compressive strain measured in each section as the test progressed. The total strain is the summation of the vertical compressive strains measured in all the coils. There seems to have been a significant increase in strain occurring in the cemented sections, suggesting they were being damaged by the loading and that the moduli of the materials were reducing. This process appears to have been finished by 500,000 load cycles. The unbound control and lime-stabilised sections initially increased in strain and then decreased. Similar results have been seen in previous unbound CAPTIF tests and suggest some softening behaviour followed by hardening as the pavement settles down.

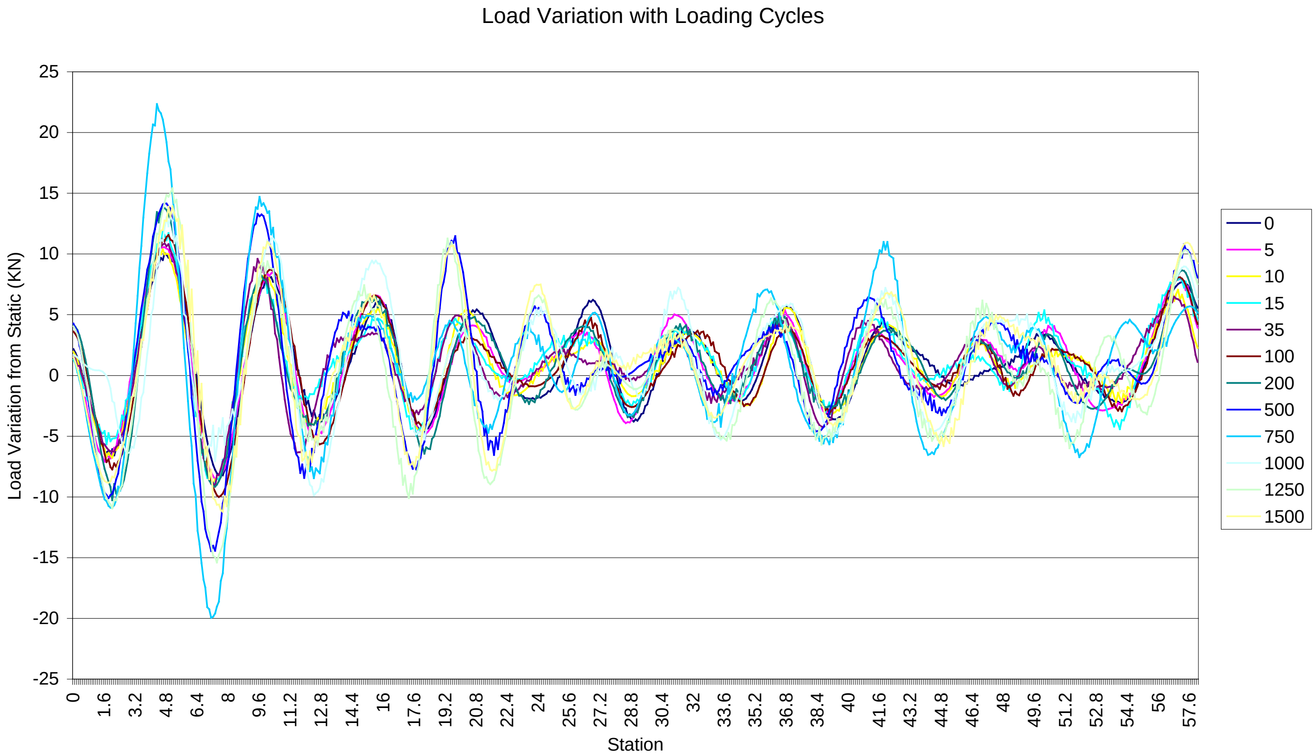
Figure 3.10 Total vertical compressive strain



3.4.3 Loading

The variation in dynamic loading around the track was recorded at each testing interval (figure 3.11). The dynamic loading was measured by knowing the static weights of the vehicles and measuring their accelerations as they pass around the track. The results suggest that change in the dynamic load would not explain the change in total vertical compressive strain seen in figure 3.10.

Figure 3.11 Dynamic vertical loading (load interval x 1000)



3.4.4 FWD testing

FWD readings were undertaken at 0 laps, 676,000 laps and 1,500,000 laps. The raw D0 results showed significant increases with loading in the cement-stabilised sections (figure 3.12). This confirmed the trends seen in the Emu data. The percentage change in D0 for each section is presented in figure 3.13. This was a significant change in behaviour when compared with the first test pavements, which did not significantly change in D0 (figure 2.3c). The figure seems to suggest, quite sensibly, that the stiffer the material is initially, the more room it has to lose that stiffness on loading.

Figure 3.12 FWD D0 results

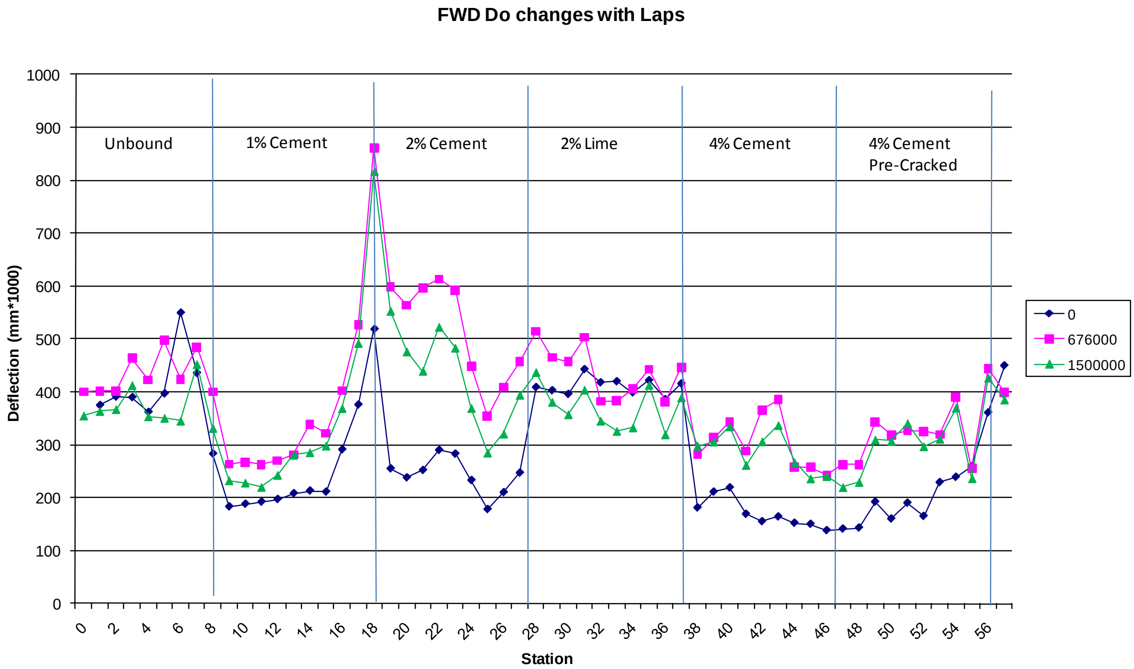
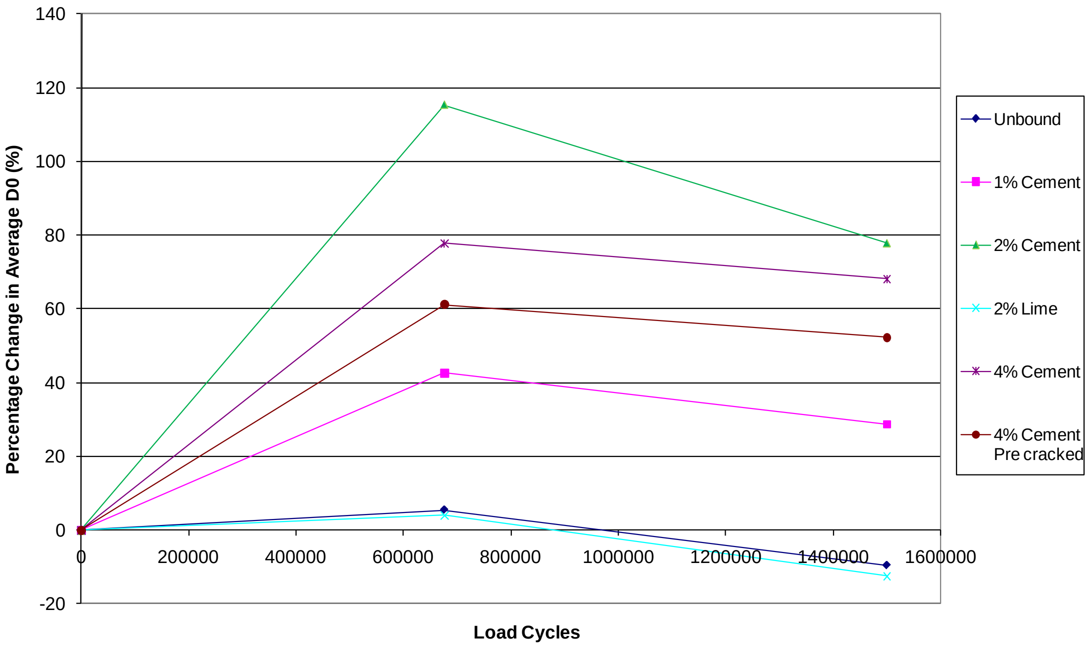
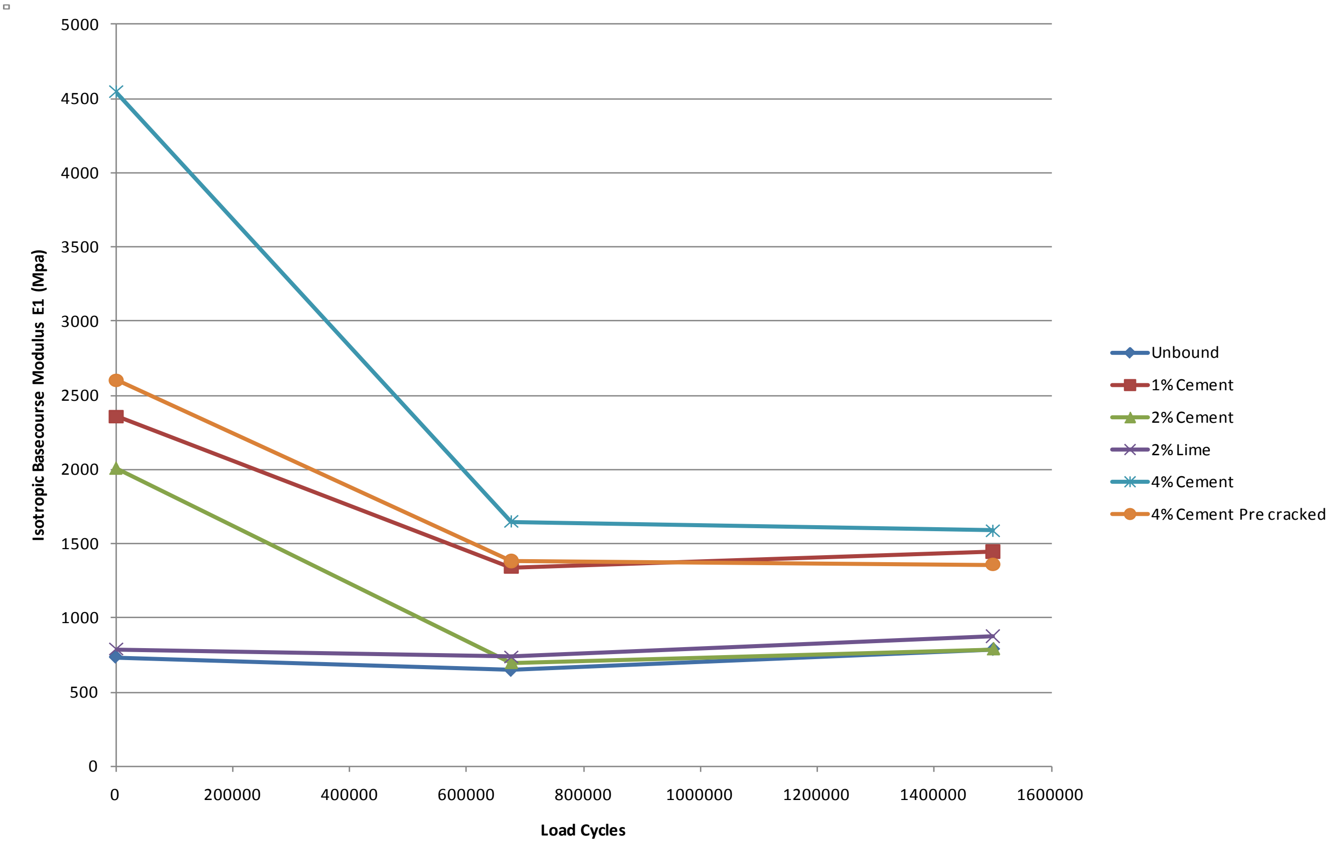


Figure 3.13 Percentage change in D0 results from initial tests



Back-analysis of the FWD readings with ELMOD showed significant decreases in the moduli of the basecourse layers stabilised with cement, confirming that the damage was in the basecourse layers (figure 3.14). The modulus of the basecourse layers reverted to something close to the 1% cement case and conformed to the Austroads concept of two phases of behaviour – an effective fatigue life phase where the initial modulus drops rapidly and an equivalent granular phase where the modulus remains relatively constant.

Figure 3.14 FWD ELMOD analysis



3.4.5 Transverse profiles

Regular transverse profiles were taken with CAPTIF's transverse profilometer to measure the functional performance of the pavement. Both VSD and rut depth were calculated from the data.

Figure 3.15 presents the average VSD with laps for each test section. Note that the poor performance of section C was related to a water pipe failure, which flooded the section. Figure 3.16 presents the 90th percentile (or worst 10%) VSD. Figures 3.17 and 3.18 present similar data for rutting.

Figure 3.15 Average VSD with laps

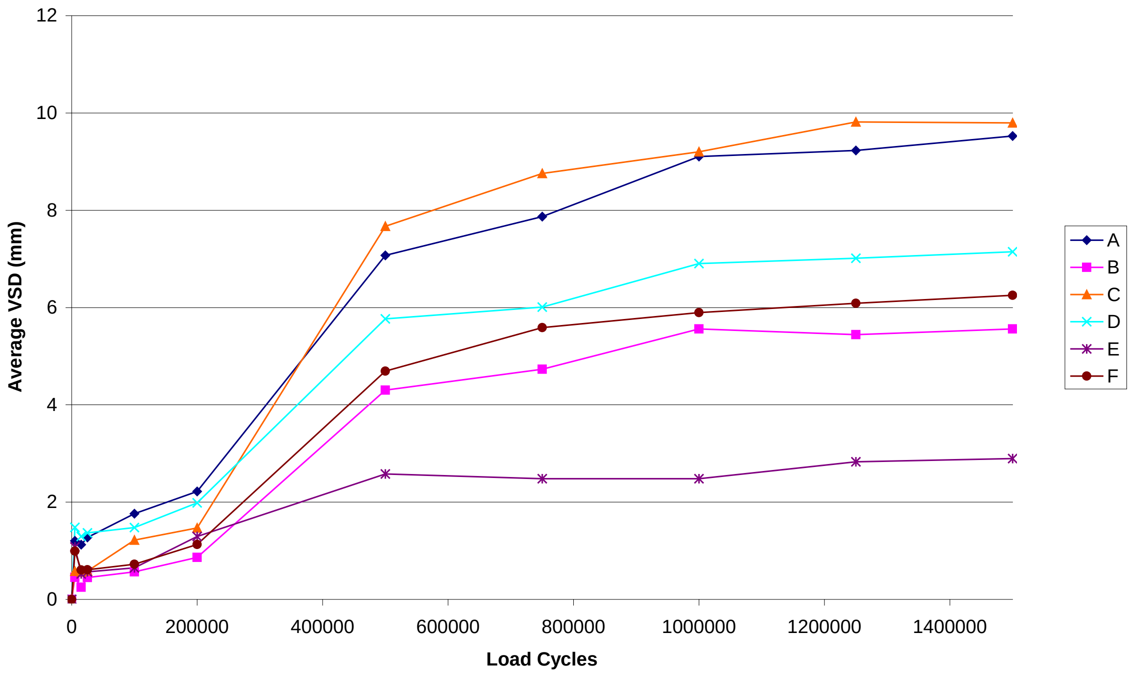


Figure 3.16 90th percentile VSD with laps

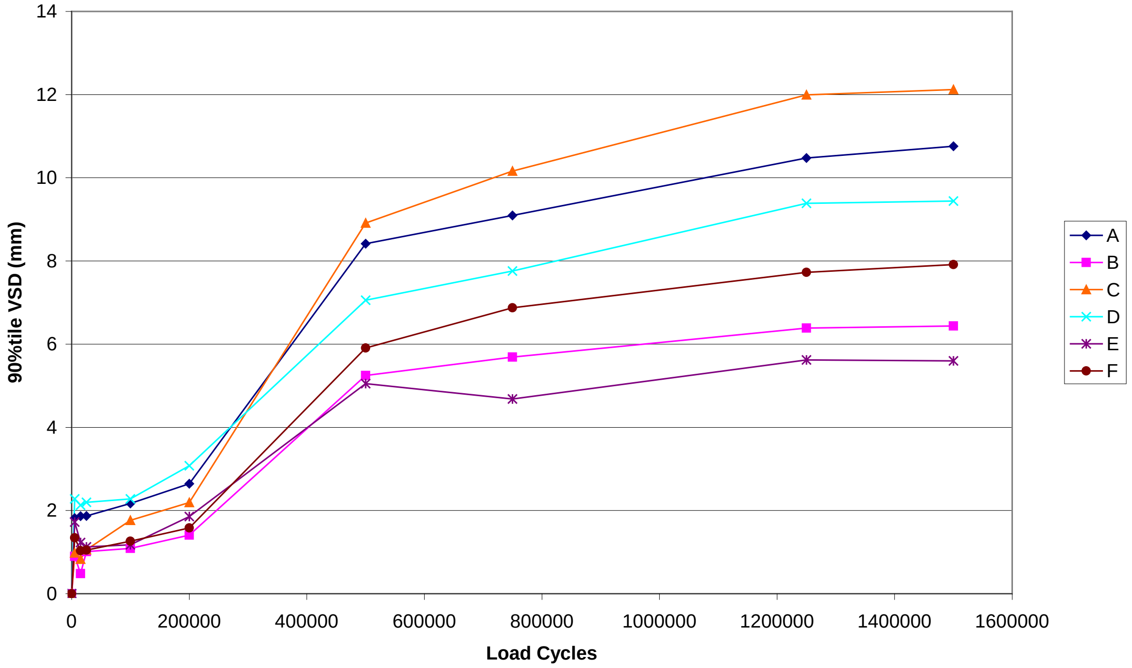


Figure 3.17 Average rutting with laps

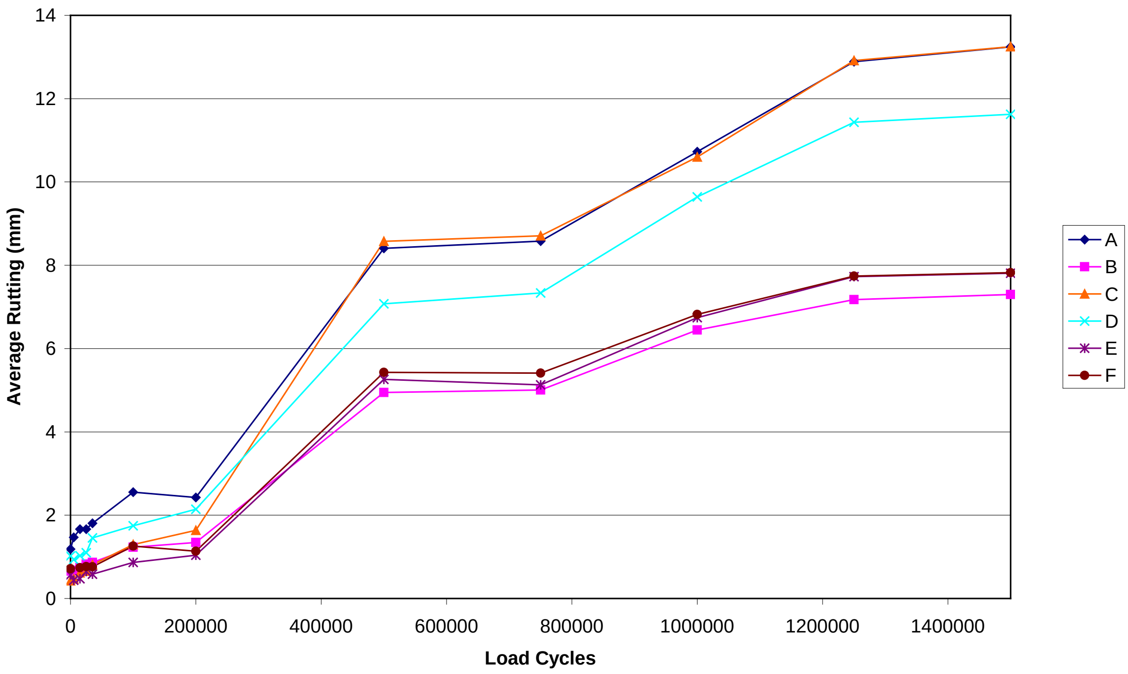
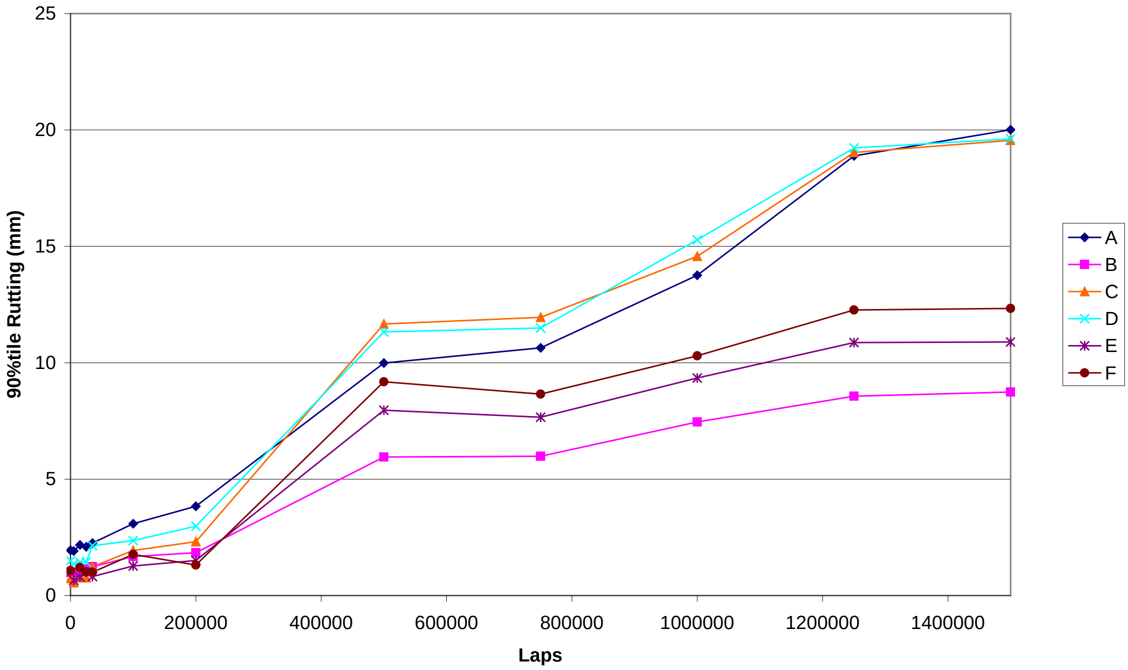


Figure 3.18 90th percentile rutting with laps

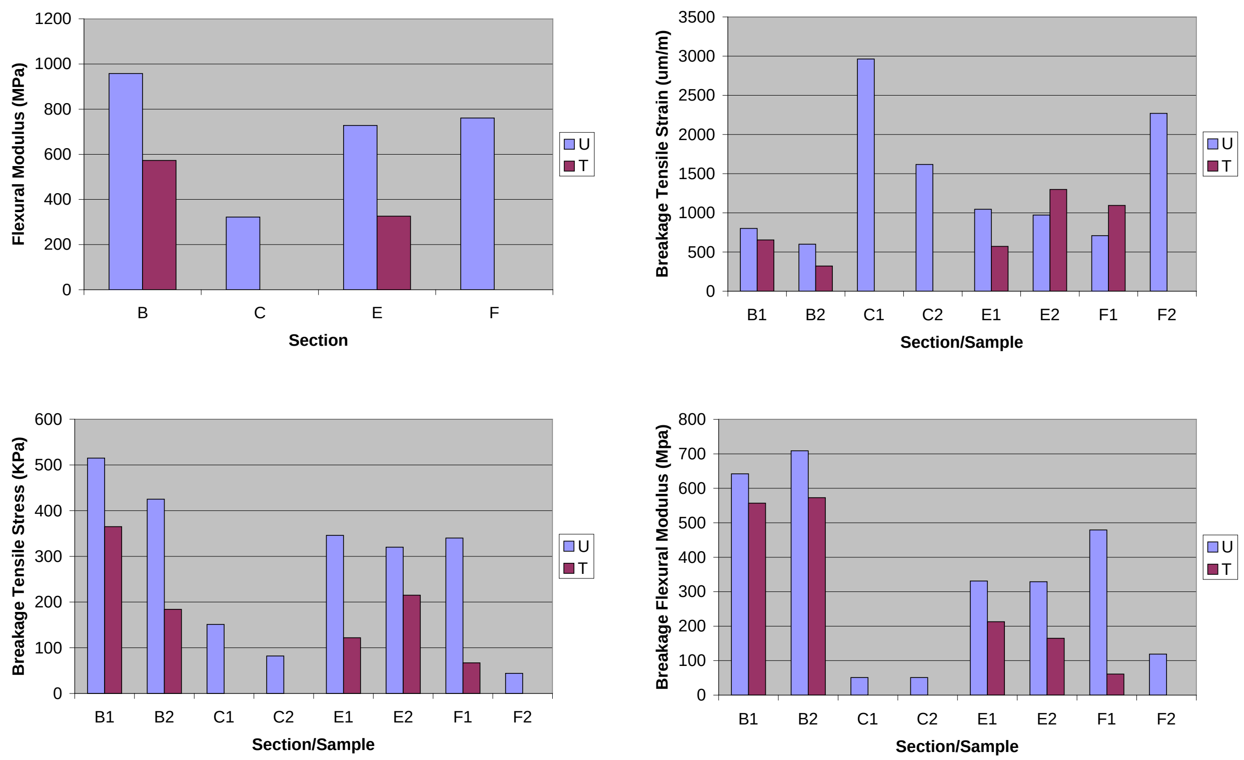


3.5 Post-mortem

Trenches were excavated as part of the post-mortem, with photos and layer profiles recorded. The photos in appendix C and layer profiles in appendix D reveal some interesting problems. In section E, half of the loading was on a patch that was laid as a result of a surface damage caused by a mechanical failure. The higher-than-expected rutting results appear to be related to this hand-laid patch not performing as well as expected. Removing the patched sections from the rutting analysis would leave the average rutting at the end of the test closer to 7mm for section E. A similar improvement would be made to the 90th percentile rutting values.

Beams were saw-cut from trafficked and untrafficked areas of each cement-modified test section and sent to Pavespec for flexural beam testing. The results in figure 3.19, while fairly inconsistent, generally show that the test sections had been damaged by the loading.

Figure 3.19 Post-mortem flexural beam tests – trafficked (T) and untrafficked (U) comparison



An attempt was also made to core the pavement – however this was not successful.

Scala penetrometer testing was repeated in the trenches and inspection of the results suggested that the subgrade had an average CBR that was closer to the 8 that was originally targeted, and similar to the original FWD testing – suggesting perhaps an error in the earlier testing, to produce quite a surprising result.

3.6 Analysis for pavement design

The data from the FWD testing was used in a CIRCLY analysis to predict the elastic strains in the pavements for comparison with the Emu data and make a prediction of performance for comparison with the rutting performance.

The CIRCLY analysis assumes that pavement reaches a terminal condition in terms of unacceptable deformation, which is generally considered to be where 10% of the pavement reaches a 20mm rut (the 90th percentile). The data from the transverse profiles was used to determine the end of life of each of the pavement sections. Figure 3.20 shows that the average VSD was related to average rutting, and figure 3.21 shows that the 90th percentile VSD and rutting were also related.

Figure 3.20 Average VSD v average rut

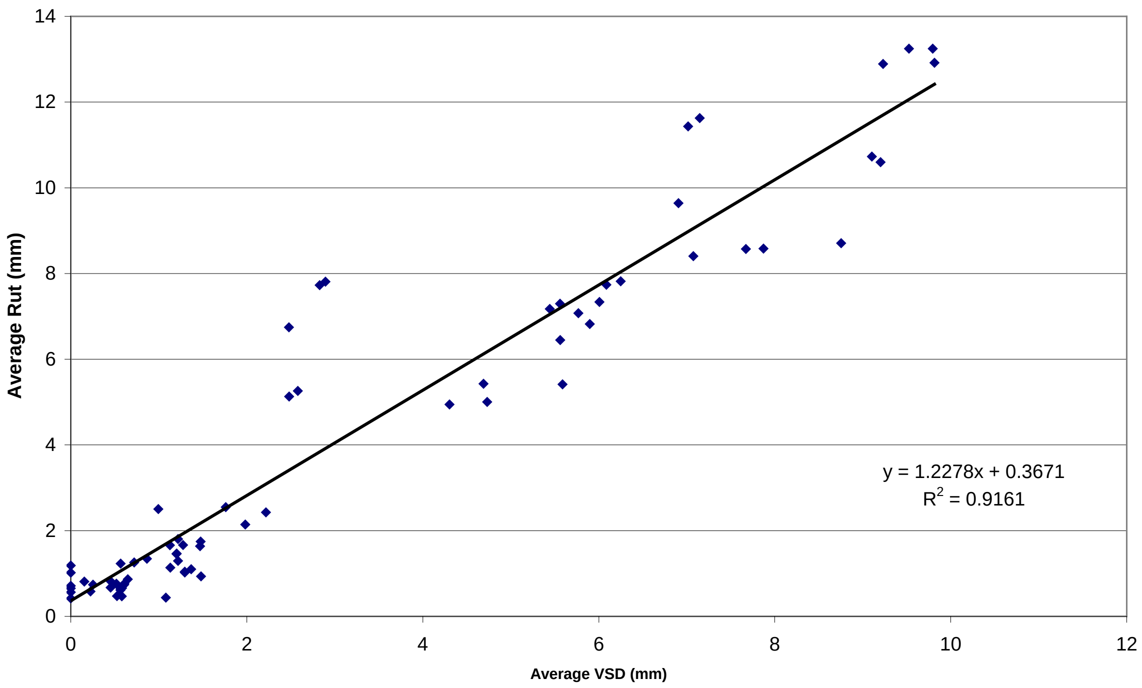


Figure 3.21 90th percentile VSD v 90th percentile rut

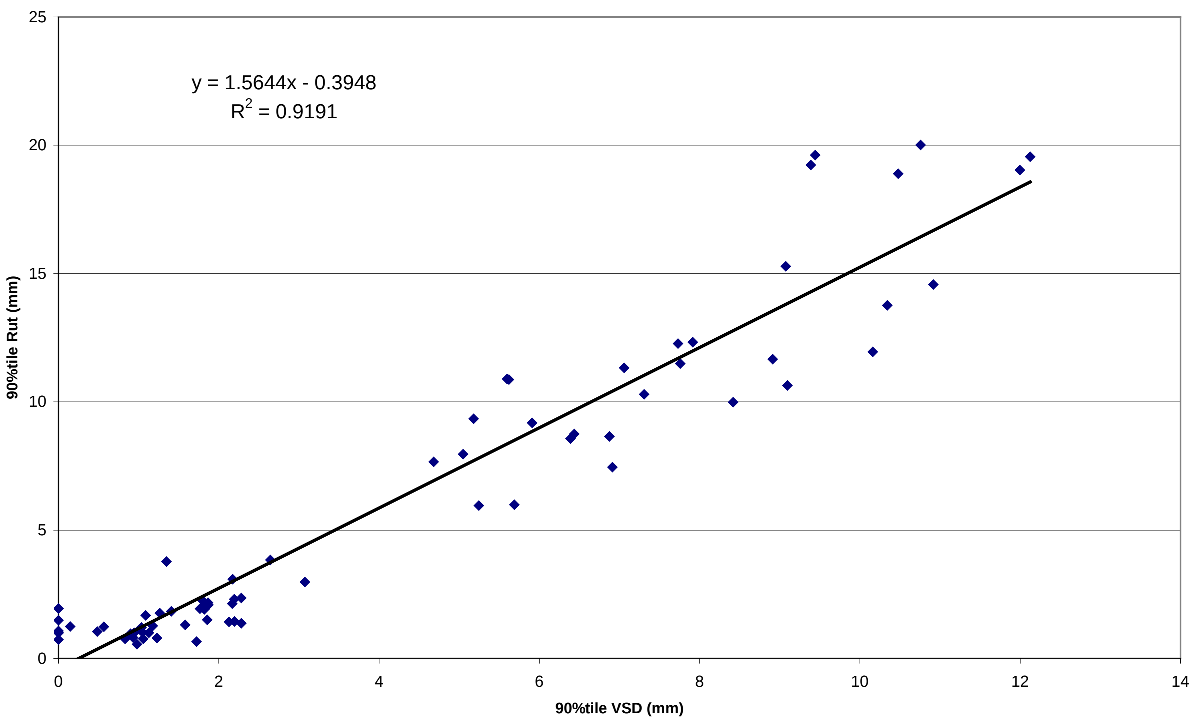


Figure 3.21 shows that a 90th percentile 20mm rut was reached when the 90th percentile VSD reached approximately 13mm.

Figure 3.22 90th percentile VSD v average VSD

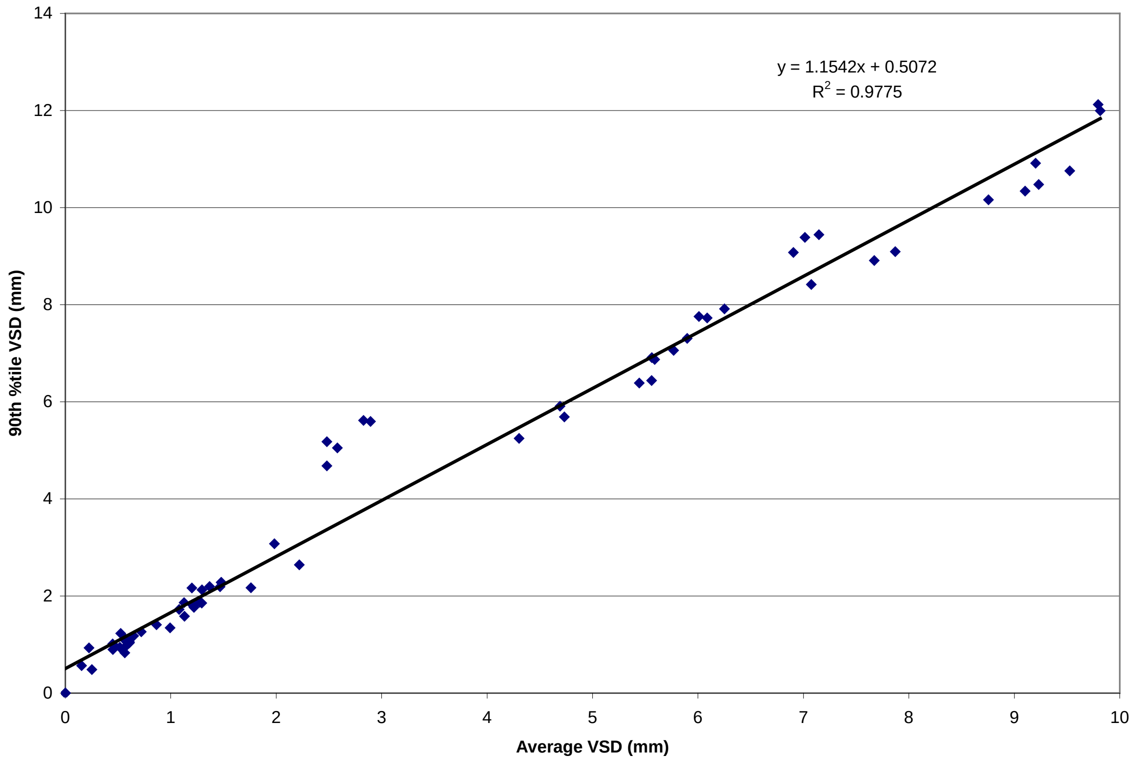
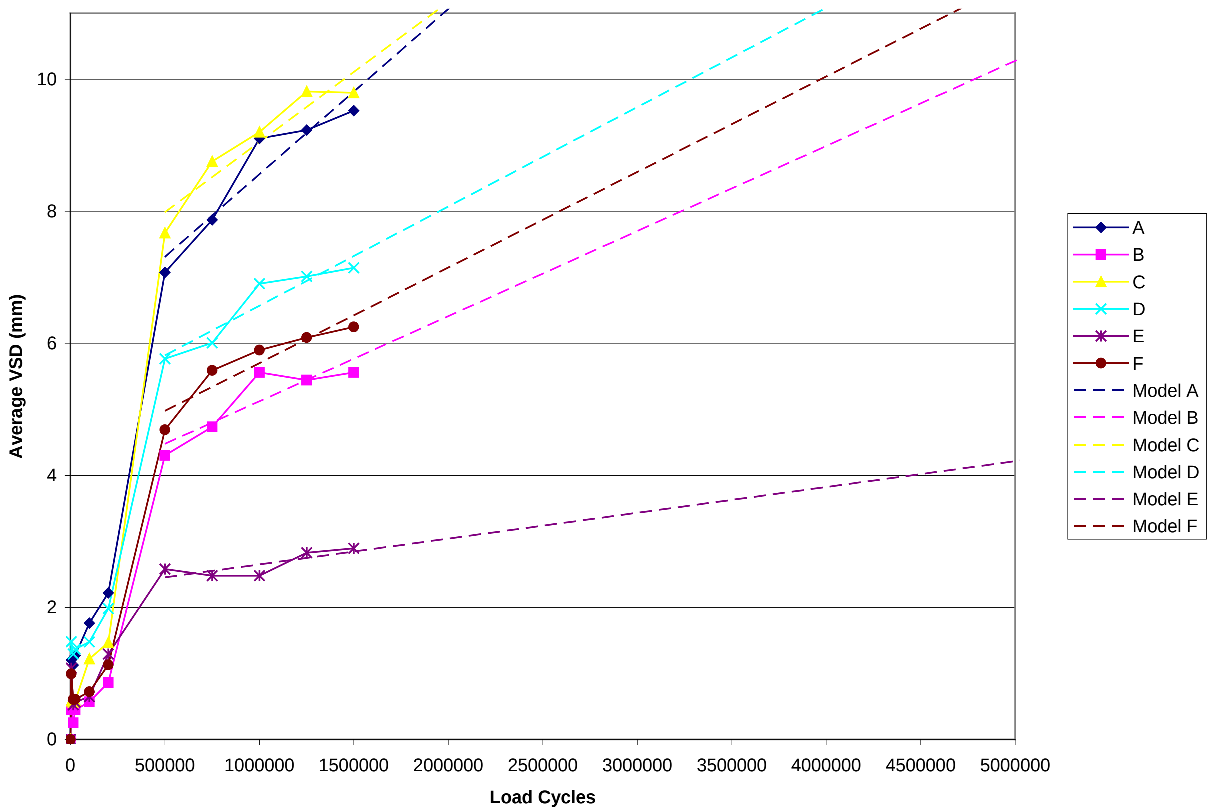


Figure 3.22 shows that a 90th percentile 13mm VSD was reached when the average VSD reached approximately 11mm.

A linear relationship between load cycles and VSD was assumed from 500,000 to 1,500,000 load cycles and a least squares regression fitted in Excel as shown in figure 3.23 following.

Finally, the terminal number of load cycles was determined by extending the VSD relationships to 11mm (20mm rut) as represented in figure 3.23.

Figure 3.23 Final design life – average VSD = 11mm



The absolute values of the extrapolations in figure 3.23 are debatable; however, the extrapolations are consistent and conservative. Alternative interpretations could be made, but it is the relative change in performance that is important and they would be consistent with these findings even if alternative extrapolations were made. Table 3.16 below provides the number of 60KN load repetitions to the terminal 20mm rut condition that were extrapolated from the average VSD values in figure 3.23. It also provides the percentage improvement in load-carrying capability from the unbound case.

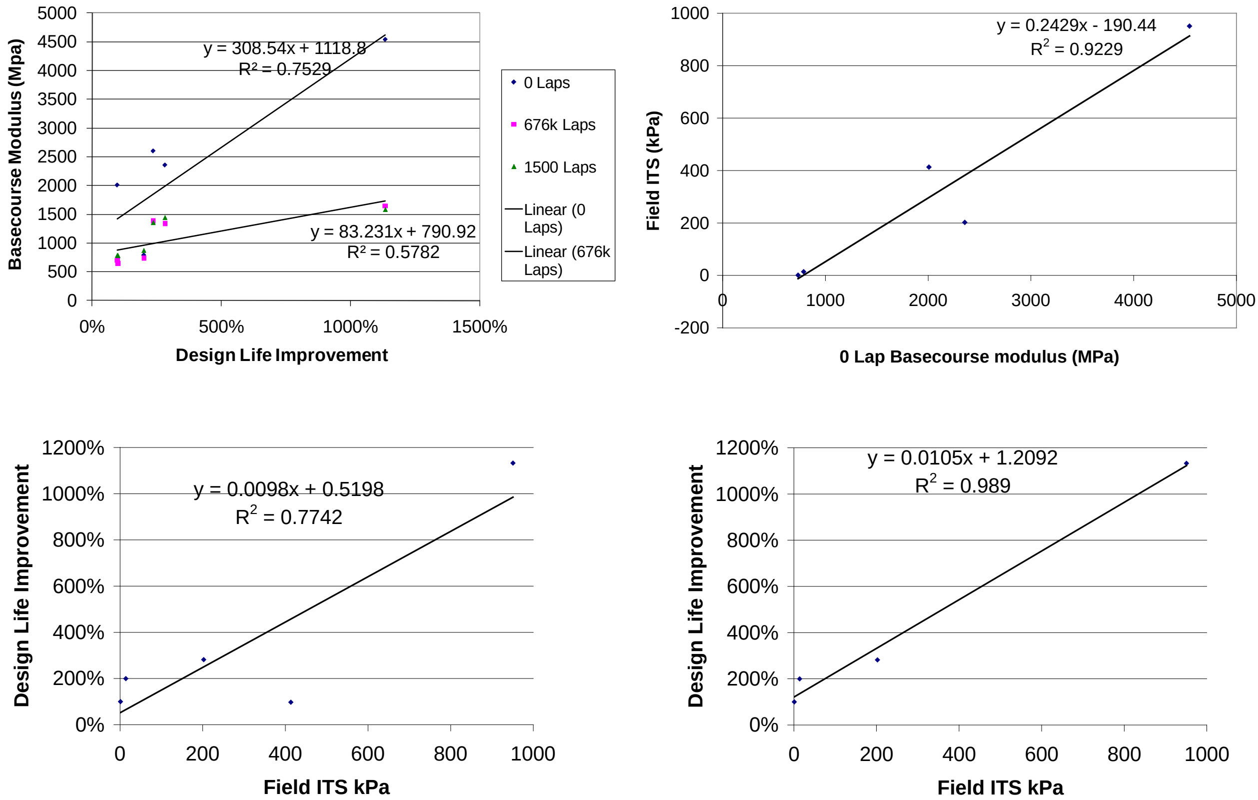
Table 3.16 Load repetitions to terminal condition extrapolated from CAPTIF measurements

Section	Material	60KN load cycles	Design life improvement (%)
A	Unbound	2.0E+06	100%
B	1% cement	5.6E+06	282%
C	2% cement	1.9E+06	97%
D	2% lime	3.9E+06	200%
E	4% cement	2.2E+07	1133%
F	4% precracked	4.7E+06	236%

3.6.1 Basic analysis

Simple review of the FWD basecourse modulus and improvement in life of the pavements in figure 3.24 shows that it should be possible to predict performance in design relatively easily. Figure 3.24 also shows that the basecourse moduli is related to ITS value and that the ITS value can be directly related to a design life improvement. This means that improvements should be predictable from mix design laboratory results. It's interesting that in the last graph of the figure, the 2% cement section has been removed as an outlier (as it failed, due to a water pipe burst) and the ITS makes an excellent prediction of the improvement on this limited data set.

Figure 3.24 Simple analysis of improvements v field and lab tests



3.6.2 Elastic strains

The calculation of elastic strains was both dependent on the method of back-analysing the FWD data in ELMOD and the method of sublayering the basecourse used in CIRCLY.

The ELMOD back-analysis from the final construction testing was used to determine an estimate of the stiffness of the basecourse layers, but no correction was made for moving from an isotropic to an anisotropic analysis, as this was a simple comparative study. The subgrade was assumed to be anisotropic, with a CBR of 11 and vertical elastic modulus of 110MPa, a horizontal modulus of 55MPa and a Poisson ratio of 0.45. The asphalt layer was assumed to be isotropic, with an elastic modulus of 3000MPa, a Poisson ratio of 0.4 (again, not strictly correct modelling to Austroads 2004). The basecourse layers were assumed to have a degree of anisotropy of 2 and a Poisson ratio of 0.35.

The remaining parameters are given in table 3.17. The results of table 3.17 suggest we used a very high modulus for the 1% cement section, and before it was used in the modelling a number of checks were applied. The construction ITS and UCS tests in tables 3.12 and 3.13 suggest that the binder content was likely to be correct and the density reported in table 3.11 was not higher than the other sections. Tables 3.9 and 3.10 don't suggest an improved subgrade either. The sensitivity of the analysis was tested by running 16 alternative models through Elmod, varying the layer thickness assumption and back-calculation procedures. The additional models resulted in very similar results – the model used simply had the lowest fitting errors.

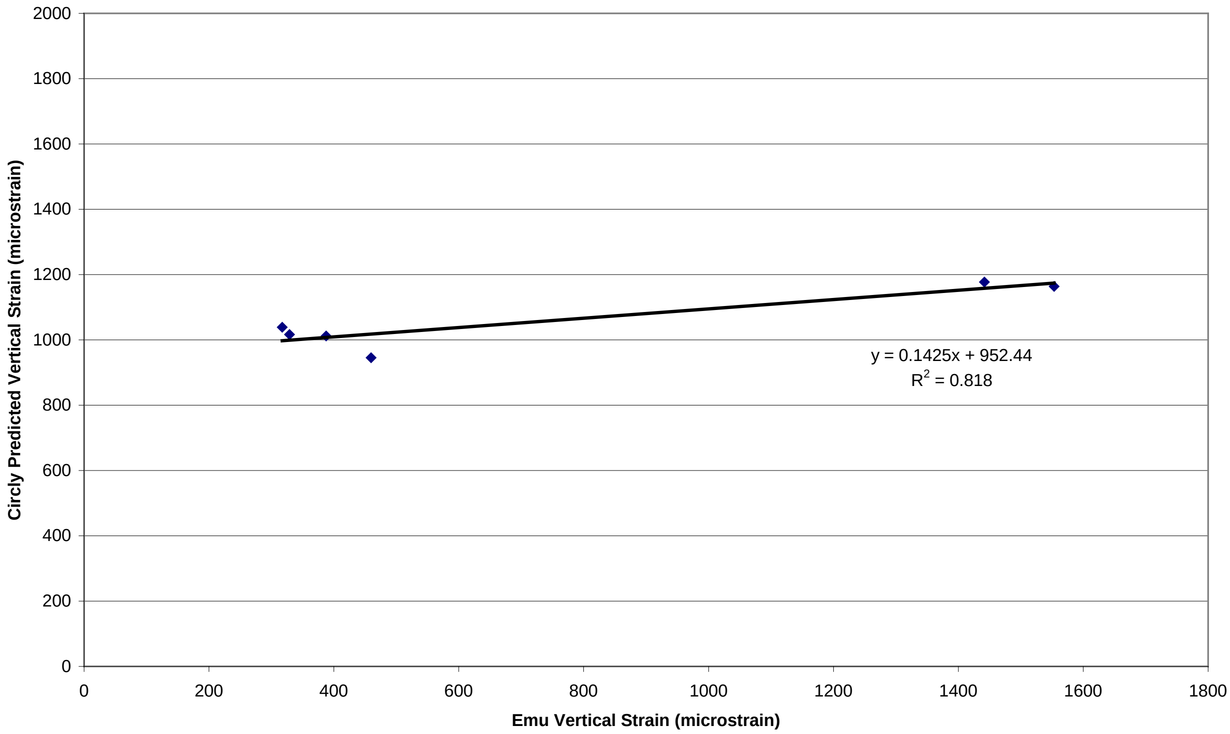
Table 3.17 CIRCLY model parameters

Material	Depths (mm)		Moduli (MPa) – 1992 sublayering		
	AC	BC	AC	BC	SG
Unbound	40	210	3000	750	110
1% cement	40	210	3000	2350	110
2% cement	40	210	3000	2000	110
2% lime	40	210	3000	800	110
4% cement	30	220	3000	4550	110
4% cement precracked	30	220	3000	2600	110

The current Austroads sublayering technique was found to be inadequate, as it severely limits the upper moduli for the basecourse layers. It was found that the 1992 procedures for sublayering provided a more realistic spread of results.

Figure 3.25 shows that the vertical elastic strains calculated by the analysis at the top of the subgrade still did not compare well with those measured with the Emu system (despite the apparently reasonable R-squared value).

Figure 3.25 Actual and predicted top of subgrade vertical strains



3.6.3 Performance

Interestingly, despite the poor elastic strain prediction, a reasonably good prediction of performance can be obtained from CIRCLY using the 1992 procedures for sublayering. Figure 3.26 suggests that the analysis was reasonably good even when including all the data.

Figure 3.26 Actual life at CAPTIF v life predicted by FWD and CIRCLY (subgrade strain)

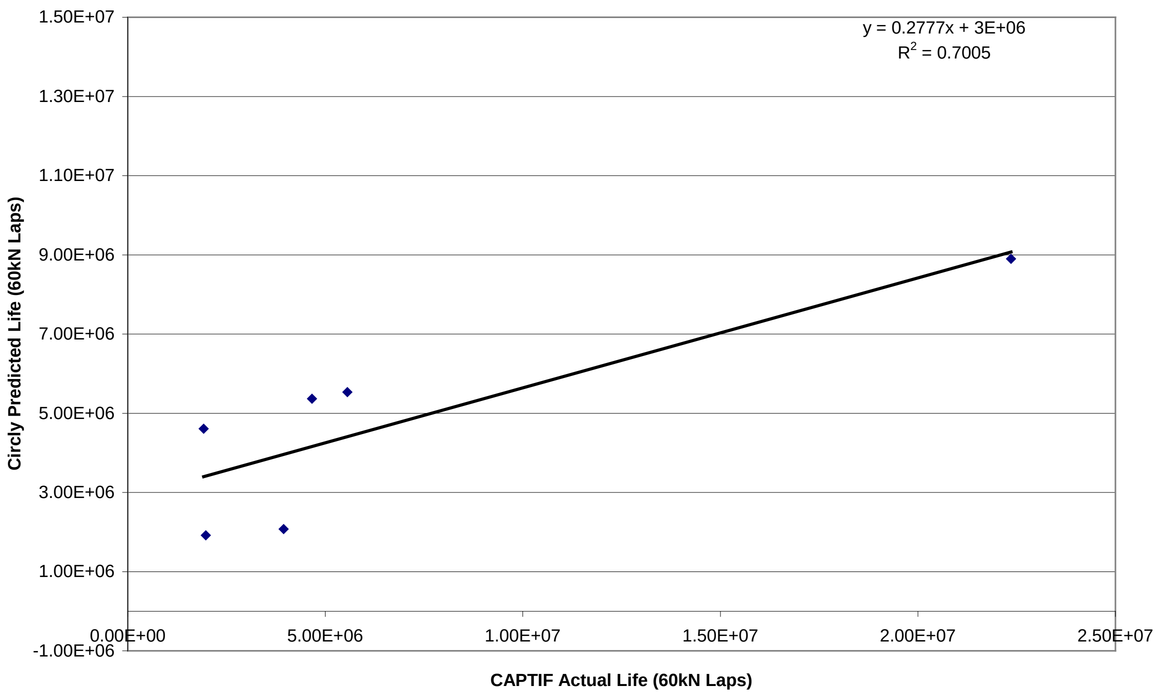


Figure 3.27 Actual life at CAPTIF v life predicted by FWD and CIRCLY (sections C and F removed)

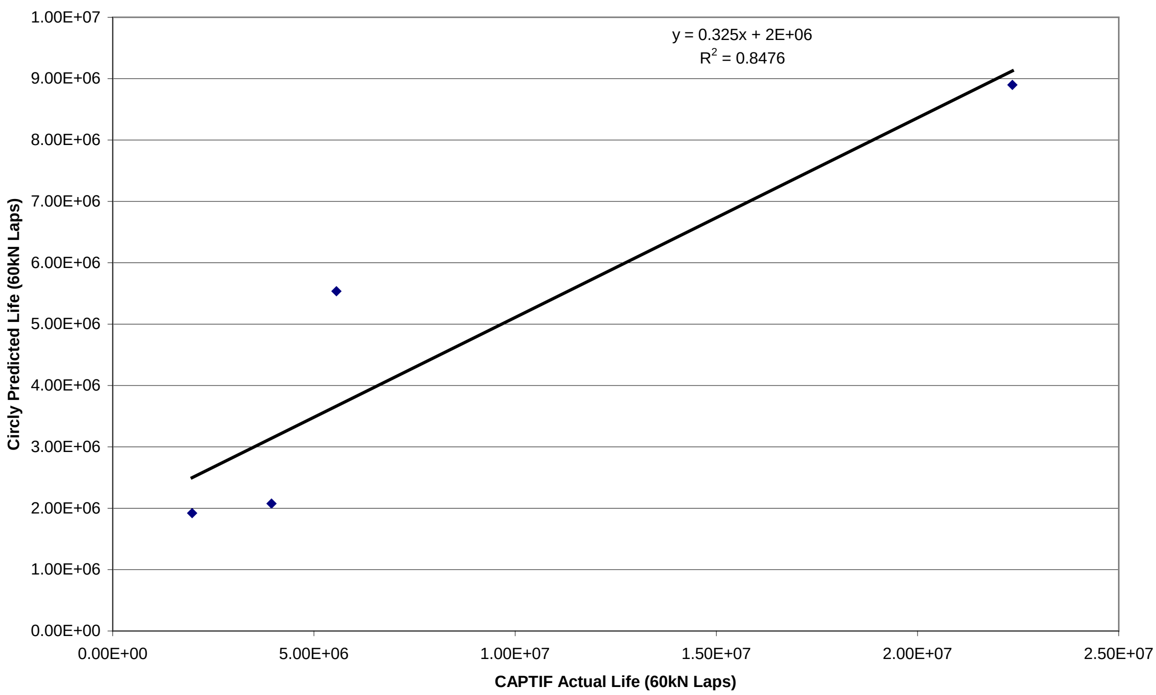


Figure 3.27 indicates that CIRCLY did a good job of predicting the performance when the precracked section, and section C (which was accidently flooded), were removed. This result combined with the fact that the ITS testing and initial modulus values show a strong correlation suggest that the performance of these test sections was more or less predictable.

3.7 Test 2 conclusions

The following conclusions can be drawn from test 2:

- The RLT test using the current stress levels in the draft T/15 specification was not able to distinguish between the materials with cement contents over 1.0%.
- The transition from modified to bound behaviour was difficult to determine; however it could be said that the 4% cement pavement was practically behaving in a bound way, as there was very little rutting observed. This provides an upper bound for laboratory-mixed UCS tests of 19MPa or an ITS of 1260KPa. This is considerably higher than the Austroads *Pavement design* (2004). A value of approximately half this would appear to be a sensible mix design limit to prevent bound behaviour, and equates to the 2% cement material compacted with the vibrating hammer.
- The testing found a strong correlation with the UCS, ITS and IT modulus results. There was a poor correlation between the IT modulus and the RLT modulus.
- During construction there was a good correlation between the laboratory-mixed and field-mixed UCS values. However the field results were approximately 80% of the laboratory values.
- During construction there was a good correlation between the laboratory-mixed and field-mixed ITS values. However the field results were approximately 70% of the laboratory values.
- Precracking the basecourse reduced the average basecourse modulus after construction by 40%.
- The Emu system showed that the cement-stabilised sections initially halved the vertical strain on the top of the subgrade. Discussion with industry suggested that the precracking would ‘heal’ and the modulus would improve. This ‘healing’ was not observed in the CAPTIF test.
- The Emu system showed that the total vertical strain measured in the cement-stabilised sections increased by 30–50% by 500,000 load cycles, and then remained constant. The non-cement sections saw an initial increase in strain and a reduction back to towards zero increase at the end of the test.
- The recordings of the variation in load (dynamic load figure 3.11) around the test track showed that the spatial variation in loading was relatively constant during the test.
- FWD testing during loading showed that the D0 increased significantly during testing in the cement-stabilised sections (supporting the Emu data). This suggested that the pavements tested were being damaged by the loading.
- Analysis of the FWD data suggests the increase in D0 in the cemented sections was due to a decrease in basecourse modulus.
- The unbound and lime sections did not increase in D0 during loading.
- The average VSD results show that the 4% cement section E was the best performer. The worst-performing sections were the unbound sections A and C, with 2% cement. However, section C was accidentally flooded when a water pipe burst and that was the cause of its poor performance.

- The precracking of section F did not fully heal. Once precracked, the 4% cement behaved in a similar manner to 1% cement uncracked.
- Post-mortem testing confirmed that the cement-stabilised test sections were, as expected, damaged by the loading.
- The basecourse modulus from initial FWD testing showed a good relationship with the load-carrying capability of the pavements.
- Field ITS testing testing showed a good relationship with improvements in the load-carrying capability of the pavements.
- Using the modulus values from the initial FWD testing and the Austroads 1992 sublaying procedures, vertical strains were calculated by CIRCLY and compared with the Emu system. A reasonable relationship was found; however the results were not directly comparable.
- Using the modulus values from the initial FWD testing and the Austroads 1992 sublaying procedures, a design loading was calculated with CIRCLY and compared with the CAPTIF testing. A reasonable relationship was found and the results were conservative.
- Without exception the stabilised sections performed better than the unbound control section.

4 Field study

4.1 Field study objectives

The field study considered the field performance of cement-stabilised pavements, particularly looking at the suitability of the Austroads pavement design criteria for bound pavements in New Zealand. Six sites were selected for detailed analysis. They had from 3–6% cement mixed and were compacted at approximately the OWC. These selected pavement sections were tested and then modelled to obtain the expected life of the pavements, which was compared with the life currently obtained.

In this study, two software packages were used for pavement modelling purposes:

- CIRCLY, based on Austroads (2004)
- PADS, based on South African design criteria (Theyse and Muthen 2001).

The key input parameters are the design traffic, the layer thicknesses, Poisson ratios (usually assumed to be 0.2 for cement-stabilised layers and 0.35 for asphalt and granular materials) and the resilient moduli of each layer. Another input parameter called the traffic multiplier is needed for CIRCLY and this depends on New Zealand's mix of heavy vehicles. The New Zealand Supplement to Austroads pavement design guide (Transit New Zealand 2005) provides information on New Zealand traffic conditions.

In this study the resilient modulus for stabilised pavement was obtained using two different methods:

- laboratory testing on cores taken from selected pavement sections
- presumptive values from the design guide.

While FWD readings were taken on the sites, they were not used in the expected life modelling, as the CAPTIF testing showed cemented layers are likely to lose stiffness over time. It was too difficult to use the FWD tests, as they ranged from possibly complete stiffness loss to what appeared to be relatively intact stiffness, and correcting the data to an initial stiffness for design would have invalidated any resulting expected design life. The FWD results did, however, provide an excellent indicator of the stiffness loss at the sites.

For each test pavement the current traffic data was obtained and an estimate of the total traffic to date was made using appropriate traffic growth factors and the pavement age.

The results from the current Austroads pavement design method were compared with South African pavement design results and the present condition of those selected pavement sections.

4.2 Sensitivity analysis for bound design

4.2.1 Model used in the sensitivity analysis

A pavement design sensitivity analysis was conducted using a design model with a chipseal-surfaced cemented layer overlaying a subgrade. The chipseal was assumed to have negligible effect on the mechanical performance of the pavement and was therefore ignored in the calculations. The design variables, including resilient modulus, layer thickness and the fatigue performance criteria, can influence the design thickness of the cemented layer. This preliminary analysis shows the effect on service life of pavement in terms of the variables such as design thickness, resilient modulus, design number of equivalent standard axles (ESAs), etc.

The sensitivity of the cemented-layer thickness to the cemented material's modulus was examined for a number of pavements. Two different subgrades (with modulus 50 or 80MPa) and five cemented materials with different moduli (of 2000, 3500, 5000, 7500 or 10,000MPa) were assumed and the cemented-material thickness calculated for the various combinations.

Table 4.1 indicates the effect on the thickness of changing the modulus of the cemented material. This table was developed assuming a design life of 10^7ESAs. The following data was used for the thickness calculation of each combination:

- It was assumed that the cemented material was isotropic, while the subgrade was anisotropic.
- The modulus of the cemented material was assumed to be constant throughout the depth and the Poisson ratio was 0.2.
- The life of the cemented material was determined by the maximum horizontal strain at bottom of layer.
- For cemented material, the damage exponent was 12 and the fatigue constant was calculated using the formula: $K = 191 + 11300/E^{0.804}$ (Austroads 2004).
- A full axle as defined by Austroads (ibid) was assumed.

Traffic multiplier for cemented materials, as per the Transit NZ Supplement (2005), was 3.6, while for subgrade materials it was 1.2.

Table 4.1 Variation in design thickness of cemented materials for a design life of 10^7ESA (>3% cement assumed as bound material) as a function of modulus.

Subgrade modulus (MPa)	Cemented-material layer	
	Modulus (MPa)	Thickness (mm)
50	2000	420
50	3500	373
50	5000	338
50	7500	295
50	10,000	280
80	2000	385
80	3500	344
80	5000	313
80	7500	273
80	10,000	260

The resulting calculated thicknesses ranged from 420–260mm.

Figure 4.1 Variation in thickness (mm) relative to the modulus (MPa) of five cemented materials (precracking phases)

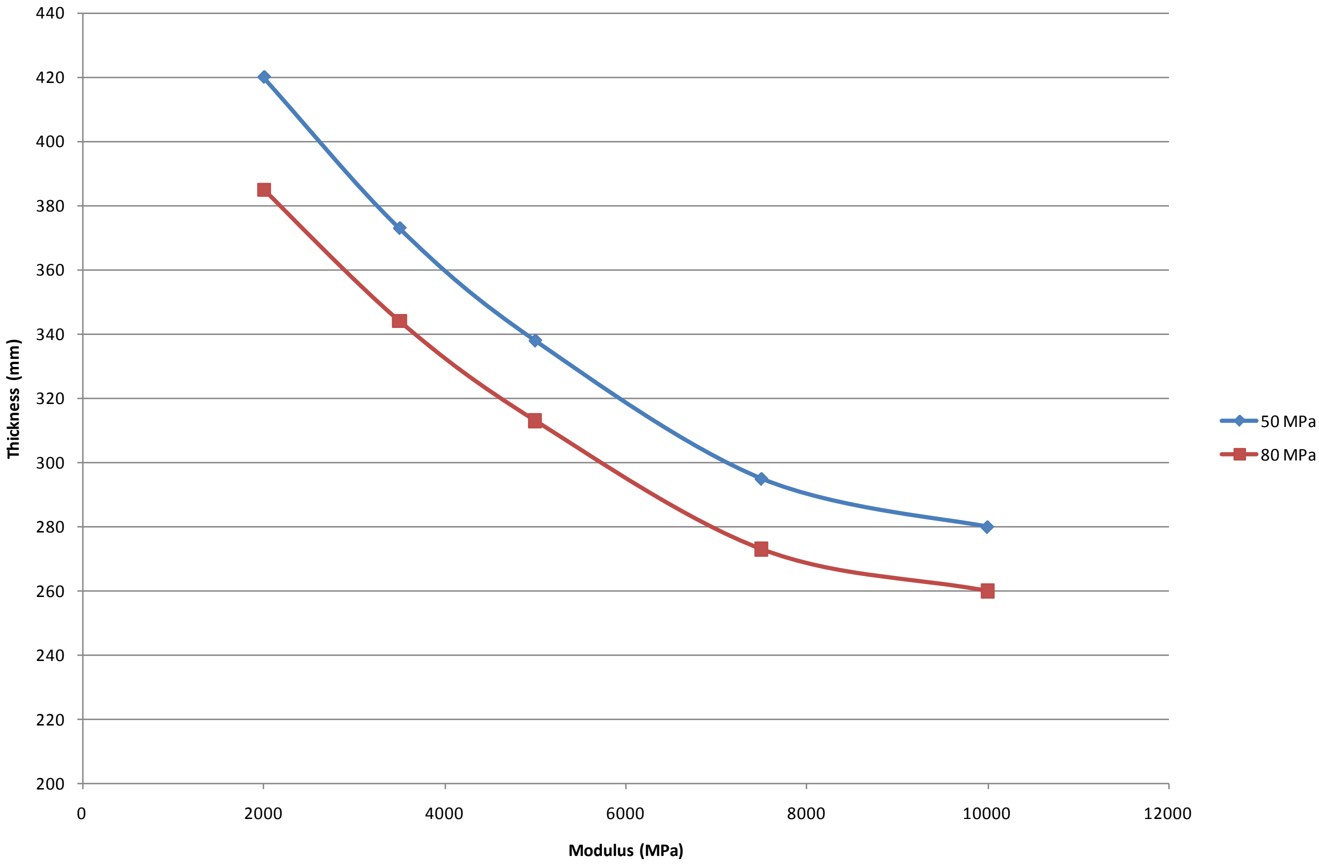


Figure 4.1 indicates that the design thicknesses changed significantly with the cemented-layer modulus and were also sensitive to the subgrade modulus.

4.2.2 Effect of traffic multiplier

The damage caused by heavy vehicles in each material was different. Austroads (2004) provides damage types for the fatigue of asphalt, the fatigue of bound cemented materials, and subgrade rutting. Each of the damage types has different number of standard axle repetitions (SAR) and it is commonly referred to as equivalent standard axles (ESA) for the deformation damage of unbound pavements. Standard axle repetitions for estimating damage to cement-bound materials are designated as SARc. The ratio between the SAR and the ESA is called the 'damage index parameter' or 'traffic multiplier' for each type of damage. The presumptive damage index parameters given in table 7.8 in Transit New Zealand (2005) were determined from New Zealand weigh-in-motion (WIM) data. These values were used for modelling the pavements in this study.

For cement-bound materials the damage mechanism is fatigue. The fatigue damage exponent and the axle load distribution were used for calculating SAR_C. The analysis results help us to understand the effect of cemented-layer thickness related to the variation on SAR_C or traffic multiplier.

Presumptive damage index parameters for New Zealand traffic-loading conditions (for cemented-layer fatigue $SAR_C/ESA = 3.6$) are given in Transit NZ (2005). The damage index (SAR_C/ESA) is used to estimate design traffic loading in SAR_C for cemented materials. The Austroads design guide (2004) gives SAR/HVAG factors, which are the product of SAR_C/ESA × ESA/HVAG. The damage index (SAR_C/HVAG) is 1.5 for a moderately loaded mix of heavy vehicles, and 28 for a relatively heavily loaded mix of heavy vehicles (log trucks, bulk tankers, etc.) (Transit NZ 2002).

The Austroads (2004) damage index (SAR_C/ESA) is 12 for rural and urban, and the damage exponent (m) is also 12. The Australian factor is significantly different from that for New Zealand roads, which is understandable considering the traffic composition for the two countries.

The assumed traffic growth is zero for the following calculation. The cemented-stabilised layer thickness of 344mm is designed for the modulus of 3500MPa, the subgrade modulus is 80MPa, and the ESA is 10^7. Table 4.2 shows the service life and cumulative damage factor (CDF) for four traffic multipliers.

Table 4.2 The effect of different traffic multipliers on the design life of cemented layers

Traffic multiplier	Thickness (mm)	CDF	Design life (years)
3	344	0.818	30.0
3.6	344	0.982	25.0
10	344	2.73	9.0
25	344	6.82	3.6

It can be seen that changes in the traffic multiplier produces sensible differences in design life. Doubling the traffic multiplier halves the life.

4.3 RAMM data analysis

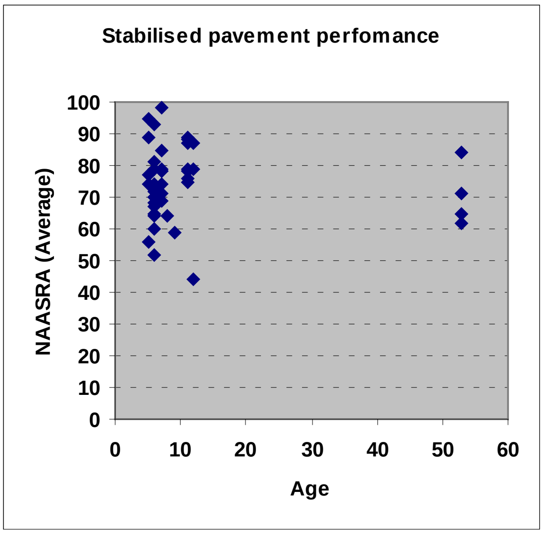
A search of the highway RAMM database was performed to locate cement-stabilised pavement sections. There were 357 cement-stabilised sections of state highway recorded in the RAMM data and those with the following conditions were selected:

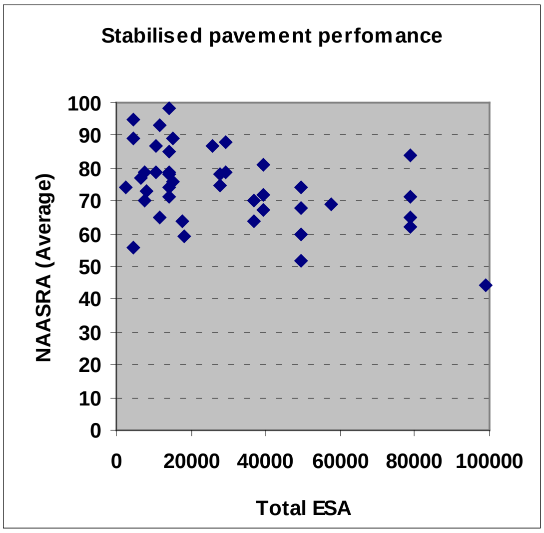
- The ages of the stabilised sections were greater or equal to five years.
- The proportion of cement used for stabilisation was greater than 2%.
- The section had a thickness greater than 100mm.
- The section was at least 200m long.

There were 39 sections that met the above criteria. The surface data obtained from this 2004 RAMM data was analysed and is shown in figure 4.2.

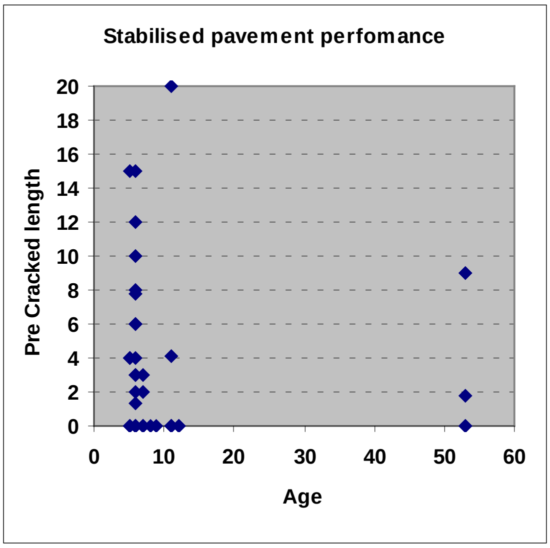
Figure 4.2 Overall performance of stabilised pavement sections chosen for analysis as measured by:

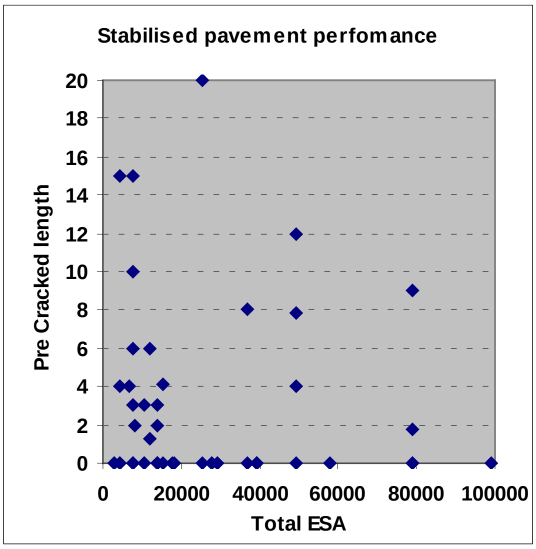
(a) Roughness (NAASRA)



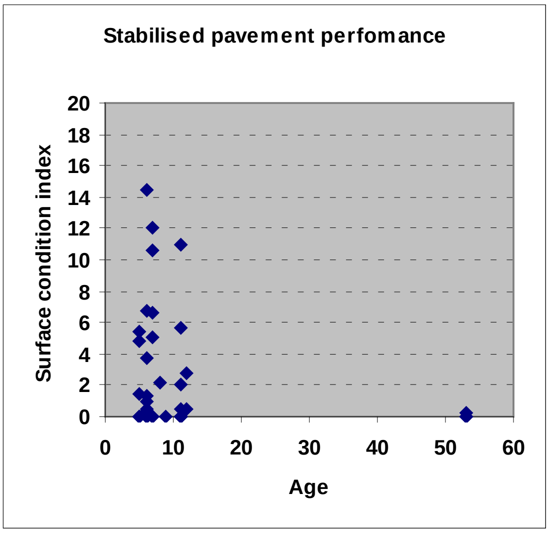


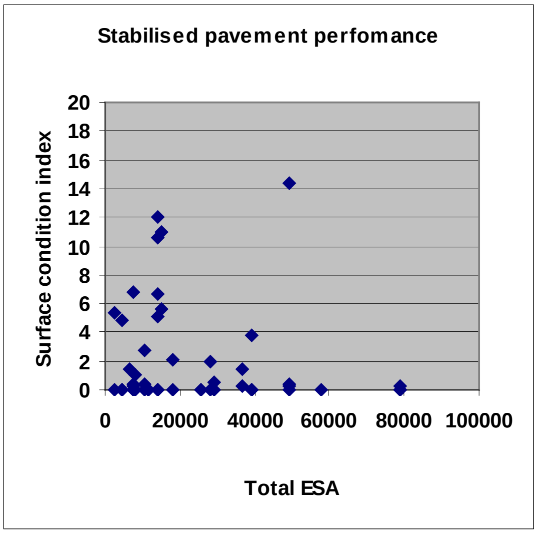
(b) Pre-cracked length (cracked length from RAMM) (mm)



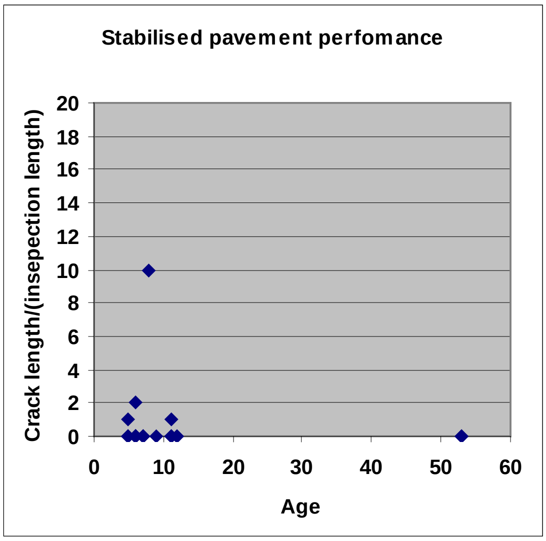


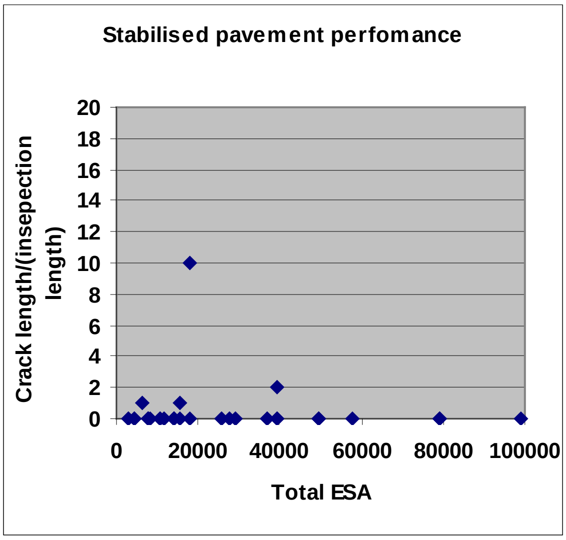
(c) Surface condition index





(d) Crack length/inspection length, for age (years) and for trafficking (total ESA)





The surface condition index (SCI) reported in RAMM is defined as follows:

SCI =Min (100, [Min (100, (4 * ACA + 0.5 * ARV + 80 * APT + 20 * APH + 1.2 * AFL))] + [3 * Min (100, Max (0, ((AGE2 – SLIF) / SLIF * 12)))]) (Equation 4.1)

Where:

SCI = surface condition index

ACA = area of cracking (derived from alligator in RAMM) in %

ARV = area of ravelling (derived from scabbing in RAMM) in %

APT = area of potholes in %

APH = area of pothole patches in %

AFL = area of flushing in %

AGE2 = surface age in years

SLIF = expected surface (design) life in years).

The graphs do not show any consistent relationship between the pavement performance and age or traffic volume. However, they were plotted for purposes of a preliminary investigation in order to identify pavements that should be investigated further. Some of the thicknesses of the stabilised layers shown in the RAMM data were zero, and a few sections were identified as recycled even though the data indicated a stabilised layer. The RAMM data was found to not be very reliable, especially for pavements constructed before 1990.

Therefore six new sections were selected through local knowledge, both on state highways and local authority roads.

4.4 Sites selected for detailed investigation

The following sites were selected for a detailed investigation:

- **SH2 Napier 678/12:** The stabilised layer was constructed in 1993. The thickness of the cement-stabilised layer was 125mm and the proportion of cement used for stabilisation was 5%. This information was obtained from the Opus Napier office. Shrinkage cracks occurred at approximately equal spacings. The surface condition can be seen in the photos in appendix E (figures E.1–E.4).
- **SH50 Napier 49/2.3:** The stabilised layer was constructed in 1993. The thickness of the cement-stabilised layer was 125mm and the proportion of cement was 5%. This information was obtained from the Opus Napier office. Shrinkage cracks occurred at approximately equal spacing. The surface condition is shown in the photos in appendix E (figures E5–E.10).
- **SH1 Hamilton 554/1.71 to 2.99 (Cobham Drive):** This was a 4-lane section. A core was taken from this section of the road after its construction in 1966 and the sample shown in appendix E (figure E.47) is in the Opus Hamilton laboratory (although its exact origin is not known). The mixture proportion was noted as 78% greywacke basecourse; 3% cement and 17% pit sand by weight; wet density 140lb/cu.ft (2243kg m^{-3}) compacted; strength 700psi (4.8MPa) at 7 days; 5in (127mm) cement-stabilised layer; and 1.5in (38mm) asphalt concrete (AC). This section had been overlaid with various AC layers over the years, and the total thickness of the asphalt concrete and the chipseal layer at the time of research was 115mm. The surface condition is shown in appendix E (figures E.32–E.34). There were several cracks that had been bandaged and several cracks were opening up.
- **SH1 Hamilton 574/8.98 to 9.21 (Whitehall):** The construction details of the site are available in RRU BC/35 (Irvine 1982a). Three test strips were constructed in 1968: sections 1 and 2 were in the left lane and section 3 was in the right lane; all sections were 61m long.

  The former Route Position (RP) system had been changed, and the old RP noted in Irvine (1982b) was 486/8.46 to 8.69.

  - section 1 (RP 486/8.460): 200mm cemented greywacke, 25–30% sand and approximately 70% TNZ M/4 basecourse
  - section 2: 250mm greywacke basecourse, same grading as for section 1
  - section 3: 250mm greywacke basecourse, compliant with TNZ M/4.

It seemed that sections 2 and 3 had been reconstructed, but the cement-treated section 1 (8.98 to 9.04) appeared to be untouched.

- **SH38 Rotorua RP 0/1.01 to 11.20 (Rerewhakaaitu):** The construction details of the site are available in RRU BC/33 (Irvine 1982b). Sections of the test strip had been reconstructed and in part widened. This was done around 1994 due to the cobble cracking of the pavement, and consisted of shoulder widening and the application of a 150mm overlay. From discussion with the consultant for the reconstruction work (Denis Whimp of Sigma Consultants, Rotorua), the reconstruction was done from RP 1.69 to 3.3, and RP 1.01 to 1.41 were not touched. Section A (RP 0/1.006–1.408) was 150mm of greywacke basecourse. We were not interested in this disturbed section. However, the details were not available in RAMM database or any other source.

  A core could not be obtained from section A and only one core was taken out of three at RP 0/1.41km (part of section B). The indirect tensile (IT) test result also indicated very low strength from this one core, compared with the core obtained from section D. It was assumed that the proportion of cement was 3% in the section between RP 0/1.09 and 0/1.4km.

- **Mangatu Road 12km, Gisborne:** The information was collected from a verbal discussion with people who observed the construction but were not involved in the design or construction. The exact records were unavailable. The stabilised layer was constructed in 1995 using a local low-strength aggregate with 3% cement. The road was mainly used to transport logs.

  There were two sections in Mangatu Road. In the first section the mix was compacted just after mixing with water, but in the second section the compaction process was delayed around four hours. The purpose of the process was to reduce the strength of the stabilised layer and to form fine or micro cracks.

  The fine cracks at 10.16km can be seen in the photo in appendix E (figure E.25). The first section had fewer potholes, shoving and fatigue cracks compared with the second section, as seen in the photos in appendix E (figures E.11–E31). No noticeable shrinkage cracks or edge failures had occurred in the two sections, but several earth slips had occurred.

4.5 FWD testing

The sites were tested with a FWD. A huge variation was noted in the modulus estimated from the back-analysis at each site. The modulus before cracking process is necessary for pavement modelling; thus these results were only really of use to determine how the site had performed.

FWD testing was carried out according to the following plans (figures 4.3 and 4.4).

Figure 4.3 FWD test strip for SH1 Whitehall RP 5748.985–9.035

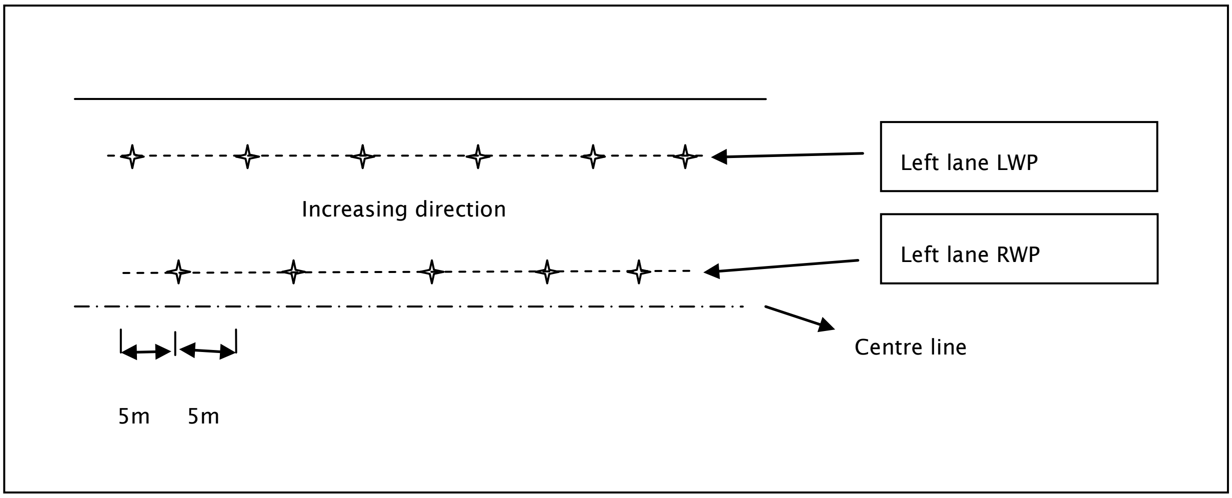
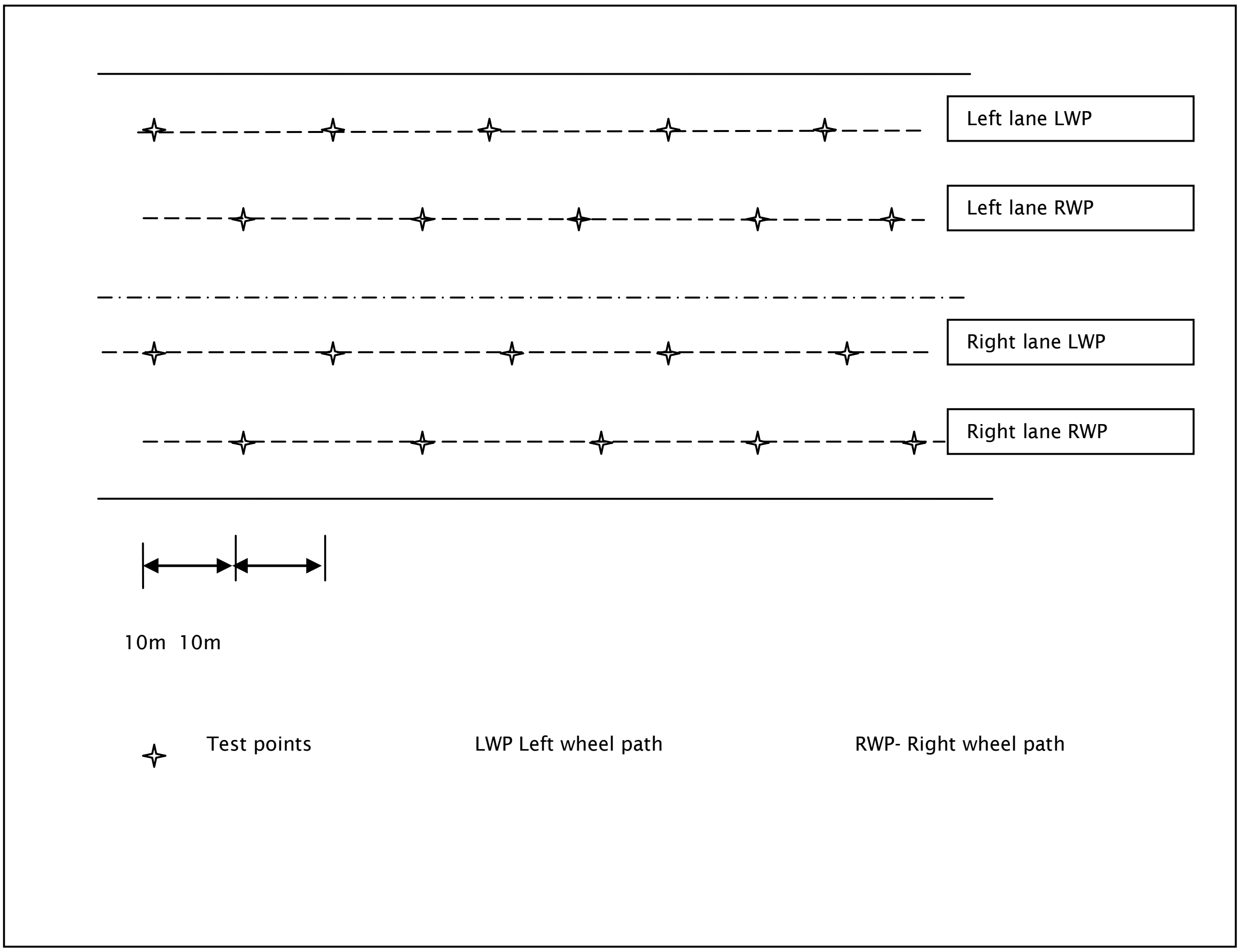


Figure 4.4 FWD test strip for each location, except the SH1 RP 574 Whitehall



The FWD results in table 4.3 indicated that relatively low cement (3%) contents can generate bound layers with high moduli. The 50th percentile values for Mangatu Road were quite high in places. However, it can be said that generally, with the exception of the Napier sites, the 90th percentile values were all at what would be considered unbound values.

Table 4.3 FWD results

Site	CSL[a]		FWD						
	Thickness (mm)	Cement (%)	Chainage	50th percentile modulus			90th percentile modulus		
				Layer 1	Layer 2	Layer 3	Layer 1	Layer 2	Layer 3
SH1 Hamilton									
554/1.71-2.99	170	3	0.180-0.281	27,100	594	337	3818	396	225
			0.901-1.051	5191	1069		1163	376	
SH38 Rerewhakaaitu									
0/1.050-1.150	310	6	1.05-1.15	610	216		388	120	
0/1.450-1.550	300	3	1.45-1.55	536	135		407	74	
			1.85-1.951	1000	732		521	228	
			2.260-2.361	922	564		643	356	
			2.65-2.751	1259	947		618	361	
Mangatu Road									
4250-4270	130 & 330	6	4.25-4.351	7400	452		683	153	
8960-8980	145 & 340	3	8.960-9.061	4332	176		432	23	
10,070-10,090	170	3	10.070-10.171	3299	567		327	122	
10,940-10,960	140	3	10.940-11.041	1492	425		604	167	
SH2 Napier									
678/12.17-12.19	150	5	12.1-12.201	8397	2220		6389	99	
SH50 Napier									
49/2.38-2.4	125	5	2.313-2.403	69,783			18,909		
SH1 Whitehall									
574/8.985-9.035			8.985-9.035	448			353		

a) Cement-stabilised layer.

4.6 Test pits and cores

The pavement layer thicknesses were measured from several test pits, which were selected from the FWD data. The site locations are detailed in table 4.4.

Table 4.4 Test locations

Site no.	Location	Lane	Wheel path
1	SH 2 RP 678/12.14	Left	Left
2	SH 50 RP 49/2.658	Left	Left
3	Mangatu Road at 4.26km	Left	Left
4	Mangatu Road at 9.04km	Right	Right
5	Mangatu Road at 10.16km	Right	Left
6	Mangatu Road at 11.00km	Left	Right
7	SH1 RP 554/1.951	Left	Left
8	SH2 RP 554/2.721	Left	Left
9	SH38 RP 0/1.09	Left	Left
10	SH38 RP 0/1.49	Left	Left
11	SH38 RP 0/1.81	Left	Left

Using these test pits, the subgrade CBR was also estimated from the Scala penetrometer. The core samples collected from these test pits were used for the other laboratory tests.

The photos of the test pits, including cores at different sites, are attached in appendix E. The test pit logs and the Scala penetrometer results are attached in appendix F.

4.7 Indirect tensile resilient modulus (ITMr) test results

The core samples were tested using the ITMr test for cement-treated material, and the results are given in table 4.5. The test method is described in Moorthy and Patrick (2005).

Table 4.5 ITMr for the core samples

Road name	Location RP	Sample no.		Density t/m^3	Modulus (MPa)	Average (MPa)	Comment
		Site no.	Central lab				
SH1	554/2.721	2A	6/06/128-2a	2.24			
	554/2.721	2B	6/06/128-2b	2.23	20,279	19,184	
	554/2.721	2B	6/06/128-b2	2.22	19,155		
	554/2.721	2C	6/06/128-2c	2.17	18,117		
SH38	0/1.09	1A	6/06/130-a				Broken
SH38	0/1.49	2A	6/06/129-2a				Broken
SH38	0/1.49	2C	6/06/129-2c	1.51	71		
SH38	0/1.81	3A	6/06/130-3a	1.60	827		
Mangatu Rd	4260	A	6/06/131-a	2.21	10,942	11,934	
Mangatu Rd		B	6/06/131-b	2.19	12,628		
Mangatu Rd		C	6/06/131-c	2.22	12,232		
Mangatu Rd	9040	A	6/06/132-a	2.21	12,670	12,252	

Road name	Location RP	Sample no.		Density t/m^3	Modulus (MPa)	Average (MPa)	Comment
		Site no.	Central lab				
Mangatu Rd		B	6/06/132-b	2.22	11,834		
Mangatu Rd		A	6/06/132-a2	2.19	3995	3995	
Mangatu Rd	10,160	A	6/06/133-a				Broken
Mangatu Rd		B	6/06/133-b	2.16			Broken
Mangatu Rd	11,000	A	6/06/134-a	2.15	6866	8191	
Mangatu Rd		B	6/06/134-b	2.15	8355		
Mangatu Rd		C	6/06/134-c	2.17	9352		
Mangatu Rd							
SH50	49/2.658	A	6/06/136-a	2.29	32,831	34,461	
SH50	49/2.658	B	6/06/136-b	2.26	35,387		
SH50	49/2.658	C	6/06/136-c	2.32	35,164		
SH2	678/12.14	A	6/06/135-a	2.33	36,335	40,340	
SH2	678/12.14	B	6/06/135-b	2.35	43,622		
SH2	678/12.14	C	6/06/135-c	2.33	41,062		

4.8 Modelling and analysis

The design traffic calculation for each of the sections is shown in appendix G. The traffic data was obtained from the NZTA website. Some of the traffic growth rates estimated from this data were not reliable and instead, reasonable assumptions of traffic growth were applied. In this study the models were prepared using 95th percentile design life reliability.

The pavement structural design calculation using the Austroads method (CIRCLY 5.1) for each of the pavement sections is shown in appendix H. The two types of model are:

- Type I model: presumed value for resilient modulus obtained for the stabilised layer
- Type II model: modulus of the stabilised layer obtained from IT tests

Structural design calculations using PADS, based on the South African mechanistic-empirical design method, for each section are shown in appendix I. These designs used a presumed resilient modulus, which is very similar to the Type I model.

There are some fundamental differences between the PADS and CIRCLY manuals. For example the South African design criteria specifies that the Poisson ratio for the stabilised material is 0.35, but in the CIRCLY model the Poisson ratio used is 0.2. Note that the presumed values suggested in the PADS manual were used for the PADS models.

The materials indicated in appendix I are as follows:

- AC continuously graded asphalt surfacing

- AG gap-graded asphalt surfacing
- C1–C4 lightly cement-treated materials
- G1–G6 granular materials
- EG4–EG6 equivalent granular materials
- Soils in-situ or imported subgrade material
- BC asphalt bases.

The PADS and CIRCLY model results are summarised in table 4.6. It can be seen that for Mangatu Road, where failure is starting to occur, the Austroads design has underestimated the traffic by several orders of magnitude. However, given the 90th percentile FWD modulus of most of the sites was now at unbound levels, the CIRCLY predictions for a number of the sites also appear unrealistically optimistic. The ITS results complicate this picture, with relatively high values in places; however, it is perhaps best considered that the FWD data does not rule out these high ITS values, but suggests that parts of these pavements (10%) are starting to approach unbound values. The PADS design appears to be a better predictor.

An improved estimate could be made using the FWD subgrade modulus estimates. The modulus of subgrade soils at SH38 Rerewhakaaitu and SH1 Cobham Drive appear to be overestimated by the Scala testing. However it is the same for both the PADS and CIRCLY design methods. It also does not address industry's main concern that the CIRCLY approach can significantly underestimate the pavement life.

Table 4.6 Summary of results

Item no.	Road section	CSL[a]		Design traffic (ESA)	PADS model I (presumed)		CIRCLY				Actual condition
		Thickness (mm)	Cement (%)		Life (ESA)	CL[b]	Model I (presumed)		Model II (ITS)		
							Life (ESA)	CL	Life (ESA)	CL	
SH1 Hamilton											
1	554/1.71-2.99	170	3	4.4E+06	2.86E+06	CSLF[c]	5.26E+06	CSLF	1.70E+09	CSL	Good
SH38 Rerewhakaaitu											
2	RP 0/1.050-1.00, 150	310	6	1.2E+07	1.31E+07	CSL	5.11E+10	CSL	-	-	Good
3	RP 0/1.450-1.550	300	3	1.2E+07	9.07E+06	CSL	1.52E+10	CSL	-	-	Good
Mangatu Road											
4	4250-4270 CSL-1	130	6	8.8E+05	3.55E+06	CSL	4.66E+09	CSL	9.72E+09	CSL	Good
	4250-4270 CSL-2	330	6	8.8E+05	3.55E+06	CSL					
5	8960-8980 CSL-1	145	3	8.8E+05	4.13E+06	CSL	1.39E+10	CSL			Good
	8960-8980 CSL-2	340	3	8.8E+05	4.13E+06	CSL			1.05E+11	CSL	
6	10,070-10,090	170	3	8.8E+05	1.34E+05	CSLF	1.15E+01	CSLF	4.11E+03	CSLF	Beginning to fail
7	10,940-10,960	140	3	8.8E+05	3.59E+04	CSLF	5.99E-+01	CSLF	1.70E+02	CSLF	Beginning to fail
SH2 Napier											
8	678/12.17-12.19	150	5	2.1E+06	3.63E+05	CSL	1.10E+01	CSLF	1.65E+08	CSL	Good
SH50 Napier											
9	49/2.38-2.4	125	5	5.0E+05	5.65E+05	CSLF	9.46E+00	CSLF	2.17E+06	CSL	Good

CSL Cement-stabilised layer
CL Critical layer
CSLF Cement-stabilised layer failed

Reasonable prediction
Non-conservative prediction
Very conservative prediction

4.9 Conclusions

This field study was performed to obtain an indication of the ability of the Austroads pavement design method to predict the performance of cement-stabilised pavements.

The following conclusions have been drawn:

1 The modelling using the ITS core values with the CIRCLY Model II (ITS) confirmed the difficultly in using field data for performance prediction and did not add to the analysis. It did confirm the decision not to use the FWD data for the analysis and the need to use CAPTIF.

2 The Austroads design guide is extremely conservative in some cases and extremely optimistic in others. The optimistic cases appear to be driven by poor estimates of subgrade modulus from Scala penetrometer testing.

3 The Mangatu Road pavement photos suggest that 3% cement is capable of a fatigue-type cracking failure, confirming the suggestion from the CAPTIF testing to limit cement to, say, 2%.

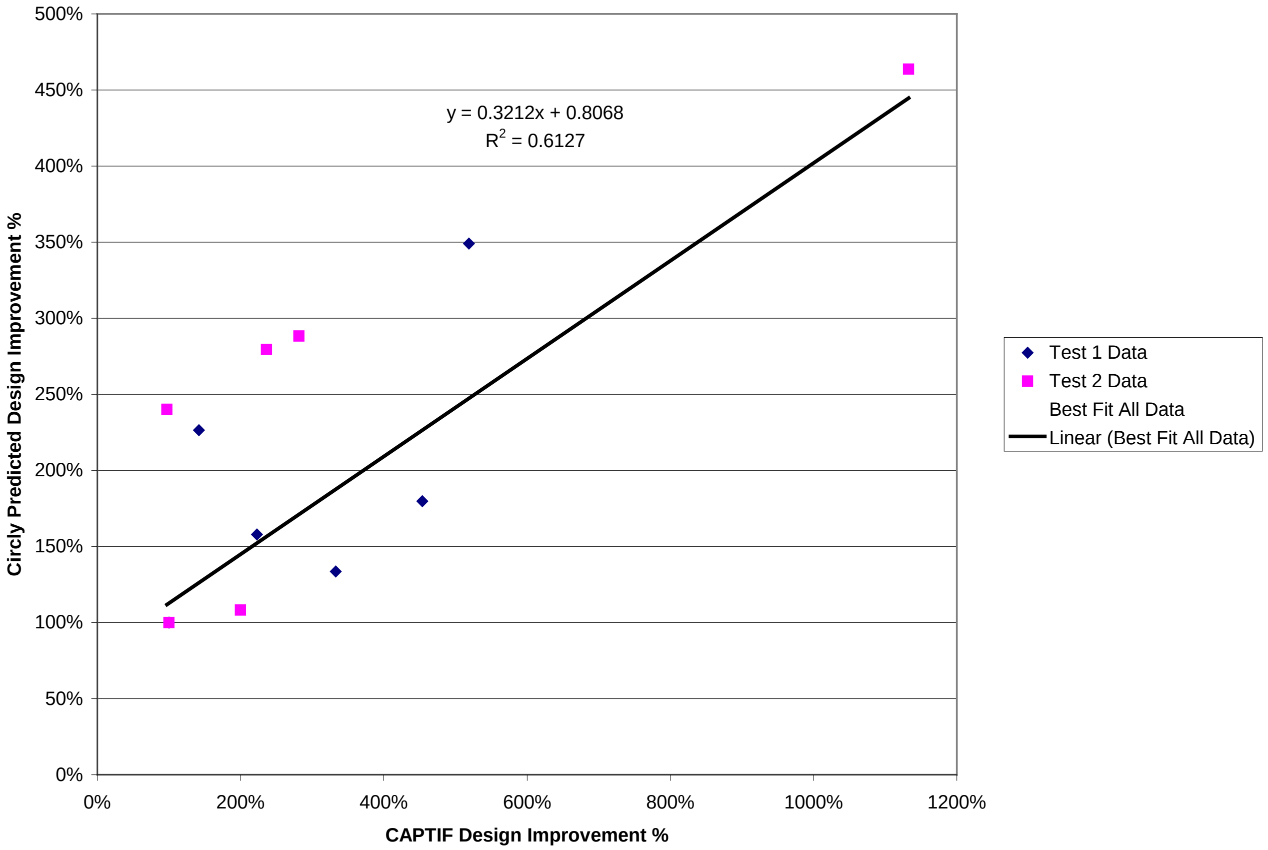
4 The South African PADS method appeared to provide more realistic results, despite using the same data. This appears to be driven by the fact the PADS method assumes very little life after cracking occurs.

5 Across-project findings

5.1 CAPTIF tests

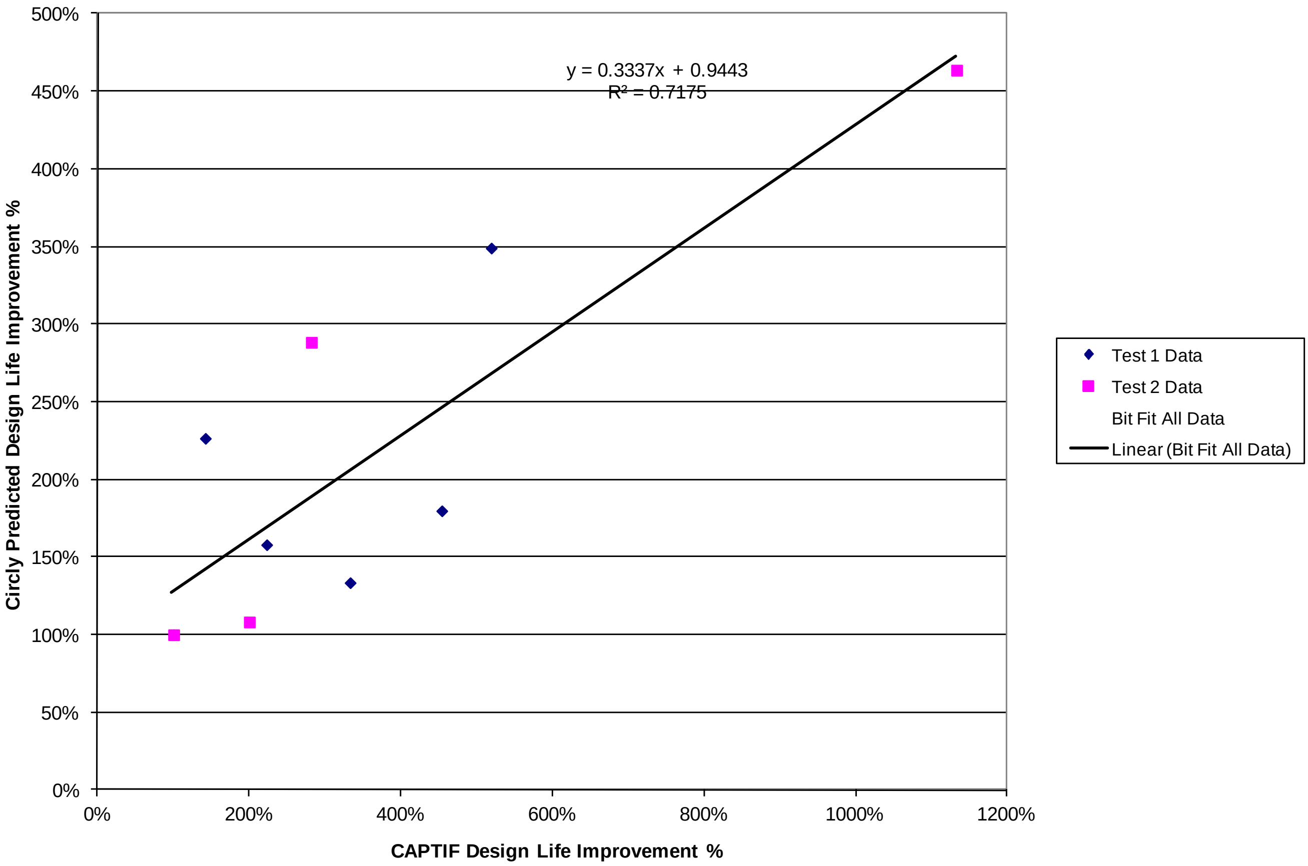
To bring the two CAPTIF tests together in one analysis, it was necessary to normalise the results by looking at the improvement in life predicted by CAPTIF and the design approach using CIRCLY. Figure 5.1 suggests that when the initial construction FWD measurements and CIRCLY were used, a relatively poor prediction of the improvement to the design was made when considering all the data.

Figure 5.1 FWD predicted design life improvement v actual design life improvement



However when the outliers of test 1 and test 2 were removed from analysis, the relationship improved (figure 5.2). The outliers were test 2 section C – 2% cement and section F 4% cement precracked.

Figure 5.2 FWD-predicted design life improvement v actual design life improvement (outliers removed)



Comparing the field ITS results with the design life improvement measured suggested there was possibly a relationship between the ITS and the improvement observed at CAPTIF (figure 5.3).

The relationship was potentially dominated by the high ITS of test 2 section E – 4% cement, and contaminated by the accidental destruction of test 2 section C – 2% cement. To test this both sections were removed from the analysis and the results plotted in figure 5.4. This showed that there was a strong relationship between ITS and improvement even when the 4% cement section was not included.

When the results were plotted in figure 5.5 with only test 2 section C – 2% removed, the relationship between ITS and improvement was very strong.

Figure 5.3 ITS v actual design life Improvement – all ITS data

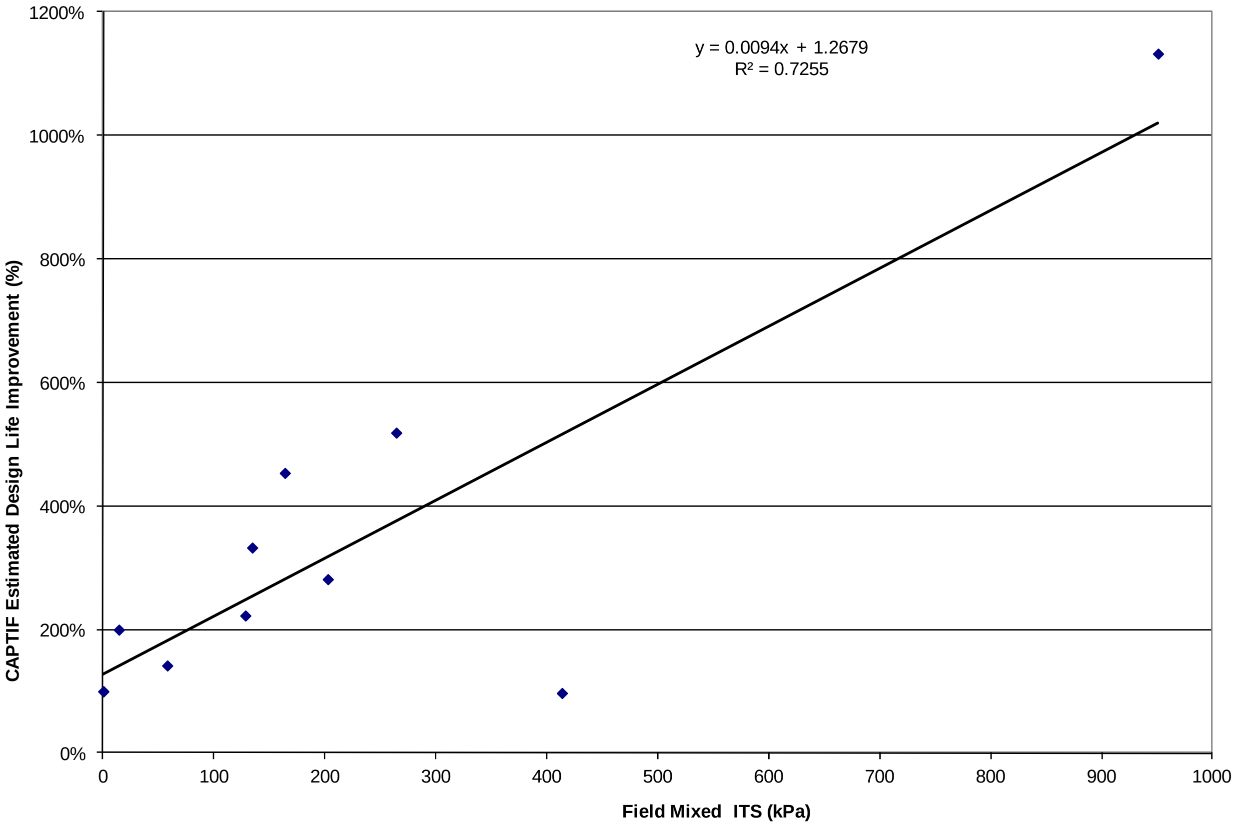


Figure 5.4 ITS v actual design life improvement (section C & E test 2 removed)

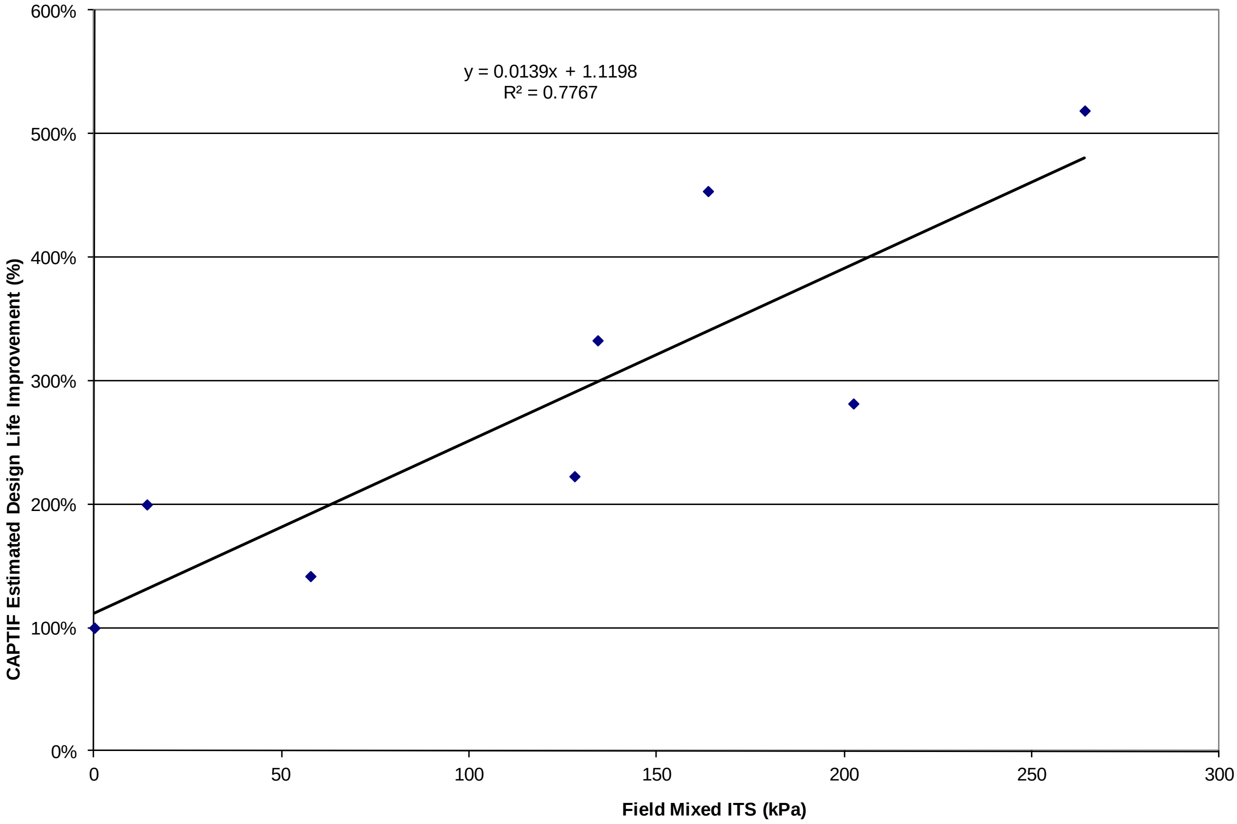
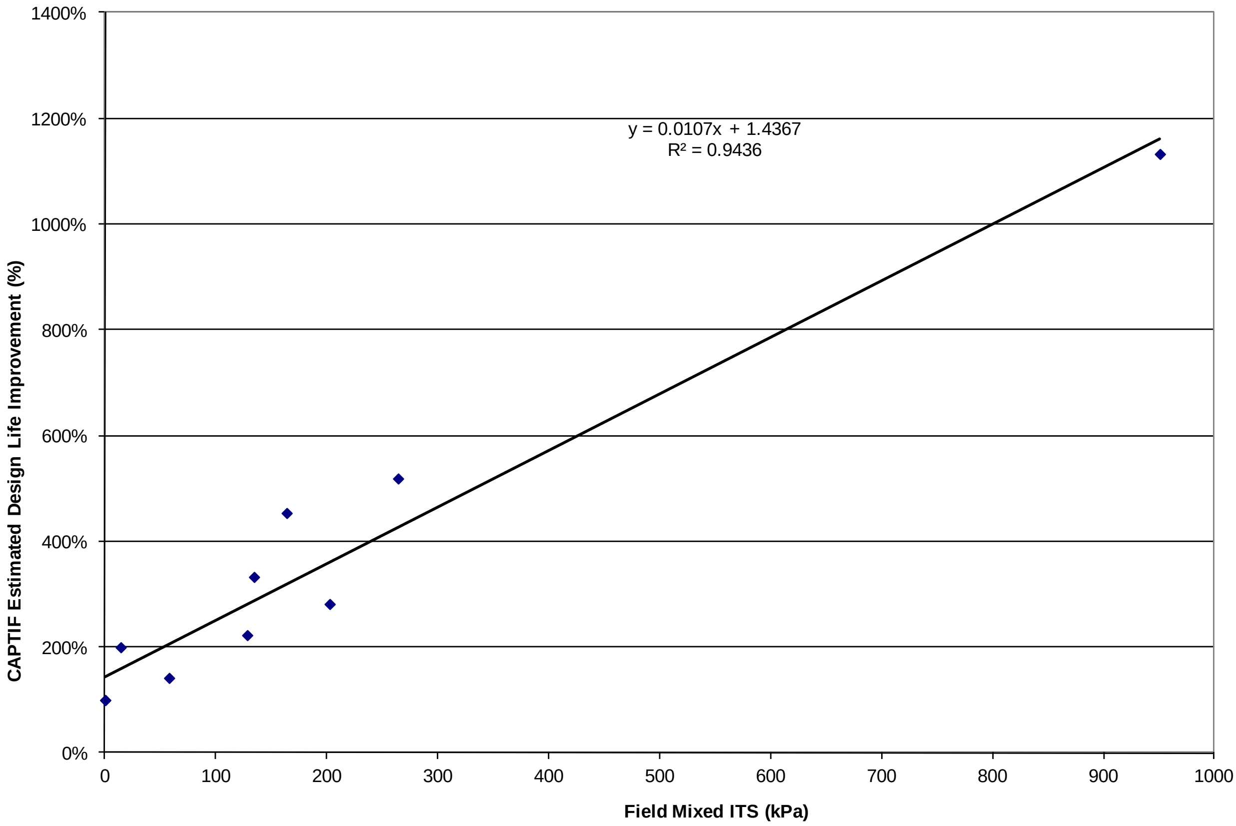


Figure 5.5 ITS v actual design life improvement (section C test 2 removed)



There was not a great relationship between the ITS results and the initial FWD results (R squared of 0.7); however, as shown in figure 5.6 following, the ITS makes a relatively good 'estimate of the improvement' in modulus that will be seen.

Figure 5.6 Field-mixed soaked ITS v modulus improvement

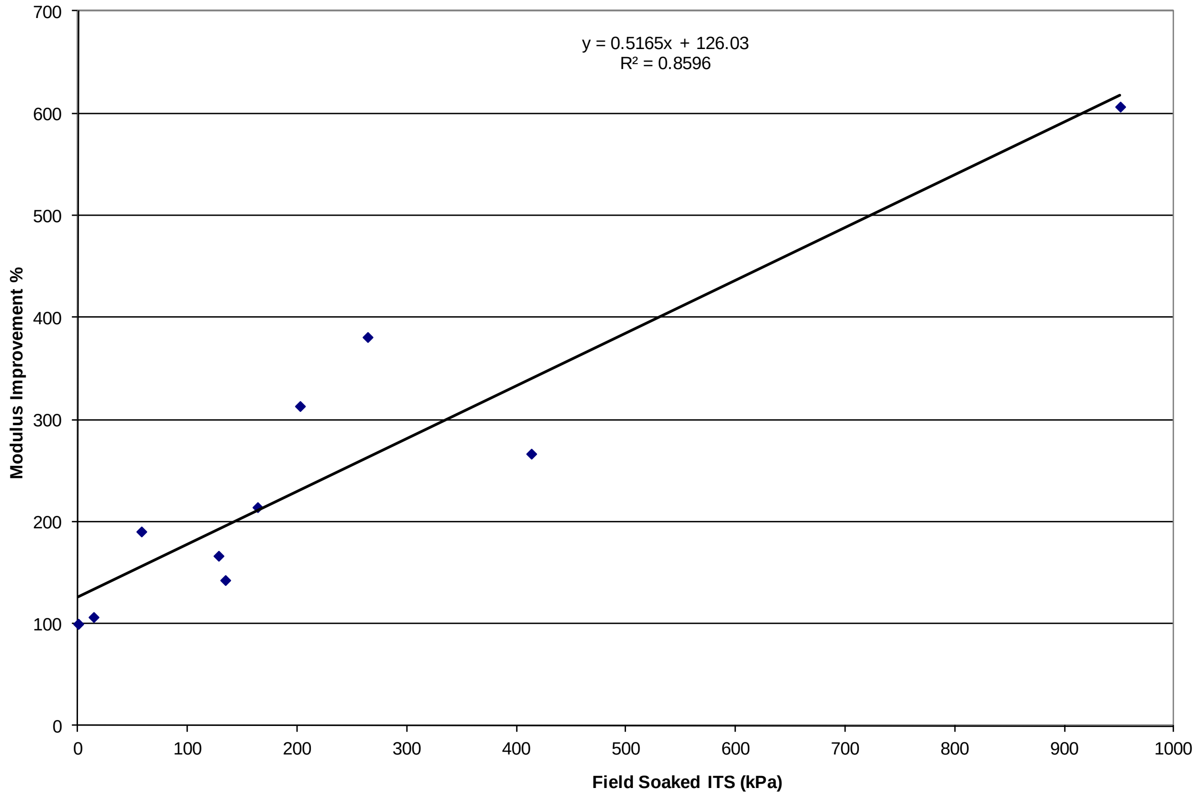


Figure 5.7 shows that a similar relationship can be found with the initial laboratory-mixed samples.

Figure 5.7 Laboratory-mixed soaked its v modulus improvement

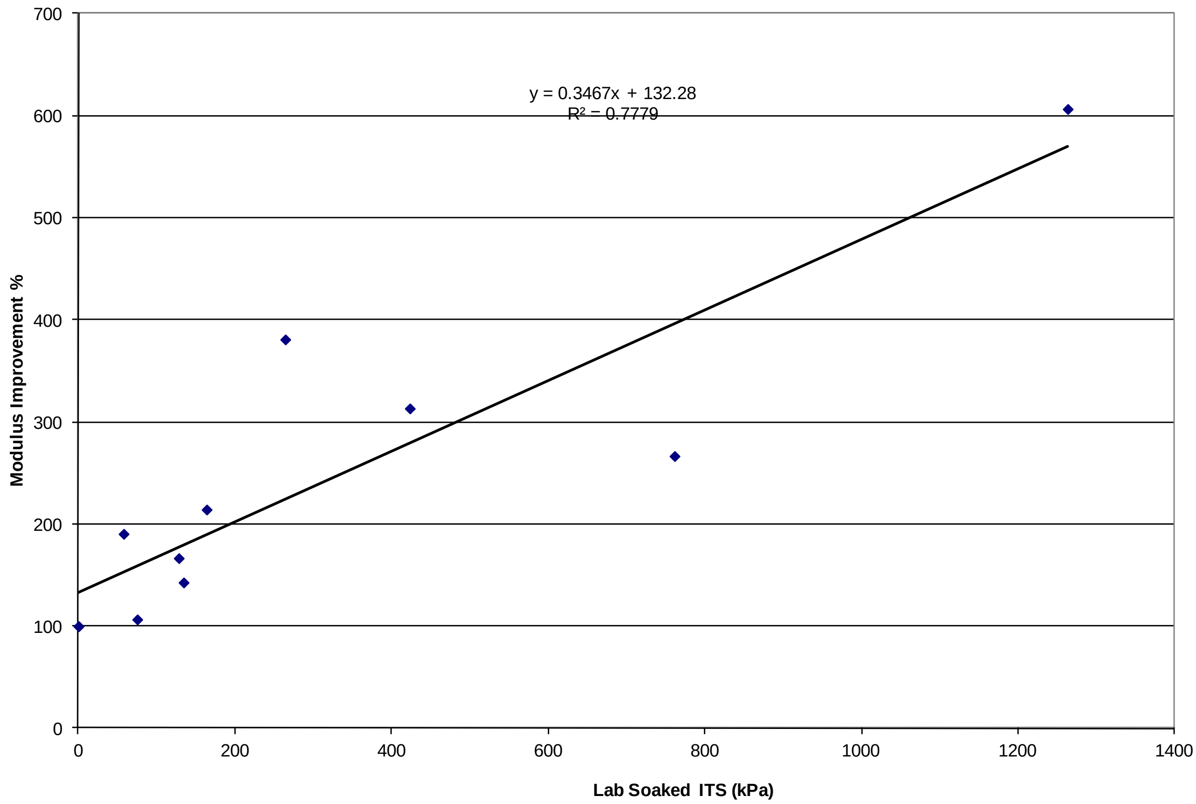
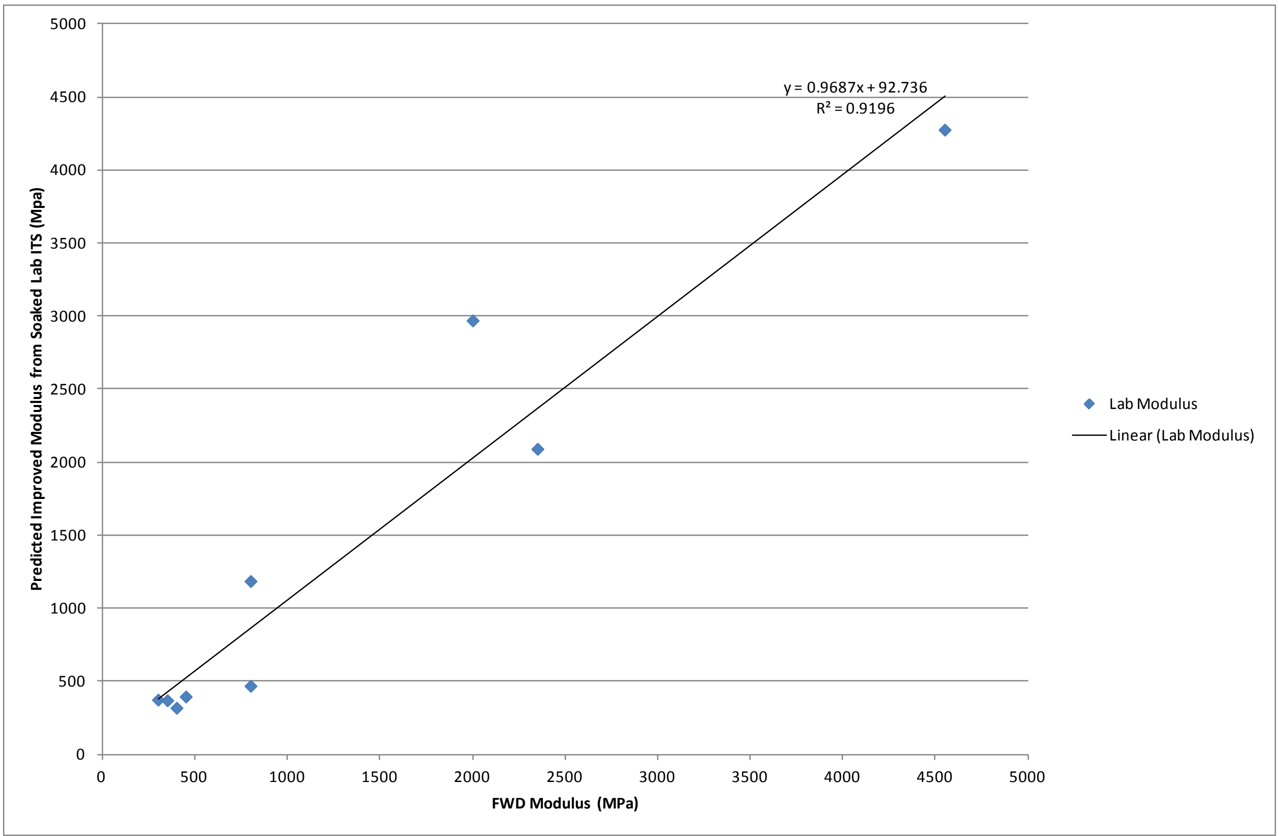


Figure 5.8 Laboratory-predicted modulus v FWD modulus



The laboratory-predicted modulus v the actual modulus provided in figure 5.8 suggests that using the laboratory-mixed soaked ITS to predict an improvement in the field modulus normalised the data quite well.

The equation from figure 5.7 earlier was used to predict the improvement in the unbound modulus from the laboratory-mixed soaked ITS tests and the final modulus was rounded to the nearest 50MPa. That value was then used in a CIRCLY analysis of the pavements. The parameters for the models are provided in table 5.1 below, along with increase in life observed at CAPTIF.

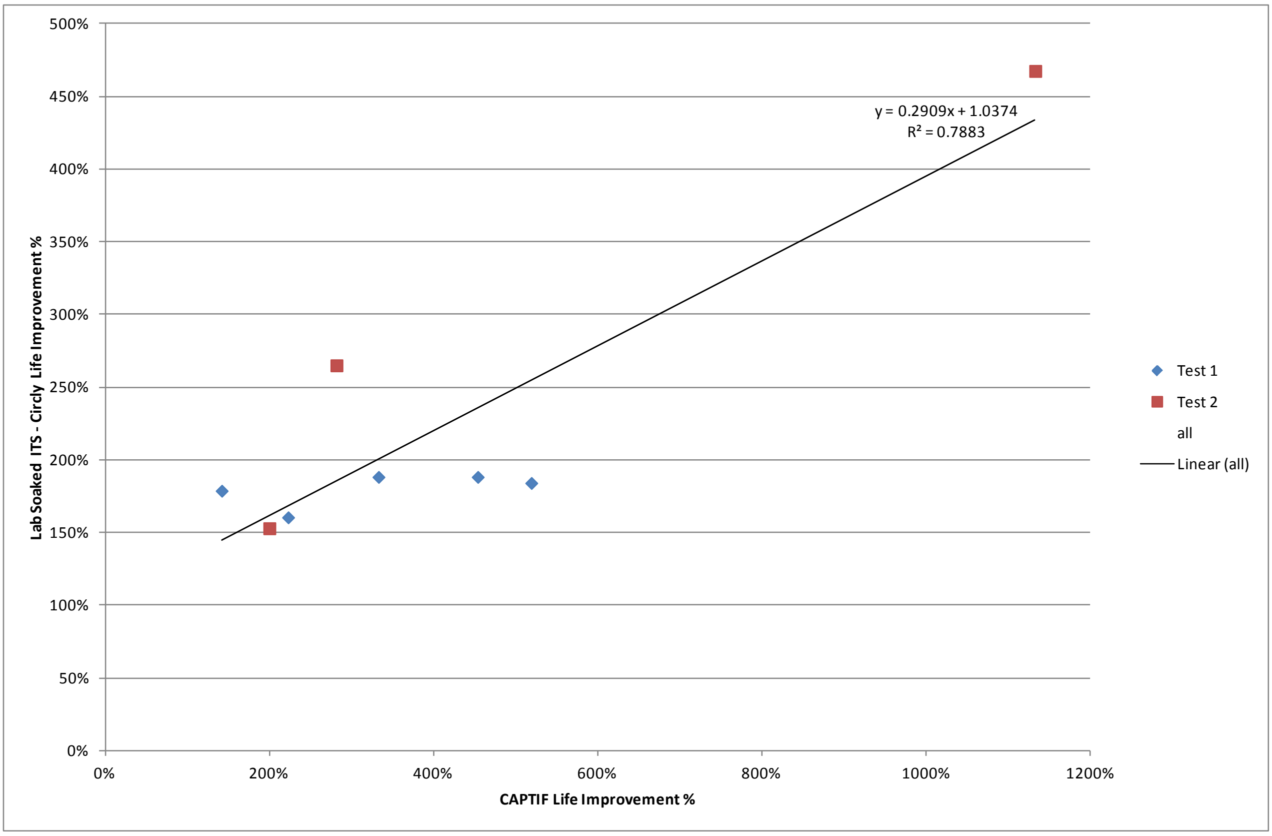
Table 5.1 Parameters for laboratory-mixed soaked ITS-based CIRCLY model

Material	Depths (mm)		ITS (KPa)	Moduli (MPa) 1992 sublayering			CIRCLY life laps	CIRCLY life % incr	CAPTIF % incr
	AC	BC	BC	AC	BC	SG			
1.2 % foamed bitumen, 1% cement	40	210	134	3000	400	110	1.45E+07	188%	333
1.4 % foamed bitumen, 1% cement	40	210	163	3000	400	110	1.45E+07	188%	454
2.8 % foamed bitumen, 1% cement	40	210	264	3000	450	110	1.42E+07	184%	519
1% cement	40	210	128	3000	350	110	1.24E+07	161%	223
Unbound (no binder)	50	190	0	3000	200	110	7.70E+06	100%	100
2.2 % foamed bitumen	50	200	57.6	3000	300	110	1.38E+07	179%	142
Unbound	40	210	0	3000	750	110	2.58E+07	100%	100

Material	Depths (mm)		ITS (KPa)	Moduli (MPa) 1992 sublayering			CIRCLY life laps	CIRCLY life % incr	CAPTIF % incr
	AC	BC	BC	AC	BC	SG			
1% cement	40	210	423	3000	2100	110	6.84E+07	265%	282
2% cement	40	210	761	3000	2950	110	1.02E+08	0	0
2% lime	40	210	75	3000	1200	110	3.95E+07	153%	200
4% cement	30	220	1263	3000	4300	110	1.21E+08	468%	1133

Figure 5.9 below shows that using the ITS to predict an improvement in the performance of an unbound material provided a very good correlation with the observed improvement at CAPTIF. But the *actual* improvement was even better than the prediction making for a conservative analysis.

Figure 5.9 Laboratory-mixed soaked ITS CIRCLY analysis v CAPTIF results (one outlier removed)



An improved approach would be to use the RLT test to estimate the initial rutting life of the unmodified material and directly apply an ITS design life improvement relationship. Figure 5.10 assumes that the subgrade is sufficiently protected and estimates the design life from draft T/15 estimate of life for the unbound base layer, multiplied by an ITS life improvement factor provided by a best fit between the laboratory ITS values and the observed improvements at CAPTIF (similar to figure 5.5.)

The improvement can be determined as follows:

*Modified layer life = unbound layer life (ie T/15 life)*ITS improvement factor* (Equation 5.1)

Where

ITS improvement factor = 0.01672*(ITS)+1.5691

ITS = T/19 soaked ITS value (50.8mm/min strain rate).

In this example the unbound life for the foamed-bitumen testing was estimated in a separate T/15 RLT test that was more representative of the actual conditions at CAPTIF, as the original T/15 testing at lower densities and a higher moisture content suggested that the material would fail almost instantaneously. In the test used for the estimate, the density was 2.23 t/m^3 and moisture content was 3.7%. The T/15 test estimated the life of the unbound layer would be 0.66 million ESA. In the cement/lime testing, T/15 testing estimated the unbound layer would last 1.22 million ESA. Figure 5.10 shows this makes a good estimate of the life improvement.

Figure 5.10 Design from T/15 and ITS

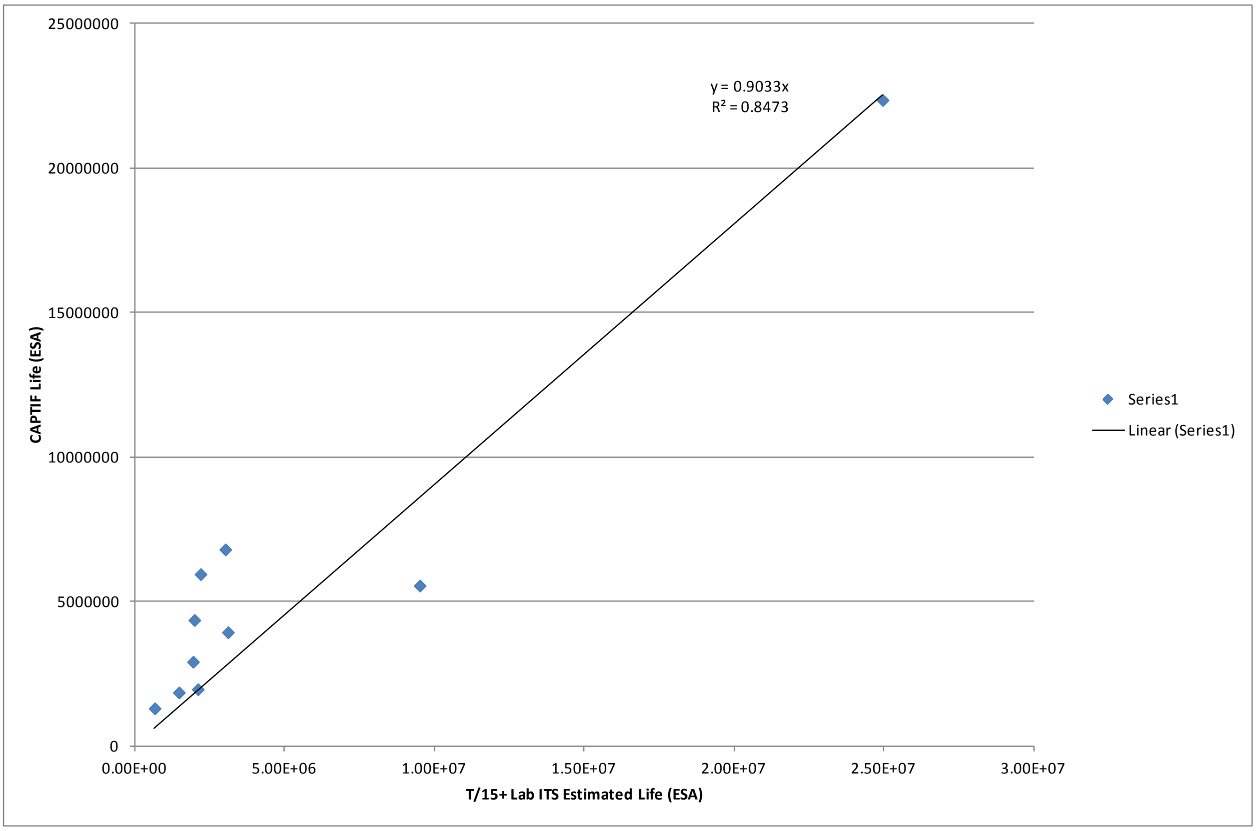
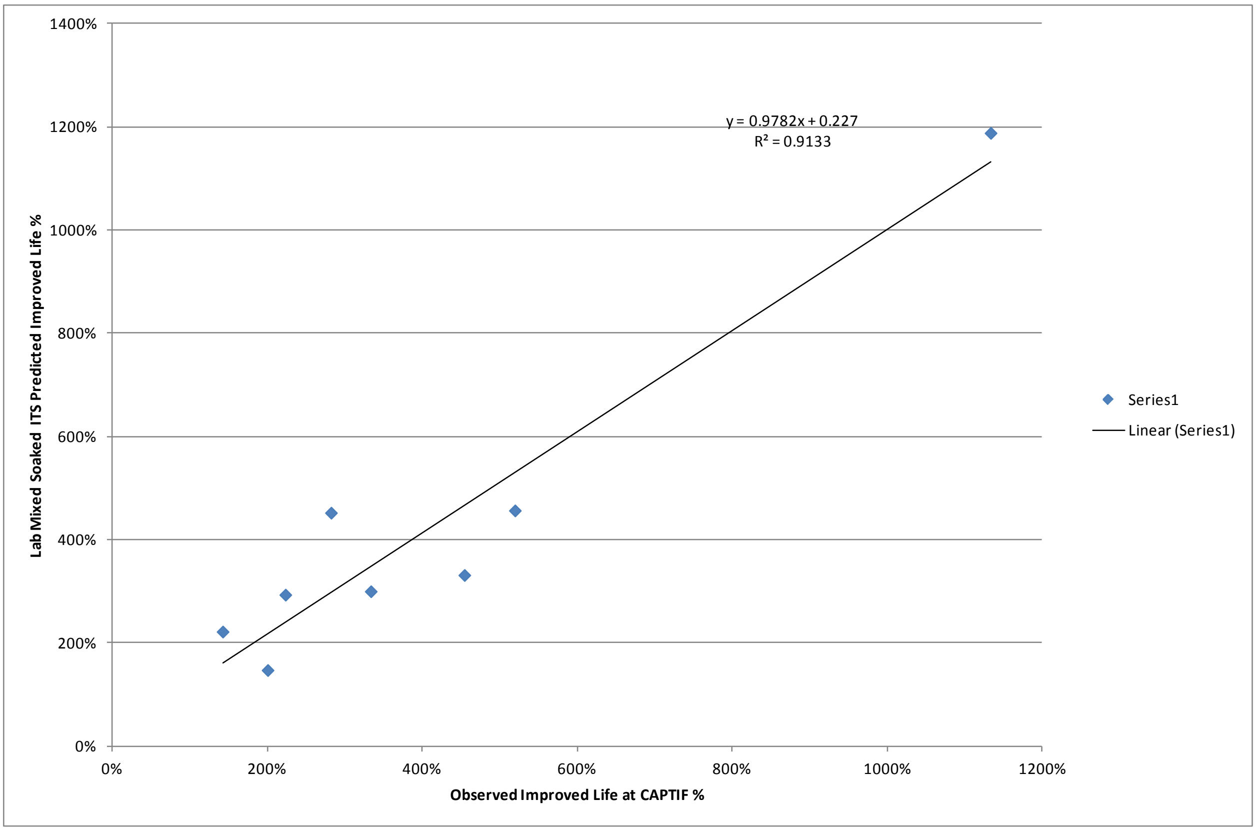


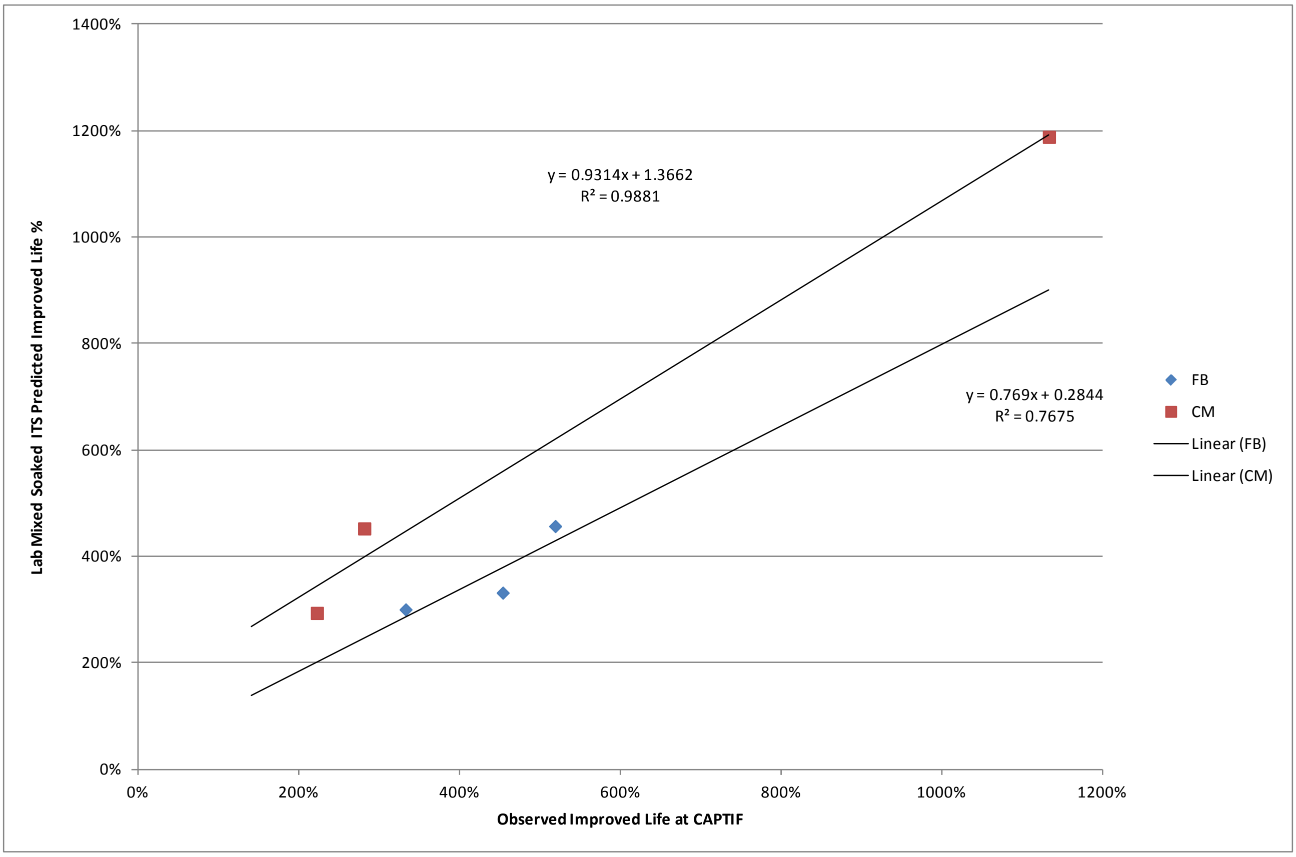
Figure 5.11 shows that part of the error in figure 5.10 came from the initial estimate of life from T/15.

Figure 5.11 Improvement estimate from T/15 and ITS



One of the key remaining questions is whether there is a discernible difference between foam-treated pavements and cement-treated pavements. Figure 5.12 plots the 1% and 4% cement-only tests and the foamed-bitumen-plus-cement tests separately. It suggests that for the same laboratory-mixed ITS data, using foamed bitumen is likely to result in an improved life. The loss of stiffness in the cement-stabilised pavements is suspected to be a significant part of the reason for the lower performance. Key to maintaining performance in the cement-modified pavements will be maintaining its stiffness.

Figure 5.12 Improvement estimate from T/15 and ITS



5.2 Field tests

The field tests reinforced the CAPTIF findings around bound pavements, in particular the concept that fatigue becomes a failure mechanism between 2% and 4% cement. The field study suggested that fatigue could occur in 3% cement mixes that had similar ITS Mr results to those at CAPTIF. The field study also confirmed the rather variable performance of the Austroads pavement design guide (2004) when predicting bound behaviour. At times it was unreasonably pessimistic and at other times overly optimistic. The South African PADS approach appeared to give more reasonable results.

5.3 Overall conclusions

The objective of this project was to improve the sustainability of New Zealand roads by undertaking the following tasks:

1. Determine the benefits of using cement- and/or lime-modified aggregates in terms of increased performance (rutting resistance) and incorporate this in a design methodology, filling a gap identified by Austroads.

2. Validate the benefits of foamed bitumen/cement-stabilised aggregates in terms of increased performance (rutting and fatigue resistance) and incorporate this in a design methodology, building on research by the University of Canterbury and/or the South African Interim Technical Guidelines.

3 Understand the continuum from unbound (no binder), modified (small amounts of binder) to bound (high amounts of binder) behaviour.

4 Review the appropriateness of the Austroads tensile strain criterion for bound aggregates, which is considered by many New Zealand designers to be overly conservative.

Tasks 1 and 2 were completed at CAPTIF and a methodology for predicting performance in design, which builds on current design procedures and laboratory tests, was proposed. Recommendations have been made for rehabilitation design and new design.

Task 3 was not completed with CAPTIF data or the field study, but the research did shed light on the issue. The research suggested that bound behaviour clearly occurred occurs at 3–4% cement contents. The CAPTIF testing showed that at 4% cement contents there was very little rutting, but significant stiffness loss. At 3% cement contents in the field, fatigue failures were observed. At CAPTIF, 4% cement contents showed significant losses of stiffness and the stiffness tended to a value of 1% cement.

It would be a prudent limit for design to start considering bound behaviour at 2% cement contents, which from the CAPTIF test would be a vibratory-hammer-prepared soaked ITS over a limit of 600KPa (when mixed and tested in the lab). This would form an upper limit for using the procedures in figure 6.6 in the next section. Above this limit there is a risk of cracking, leading to water entering the pavement, and potentially rapid failure and difficult repairs. Below this limit there is a risk of the stiffness reducing and the performance not being as good as estimated (this is considered in the figure 6.6 procedures). These values are considerably higher than those proposed by Austroads but are, in part, a function of sample preparation.

Task 4 was completed with a review of field performance and CAPTIF data. The Austroads tensile strain criterion appeared to produce inappropriate results for New Zealand conditions and the South African approach appeared to produce more appropriate results and should be further investigated.

6 Project discussion

The empirical approach recommended in section 5 needs further validation and there may be additional design requirements needed.

6.1 Stiffness loss

In particular, the approach outlined in section 5 did not explain the loss of stiffness seen in the cemented sections. While these sections performed better than the control sections in terms of rutting, the stiffness of the layers reverted back to almost unbound values over the course of the project. The apparent difference in behaviour in the test 1 cemented section is perhaps explained by the fact that there was little strength gain to lose in the first test. It is notable that the foamed-bitumen section did not appear to lose strength, despite being reasonably strong initially and having higher initial central deflections in test 1.

6.2 Fatigue testing

Fatigue testing of the cement materials sheds some light on the loss of stiffness behaviour seen at CAPTIF. The testing was completed as a collaborative effort between this project and the NZTA report 463 ‘Development of tensile fatigue pavement design criteria for aggregates bound by stabilising agents’. (Arnold et al 2011).

Figures 6.1–6.3 show that the cement-treated material was very brittle at low cement contents; that is to say, it had extremely high exponents, and as such it did not fatigue. Instead, it was either below a threshold it could tolerate or it failed practically instantly. Literature on the fatigue behaviour of brittle materials (Richie 1999) suggests that by analogy, at least, such materials should be designed below the threshold. Our research suggests that as the cement content increases, the exponent lowers in the laboratory tests and in theory, at least, the material can treated as capable of fatigue.

The fatigue behaviour of foamed bitumen is shown in figure 6.4 following. While this was not an example of the material tested at CAPTIF, it suggested that the bitumen was making the material ductile; ie the exponent was low for the cement content. This change in behaviour would possibly explain the good performance of the relatively stiff foamed-bitumen material at CAPTIF.

Figure 6.5 also shows the change in behaviour, from brittle to ductile, as more bitumen was added to a cemented sample in a beam breakage test. The material could still carry load and deform after static failure had occurred.

Figure 6.1 1% cement fatigue tests – outlier may in fact be real data

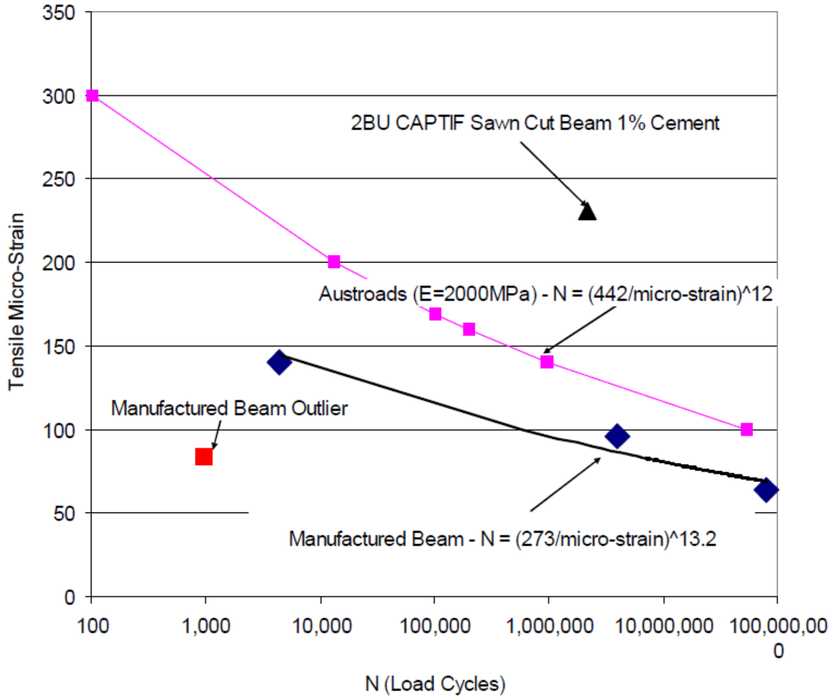


Figure 6.2 2% cement fatigue tests

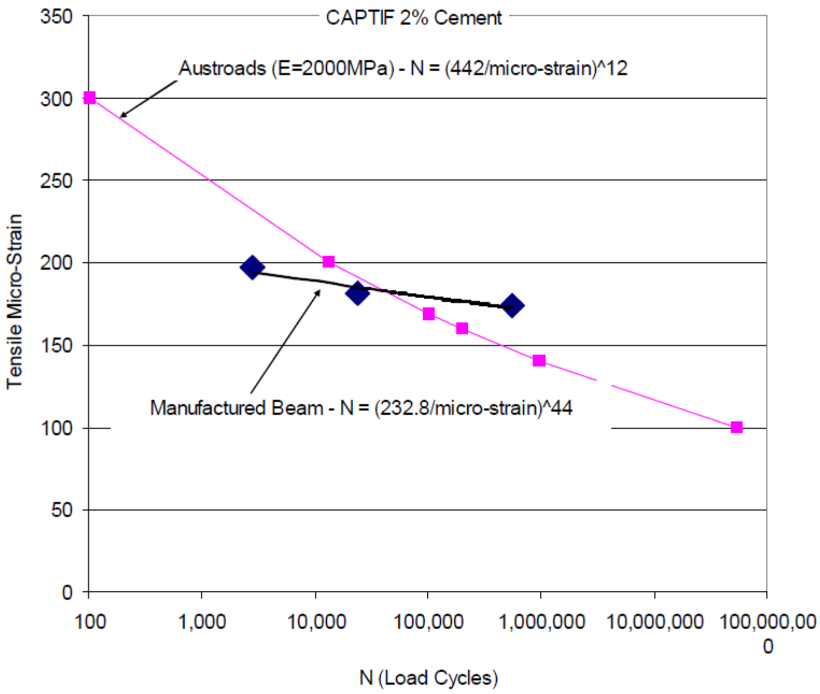


Figure 6.3 4% cement fatigue tests

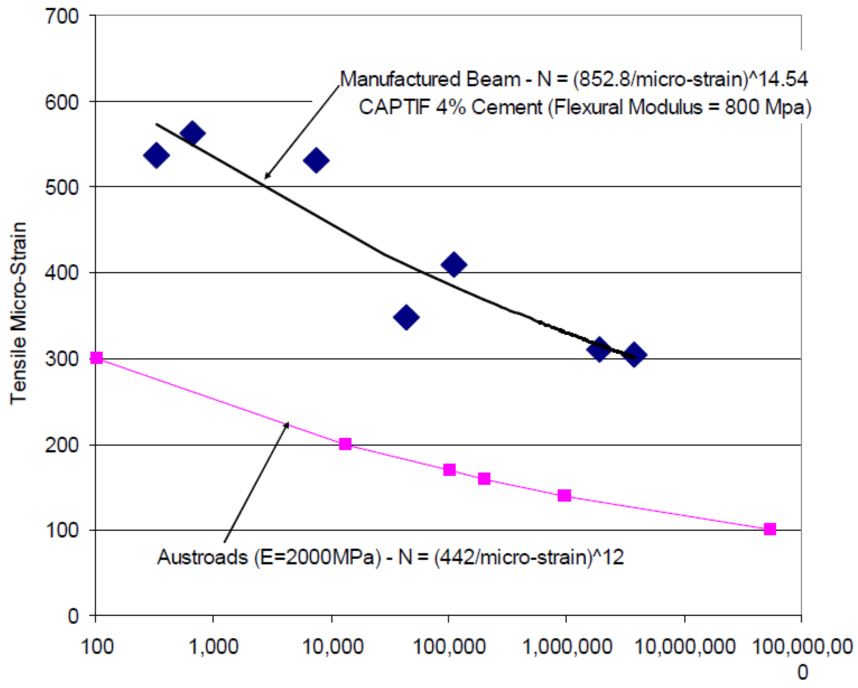


Figure 6.4 Typical foamed-bitumen fatigue tests (not from this project)

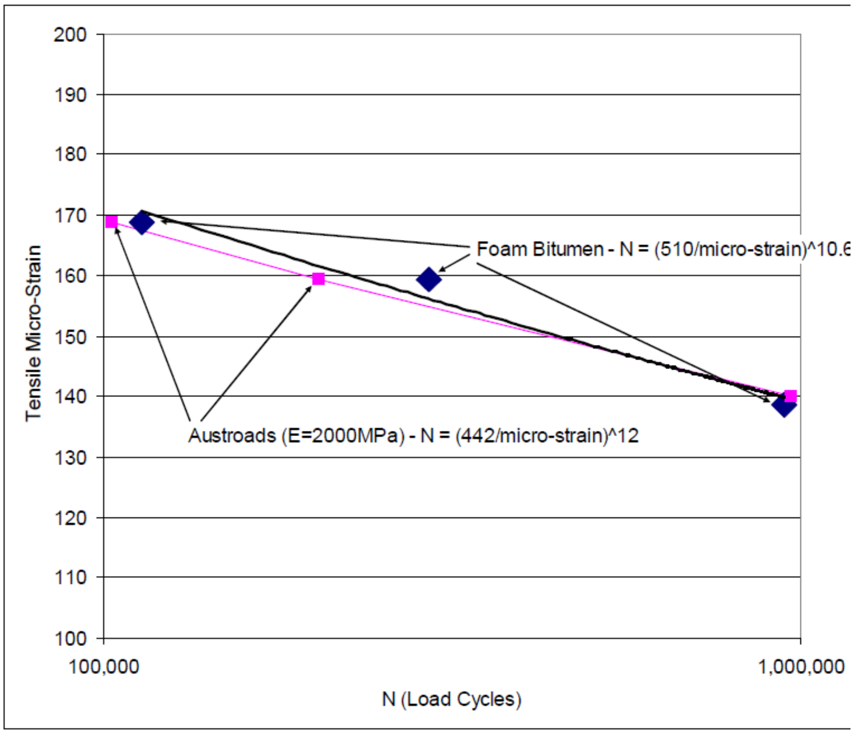
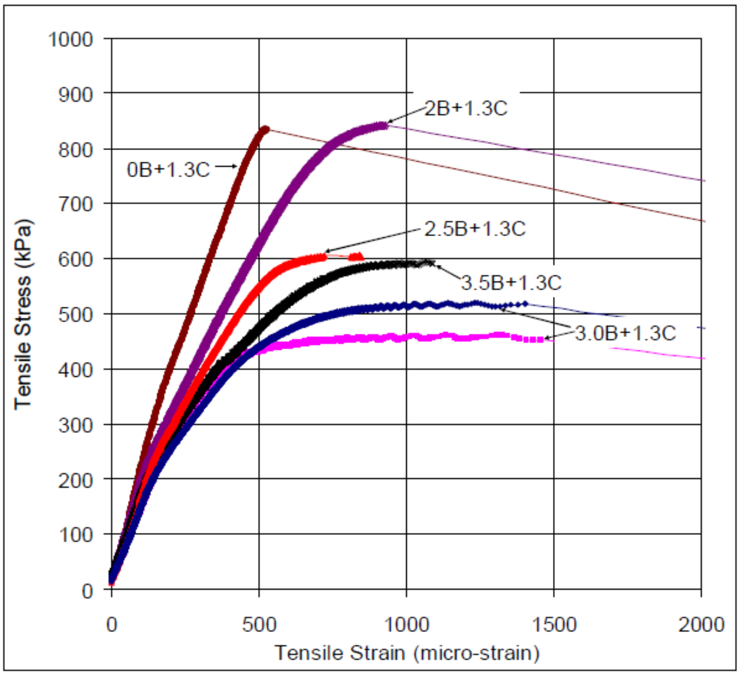


Figure 6.5 Typical change from brittle to ductile behaviour on increasing bitumen content



6.3 Rate of stiffness loss

The fatigue testing did, to a certain extent, explain the differences between the foamed-bitumen and cemented sections. However it did not explain completely the rate of stiffness loss observed in test 2. The Emu coils results in figure 3.10 suggested that all the cemented materials were losing stiffness at fairly constant rate, while the laboratory fatigue tests suggested they should be quite different.

Conventional fatigue theory has a potential answer for this. Richie (1999) provides the following general equation of crack growth-rate fatigue behaviour – it does not deal with elastic conditions where the load is very low or static failure when the load is very high, but it provides a model for describing both brittle and ductile behaviour in fatigue.

$$da/dN = C'(K_{max})^n(\Delta K)^p \qquad \text{(Equation 6.1)}$$

Where

da/dN	=	crack growth rate
C'	=	is a constant equal to C $(1-R)^n$ and (n+p)=m
K_{max}	=	maximum applied stress intensity
ΔK	=	$K_{max} - K_{min}$
R	=	K_{min}/K_{max} (for positive R)
C, n, p	=	material scaling constants.

Implicit in the equation is that subcritical crack growth can be characterised in terms of some governing parameter (often thought of as a cracking driving force) that describes local conditions in terms of loading parameters, crack size and geometry.

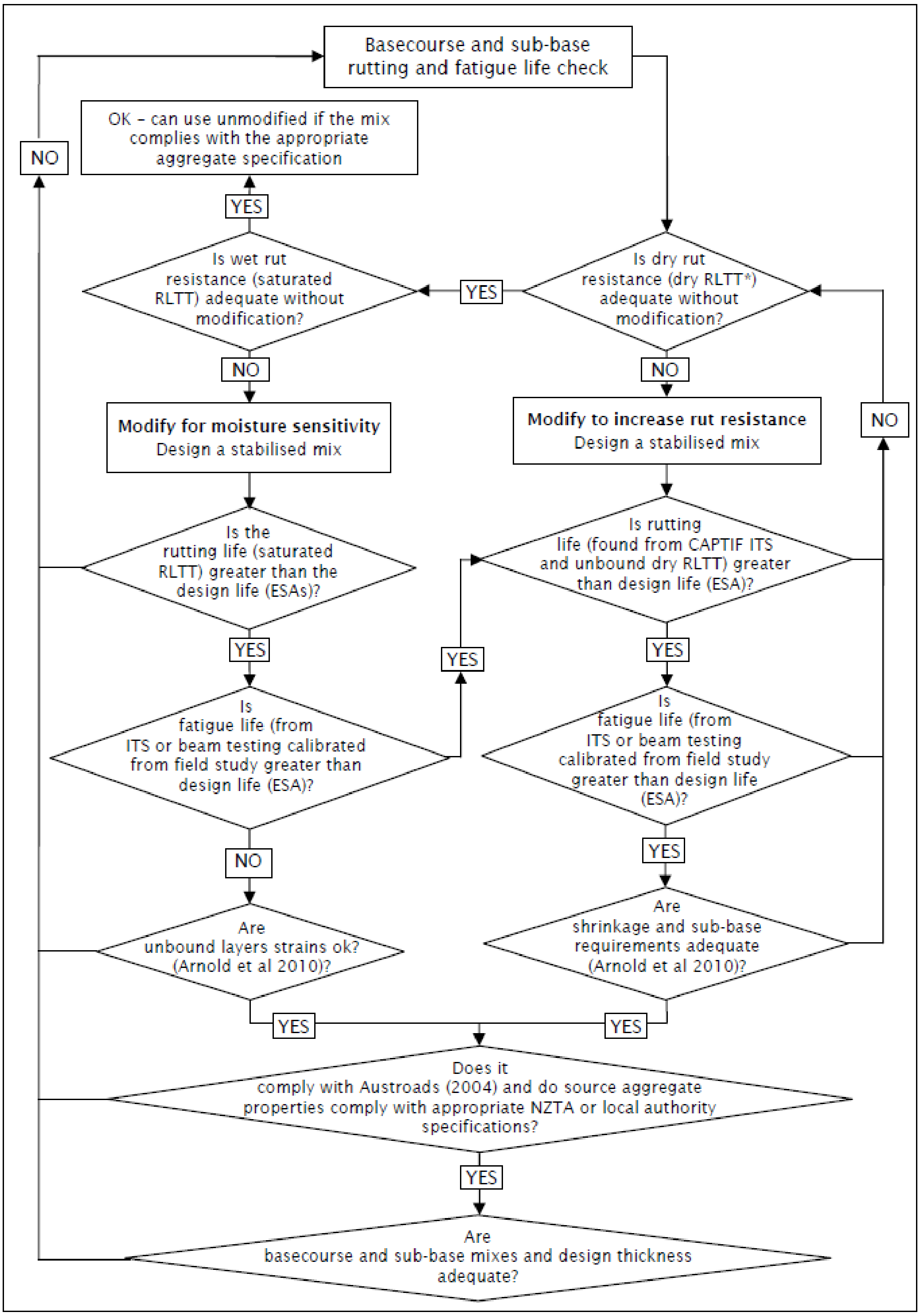
In ductile materials the behaviour of the materials is controlled by intrinsic mechanisms (in front of the crack tip). For example, transgranular ductile striation mechanisms characterise ductile fatigue failure in metals. It has been suggested that such local crack growth occurs via a mechanism of opening and blunting of the crack tip on loading, followed by resharpening of the tip on unloading and closing. Blunting occurs where a plastic zone forms in front of the tip and is sufficiently large when compared to the microstructural dimensions forcing shear to occur on two slip planes roughly 45° to the crack plane. Intrinsic mechanisms are an inherent property of the material and are active irrespective of the length of crack or geometry of the test specimen; under monotonic loading they control the driving forces to initiate cracking. Under cyclic loading the crack advance is controlled by ΔK. So in the general equation n is much less than p and changes in load can result in changes in length of life.

In brittle materials the behaviour of the materials is controlled by extrinsic mechanisms (behind the crack tip), which shield the crack tip from the driving force and the crack advances by effectively static modes (ie cleavage, intergranular cracking or microvoid coalescence). Crack advance is controlled by K_{max}. Bridging grains in the crack assist performance by reducing K_{max} and bridging degradation is controlled by ΔK. This would explain the high exponents of the material with 1% and 2% cement contents – effectively, n is much greater than p. This means that the performance cannot be changed by small changes in stress as with ductile behaviour – either the material works or it doesn't. At higher cement contents the effect of bridging would be more durable (as seen in the laboratory). However, an explanation of the difference between the fatigue-like laboratory behaviour and brittle behaviour at CAPTIF is perhaps that the crack geometry dominates the field behaviour; ie compaction on a soft substrate could create stress concentrations not seen in the laboratory-manufactured specimens. This would also explain the disparity in the relative performance between the beam breakage tests where samples were saw-cut in the post-mortem (see figure 3.18), and ITS test results where samples were compacted into moulds (see figure 3.8). It does not explain the overall excellent fatigue performance of the 1% cement beam saw-cut from CAPTIF tested for fatigue, but that may just be a function of longer curing.

6.4 Pavement design using stabilised aggregates

Three NZTA research projects have recently been completed on stabilised aggregates. The key researchers met to discuss how the outputs of their research could be used in a pavement design process. A flow chart that captured all aspects of a design check for rutting and cracking was developed. The rutting check was based on earlier research using the RLT test (Arnold and Werkemeister 2010; Arnold et al 2010) modified by ITS testing from CAPTIF; checking the fatigue life was based on the three recent research projects on stabilised aggregates. Fatigue life is the number of load cycles when the stabilised aggregate returns to being an unbound aggregate. The flow chart detailing a current 'draft' design process is shown in figure 6.6. This design process will be finalised in the next edition of the New Zealand supplement to the Austroads pavement design guide.

Figure 6.6 Design process for modified pavement layers



*RLTT = repeated load triaxial testing

This research relates to the flow chart step ‘Is rutting life (found from CAPTIF ITS and unbound dry RLTT) greater than design life (ESA)?’. To complete this check the following steps would be taken:

1 A pavement design using the Austroads guide would be undertaken to check, for example, that the subgrade was sufficiently protected and any asphalt layers were unlikely to fatigue, using a conservatively estimated modulus for the modified layer.

2 The unbound aggregate for a new pavement design would be tested in the RLT to the unsoaked NZTA T/15 test and assigned a design life, or a conservative estimate of unbound life would be made if using the standard NZTA B/5 aggregate requirements. In a rehabilitation situation, an estimate of the traffic loading to failure would be taken as the unbound life where aggregate complies with the NZTA B/5 aggregate requirements.

3 Laboratory-mixed ITS samples would be prepared, using the NZTA T/19 specification, from the new or existing aggregate and tested in a soaked condition.

4 The improvement in the basecourse life would be determined from the unbound life determined in step 2 and the soaked ITS values in step 3, using the following formula:

*Modified layer life =unbound layer life (ie T/15 life)*ITS improvement factor* (Equation 6.2)

Where:

ITS improvement factor = 0.01672*(ITS)+1.5691
ITS = T/19 soaked ITS value (50.8mm/min strain rate)

5 Additional checks would then be made for fatigue resistance (Arnold et al (2011), to ensure the performance was maintained, and for shrinkage, to ensure the materials would not create shrinkage cracks and cause maintenance issues – but ideally, the laboratory-based soaked ITS value should be kept below 600KPa, to prevent any fatigue failure leading to surface cracking.

7 Conclusions and recommendations

This chapter indicates the extent to which the project's objectives were fulfilled by this research, and makes a number of recommendations.

- Objective 1: *Determine the benefits of using cement- and/or lime-modified aggregates in terms of increased performance (rutting resistance) and incorporate this in a design methodology, filling a gap identified by Austroads.*

  This research showed that modifying the tested aggregates with 1% cement could reduce rutting and improve the rutting life of the pavement by 200–300% compared with the unbound pavement. However, stiffness loss occurred during this testing. Section 6.4 presented a design methodology that can be used with initial laboratory data to estimate the initial improvement in rutting performance. Further research is needed into the implications of this stiffness loss in cement-only materials. Figure 6.6 presented a wider approach that included other recent NZTA research to address the stiffness loss.

- Objective 2: *Validate the benefits of foamed bitumen/cement-stabilised aggregates in terms of increased performance (rutting and fatigue resistance) and incorporate this in a design methodology, building on research by the University of Canterbury and/or the South African Interim Technical Guidelines.*

  This research showed that modifying aggregates with foamed bitumen and cement reduced rutting and created a 500% improvement in life compared with the unbound pavement, without any loss of stiffness (fatigue) during the project. The design methodology presented in section 6.4 can also be used for foamed bitumen. Figure 6.6 presented a wider approach, including other recent NZTA research, which explained why stiffness loss was not observed.

- Objective 3: *Understand the continuum from unbound (no binder), modified (small amounts of binder) to bound (high amounts of binder) behaviour.*

  This research project did not result in a better understanding of the continuum from modified to bound behaviour; however, it provided some useful findings. The research suggested that bound behaviour clearly occurred at 3–4% cement contents. The CAPTIF testing showed that at 4% cement contents there was very little rutting, but significant stiffness loss. At 3% cement contents in the field, classical fatigue failures were observed, where stiffness was lost and cracking was apparent in the pavement surface. At CAPTIF, materials with 4% cement showed significant losses of stiffness and the stiffness tended to a value of stiffness observed in 1% cement pavements.

  It would be a prudent limit for design to start considering bound behaviour at 2% cement contents, which from the CAPTIF test would be a vibratory-hammer-prepared soaked ITS over a limit of 600KPa (when mixed and tested in the lab). This would form an upper limit for using the procedures in figure 6.6. Above this limit there is a risk of cracking, leading to water entering the pavement, and potentially rapid failure and difficult repairs. Below this limit there is a risk of the stiffness reducing and the performance not being as good as estimated (this is considered in the figure 6.6 procedures). These values are considerably higher than those proposed by Austroads but are, in part, a function of sample preparation.

- Objective 4: *Review the appropriateness of the Austroads tensile strain criterion for bound aggregates, which is considered by many New Zealand designers to be overly conservative.*

  The CAPTIF test and field study suggested the Austroads tensile strain criterion appeared to produce inappropriate results for New Zealand conditions, and the South African approach appears to produce more appropriate results. Materials with 4% cement, tested at CAPTIF, led to a 1000% increase in rutting life compared with the unbound pavement; however there was significant stiffness loss. Research by Arnold (2012) produced a laboratory-based approach to the design of bound materials, and this is given in figure 6.6.

The current Austroads design procedures did not accurately predict the improvement in the modified materials (as already noted by Austroads (Foley 2001); however it was conservative.

The benefits of modified aggregates can be included in current Austroads design approach with the empirical procedures laid out in section 6.4 of this report. These preliminary recommendations have been made for rehabilitation design and new design. The procedures can be used to estimate design life from laboratory-mix design data on any proposed material.

7.1 Summary of findings and recommendations regarding foamed bitumen

- The addition of foamed bitumen significantly improved the performance of materials with 1% cement studied in this research.
- The current design methods for foamed-bitumen pavements are over-conservative. However, the blanket use of an unsublayered 800Mpa modulus appears not to be appropriate as a model, and an alternative approach is suggested in section 5.
- The pavement behaved in a ductile manner even when water was deliberately introduced.
- Foamed-bitumen contents that maximise ITS in pavements (while retaining ductility) should be adopted where there is a potential risk of water introduction into the pavement layers. Note that this research only tested foamed bitumen with 1% cement content.
- The RLT test using the current stress levels in the draft T/15 specification was not able to detect the effect of foamed bitumen in materials with cement, and complementary tests should be conducted to assess the properties of foamed-bitumen mixes.
- The stiffness loss that would be expected from laboratory fatigue beam testing was not observed at CAPTIF.
- Given that there appeared to be no stiffness loss with loading the empirical design approach given in section 5, this can be safely used (as long as ductile behaviour is maintained), but further validation and consideration given to the full procedures in section 6.4 is suggested.

7.2 Summary of findings and recommendations regarding cement/lime

- The RLT test using the current stress levels in the draft T/15 specification was not able to predict the change in performance of the materials as cement contents rose over 1.5%.

- The testing found a strong correlation with the UCS, ITS and IT modulus results. There was a poor correlation between the IT modulus and the RLT modulus.
- During construction there was a good correlation between the laboratory-mixed and field-mixed UCS values. However the field results were approximately 80% of the laboratory values.
- During construction there was a good correlation between the laboratory-mixed and field-mixed ITS values. However the field results were approximately 70% of the laboratory values.
- Precracking the basecourse reduced the average basecourse modulus after construction by 40%.
- The precracked cement section did not fully heal. The 4% cement, once precracked, behaved in a similar manner to 1% cement uncracked.
- The cement-modified pavements behaved in a brittle manner. When water was accidentally introduced to one section it quickly reverted to unbound stiffnesses. However wet cement-modified pavements did perform better than wet unbound pavements.
- Post-mortem testing confirmed that the cement-stabilised test sections were, as expected, damaged by the loading.
- The basecourse modulus from initial FWD testing showed a good relationship with the load-carrying capability of the pavements.
- ITS testing on field-obtained samples showed a good relationship with improvements in the load-carrying capability of the pavements.
- The stiffness loss that would be expected from laboratory fatigue-beam testing was observed at CAPTIF. However that stiffness loss occurred across all cemented sections in a relatively uniform number of loads. That was not expected from the laboratory tests. This may have been due to the brittle nature of the material.
- Given that there appeared to be stiffness loss with loading, the empirical design approach given in section 5 should be further validated before wide use. However, the research by Arnold et al (2012) suggests limits that should prevent stiffness from degrading significantly (figure 6.6).

The results of this research, when validated, will increase the utilisation of locally available materials and promote the recycling of existing materials. This will in turn reduce the cost of pavement construction, rehabilitation and maintenance, without compromising performance.

This research will also reduce the travel-time delays associated with rehabilitation work in terms of frequency and duration. Modified materials have the potential to last longer before needing rehabilitation and the construction techniques associated with modified materials, such as in-situ modification, are very fast when compared to traditional overlays, taking days rather than months to compete projects. In addition, these stabilised pavements will better resist the impacts of any changes in the mass limits of heavy vehicles and increasing traffic volumes, and will provide improved performance in wet conditions.

8 References

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Appendix A Test 1 post-mortem photos


STA11


STA16


STA21


STA25


STA31

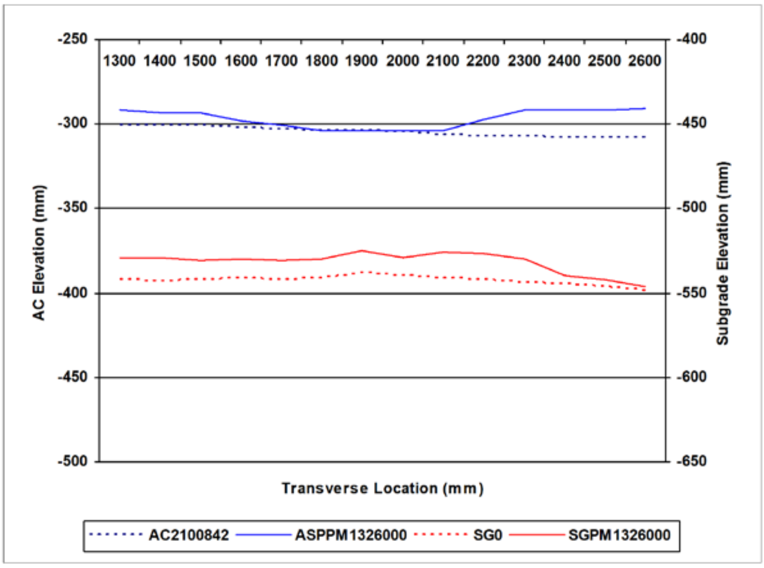

STA33

STA 40 – Incorrectly labelled as STA 04


STA50


STA54

Appendix B Test 1 post-mortem trench profiles



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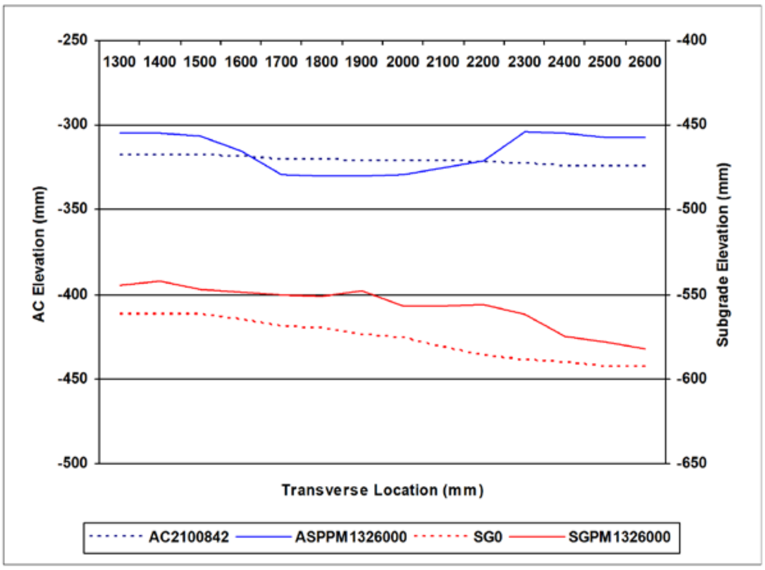

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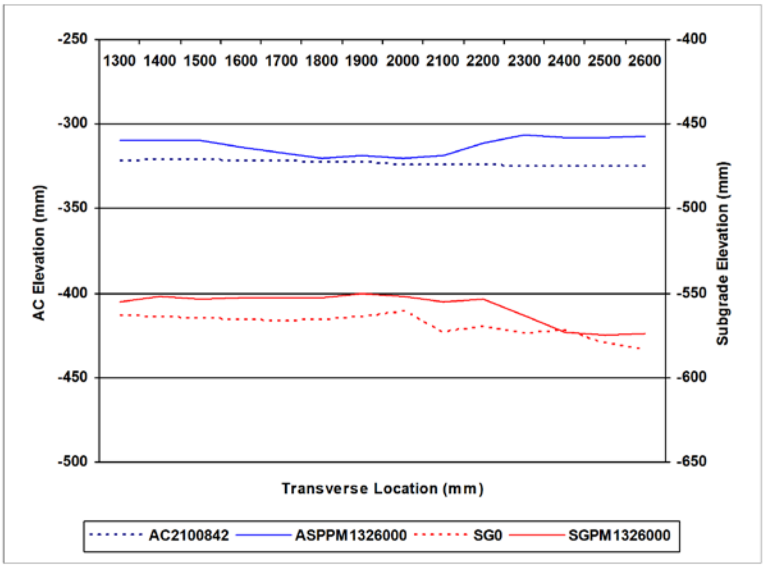

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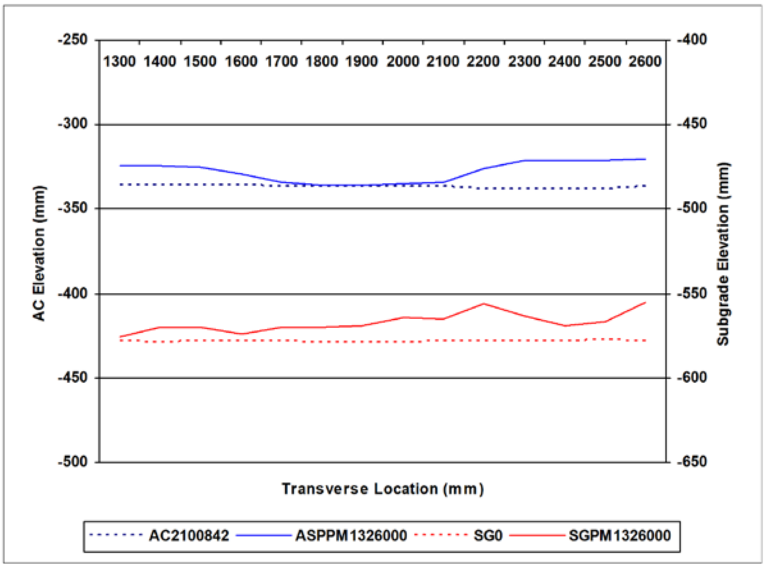

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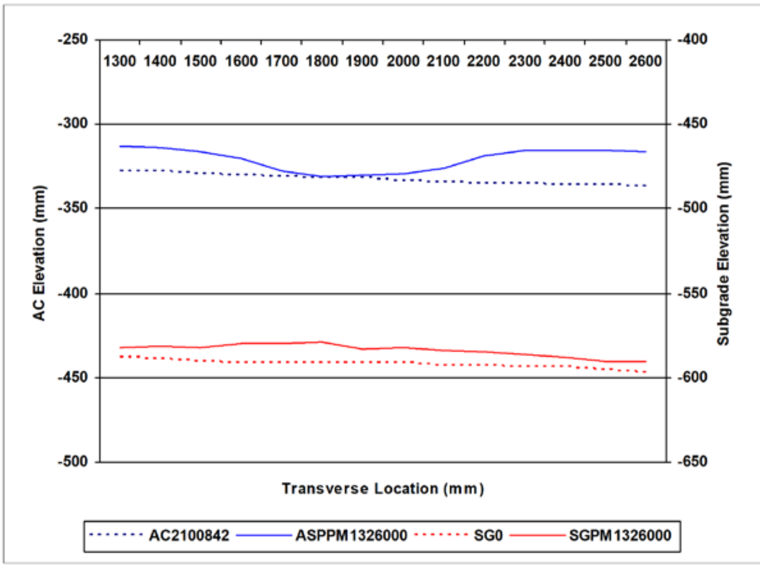

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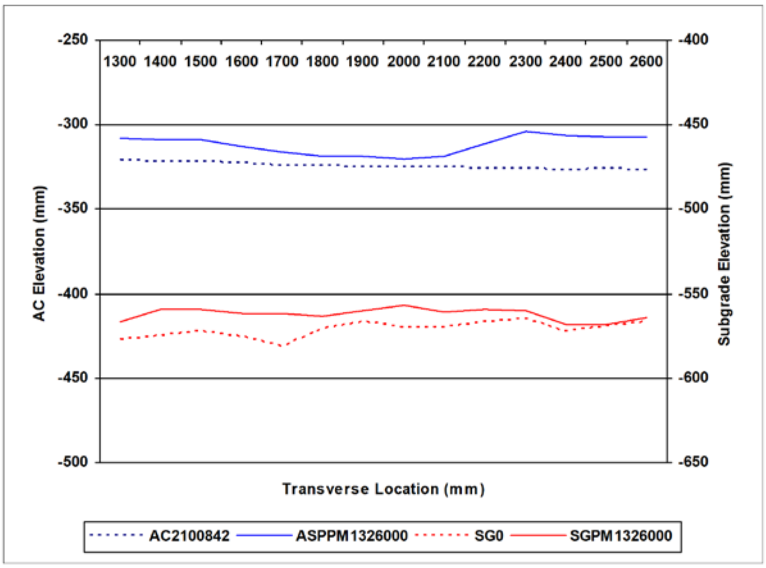

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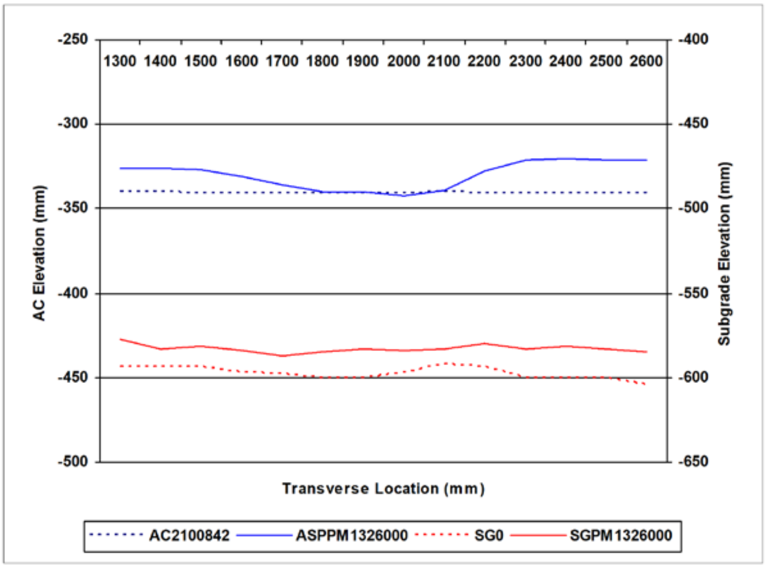

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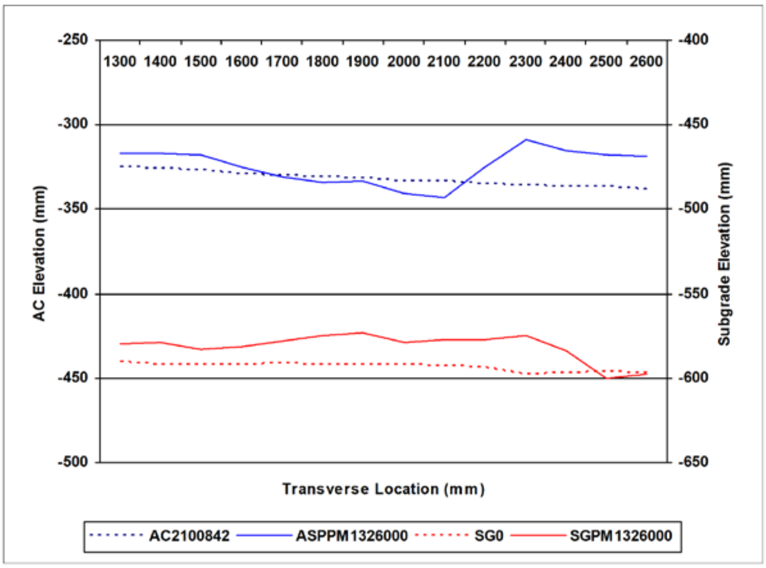

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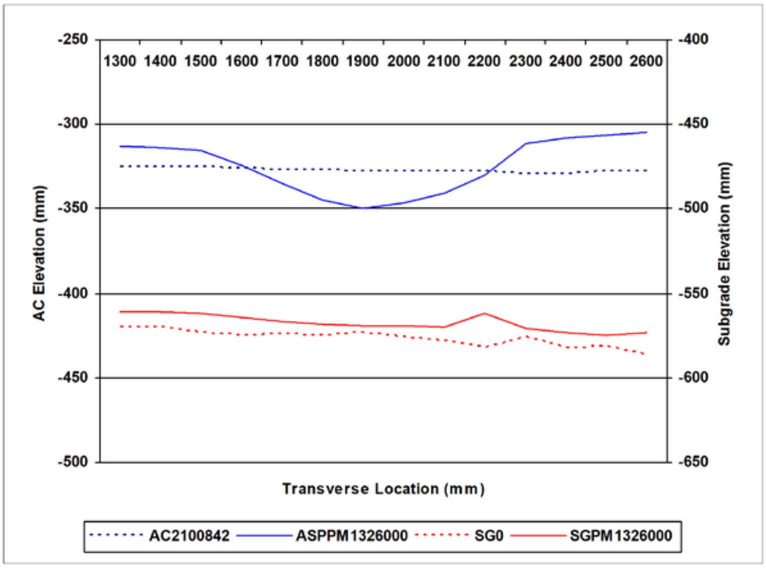

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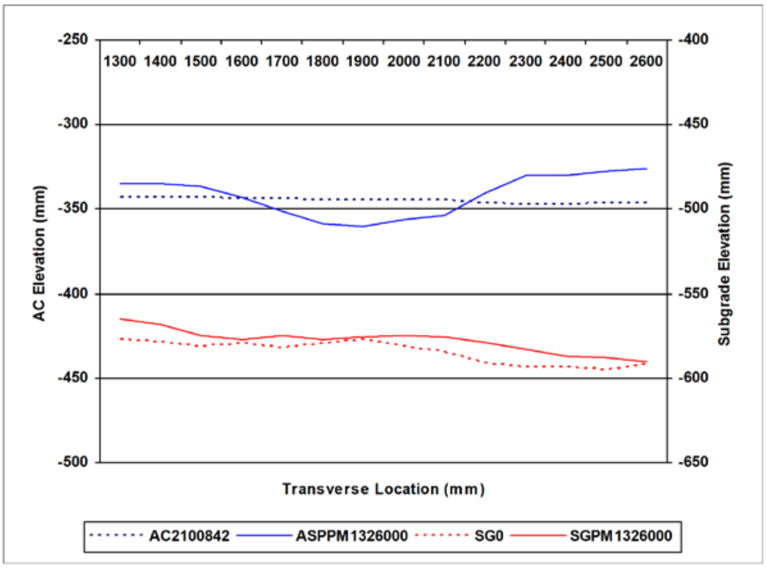

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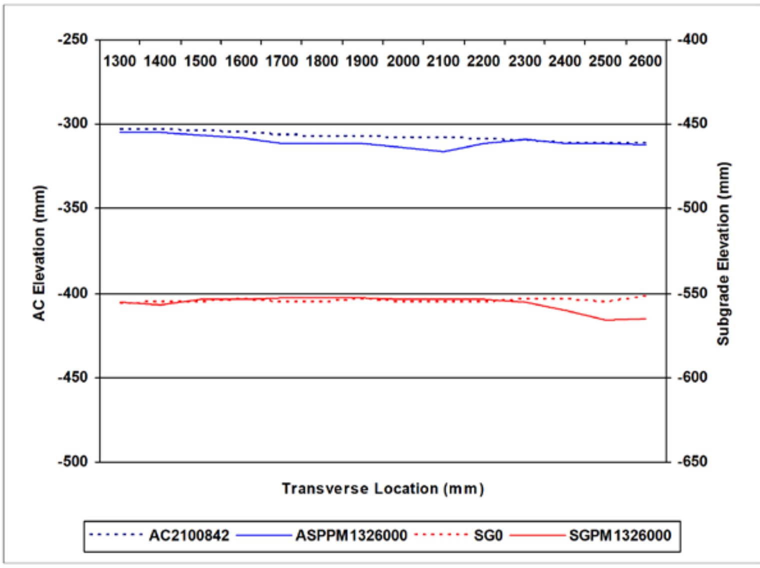

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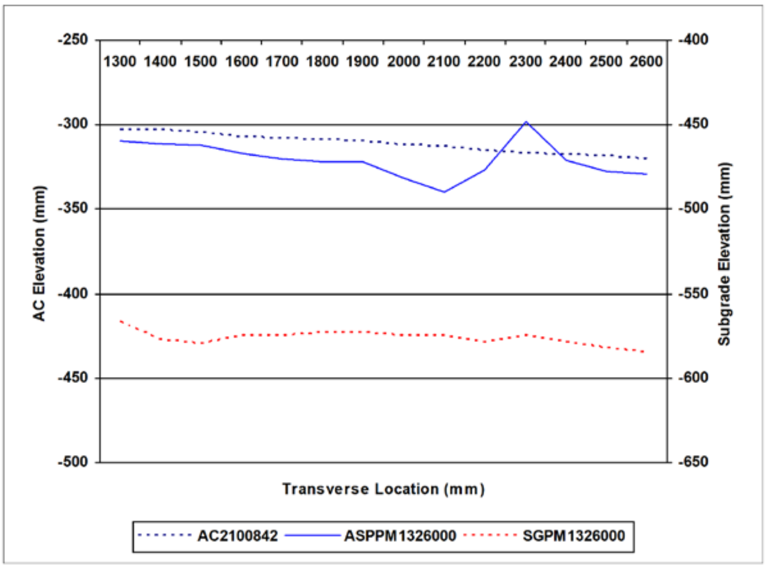

Station 54

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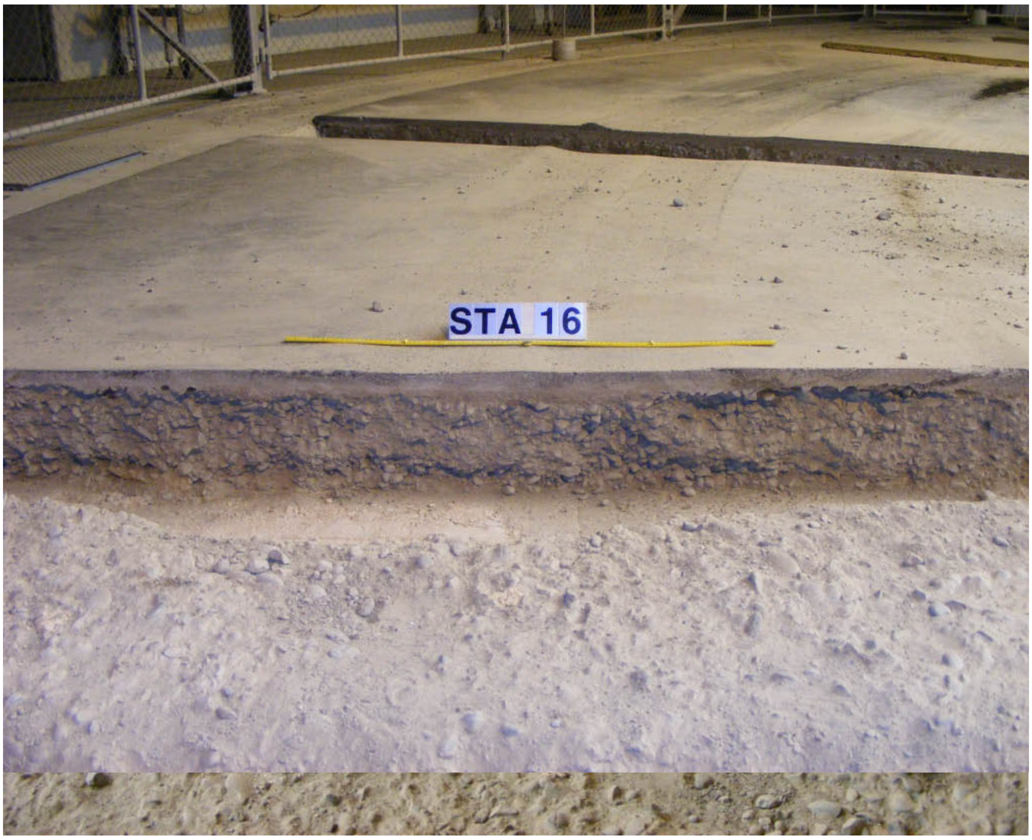
Appendix C Test 2 post-mortem photos


STA 06


STA 13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STA 19


STA 24


STA 26


STA 31


STA 32


STA 36


STA 39


STA 42

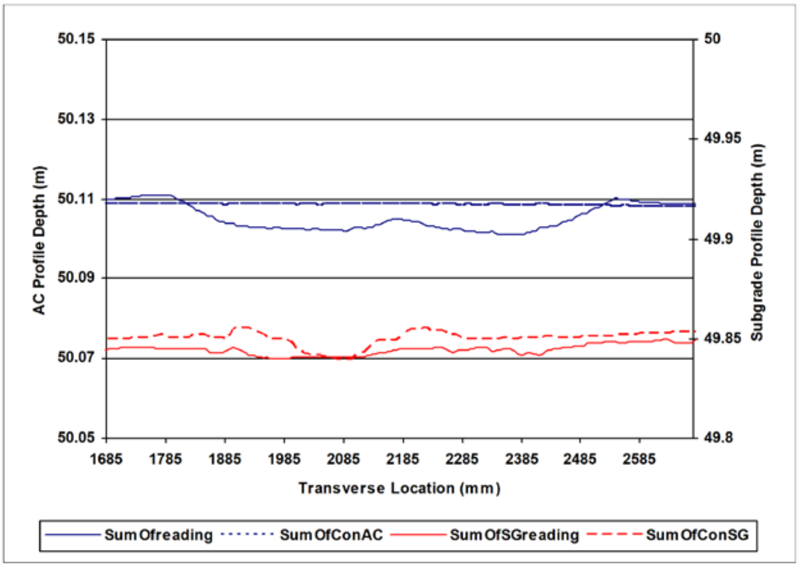

STA 43


STA 49


STA 51


STA 56

Appendix D Test 2 post-mortem trench profiles



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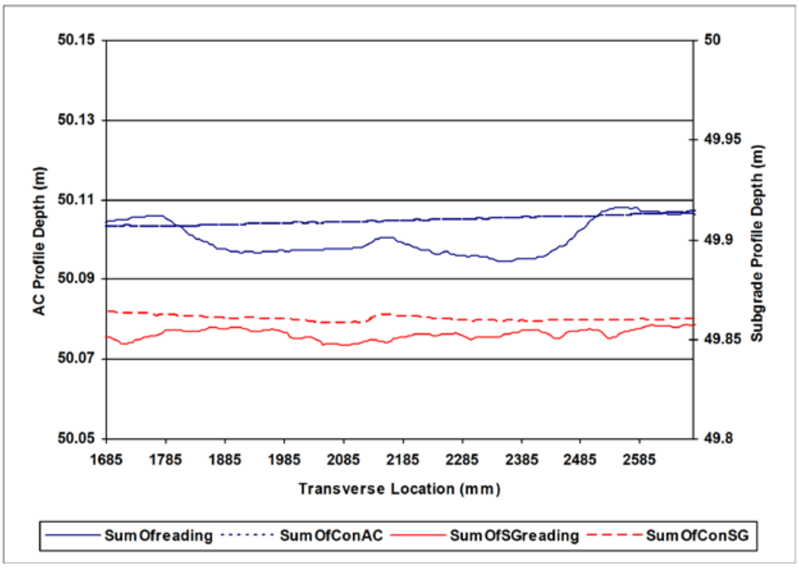

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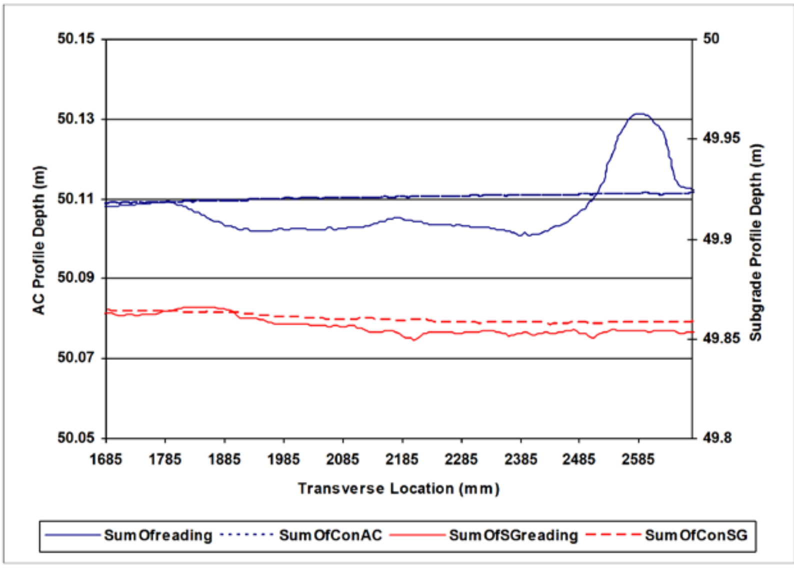

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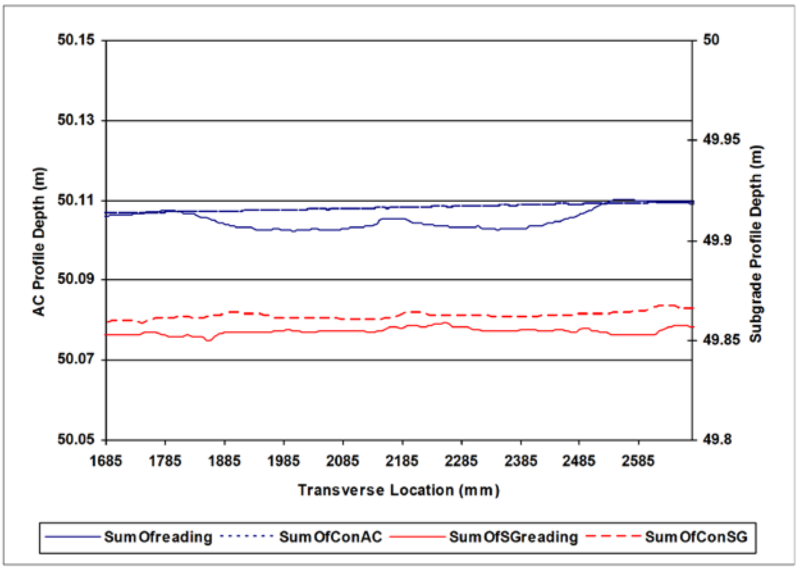

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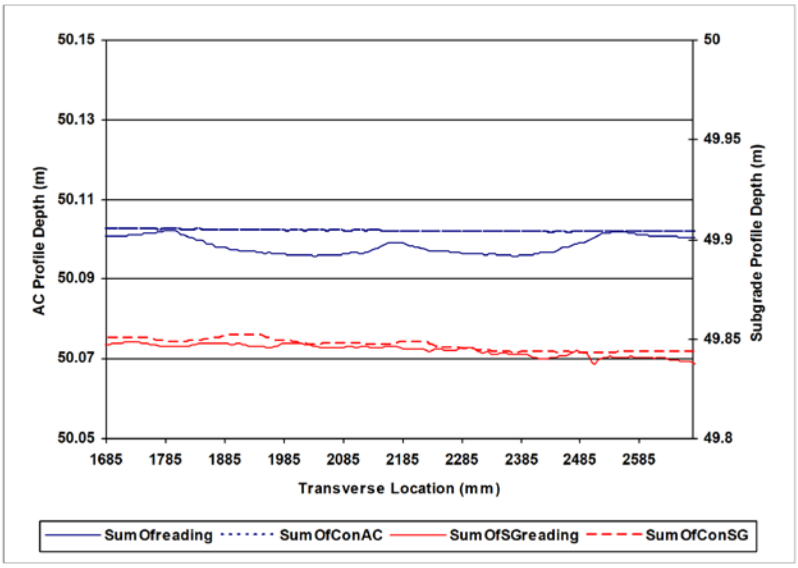

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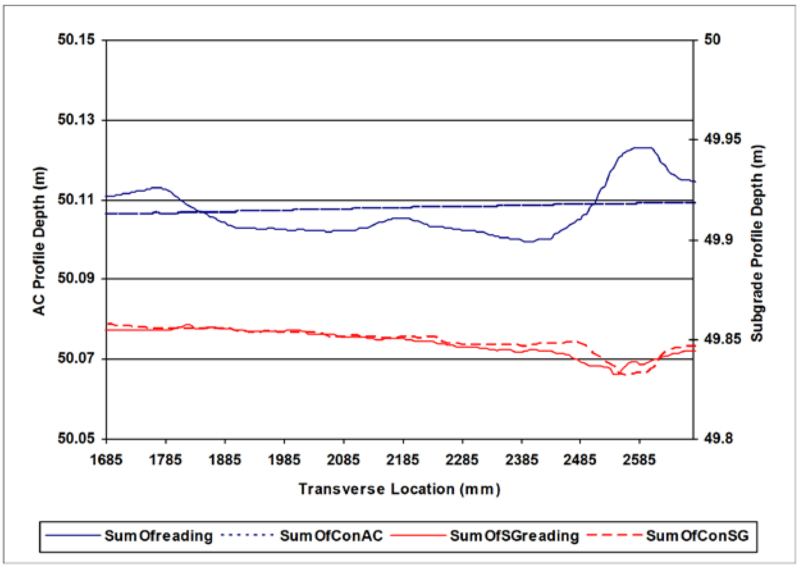

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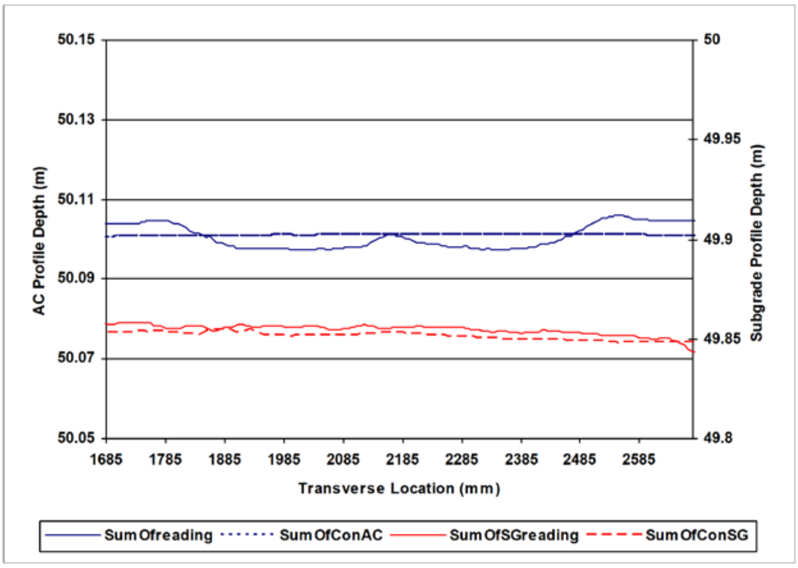

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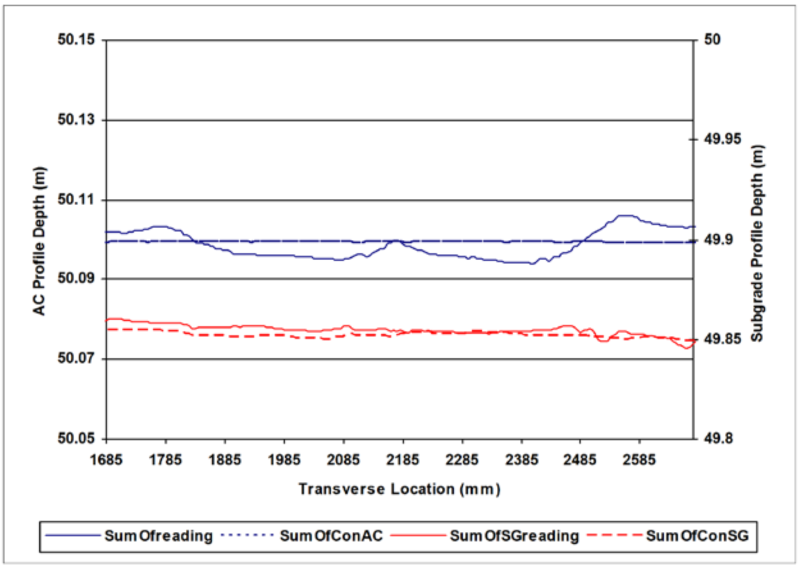

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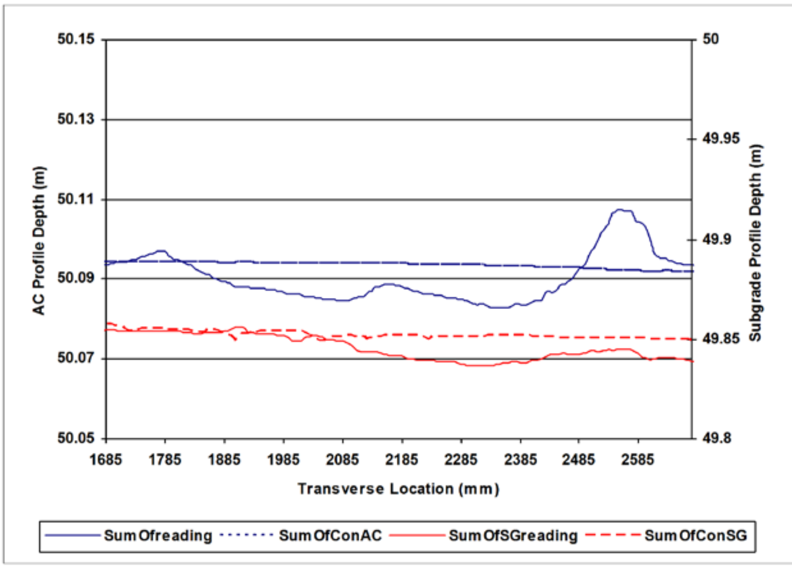

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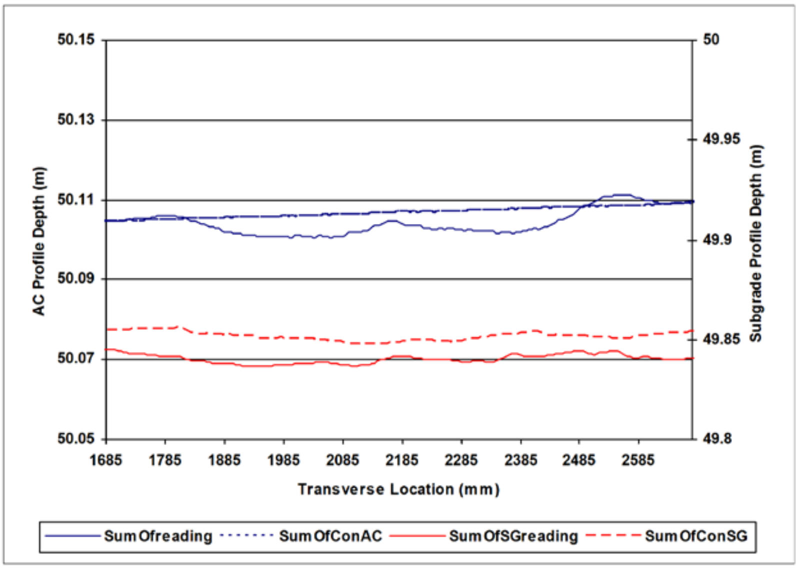

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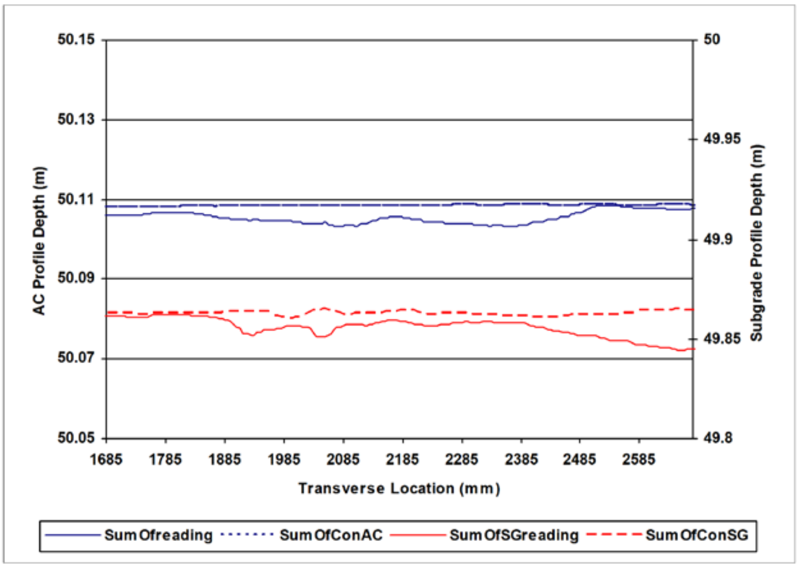

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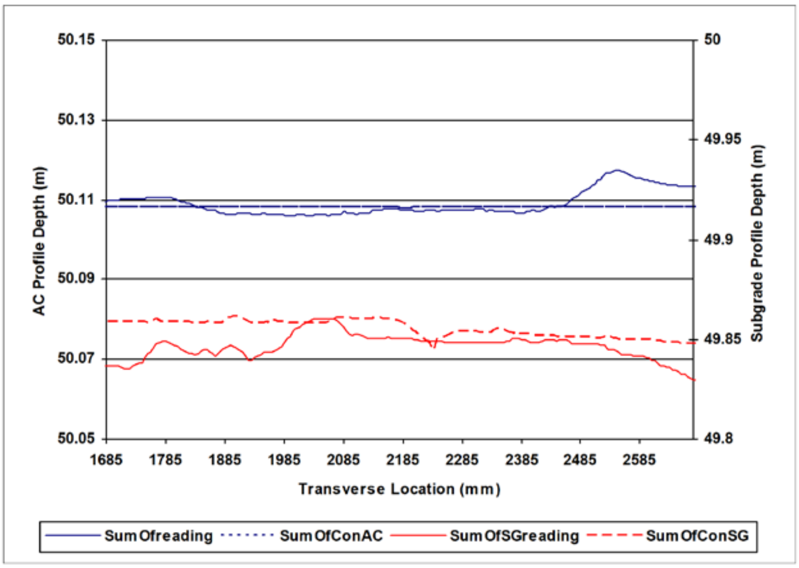

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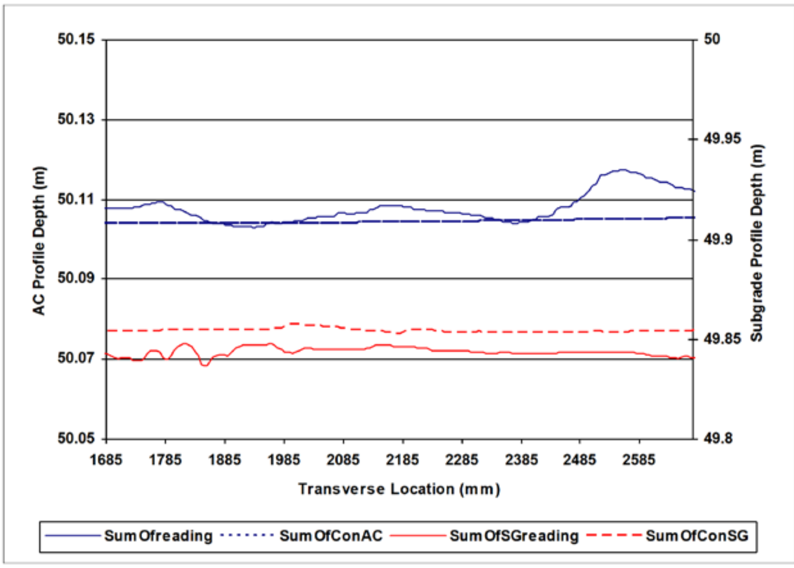

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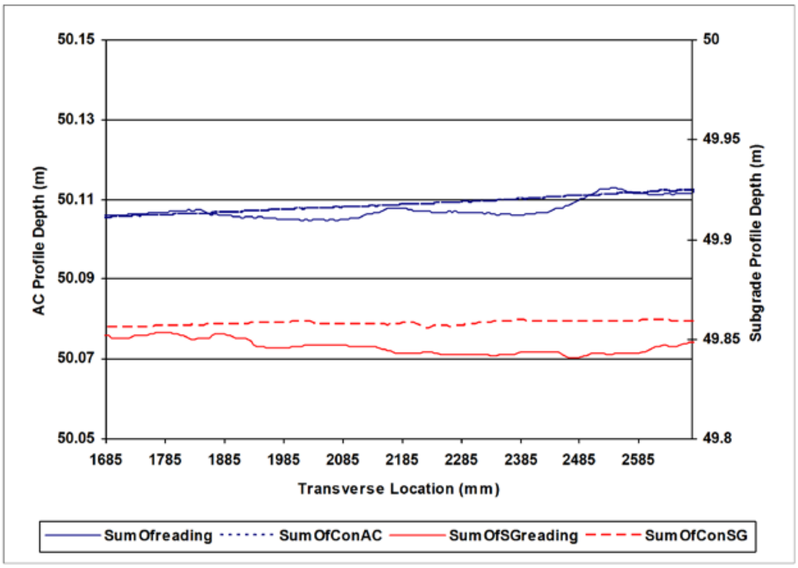

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Appendix E Field site photos

E.1 Site surface photos

Figure E.1 SH2 RS 678\12.17 – stabilised layer visible through the shoulder

Figure E.2 SH2 RS 678\12.17 – stabilised layer visible through the shoulder (a close-up view)

Figure E.3 SH2 RS 678\12.17 – a cracked pavement including shoulder

Figure E.4 SH2 RS 678\12.17 – a cracked pavement including shoulder (close-up photo)

Figure E.5 SH50 RS 17/12.31 – a few cracks appear up to the centreline from the edge

Figure E.6 SH50 RS 17/12.31 – the cracks appear to be equally spaced

Figure E.7 SH50 RS 17/12.31 – cracks have appeared between the layers

Figure E.8 SH50 RS 17/12.31 – the cracks appear to be approximately equally spaced (increasing direction)

Figure E.9 SH50 RS 17/12.31 – the cracks appear to be approximately equally spaced (decreasing direction)

Figure E.10 SH50 RS 49/1.23 – the cracks appear in longitudinal and transverse directions

Figure E.11 Gisborne local road – material used for Mangatu Road (from Mangamaia)

Figure E.12 Mangatu Road 4.25km – overall a strong shoulder

Figure E.13 Mangatu Road cross-section at 5.82km – an earth slip site (side view)

Figure E.14 Mangatu Road cross-section at 5.82km – an earth slip site (decreasing direction)

Figure E.15 Mangatu Road 8.96km

Figure E.16 Mangatu Road 8.96km (a close-up)

Figure E.17 Mangatu Road 9.29km

Figure E.18 Mangatu Road 9.29km (a close-up)

Figure E.19 Mangatu Road 9.29km – an overall view (increasing direction)

Figure E.20 Mangatu Road 10.07km (decreasing direction)

Figure E.21 Mangatu Road 10.07km (a close-up)

Figure E.22 Mangatu Road 10.07km – another pothole

Figure E.23 Mangatu Road 10.07km (increasing direction)

Figure E.24 Mangatu Road 10.07km – alligator cracks

Figure E.25 Mangatu Road 10.16km – visible fine cracks

Figure E.26 Mangatu Road 10.94km – overall strong edges

Figure E 27 Mangatu Road 10.94km – a shear and fatigue failure

Figure E.28 Mangatu Road 10.94km (a close-up)

Figure E.29 Mangatu Road 10.94km – shear failure with strong shoulder support

Figure E.30 Mangatu Road 12.43km

Figure E.31 Mangatu Road 12.43km – level difference in this crack indicates a slip in the near future)

Figure E.32 SH1 RP 554/2.711 – cracks have been bandaged

Figure E.33 SH1 RP 554/ 1.09 – cracks have appeared

Figure E.34 SH1 RP 554/1.09 –several cracks have appeared

E.2 Cores and test pit photos

Figure E.35 Core from SH2 RP 678/12.14

Figure E.36 Pit at SH2 RP 678/12.14

Figure E.37 The three pits at SH2 RP 678/12.14

Figure E.38 Mangatu Road core and the surface of the pavement at 4.260km

Figure E.39 Mangatu Road test pit at 4.260km

Figure E.40 Mangatu Road test pit and testing at 4.260km

Figure E.41 Mangatu Road surface and a core from 1st layer at 9.040km

Figure E.42 Mangatu Road surface and a core from 2nd layer at 9.040km

Figure E.43 A test pit at Mangatu Road 9.040km (yhe location of two cores)

Figure E.44 A test pit at Mangatu Road 10.16km

Figure E.45 A test pit at Mangatu Road 11km

Figure E.46 A core at Mangatu Road 11km

Figure E.47 Core obtained near the SH1 RP 554 /1.09 during late 1960

Figure E.48 A core from Cobham Dr SH1 RP 554/1.951

Figure E.49 A core pit from Cobham Drive SH1 RP 554/2.711

Figure E.50 A core from Cobham Drive SH1 RP 554/2.711

Figure E.51 A core at SH38 RP 0/1.09

Figure E.52 The 2nd layer at SH38 RP 0/1.09

Figure E.53 The core pit at SH38 RP 0/1.09

Figure E.54 The three core pits at SH38 RP 0/1.09

Figure E.55 A core from SH38 RP 0/1.49 location A

Figure E.56 2nd core from SH38 RP 0/1.49 location B

Figure E.57 A core pit at SH38 RP 0/1.49 location B

Figure E.58 An over-layered section at SH38 0/1.935

Figure E.59 A core from the over-layered section at SH38 0/1.935

Appendix F Test pit data

PAVEMENT INVESTIGATION LOG
TEST REPORT

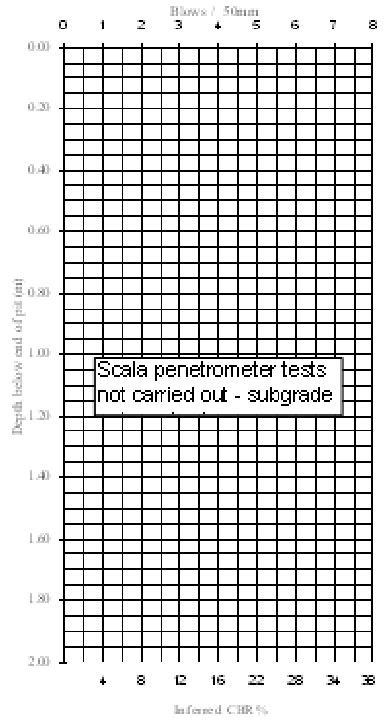
Project :	**TNZ Project - Stabilised Pavement Layers**
Location :	**SH2 - RP 678/12.14 Te Hauke**
Client :	**Transit NZ**
Client Agent:	**Haran Moorthy - Opus / Central Lab**
Sampled by :	**A.Ching**
Date Sampled :	**28/06/06**
Sampling method :	**Core drill**
Sample description :	**Cement stabilised gravel (basecourse)**

Project No :	**5-21092.00 - 003NL**
Lab Ref No :	**N06.571/1**
Client Ref No :	

Depth (mm)	Pavement Description
25	CHIPSEAL
175	GRAVEL Cement stabilised
250	GRAVEL, Grey, Crushed, Tightly packed, Moist, Well Graded 40mm max siz
450	GRAVEL, Reddish/Brown, Tightly paked, Moist, Well graded 75mm max size, Sub Rounded
	End of testpit at 450mm

Basecourse sample recovered at: 25 - 175
Subbase sample recovered at :
Subgrade sample recovered at :
Depth at which scala penetrometer started :

Scala Penetrometer



Densities	Basecourse	Subgrade
NDM results at:		
Wet density (t/m³):		
Dry density (t/m³):		
Water content %:		

Test Methods	**Notes**
Determination of Penetration Resistance of a Soil, NZS 4402 : 1988, Test 6.5.2 Insitu Density : NZS :4407 : 1991 : Test 4.2.2 for Backscatter Mode Inferred CBR values taken from Austroads pavement design manual 1992	

Date tested : 28/06/06
Date reported : 12/07/06

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Approved
A. Ching
Designation : *Laboratory Manager*
Date : 12/07/06

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PAVEMENT INVESTIGATION LOG TEST REPORT

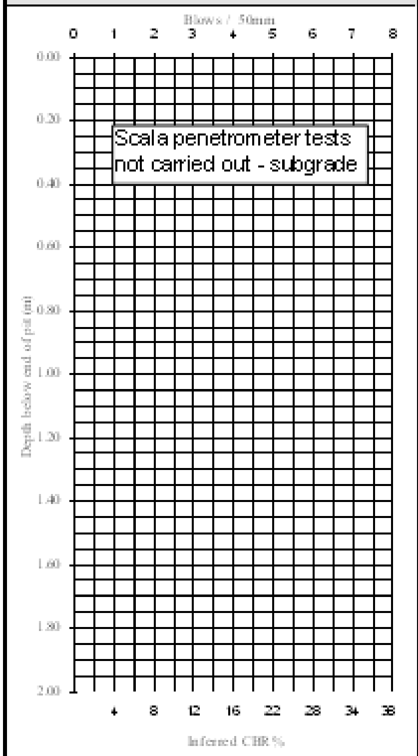
Project :	**TNZ Project - Stabilised Pavement Layers**
Location :	**SH50 - RP 49/2.658 Manga-o-nuku**
Client :	**Transit NZ**
Client Agent:	**Haran Moorthy - Opus / Central Lab**
Sampled by :	**A.Ching**
Date Sampled :	**28/06/06**
Sampling method :	**Core drill**
Sample description :	**Cement stabilised gravel (basecourse)**

Project No :	5-21092.00 - 003NL
Lab Ref No :	N06.571 / 2
Client Ref No :	

Depth (mm)	Pavement Description
12	CHIPSEAL
115	GRAVEL Cement stabilised
	End of testpit at 115mm **The actual depth of the cement stabilised layer is 125 mm**

Basecourse sample recovered at : 12 - 115
Subbase sample recovered at :
Subgrade sample recovered at :
Depth at which scala penetrometer started :

Scala Penetrometer



Densities	Basecourse	Subgrade
NDM results at :		
Wet density (t/m³):		
Dry density (t/m³):		
Water content %:		

Test Methods	Notes
Determination of Penetration Resistance of a Soil, NZS 4402 : 1988, Test 6.5.2 Insitu Density : NZS : 4407 : 1991 : Test 4.2.2 for Backscatter Mode Inferred CBR values taken from Austroads pavement design manual 1992	

Date tested : 28/ 06/ 06
Date reported : 12/ 07/ 06

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Approved
A. Ching
Designation : *Laboratory Manager*
Date : 12/ 07/ 06

CSF 2067 (04/ 05/ 2004)

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PAVEMENT INVESTIGATION LOG TEST REPORT

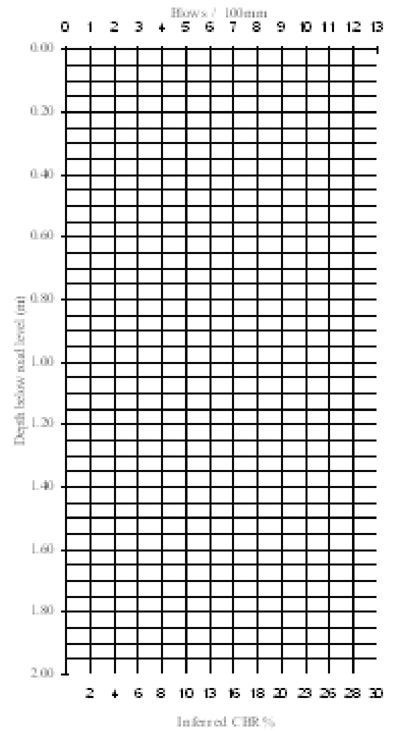
Project : **The Design of Stabilised Pavements in New Zealand**
Location : **Mangatu Road RP 4.260 LHS OWT, Gisborne**
Client : **Haran A Moorthy, Opus Central Laboratory**
Contractor : **Opus Laboratory Gisborne**
Sampled by : **Opus Laboratory Gisborne**
Date Sampled : **29/06/06**
Sampling method : **NZS : 4407 : 2.4.8.2**

Project No :	**52109200.002GL**
Lab Ref No :	**06/284**
Client Ref :	**Haran Moorthy**

Depth (mm)	Pavement Description Width 6.5m
0	Chipseal - Slightly Flushed
20	Grey Sandy RIVER GRAVEL (heavily stabilised) Very Dense; Dry; Non Plastic Max Size AP20; Well Graded; Partially Crushed
150	Grey Sandy RIVER GRAVEL (stabilised) Dense; Dry; Non Plastic Max Size AP50; Well Graded; Uncrushed
480	Grey Sandy RIVER GRAVEL (unstabilised) Dense; Dry; Non Plastic Max Size AP50; Well Graded; Uncrushed
540	Subgrade not encountered
	Comment: Refusal of Scala Penetrometer at 500mm

Basecourse sample recovered at :	-
Subbase sample recovered at :	-
Subgrade sample recovered at :	-
Depth at which scala penetrometer started :	500mm

Scala Penetrometer



Densities

	Basecourse	Subgrade
NDM results at:	-	-
Wet density (t/m^3):	-	-
Dry density (t/m^3):	-	-
Water content %:	-	-

Test Methods	Notes
Determination of Penetration Resistance of a Soil, NZS4402 : 1988, Test 6.5.2 Insitu Density : NZS : 4407 : 1991 : Test 4.2.2 for Backscatter Mode Inferred CBR values taken from Austroads pavement design manual 1992	

Date tested : 29/06/06
Date reported : 27/07/06

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Peter Carlyle
Laboratory Manager

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PAVEMENT INVESTIGATION LOG
TEST REPORT

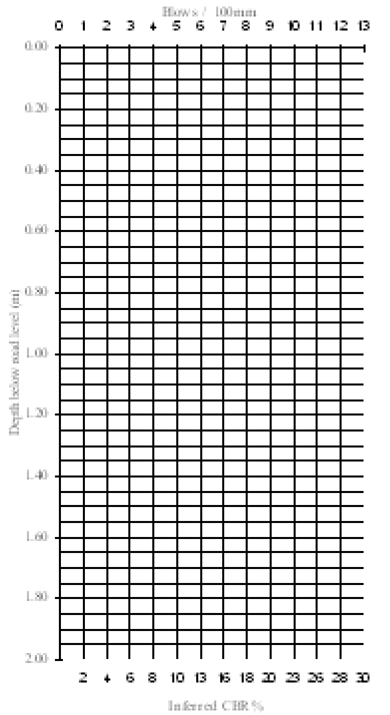
Project :	**The Design of Stabilised Pavements in New Zealand**
Location :	**Mangatu Road RP 9.040 RHS OWT, Gisborne**
Client :	**Haran A Moorthy, Opus Central Laboratory**
Contractor :	**Opus Laboratory Gisborne**
Sampled by :	**Opus Laboratory Gisborne**
Date Sampled :	**29/06/06**
Sampling method :	**NZS : 4407 : 2.4.8.2**

Project No :	**52109200.002GL**
Lab Ref No :	**06/284**
Client Ref :	**Haran Moorthy**

Depth (mm)	Pavement Description	Width 6.6m
0	Chipseal - Flushed	
15	Grey Sandy RIVER GRAVEL (heavily stabilised) Very Dense; Dry; Non Plastic Max Size AP20; Well Graded; Partially Crushed	
160	Grey Sandy RIVER GRAVEL (stabilised) Dense; Dry; Non Plastic Max Size AP50; Well Graded; Uncrushed	
500	Subgrade not encountered	
	Comment: Refusal of Scala Penetrometer at 500mm	

Basecourse sample recovered at :	-
Subbase sample recovered at :	-
Subgrade sample recovered at :	-
Depth at which scala penetrometer started :	500mm

Scala Penetrometer



Densities

	Basecourse	Subgrade
NDM results at :	-	-
Wet density (t/ m³):	-	-
Dry density (t/m³):	-	-
Water content %:	-	-

Test Methods	Notes
Determination of Penetration Resistance of a Soil, NZS 4402 : 1988, Test 6.5.2 Insitu Density : NZS : 4407 : 1991 : Test 4.2.2 for Backscatter Mode Inferred CBR values taken from Austroads pavement design manual 1992	

Date tested : 29/ 06/ 06
Date reported : 27/ 07/ 06

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CSF 2067 (08/ 04/ 03)

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PAVEMENT INVESTIGATION LOG
TEST REPORT

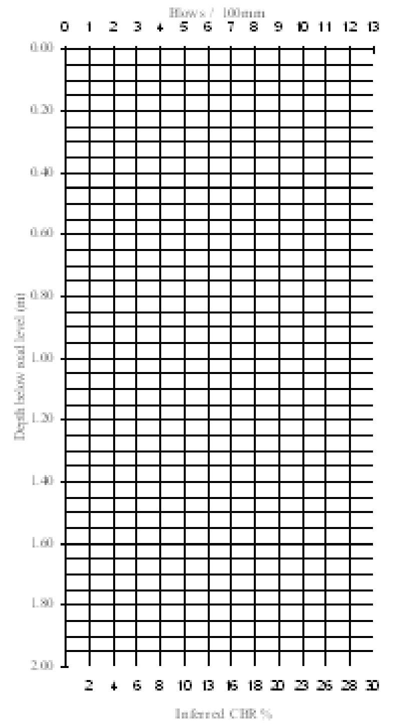
Project :	**The Design of Stabilised Pavements in New Zealand**
Location :	**Mangatu Road RP 10 160 RHS OWT, Gisborne**
Client :	**Haran A Moorthy, Opus Central Laboratory**
Contractor :	**Opus Laboratory Gisborne**
Sampled by :	**Opus Laboratory Gisborne**
Date Sampled :	**29/06/06**
Sampling method :	**NZS : 4407 : 2.4.8.2**

Project No :	**52109200.002GL**
Lab Ref No :	**06/284**
Client Ref :	**Haran Moorthy**

Depth (mm)	Pavement Description — Width 6.5m
0	Chipseal - Flushed; Rutted
20	Grey Sandy RIVER GRAVEL (stabilised) Dense; Damp; Non Plastic Max Size AP20; Well Graded; Partially Crushed
190	Whitish Grey Clayey SILTSTONE (w/ some 20mm River Gravels) Medium Dense; Damp; Plastic Max Size AP65; Gap Graded (excessive fines); Partially Crushed
340	Tan SILT Hard; Damp; Non Plastic Comment: Refusal of Scala Penetrometer at 300mm

Basecourse sample recovered at :	-
Subbase sample recovered at :	-
Subgrade sample recovered at :	-
Depth at which scala penetrometer started :	300mm

Scala Penetrometer



Densities

	Basecourse	Subgrade
NDM results at :	-	-
Wet density (t/m³):	-	-
Dry density (t/m³):	-	-
Water content %:	-	-

Test Methods	**Notes**
Determination of Penetration Resistance of a Soil, NZS 4402 : 1988, Test 6.5.2 Insitu Density : NZS : 4407 : 1991 : Test 4.2.2 for Backscatter Mode Inferred CBR values taken from Austroads pavement design manual 1992	

Date tested : 29/06/06
Date reported : 27/07/06

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Approved

Peter Carlyle
Laboratory Manager

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PAVEMENT INVESTIGATION LOG
TEST REPORT

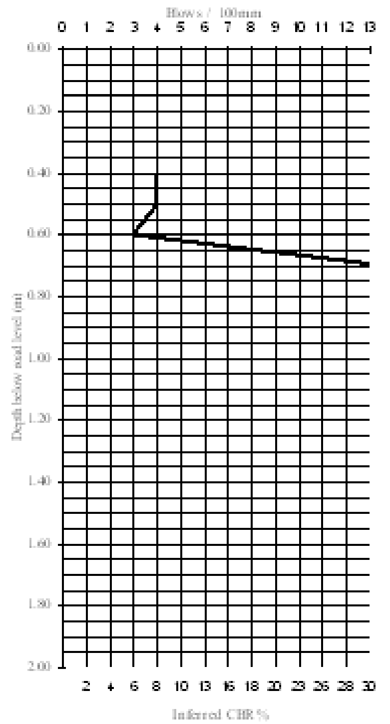
Project :	**The Design of Stabilised Pavements in New Zealand**
Location :	**Mangatu Road RP 11.000 LHS OWT, Gisborne**
Client :	**Haran A Moorthy, Opus Central Laboratory**
Contractor :	**Opus Laboratory Gisborne**
Sampled by :	**Opus Laboratory Gisborne**
Date Sampled :	**29/06/06**
Sampling method :	**NZS : 4407 : 2.4.8.2**

Project No :	**52109200.002GL**
Lab Ref No :	**06/284**
Client Ref :	**Haran Moorthy**

Depth (mm)	Pavement Description — Width 6.7m
0	Chipseal - Slightly Flushed
20	Grey Sandy RIVER GRAVEL (stabilised) Dense; Damp; Non Plastic Max Size AP20; Well Graded; Partially Crushed
160	Whitish Grey Clayey SILTSTONE (w/ some 20mm River Gravels) Medium Dense; Damp; Plastic Max Size AP65; Gap Graded (excessive fines); Partially Crushed
380	Brown Clayey SILT (w/ SILTSTONE) Stiff; Damp; Slightly Plastic
	Comment: Refusal of Scala Penetrometer at 700mm

Basecourse sample recovered at :	-
Subbase sample recovered at :	-
Subgrade sample recovered at :	-
Depth at which scala penetrometer started :	400mm

Scala Penetrometer



Densities

	Basecourse	Subgrade
NDM results at:	-	-
Wet density (t/m^3):	-	-
Dry density (t/m^3):	-	-
Water content %:	-	-

Test Methods	**Notes**
Determination of Penetration Resistance of a Soil, NZS 4402 : 1988, Test 6.5.2 Insitu Density : NZS : 4407 : 1991 : Test 4.2.2 for Backscatter Mode Inferred CBR values taken from Austroads pavement design manual 1992	

Date tested : 29/06/06
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Laboratory Manager

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F.1 Notes and data relating to cores at Cobham Drive and SH38

F.1.1 SH1 Cobham Drive site 2

The southern-most site was done first. This was at SH1 Cobham Drive RP 554/2.711 – also referred to as site 2. Three cores were drilled here, the initial core (2a) being drilled on the location, then the second core (2b) drilled 5m further at RP 554/2.716. The third core, core 2c, was drilled at RP 554/2.721. All cores were drilled in the right wheel path on the left lane.

The site from which core 2a was taken was used for a Scala test. The results are shown on the appropriate results sheet. An auger was done to achieve a total depth of 700mm. Results of the log are as follows.

Table F.1 Log of site 2 Cobham Drive

	Friction mix	35mm
	PMB friction mix	30mm
	Mix 10	50mm
Scala ↓	CTB	160mm
	Pit sand	

F.1.2 SH1 Cobham Drive site 1

This site was located at RP 554/1.951. The core was drilled in the left wheel path in the left lane. After the drill penetrated into the CTB, it became jammed and then broke, the holding bolts being ripped from the drill mounting and thus ripping out the wiring. The hole was completed with the jack hammer to the interface of the pit sand and CTB. A Scala test was done at this point.

Further auguring was then done to a total depth of 700mm, at which depth a clay silt change was noticed. Because of this change, a small Scala was done at this depth too. This site was logged as follows.

Table F.2 Log of site 1 Cobham Drive

	Friction course	35mm
	PMB friction course	30mm
	Mix 10	55mm
Scala 1🡻	CTB	170mm
	Sand	210mm
Scala 2🡻	Clay/silt	30mm

No further testing could be done on this site because of drill breakage.

F.1.3 SH38

On this location, Rotorua Concrete Cutting Services were used to cut the cores. Traffic management was undertaken by 50-Fifty Traffic Management Ltd. The first site was tested after measurement from the intersection, and then relocated using the FWD marks.

F.1.3.1 Site 1 SH38 RP0/1.09

Each core disintegrated on being retrieved from the barrel, as they each showed signs of weakness. As the pieces broke off and the depth of the hole was soon exceeded by the barrel length, a smaller 100mm diameter barrel was used.

Table F.3 Log of site 1 SH38 RP 0/1.09 (1A)

	Chip seals	30mm
	CTB layer 1	140mm
Scala 🡻	CTB layer 2	180mm
	Coarse pumice sand	150mm

Table F.4 Log of site 1 SH38 RP 0/1.09 (1C)

	Chip seals	40mm
	CTB layer 1	140mm
Scala ↓	CTB layer 2	160mm
	Coarse pumice sand	

F.1.3.2 Site 2 SH38 RP0/1.49

The problems with core recovery continued with these holes.

Coring was not done in sequential order but in the pattern as shown, with about 2m between each location.

Table F.5 Log of site 2 SH38 RP 0/1.49 (2B)

← Murapara

2B 2A 2C

	Chipseals	40mm
	Harder CTB	150mm
Scala ↓	Softer CTB	150mm
	Coarse pumice sand	200mm

F.1.3.3 Site 3 S 38 RP0/1.81

This location was almost opposite the entrance to the Waipa Mill. The pavement had been overlaid and again, core recovery was difficult due to the weakness of the material. Measurement was taken from FWD marks.

Table F.6 Log of site 3 SH38 RP 0/1.81 (3A)

	Chipseal	20mm
	Basecourse 25mm?	120mm
	Chipseal	40mm
Scala ↓	CTB	80mm
	Dense coarse pumice sand	

After this site was cored another site 20 metres towards SH5 was chosen, basically because of the seal change. The first core showed similar material and layers as Site 3 and no further cores were taken.

Appendix G Design traffic calculations

DESIGN TRAFFIC CALCULATIONS FOR SH2 RP 678/12.17

Box 1 - Design Parameters

NB Data is required to be inputted in all yellow cells

Design period (yrs)	(P)	13	
Annual average daily traffic	(AADT)	3664	
Direction factor	(DF)	0.5	*refer Austroads Section 7.4.1*
% heavy vehicles	(%HV)	13%	
Lane distribution factor	(LDF)	1	*refer Austroads Table 7.3*
Heavy vehicle growth rate	(R)	4%	
Cumulative growth factor	(CGF)	13.0	*AUSTROADS Section 7.4.5*
Arithmetic growth factor	(AGF)	13.0	*NZ Supplement - Section 7.1*
No. of axle groups per heavy vehicle	(N_{HVAG})	2.4	*Presumptive value from NZ Supplement*

Box 2 - Total no. of heavy vehicle axle groups (N_{DT})

		Cumulative	Arithmetic
N_{DT} = 365 x (AADT*DF) x %HV/100 x LDF x CGF x N_{HVAG}	N_{DT} =	2.7E+04	2.72E+04

Box 3 - Design traffic for flexible pavements

Presumptive Damage Index Values
from NZ Supplement Appendix 7.4
using Table 7.4.1 traffic distribution

Damage Index	Value
ESA/HVAG	0.6
ESA/HV	1.4
SARa/ESA	1
SARs/ESA	1.2
SARc/ESA	3.6

Design no. of ESA of loading (DESA) = ESA/HVAG x N_{DT}

		Cumulative	Arithmetic
	DESA =	1.6E+04	1.6E+04

The DESA is the design traffic for unbound granular pavements with thin bituminous surfacings

(AUSTROADS figure 8.4)

For other flexible pavements, the design
number of Standard Axle Repetitions (DSAR)
for each distress type needs to be estimated

		Cumulative	Arithmetic
DSARa = DESA x SARa/ESA	DSARa =	1.6E+04	1.6E+04
DSARs = DESA x SARs/ESA	DSARs =	2.0E+04	2.0E+04
DSARc = DESA x SARc/ESA	DSARc =	5.9E+04	5.9E+04

DESIGN TRAFFIC
CALCULATIONS FOR SH50 RP
49/2.38

Box 1 - Design parameters

NB Data is required to be inputted in all yellow cells

Design period (yrs)	(P)	13	
Annual average daily traffic	(AADT)	1229	
Direction factor	(DF)	0.5	*refer Austroads Section 7.4.1*
% heavy vehicles	(%HV)	10	
Lane distribution factor	(LDF)	1	*refer Austroads Table 7.3*
Heavy vehicle growth rate	(R)	3	
Cumulative growth factor	(CGF)	15.6	*AUSTROADS Section 7.4.5*
Arithmetic growth factor	(AGF)	15.5	*NZ Supplement - Section 7.1*
No. of axle groups per heavy vehicle	(N_{HVAG})	2.4	*Presumptive value from NZ Supplement*

Box 2 - Total no. of heavy vehicle axle groups (N_{DT})

N_{DT} = 365 x (AADT*DF) x %HV/100 x LDF x CGF x N_{HVAG}

	Cumulative	Arithmetic
N_{DT} =	8.4E+05	8.36E+05

Box 3 - Design traffic for flexible pavements

Presumptive Damage Index
Values from NZ Supplement
Appendix 7.4 using Table 7.4.1
traffic distribution

Damage Index	Value
ESA/HVAG	0.6
ESA/HV	1.4
SARa/ESA	1
SARs/ESA	1.2
SARc/ESA	3.6

Design no. of ESA of loading (DESA) = ESA/HVAG x N_{DT}

	Cumulative	Arithmetic
DESA =	5.0E+05	5.0E+05

The DESA is the design traffic for unbound granular pavements with thin bituminous surfacings (AUSTROADS figure 8.4)

For other flexible pavements, the design number of Standard Axle Repetitions (DSAR) for each distress type needs to be estimated

DSARa = DESA x SARa/ESA

DSARs = DESA x SARs/ESA

DSARc = DESA x SARc/ESA

	Cumulative	Arithmetic
DSARa =	5.0E+05	5.0E+05
DSARs =	6.1E+05	6.0E+05
DSARc =	1.8E+06	1.8E+06

DESIGN TRAFFIC CALCULATIONS FOR MANGATU ROAD

Box 1 - Design parameters

NB Data is required to be inputted in all yellow cells

Design period (yrs)	(P)	16	
Annual average daily traffic	(AADT)	200	
Direction factor	(DF)	0.5	*refer Austroads Section 7.4.1*
% heavy vehicles	(%HV)	50	
Lane distribution factor	(LDF)	1	*refer Austroads Table 7.3*
Heavy vehicle growth rate	(R)	3	
Cumulative growth factor	(CGF)	20.2	*AUSTROADS Section 7.4.5*
Arithmetic growth factor	(AGF)	19.8	*NZ Supplement - Section 7.1*
No. of axle groups per heavy vehicle	(N_{HVAG})	2.4	*Presumptive value from NZ Supplement*

Box 2 - Total no. of heavy vehicle axle groups (N_{DT})

N_{DT} = 365 x (AADT*DF) x %HV/100 x LDF x CGF x N_{HVAG}

	Cumulative	Arithmetic
N_{DT} =	8.8E+05	8.69E+05

Box 3 - Design traffic for flexible pavements

Presumptive Damage Index Values
from NZ Supplement Appendix 7.4
using Table 7.4.1 traffic distribution

Damage Index	Value
ESA/HVAG	0.6
ESA/HV	1.4
SARa/ESA	1
SARs/ESA	1.2
SARc/ESA	3.6

		Cumulative	Arithmetic
Design no. of ESA of loading (DESA) = ESA/HVAG x N_{DT}	DESA =	5.3E+05	5.2E+05

The DESA is the design traffic for unbound granular pavements with thin bituminous surfacings

(AUSTROADS figure 8.4)

For other flexible pavements, the design number of Standard Axle Repetitions (DSAR) for each distress type needs to be estimated

		Cumulative	Arithmetic
DSARa = DESA x SARa/ESA	DSARa =	5.3E+05	5.2E+05
DSARs = DESA x SARs/ESA	DSARs =	6.4E+05	6.3E+05
DSARc = DESA x SARc/ESA	DSARc =	1.9E+06	1.9E+06

DESIGN TRAFFIC CALCULATIONS FOR
SH1 554/9.71

Box 1 - Design parameters

NB Data is required to be inputted in all yellow cells

Design period (yrs)	(P)	40	
Annual average daily traffic	(AADT)	2000	
Direction factor	(DF)	0.5	*refer Austroads Section 7.4.1*
% heavy vehicles	(%HV)	11	
Lane distribution factor	(LDF)	1	*refer Austroads Table 7.3*
Heavy vehicle growth rate	(R)	3	
Cumulative growth factor	(CGF)	75.4	*AUSTROADS Section 7.4.5*
Arithmetic growth factor	(AGF)	64.0	*NZ Supplement - Section 7.1*
No. of axle groups per heavy vehicle	(NHVAG)	2.4	*Presumptive value from NZ Supplement*

Box 2 - Total no. of heavy vehicle axle groups (NDT)

		Cumulative	Arithmetic
NDT = 365 x (AADT*DF) x %HV/100 x LDF x CGF x NHVAG	NDT =	7.3E+06	6.17E+06

Box 3 - Design traffic for flexible pavements

Damage index	Value
ESA/HVAG	0.6
ESA/HV	1.4
SARa/ESA	1
SARs/ESA	1.2
SARc/ESA	3.6

		Cumulative	Arithmetic
Design no. of ESA of loading (DESA) = ESA/HVAG x NDT	DESA =	4.4E+06	3.7E+06

(AUSTROADS figure 8.4)

type needs to be estimated

		Cumulative	Arithmetic
DSARa = DESA x SARa/ESA	DSARa =	4.4E+06	3.7E+06
DSARs = DESA x SARs/ESA	DSARs =	5.2E+06	4.4E+06
DSARc = DESA x SARc/ESA	DSARc =	1.6E+07	1.3E+07

DESIGN TRAFFIC CALCULATIONS FOR SH38 RP 0/1.09

Box 1 - Design parameters

NB Data is required to be inputted in all yellow cells

Design period (yrs)	(P)	41	
Annual average daily traffic/lane	(AADT)	465	
Direction factor	(DF)	1	*refer Austroads Section 7.4.1*
% heavy vehicles	(%HV)	10%	
Lane distribution factor	(LDF)	1	*refer Austroads Table 7.3*
Heavy vehicle growth rate	(R)	3	
Cumulative growth factor	(CGF)	78.7	*AUSTROADS Section 7.4.5*
Arithmetic growth factor	(AGF)	66.2	*NZ Supplement - Section 7.1*
No. of axle groups per heavy vehicle	(N_{HVAG})	2.4	*Presumptive value from NZ Supplement*

Box 2 - Total no. of heavy vehicle axle groups (N_{DT})

N_{DT} = 365 x (AADT*DF) x %HV/100 x LDF x CGF x N_{HVAG}

	Cumulative	Arithmetic
N_{DT} =	3.2E+04	2.70E+04

Box 3 - Design traffic for flexible pavements

Presumptive Damage Index Values from NZ Supplement Appendix 7.4 using Table 7.4.1 traffic distribution

Damage Index	Value
ESA/HVAG	0.6
ESA/HV	1.4
SARa/ESA	1
SARs/ESA	1.2
SARc/ESA	3.6

		Cumulative	Arithmetic
Design no. of ESA of loading (DESA) = ESA/HVAG x N_{DT}	DESA =	1.9E+04	1.6E+04

The DESA is the design traffic for unbound granular pavements with thin bituminous surfacings

(AUSTROADS figure 8.4)

For other flexible pavements, the design number of Standard Axle Repetitions (DSAR) for each distress

type needs to be estimated

		Cumulative	Arithmetic
DSARa = DESA x SARa/ESA	DSARa =	1.9E+04	1.6E+04
DSARs = DESA x SARs/ESA	DSARs =	2.3E+04	1.9E+04
DSARc = DESA x SARc/ESA	DSARc =	6.9E+04	5.8E+04

Appendix H Models using AUSTROADS pavement design method

Table H.1 SH2 RP 678/12.17

CIRCLY MODEL					
INPUT				OUTPUT	Service life
Multiplier	Material type	ID	Thick (mm)	CDF	
	Method I: Presumed value				
3.6	Cement stabilised	Cement2000	150	2.46E+03	1.10E+01
	Unbound granular04	Gran_500	75		
1.2	Subgrade (Austroads 2004)	Sub_CBR10	0	4.12E-04	6.55E+07
	Method II: Modulus obtained from IT test				
3.6	Cement stabilised	Cement40300	150	1.64E-04	1.65E+08
	Unbound granular04	Gran_500	75		
1.2	Subgrade (Austroads 2004)[a]	Sub_CBR10	0	6.96E-09	3.88E+12

a) Assumed as subgrade soil.

Table H.2 SH50 RP 49/2.38

CIRCLY MODEL					
INPUT				OUTPUT	Service life
Multiplier	Material type	ID	Thick (mm)	CDF	
	Method I: Presumed value				
3.6	Cement stabilised	Cement5000	125	8.88E+04	9.46E+00
	Unbound granular04	Gran_500	75		
1.2	Subgrade (Austroads 2004)	Sub_CBR10	0	3.34E-03	2.51E+08
	Method II: Modulus obtained from IT test				
3.6	Cement stabilised	Cement34500	125	3.87E-01	2.17E+06
	Unbound granular04	Gran_500	75		
1.2	Subgrade (Austroads 2004)	Sub_CBR10	0	2.70E-06	3.11E+11

Table H.3 SH1 RP 554/1.71

CIRCLY MODEL					
INPUT				OUTPUT	Service life
Multiplier	Material type	ID	Thick (mm)	CDF	
	Method I: Presumed value				
1	Asphalt	Asph2000	120	9.28E-05	1.62E+11
3.6	Cement stabilised	Cement2000	170	2.85E+00	5.26E+06
	Unbound granular04	Gran_700	210		
1.2	Subgrade (Austroads 2004)	Sub_CBR12	0	6.28E-05	2.39E+11
	Method II: Modulus obtained from IT test				
1	Asphalt	Asph2000	120	1.50E-30	1.00E+37
3.6	Cement stabilised	Cement19200	170	8.82E-03	1.70E+09
	Unbound granular04	Gran_700	210		
1.2	Subgrade (Austroads 2004)	Sub_CBR12	0	8.47E-07	1.77E+13

Table H.4 SH38 RP 0/1.09

CIRCLY MODEL					
INPUT				OUTPUT	Service life
Multiplier	Material type	ID	Thick (mm)	CDF	
	Method I: Presumed value				
3.6	Cement stabilised	Cement5000	310	6.26E-07	5.11E+10
	Unbound04	Gran_650	0		
	Method II: Unable to get modulus from IT test				

Table H.5 SH38 RP 0/1.49

CIRCLY MODEL					
INPUT				OUTPUT	Service life
Multiplier	Material type	ID	Thick (mm)	CDF	
	Method I: Presumed value				
3.6	Cement stabilised	Cement2000	300	2.11E-06	1.52E+10
	Unbound04	Gran_650	0		
	Method II:Unable to get modulus from IT test				

Table H.6 Mangatu Rd 4.3km

CIRCLY MODEL					
INPUT				OUTPUT	Service life
Multiplier	Material type	ID	Thick (mm)	CDF	
	Method I: Presumed value				
3.6	Cement stabilised	Cement10000	130	1.89E-04	4.66E+09
3.6	Cement stabilised	Cement2000	330	9.48E-05	9.28E+09
	Unbound gran(AU 2004)	Gran_100	60		
1.2	Subgrade (select Mat)	subsltCB10	0	4.78E-08	1.84E+13
	Method II: Modulus obtained from IT test				
3.6	Cement stabilised	Cement11900	130	9.05E-05	9.72E+09
3.6	Cement stabilised	Cement2000	330	7.03E-05	1.25E+10
	Unbound gran(AU 2004)	Gran_100	60		
1.2	Subgrade (select Mat)	subsltCB10	0	3.96E-08	2.22E+13

Table H.7 Mangatu Rd 9.001km

CIRCLY MODEL					
INPUT				OUTPUT	Service life
Multiplier	Material type	ID	Thick (mm)	CDF	
	Method I: Presumed value				
3.6	Cement stabilised	Cement10000	145	6.32E-05	1.39E+10
3.6	Cement stabilised	Cement2000	340	3.15E-05	2.79E+10
1.2	Subgrade (select Mat)	subsltCB10	0	4.23E-08	2.08E+13
	Method II: Modulus obtained from IT test				
3.6	Cement stabilised	Cement12200	145	3.66E-08	2.40E+13
3.6	Cement stabilised	Cement3500	340	8.42E-06	1.05E+11
1.2	Subgrade (select Mat)	subsltCB10	0	6.15E-09	1.43E+14

Table H.8 Mangatu Rd 10.12km

CIRCLY MODEL					
INPUT				OUT PUT	Service Life
Multiplier	Material type	ID	Thick (mm)	CDF	
	Method I:Presumed value				
3.6	Cement Stabilised	Cement2000	170	7.68E+04	1.15E+01
1.2	Subgrade (Selected Mate)	subsltCB10	150	8.17E-03	1.08E+08
1.2	Subgrade (Isotropic)	cbr6	0	4.11E+00	2.14E+05
	Method II: Modulus obtained from IT test				
3.6	Cement Stabilised	Cement8200	170	2.14E+02	4.11E+03
1.2	Subgrade (Selected Mate)	subsltCB10	150	2.39E-05	3.68E+10
1.2	Subgrade (Isotropic)	cbr6	0	2.56E-02	3.44E+07

Table H.9 Mangatu Rd 11km

CIRCLY MODEL					
INPUT				OUTPUT	
Multiplier	Material type	ID	Thick (mm)	CDF	Service life
	Method I: Presumed value				
3.6	Cement stabilised	Cement2000	140	1.47E+06	5.99E-01
1.2	Subgrade (selected mate)	subsltCB10	160	6.08E-02	1.45E+07
1.2	Subgrade (isotropic)	cbr6	0	1.86E+01	4.73E+04
	Method II: Modulus obtained from IT test				
3.6	Cement stabilised	Cement8200	140	5.17E+03	1.70E+02
1.2	Subgrade (selected mate)	subsltCB10	160	2.02E-04	4.36E+09
1.2	Subgrade (isotropic)	cbr6	0	1.77E-01	4.97E+06

Appendix I Models using South African pavement design method

Table I.1 SH2 RP 678/12.17

PADS MODEL					
INPUT				OUTPUT	
Poisson's ratio	Material type	Modulus (MPa)	Thick (mm)	Service life	Cemented life
	Method I: Presumed value				
0.35	C1	2000	150	2.48E+06	3.63E+05
0.35	G2	500	75	9.60E+05	
0.35	Soil[a]	100	0	1.67E+08	

a) Assumed as subgrade soil.

Table I.2 SH50 RP 49/2.38

PADS MODEL					
INPUT				OUTPUT	
Posisson's ratio	Material type	Modulus (MPa)	Thick (mm)	Service Life	Cemented life
	Method I:Presumed value				
0.35	C1	5000	125	1.31E+07	5.65E+05
0.35	G1	500	75	9.18E+05	
0.35	Soil	100	0	6.91E+08	

Table I.3 SH1 RP 554/1.71

PADS MODEL					
INPUT				OUTPUT	
Poisson's ratio	Material type	Modulus (MPa)	Thick (mm)	Service life	Cemented life
	Method I:Presumed value				
0.35	AC	2000	120	2.89E+06	
0.35	C2	2000	170	1.17E+10	2.86E+06
0.35	G1	700	210	2.78E+11	
0.35	Soil	120	0	1.20E+12	

Table I.4 SH38 RP 0/1.09

PADS MODEL					
INPUT				OUTPUT	
Poisson's ratio	Material type	Modulus (MPa)	Thick (mm)	Service life	Cemented life
	Method I:Presumed value				
0.35	C1	5000	310	1.31E+07	1.31E+07
0.35	G1	650	0	1.00E+15	

Table I.5 SH 38 RP 0/1.49

PADS MODEL					
INPUT				OUTPUT	
Poisson's ratio	Material type	Modulus (MPa)	Thick (mm)	Service life	Cemented life
	Method I:Presumed value				
0.35	C1	2000	300	9.12E+06	9.07E+06
0.35	G1	650	0	1.00E+15	

Table I.6 Mangatu Rd 4.26km

PADS MODEL					
INPUT				OUTPUT	
Poisson's ratio	Material type	Modulus (MPa)	Thick (mm)	Service life	Cemented life
	Method I:Presumed value				
0.35	C1	10000	130	5.43E+06	3.55E+06
0.35	C2	2000	330	4.46E+06	3.55E+06
0.35	G2	100	60	1.00E+15	
0.35	Soil	100	0	2.88E+10	

Table I.7 Mangatu Rd 9.001km

PADS MODEL					
INPUT				OUTPUT	
Poisson's ratio	Material type	Modulus (MPa)	Thick (mm)	Service life	Cemented life
	Method I:Presumed value				
0.35	C1	10000	145	5.47E+06	4.13E+06
0.35	C2	2000	340	6.24E+06	4.13E+06
0.35	Soil	100	0	4.26E+06	

Table I.8 Mangatu Rd 10.12km

PADS MODEL					
INPUT				OUTPUT	
Posisson's ratio	Material type	Modulus (MPa)	Thick (mm)	Service life	Cemented life
	Method I:Presumed value				
0.35	C1	2000	170	7.52E+05	1.34E+05
0.35	G2	100	150	1.42E+07	
0.35	Soil	60	0	1.82E+05	

Table I.9 Mangatu Rd 11km

PADS MODEL					
INPUT				OUTPUT	
Posisson's ratio	Material type	Modulus (MPa)	Thick (mm)	Service life	Cemented life
	Method I:Presumed value				
0.35	C1	2000	140	3.31E+06	3.59E+04
0.35	G2	100	160	4.91E+06	
0.35	Soil	60	0	5.50E+04	