

# **The relationship between vehicle axle loadings and pavement wear on local roads**

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**Keywords:** accelerated pavement testing (APT), CAPTIF, loading, New Zealand, pavement, pavement performance, roads, road user charges, thin-surfaced pavements, traffic, vehicles

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# Abbreviations and acronyms

AASHO	American Association of State Highway Officials
APT	accelerated pavement testing
CAPTIF	Canterbury Accelerated Pavement Testing Indoor Facility
CBR	California bearing ratio
ESA	equivalent standard axle
HSD	high-speed data
IRI	international roughness index
LTPP	long-term pavement performance
RLT	repeated load triaxial
RUC	road user charges
SI	structural index
SLAVEs	simulated loading and vehicle emulators
SNP	modified pavement structural number
TRL	Transport Research Laboratory
VSD	vertical surface deformation

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# Executive summary

## Introduction

Road user charges for heavy vehicles vary chiefly according to the amount of road damage assumed to be caused by varying vehicle axle loads. The current approach to determining how much damaged is caused by a particular axle has its origins in the American Association of State Highway Officials (AASHO) road test conducted in the late 1950s. The American research found that doubling an axle load did not have a linear effect and double the damage; damage increased as a power function with an exponent of 4. Often known as the 'Fourth Power Law' the research suggested that doubling the load would do 2 to the power of 4 more damage, so 16 times the damage! While ground breaking at the time, the AASHO road test was conducted with vehicles that bear little resemblance to those used today and the test was on a very limited range of materials and in a freeze-thaw climate that does not represent most of New Zealand.

There is some information on the extent to which road damage varies with axle loadings on state highways, but little reliable evidence on the wear characteristics of New Zealand local road pavements. This makes it difficult to establish appropriate levels of charges for heavy vehicles on a network average basis.

The purpose of this research was to provide reliable evidence on the wear characteristics of New Zealand local road pavements from accelerated pavement loading studies at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF). The data was extended with rut depth modelling with repeated load triaxial data and validated with field data from the nationwide long-term pavement performance (LTPP) sites.

The key objectives of the research were to:

- measure the number of wheel loads to fail a pavement in rutting for a range of axle loads on a range of typical local road pavements and materials using accelerated pavement testing at CAPTIF
- validate the accelerated pavement testing with field data obtained from LTPP sites being monitored over time
- determine how to take account of the environmental component of pavement wear in terms of rutting
- develop a methodology/relationship for determining the damage law exponent used to calculate the component of road user charges relating to pavement rutting (ie pavement renewals and structural repairs) for actual roads.

## Accelerated pavement tests

This report presents the analysis of results of accelerated pavement tests undertaken at CAPTIF as part of a three-year mass limits study. Three pavements were built at CAPTIF on subgrades that ranged from a strong sand to a weak clay. Each test pavement used a consistent subgrade with two different aggregate types at two different pavement depths. The three pavements resulted in a total of 12 pavement test sections. The aim of the testing was to compare the effect of mass on pavement wear over a range of 12 pavement types which are more typical of those found on local authority low-volume roads in New Zealand. The two 'vehicles' at CAPTIF, simulated loading and vehicle emulators known as SLAVEs, were configured with identical suspensions but with different axle loads. One was loaded to 30kN to simulate a 6 tonne axle load, while the other was loaded to 20kN to simulate a 4 tonne axle load. After a million load cycles (or when failure occurred) the plan was to follow with a loading pair of 8 versus 12 tonne axle loads; however, this was only successful for CAPTIF test number 2 as the other CAPTIF tests failed before the additional loading could be applied. The test was conducted with the two SLAVEs trafficking parallel independent wheel paths so the relative wear generated by the two could be compared.

## Analysis

Vertical surface deformation (VSD) or rutting was used as the main measure of pavement wear. VSD under the two loading regimes was measured at each 1m station. These VSD measurements were analysed to determine the number of wheel passes to obtain a VSD of 15mm. If necessary the VSD measurements were extrapolated using a linear best fit to the data from 100k to 500k to determine the number of wheel passes to reach a VSD of 15mm which was used to define the pavement's end of life. The pavement segment's end of life for both the heavy (6 tonne,  $P_{6t}$ ) and light wheel path (4 tonne,  $P_{4t}$ ) was determined when 50% of the pavement segment equalled or exceeded the end of life criteria (ie 15mm). From the pavement lives for the two wheel paths ( $N_{4t}$  and  $N_{6t}$ ) the damage law exponent,  $n$ , was determined as per the following equation:

$$n = \frac{\log\left(\frac{N_{6t}}{N_{4t}}\right)}{\log\left(\frac{P_{4t}}{P_{6t}}\right)}$$

The damage law exponent,  $n$ , is used to calculate the traffic loading in terms of equivalent standard axles (ESAs) for pavement design and deterioration modelling. It is also used in determining the level of road user charges where a current value of 4 is assumed, hence the Fourth Power Law. Therefore, analysing the results in terms of determining the damage law exponent value has the advantage that it can be used directly in current methods of pavement design, deterioration modelling and road user charges.

## Rut depth modelling

All 12 CAPTIF pavements were modelled to predict rutting from repeated load triaxial testing. The results of this modelling showed the rut depth modelling calculate close to the same rut depths measured in the CAPTIF tests, this validated the approach. Resilient and permanent strains measured in the aggregate and subgrade layers at CAPTIF also matched the predicted outcomes of the models boosting the confidence in the model. There was only one exception, in CAPTIF test 3 one section failed rapidly and it was difficult to predict rutting with this unstable behaviour. Additional rut depth models were developed for a range of LTPP pavements and selected other loads and tyre pressures to extend the data set.

## Long-term pavement performance

CAPTIF rut rates and deflections were compared those of similar LTPP pavements and the following observations were made:

- As expected, the pavement strength from the LTPP sites covered a much wider range compared with the CAPTIF dataset.
- The deterioration rate of sites with inadequate drainage was approximately 2.5 times greater than that of sites with adequate drainage.
- The median rut rate on the CAPTIF experiment was higher than that of the LTPP sections, which could be explained by the difference in loading conditions between these two datasets.
- Given the significant variation in rut rates observed on road networks, the rut rates of the two datasets could be considered to correlate relatively well.



## Results

- A relationship was found between pavement life tested at CAPTIF plus the rut depth modelling and the damage law exponent for the 4 and 6 tonne equivalent axle loads (see figure 4.15). It can be seen that for short-life pavements the damage law exponent increases. A reason for this is because short-life pavements have low strength materials that can fail rapidly in shear if the 6 tonne axle load exceeds the strength of the material while the 4 tonne axle does not fail the material in shear. This results in large differences in the number of wheel passes to cause failure and thus a high damage exponent.
- Measured strains at CAPTIF correlate with the predictions from rut depth modelling in terms of which materials (aggregates, sand or subgrade) showing the highest contribution of rutting.

## Recommendations

- The main recommendation is to consider using a damage law exponent based on the data provided in figure 9.4. There is a clear trend between damage law exponent and life.
- On average, state highways with a 25-year design traffic loading of greater than 1 million ESAs should consider using a damage law exponent of approximately 2; however, designing for the heaviest commercial vehicles operating on local low-volume roads with a lower life would need to consider a damage law exponent closer to 6. With the scatter in the results it might be prudent to consider a more conservative value for routine design.
- This research has provided a relationship between damage law exponent and pavement life which together with a known heavy vehicle traffic count can be used to determine an appropriate rut wear component cost for travelling on any section of road for any heavy vehicle type. Future technologies based on a GPS system for spatial charging of heavy vehicles could accommodate this approach for a more accurate user pays system. However, initial calculations show this user pays approach results in the pavement wear costs of a B-train logging truck being 30 times higher on a low-volume weak road than on a highly trafficked strong road. This is mostly due to there being more trucks on the highly trafficked road to pay for pavement rehabilitation when compared with a lowly trafficked road;
- The environmental component of damage can be taken into account with the estimate of pavement life which is then used in an equation to determine the damage law exponent. For example a pavement that has poor drainage in a high rainfall area will have a lower pavement life than the same pavement in a low rainfall area with good drainage. The data from the LTPP sites can be used to determine how environmental factors affect the pavement life.
- If a GPS-based road user charges system was developed, this research could be used as the basis for a simpler location-based system. This would have appropriate road user charge exponents for each of the six road classifications used in the New Zealand wide one network classification criteria, which classifies roads as either: national (normal and high volume), regional, arterial, primary collector, secondary collector, access (normal and low volume) depending on traffic and use.
- The current road user charges system could also be reviewed using a weighted average approach with the six road classification types used to assist in determining an appropriate exponent to use in pavement structural/rutting wear calculations. This would not alter the total road user charges required from industry but may alter the incremental cost of running lightly loaded truck axles vs heavily loaded axles.

## Abstract

In New Zealand heavy vehicles are charged for using the road based on the damage caused passing over the road. The current approach to charging has its origins in American research that found doubling an axle load increased the damage as a power function with an exponent of 4, known as the Fourth Power Law. This was developed with limited pavement and vehicle load types not representative of most of the roads in New Zealand. This research provided reliable evidence on the wear characteristics of New Zealand local road pavements from accelerated pavement loading studies at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF). The aim was to determine the relative damage on different pavement types/strengths. The data was extended with rut depth modelling with repeated load triaxial data and validated with field data from the nationwide long-term pavement performance sites. A relationship was found between pavement life tested at CAPTIF plus the rut depth modelling and the damage law exponent for the 4 and 6 tonne equivalent axle loads. For short-life pavements the damage law exponent increased.

# 1 Introduction

## 1.1 Background

Road user charges for heavy vehicles vary chiefly according to the amount of road damage assumed to be caused by varying vehicle axle loads. The current approach to determining how much damage is caused by a particular axle has its origins in the American AASHO road test conducted in the late 1950s. The American research found that doubling an axle load did not have a linear effect and double the damage, damage increased as a power function with an exponent of 4. Often known as the 'Fourth Power Law' the research suggested that doubling the load would do 2 to the power of 4 more damage, so 16 times the damage! While ground breaking at the time, the AASHO road test was conducted with vehicles that bear little resemblance to those used today and the test was on a very limited range of materials and in a freeze-thaw climate that does not represent most of New Zealand.

There is some information on the extent to which road damage varies with axle loadings on state highways, but little reliable evidence on the wear characteristics of New Zealand local road pavements. This makes it difficult to establish appropriate levels of charges for heavy vehicles on a network average basis.

The purpose of this research was to provide reliable evidence on the wear characteristics of New Zealand local road pavements from accelerated pavement loading studies at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF). That data is extended with rut depth modelling with repeated load triaxial (RLT) data and validated with field data from the nationwide long-term pavement performance (LTPP) sites.

## 1.2 Objectives

The objective of this research was to provide a comprehensive picture of the load-wear relationships for New Zealand roads. Previous research here and in Australia has only considered loads above the current legal limit on state highways. However that research has indicated there may be a significantly different relationship on local roads and below the legal limit. This research will fill in the gaps below the legal limit and on local roads.

The key objectives of the proposed research were to:

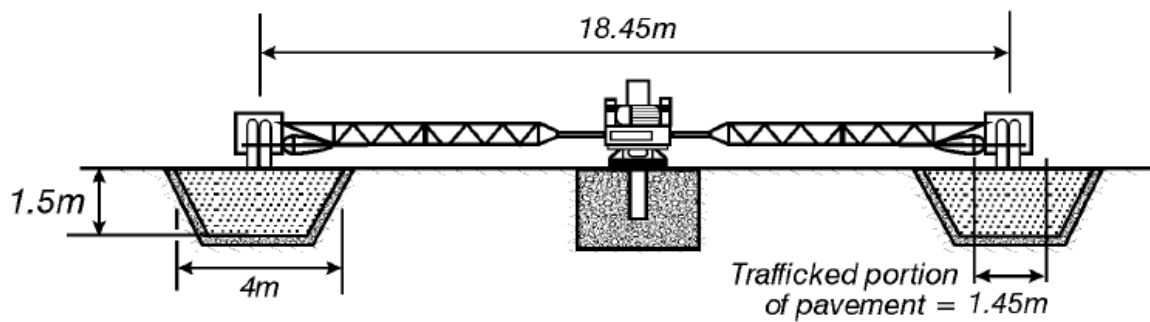
- measure the number of wheel loads to fail a pavement in rutting for a range axle loads on a range of typical local road pavements and materials using accelerated pavement testing (APT) at CAPTIF
- validate the APT with field data obtained from LTPP sites being monitored over time
- determine how to take account of the environmental component of pavement wear in terms of rutting
- develop a methodology/relationship for determining the damage law exponent used to calculate the component of road user charges (RUC) relating to pavement rutting (ie pavement renewals and structural repairs) for actual roads.

A key aim/objective of the CAPTIF tests was to cover the full range of low-volume road variables to validate and develop an appropriate rut depth prediction model that could be used on a range of wheel loads and pavement types.

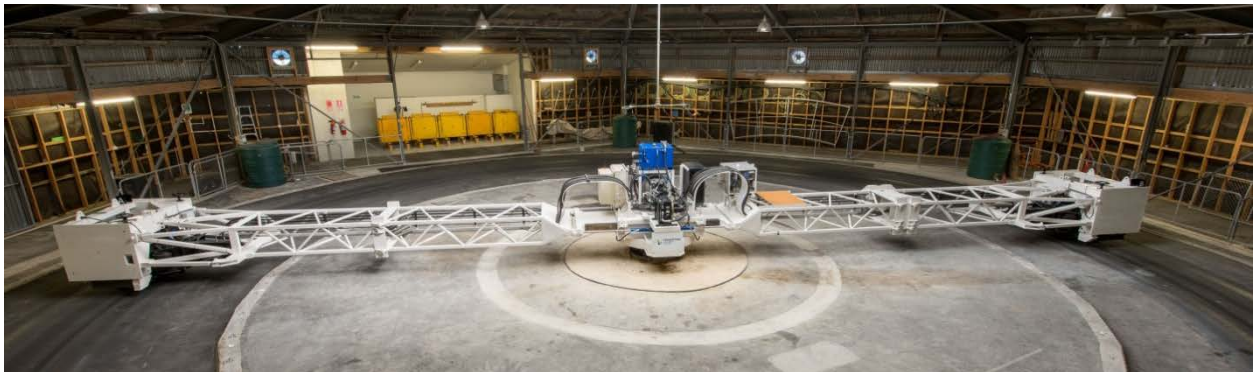
## 1.3 CAPTIF facility

CAPTIF is the NZ Transport Agency's accelerated pavement indoor testing facility, which has been in use since 1969 (William and Paterson 1971). In 1984 the first testing machine was replaced with the current machine (Pidwerbesky 1995). CAPTIF is housed in a hexagon shaped building 26m wide and 6m high. An annular concrete tank, 1.5m deep and 4m wide, confines the bottom and sides of the track (figures 1.1 and 1.2).

**Figure 1.1** CAPTIF test track



**Figure 1.2** CAPTIF - NZ Transport Agency's pavement testing facility



## 2 Literature review

### 2.1 Introduction

The very nature of this project required an outcome that would be robustly defensible technically. It was therefore imperative that the research was strongly founded on accepted procedures and practices. The first stage considered all previous work completed in this area, concepts and principles used elsewhere and any new technologies that existed for the measurement and analysis of the experiment. There were few areas of this research that were completely 'new' with many areas seeing similar work completed on stronger pavements at higher loads. The aim of the literature review was to incorporate previous work as far as possible, identify and address areas of limitation or data shortages and apply any new technology to the research methodology where practical. The literature review covers relevant information to this project in terms of determining appropriate load equivalency factors and damage law exponents.

### 2.2 Origins of the Fourth Power Law (AASHO test)

The TERNZ and Covec (2008) report *Heavy road user charges investigation – stage 1. Information search and review* provides an excellent summary on the origins of the Fourth Power Rule (also referred to in this report as the Fourth Power Law). Conclusions found in reviewing the origins of the Fourth Power Law are:

*The AASHO models of LEF and pavement performance are empirical (i.e. based on a statistical fit to observed data) and are thus not necessarily valid in situations where the environment, traffic, materials, pavement type and pavement construction methods are significantly different from the original test conditions. In spite of these shortcomings the concept of LEFs and the use of the fourth power relationship to determine LEFs have been widely applied both for pavement design and for road user cost allocation. They provide a relatively simple means of characterising the road-wear potential of a vehicle or a traffic stream with a single number, the number of ESALs applied. This approach has a close parallel in design for fatigue in metals where Miner's rule can be used to convert a spectrum of stress cycles into an equivalent number of cycles of a specific stress.*

### 2.3 CAPTIF research

A significant amount of pavement testing research funded by the NZ Transport Agency (the Transport Agency) has been conducted at CAPTIF on a range of tyre loads, pavements and surfacing types. The pavement data collected during testing is primarily rutting, roughness, deflections and in situ strains versus number of load cycles. Surfacing data collected is surface texture, cracking and potholing versus the number of load cycles. Long after a pavement testing project has been completed, PhD students have continued to use the data to develop and refine models that determine rutting and stresses and strains in the pavement, most notably Arnold 2004; Werkmeister 2004; Stevens 2005; Alvaro 2009. The PhD projects of Arnold, Werkmeister and Stevens all aimed to produce a method to calculate the amount of permanent deformation (rutting at CAPTIF) from repeated load triaxial (RLT) testing and a finite element model of the pavement. The CAPTIF data was used to validate their permanent deformation models. Werkmeister and Stevens determined the permanent deformation from resilient strains calculated in the pavement while Arnold used stresses. The later Arnold stress-based model was used for this project and was further validated and refined with CAPTIF low-volume road pavement test data. Models developed in Transport Agency research projects conducted by Arnold and Werkmeister (2010a; 2010b) have resulted

in the same answers. The Arnold model was used in this research as it is simpler and does not rely on complex non-linear elastic moduli models. Further, the Arnold model has been used to produce a significant number of pavement models with different depths, aggregate quality and subgrade strengths which as shown later is a very useful database for initial prediction of rutting for different wheel loads. All the models including the Arnold model can be refined and validated further with more CAPTIF data but one major assumption is how the rutting predictions are extrapolated for the lower loads that do not reach a rut depth failure criterion. The extrapolation method adopted was linear, after a linear rate of rutting was established in the rut depth measurements. This method of linear extrapolation was recommended in previous research on the effect of increases in mass limits on state highways at CAPTIF (Arnold et al 2005c) which is of direct relevance to this research project. Although the testing only looked at axle loads greater than the current legal limit (8.2 tonne dual tyred axle) and used a limited range of aggregate and subgrade types the main findings and information of relevance to this research project are summarised below:

- Vertical surface deformation (VSD) proved to be the most useful measure for monitoring pavement wear at CAPTIF.
- A limited range of pavements was tested at CAPTIF, where the subgrade used for all pavement test segments was a silty clay with a California bearing ratio (CBR) of 11%, while the aggregate type varied and pavement thickness was close to 300mm apart from two segments with a thickness of 250mm each.
- Pavement life in terms of the number of wheel passes until the VSD was equal to 15mm was best estimated by a best fit linear projection to the data from 100k to 500k (after the initial compaction period).
- This linear projection was effectively the compaction wear model proposed in previous mass limits reports ( $VSD = \text{initial compaction} + \text{number of loads} \times \text{rutting/wear rate}$ ) (de Pont et al 2002).
- A power model was also fitted to the data to determine the pavement life but its use was discounted due to unrealistic pavement lives being predicted. Also, the HDM III deterioration models assumed a linear model for rut depth and thus the linear projection was best.
- The end of life for the pavement segment was defined as the number of wheel passes when 10% of the area had a VSD greater or equal to 15mm. This was calculated as the 10th percentile value of lives for each individual station in each segment and wheel path.
- Based on the pavement segment lives predicted in the heavy (50 or 60kN) and light (40kN) wheel paths, the damage law exponents calculated ranged from 1.1 to 3.4.
- The lowest damage law exponent of 1.1 was calculated for the Cptf\_D01 pavement constructed with recycled crushed concrete.

**Table 2.1 Summary of test pavements used in the CAPTIF mass limits study**

New ID	Pavement thickness <sup>(a)</sup> (mm)	Aggregate short description	Aggregate description	Subgrade type
Cptf_A03	320	Montrose Class 2	A 20mm max size rhyolite from Montrose, Victoria, Australia.	Silty clay (CBR=11%)
Cptf_B03	250			
Cptf_C03	250	AP40 TNZ M/4	A 40mm max size alluvial greywacke gravel from Canterbury, New Zealand.	
Cptf_D03	320			
Cptf_E03	320	Rounded AP40 TNZ M/5	A 40mm max size uncrushed rounded river gravel from Canterbury, New Zealand.	
Cptf_A01	300	AP40 TNZ M/4	A 40mm max size alluvial greywacke gravel from Canterbury, New Zealand.	
Cptf_B01	300	AP40 TNZ M/4 + fines	A 40mm max size alluvial greywacke gravel contaminated with 10% by mass of silty clay fines from Canterbury, New Zealand.	
Cptf_C01	300	Montrose Class 2	A 20mm max size rhyolite from Montrose, Victoria, Australia.	
Cptf_D01	300	Recycled concrete	Recycled crushed concrete from Auckland building demolition sites.	

<sup>(a)</sup> Pavements include thickness of approximately 25mm of asphalt surfacing.

- A damage law exponent of 3.2 was calculated for the pavement constructed with rounded aggregate that had the shortest life (Cptf\_E03) which suggested the damage law exponent was related to pavement strength.
- Without exception for all new pavements a significant amount of deformation occurred in the first 150,000 cycles.
- This initial deformation could be predicted adequately using the same method for calculating the secondary compaction rut depth as per the HDM III models in deterioration modelling as the damage law exponent for the compaction portion was the same as that determined from the pavement lives and therefore the ESAs calculated in the formula for rut depth due to secondary compaction was correct.
- With the exception of three pavement segments increasing load on the wheel path previously trafficked with 1 million passes of the 40kN wheel resulted in additional secondary compaction similar in magnitude to what would occur on a newly constructed pavement for the same load.
- Two of the three pavements that did not suffer secondary compaction due to an increased load on an already trafficked pavement were constructed with the Australian aggregate from Montrose, Victoria.
- The other pavement that did not suffer secondary compaction due to an increased load on an already trafficked pavement was constructed with New Zealand AP40 aggregate complying with TNZ M/4 but contaminated with 10% by a mass of silty clay fines.
- During the mass limits study two tests on chipseal surfacings were conducted. These tests were 8 versus 10 tonne and an 8 versus 12 tonne test. Conclusions from these tests were:

- Relative damage in terms of deterioration of chipseal texture depth was compared using the damage law exponent method, where in the 8 versus 10 tonne test a damage law exponent of 9 was calculated, while a value of 3.5 was calculated in the 8 versus 12 tonne test.
- Should mass limits increase there may be a case on specific routes of relatively high strength pavements that the damage law exponent value be reduced from the current value of 4.
- Further, pavement modelling work should be conducted to determine appropriate damage law exponents for a greater range of tyre types, loads and pavements, in particular RLT tests are required for weaker subgrade materials in order to develop an appropriate model for pavements with weak subgrades. These pavement models should be validated with later tests at CAPTIF if appropriate.

## 2.4 Variations to the Fourth Power Law (TERNZ and Covec 2008)

TERNZ and Covec (2008) undertook a literature review and found many researchers, when re-analysing the AASHO road test data, could not find a single power law relationship, as the power exponents were dependent on the pavement type, the pavement strength, the type of distress and the terminal level.

TERNZ and Covec (2008) reported on an OECD (1982) report that presented an overview of power law based load equivalency factors and gives a range of 3–4 for flexible pavements and 8–12 for rigid pavements. Cebon (1999) also presents a summary of pavement damage exponents found by other researchers which includes exponents of 1.3–4.1 for flexible pavements and 11–33 for rigid pavements.

## 2.5 Vehicle dynamics effects (TERNZ and Covec 2008)

The Fourth Power Law formula is based on the static axle load but, of course, axle loads are dynamic. As a vehicle travels along a road the axles bounce up and down on the tyres and the vehicle body bounces up and down on the suspension and so the forces applied to the road by the tyres vary.

Research element one of the DIVINE project was an APT which was undertaken at CAPTIF in Christchurch. Although the pavement used for this was asphaltic concrete rather than chipseal it was relatively thin (85mm) and the basecourse and subgrade materials were those typically used in New Zealand. Two additional simulated<sup>1</sup> chipseal pavements were tested in conjunction with the DIVINE test (de Pont et al 1999). For all three tests, the two CAPTIF loading units were loaded to the same static load but fitted with different suspensions and run in adjacent parallel paths on the same pavement. An analysis of the results of these tests by de Pont and Steven (2000) showed the VSD of the pavement was correlated with both the applied loads and the pavement stiffness. Multiplying the pavement stiffness by the applied load to give a single parameter which approximates strain improved the correlation. Fitting a power relationship between this strain parameter and the VSD for each of the tests gives exponents between 1 and 2. Thus these dynamic loading tests suggest that if a power law is appropriate its exponent should be about 1.5 rather than 4.

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<sup>1</sup> Pavement tests at CAPTIF at the time used a 25mm AC surface layer to simulate chipseal. At this thickness the surfacing contributes very little to the pavement strength and so the performance of the basecourse and subgrade layers should be substantially the same as it would be with chipseal



## 2.6 TRL report CPR310 (Newton and Ramdas 2009)

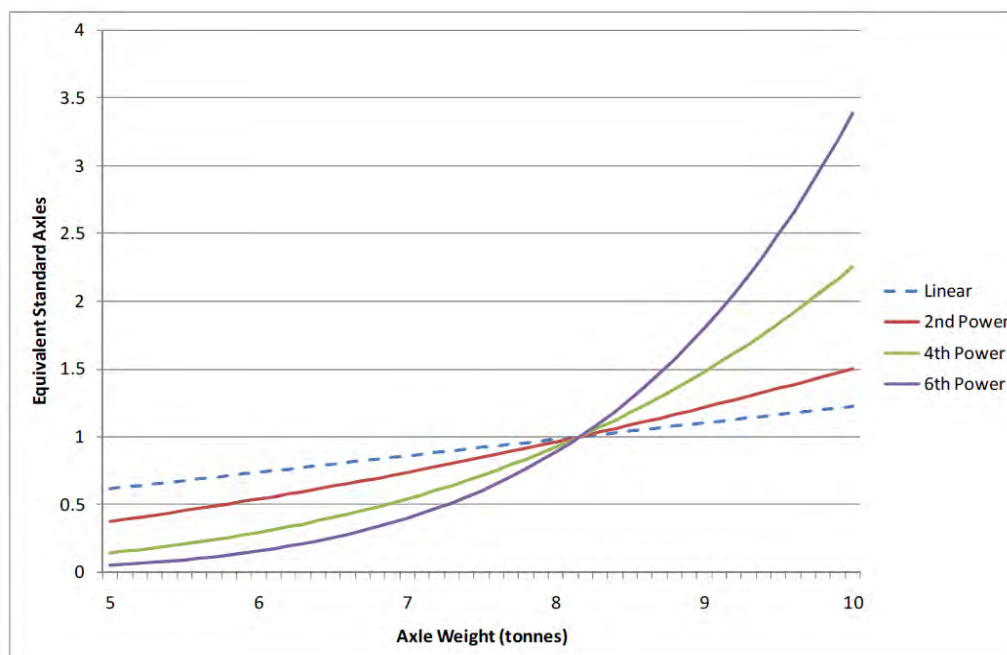
A Transport Research Laboratory (TRL) report (Newton and Ramdas 2009) reviewing the Fourth Power Law for calculating RUC, reported a range of damage exponents is possible based on past studies in New Zealand and a 2006 study to identify wear factors for designing road pavements on the UK's strategic road network (table 2.2).

**Table 2.2 Power law exponents for different modes of deterioration on UK's strategic road network (Atkinson et al 2006)**

Mode of deterioration		Range of exponents
<b>Flexible pavements</b>		
	Non-structural rutting	1.0 – 1.5
	Cracking	1.3 – 3.1
	Serviceability	4.4
	Rutting	4.0 – 9.6
	Asphalt fatigue	4 – 5
<b>Rigid pavements</b>		
	Rigid pavement cracking	5.5 – 18.0
	Faulting at joints	0.7
<b>Subgrade</b>		
	Deformation	4.0 – 7.4

The TRL authors also commented on the insufficient number of pavement types and wheel loadings tested at CAPTIF and thus questioned the robustness of the results to apply to New Zealand RUC in calculating ESA. They further state in the conclusions that there is incredible amount of uncertainty in the calculation of ESA and although this has minimal effect for pavement design it is questionable to use the ESA calculation for RUC.

**Figure 2.1 The effect of different damage law exponents on ESA**

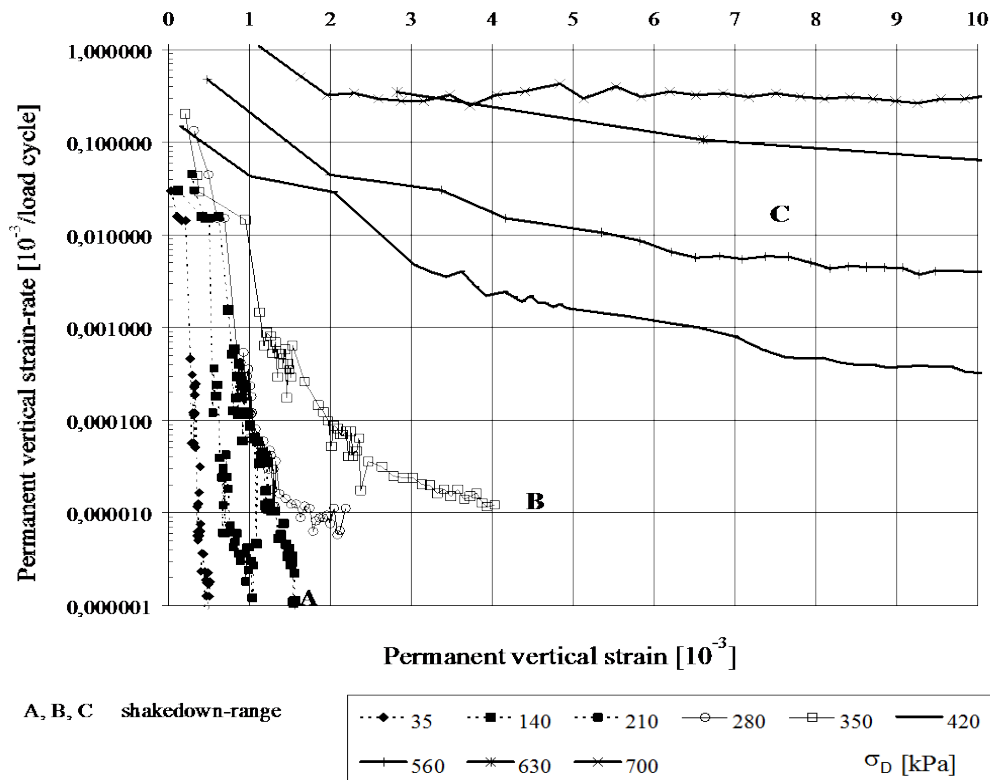


In the TRL report a plot shows the effect of various damage exponents on the ESA calculated for different axle weights (figure 2.1). As can be seen, for a damage exponent of 2.0 all the axle weights less than the standard axle weight of 8.2 tonnes cause more damage (ie would be charged more) compared with a damage exponent of 4.0. For axle loads greater than the standard axle of 8.2 tonnes the effect of a lower damage exponent of 2.0 is to reduce the damage caused (ie higher axles are charged less).

## 2.7 Rut accumulation and power law models for low-volume pavements under mixed traffic. TRB paper 08-2661 (Dawson 2008)

Dawson (2008) illustrated that it is not possible to define the relative damaging effect of different wheel loads with a single damage law exponent. For low wheel loads the response is nearing 'shakedown' (where for each load application the rate of deformation continues to decrease until no further permanent deformation occurs) and the rutting may never reach a failure condition (also described as range A behaviour). For medium wheel loads the permanent deformation increases linearly with application of each load cycle until failure occurs (range B). With loads close to the maximum shear strength of the pavement or subgrade materials rutting failure occurs very quickly (range C). The different permanent strain responses with loads can be seen in RLT test results plotted in terms of permanent vertical strain versus permanent strain rate (figure 2.2). Comparing a load in range A with a load in range C would create very large exponents, while comparing loads that are both in range A would create exponents approaching zero.

Figure 2.2 Permanent vertical strain of granodiorite,  $\sigma_3 = 70\text{kPa}$

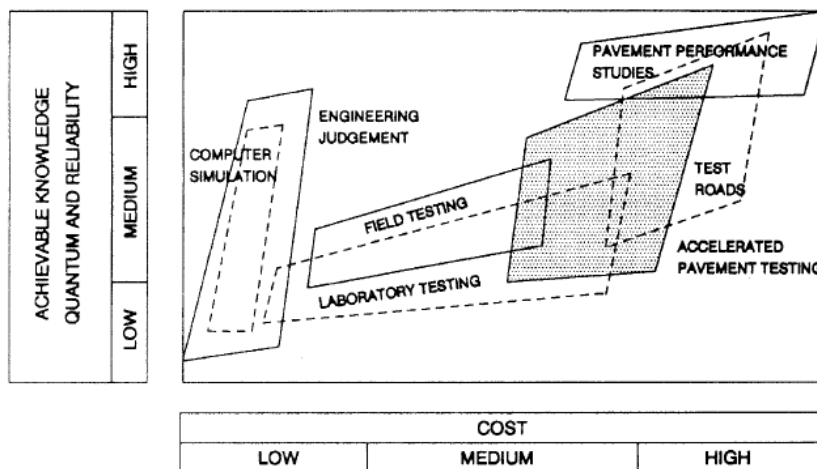


## 2.8 Long-term pavement performance studies

### 2.8.1 Understanding pavement behaviour

In their attempts to truly understand pavement behaviour, engineers have devised a number of testing methods that all vary in terms of cost, how accurately they simulate pavement behaviour and what we can learn from them. Hugo et al (1991) attempted to relate these concepts graphically (figure 2.3). The figure shows that higher cost experiments/tests normally provide more understanding of actual pavement behaviour. However, it is also recognised that each of these methods still has some inherent limitations given the range of influencing factors included in the experiment.

**Figure 2.3 Interrelationship between pavement engineering facets that collectively and individually contribute to knowledge (Hugo et al 1991)**



For example, CAPTIF is ideal for research application since it is possible to design, build and fail the exact pavement and loading configuration required to answer a given research question. However, the accelerated testing normally excludes other factors such as the influence of environmental factors on the deterioration of the pavements. The LTPP process monitors actual in-service pavements in their natural environment. However, little or no control exists on the environment or loading. LTPP programmes can best yield useful data after a long in-service life and the in-service nature of the site means learning about early failure boundaries is generally not practical. The ideal situation then is to link the results from the accelerated facilities with those from LTPP programmes.

It should also be kept in mind that the intent of APT programmes is different from LTPP programmes. Whereas, the APT allows for fundamental research into a detailed pavement response to wheel loads through sophisticated instrumentation, the aim of LTPP programmes is normally aimed at an asset management modelling. With LTPP, statistical techniques are used to analyse condition performance measures and developed pavement deterioration models that include the general loading conditions and other environmental factors in the forecasting of deterioration over time.

LTPP provides an excellent opportunity to validate the results of the APT. There have been earlier projects that link APT studies to LTPP studies in Australia (Sharp and Clayton 2001). The first link between CAPTIF and the New Zealand LTPP programme was undertaken by Henning et al (2004). The intent of this research project was to develop pavement deterioration models on the basis of accelerated data and then to calibrate it on the basis of LTPP data.

## 2.8.2 Previous research on the New Zealand LTPP programme

A number of Transport Agency and university research projects have utilised the New Zealand LTPP data. The most relevant of these are summarised in table 2.3.

**Table 2.3 Relevant research projects that used the LTPP data**

<b>Study name</b>	<b>NZ Transport Agency research report 319 'Benchmarking pavement performance between Transit's LTPP and CAPTIF programmes' (Henning et al 2007)</b>
Description	This research project developed a rut progression model by considering both the LTPP and the CAPTIF dataset. A three stage rut model was developed to represent the: a) shake-down stage; b) stable deterioration stage and c) the probability to go into an accelerated deterioration phase. The fundamental component of the model was developed using the CAPTIF data. Subsequently, the model was calibrated and refined using the LTPP database. A significant outcome of this model was its ability to mirror two potential failure modes of pavement rutting. First, some pavement behaved exactly as designed and would slowly rut over time, until it reached a terminal rut depth – (stages a and b). Others would last for some time but due to overloading or moisture increase might have a sudden or rapid failure towards the end of their lives (stage c).
Relevance	This research was directly relevant to the current project as it provided the basis of linking CAPTIF outcomes to field results. Also, it provided a methodology for rut progression models given different failure behaviour.
<b>Study name</b>	<b>NA Transport Agency research report 401 'Rationalisation of the structural capacity definition and quantification of roads based on falling weight deflectometer tests' (Henning et al 2010)</b>
Description	Pavement performance in terms of structural capacity is generally measured worldwide by the adjusted structural number (SNP) in order to perform long-term planning of road maintenance. However, SNP is not able to give any indication of how a particular pavement structure would behave for a given layer configuration. As a replacement for SNP, an alternative structural parameter, termed structural index (SI) was proposed. For each of the currently recognised structural distress modes (ie rutting, roughness, cracking and shear) a corresponding SI was developed. Each SI is mechanistically derived and has the same range and general distribution as the traditional SNP.
Relevance	In testing its applicability, it was found that SIs are extremely useful in not only describing the over-all health of a network but also highlighting the most probable causes of failure. For example, although a network might be 'strong' from a rutting perspective, it might consist of a large portion of stiff pavements that could show early failure due to cracking issues. Therefore, this concept was used to classify the typical pavements into expected 'like-behaving' groups.
<b>Study name</b>	<b>Identifying pavement deterioration by enhancing the definition of road roughness (Brown et al 2010)</b>
Description	Road roughness is an essential input into determining the RUC for driving on surfaces with different roughness levels. Currently New Zealand uses the international roughness index (IRI) to quantify the ride quality as 'perceived' by vehicles. Unfortunately the IRI does not effectively quantify the 'roughness' as experienced by truck drivers. This research project developed a technique for analysing roughness profiles from

	measurements to determine the deterioration of the road in terms of different wave lengths. From this, the deterioration of the road in terms of roughness is more effectively quantifiable.
Relevance	This project was directly related to the intended research as road roughness is a primary decision driver for when maintenance is being undertaken on roads. It is also a primary input into the life-cycle cost aspects of road maintenance planning.
<b>Study name</b>	<b>University of Auckland Master of Engineering thesis 'The development and assessment of two statistical techniques for application of long term pavement performance studies' (Ma 2011)</b>
Description	<p>The main objective for this study was to develop statistical techniques to interpret condition measurements and understand the performance trend of individual LTPP sites.</p> <p>The study had a successful outcome as it was able to assess the condition change of LTPP sites and to categorise the changes on the basis of some independent variables. This stratification was then used to assess the applicability of the original LTPP design matrix on the bases of the actual performance.</p> <p>The investigation started with selecting methodologies for the purpose of analysing time series data. Both performance index and Minkowski distance methods were chosen with the intention to trial both methodologies on LTPP performance data. Roughness and rutting data were used in the analyses. Performance index and Minkowski distance were calculated for each LTPP site for roughness and rutting data separately. The two profiles of results showed similar trend.</p>
Relevance	This study was useful in the stratification of LTPP sites that could then also be used to classify all low-volume roads according to their expected behaviour.
<b>Study name</b>	<b>University of Auckland PhD study presented as conference paper 'Improved network understanding – a diagnostic approach to the risk of pavement failure' (Schlotjes et al 2011)</b>
Description	<p>This research developed a risk index that calculates the probability of failure of low-volume roads in New Zealand based on inventory, condition and environmental data. For this research fundamental failure paths were developed for the most prominent failure modes of low-volume roads including rutting, cracking and shear. The data was then analysed using a number of data mining techniques, which yielded the failure paths that came out statistically significant on the LTPP data base.</p> <p>The combination between the failure paths and the statistical probabilities were then used to calculate an over-all risk index for given road sections.</p>
Relevance	This research is valuable in a) having an in-depth understanding of the failure mechanisms associated with low-volume roads in New Zealand. In addition to that, it is also helpful in stratifying the low-volume roads, which according to it are most likely to fail.

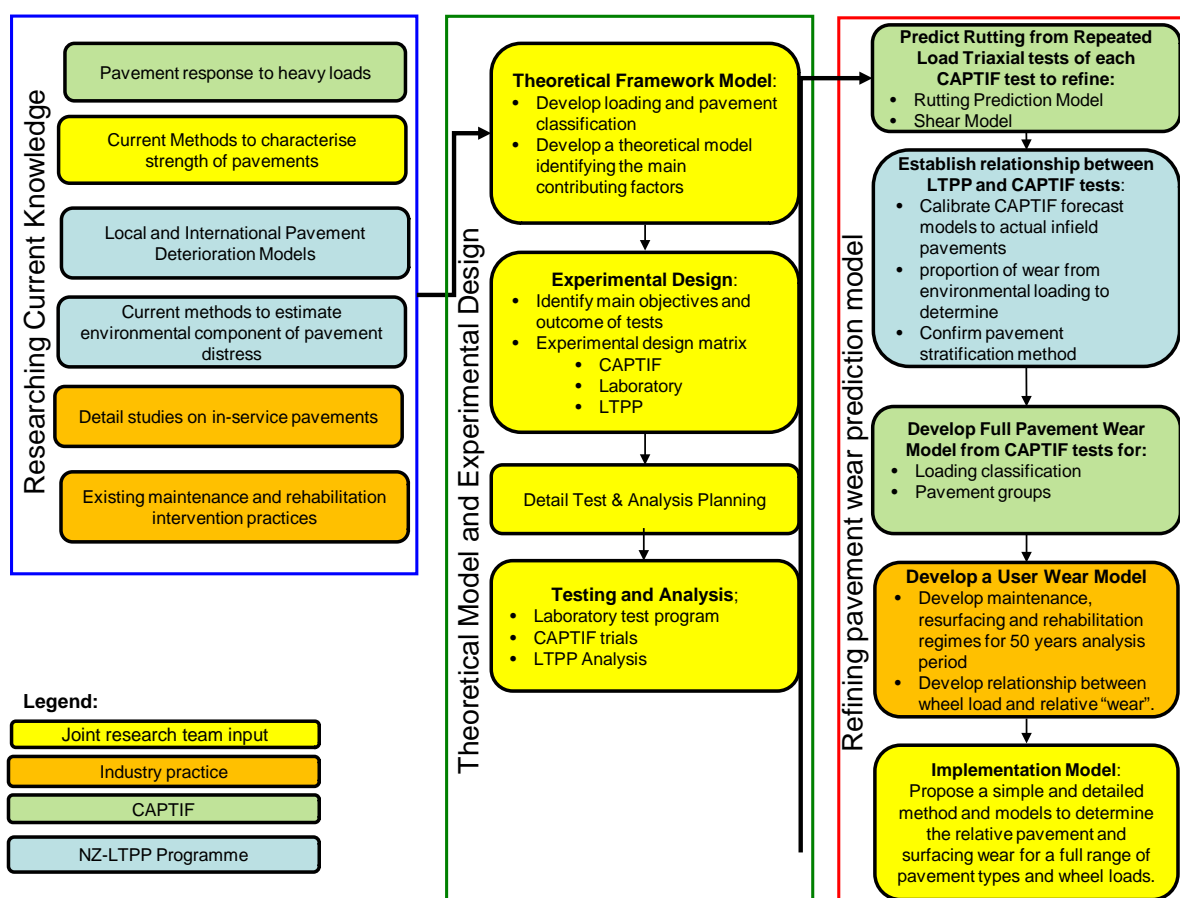
## 3 Methodology

### 3.1 Over-all research layout

The over-all research methodology is presented in figure 3.1. It shows the research consisted of three main stages:

- Stage 1 consisted of taking all existing knowledge and experience in the literature review, both local and international to feed into the detailed methodology of both stages 1 and 2.
- Stage 2 aimed at developing a theoretical model format for predicting road wear on the basis of heavy vehicle loading. The goal was not only to focus on the model establishment but also to gain more clarity on the input parameters that might influence the model outcome. In establishing these factors, the CAPTIF trials were designed to be more robust and outcome focused. The model was then calibrated with CAPTIF data and validated using the LTPP data.
- Stage 3 involved the final refinement and establishment of an appropriate power law for New Zealand's user charge model as per the objective of this project.

Figure 3.1 Research methodology



### 3.1.1 Stage 1: Planning and research of current knowledge

It was imperative for this research to be strongly founded on accepted procedures and the practices. The first stage of the project considered all previous work completed in this area, concepts and principles used elsewhere and any new technologies that existed for the measurement and analysis of the experiment. Few areas of this research were completely 'new' with many areas seeing similar work completed on stronger pavements at higher loads. The aim of stage 1 was to incorporate previous work as far as possible, identify and address areas of limitation or data shortages and apply any new technology to the research methodology where practical.

### 3.1.2 Stage 2: Theoretical model and experimental design

#### 3.1.2.1 Theoretical framework model and experimental design

The stage 1 work flowed on to the development of a theoretical framework for the models. The model allowed the refinement of the experimental design for CAPTIF testing, laboratory testing and the LTPP analysis.

#### 3.1.2.2 CAPTIF accelerated pavement trials

The research replicated, in relation to local roads, work previously done to estimate the effect on pavement wear of increasing vehicle axle loadings on state highways, using CAPTIF in Christchurch

Three CAPTIF tests were required. Two tests to examine local road behaviour of a variety of marginal aggregates – one test above and one below the legal axle limits – and one test to examine state highway behaviour below the legal limit, as this was not explored in the mass limits tests. The work was validated by comparing the detailed CAPTIF findings with the laboratory tests and field observations from the Transport Agency and local authority LTPP sites.

#### 3.1.2.3 Laboratory test programme

While the CAPTIF pavement tests were being conducted a significant programme of laboratory work was undertaken. Combined with modelling this determined an appropriate number of local road pavement types/categories to test and then predicted their rutting behaviour. The new CAPTIF data provided an input to calibrate parts of the model and pavement wear prediction method proposed.

#### 3.1.2.4 LTPP analysis

New Zealand's LTPP programme is unique in the sense that extremely accurate road condition data has been collected that has resulted in a very detailed and robust dataset of road conditions over 10 years. Research on this data has demonstrated that low-volume roads often only display minor condition changes over time, yet with either moisture or increased loading they display rapid deterioration. The data was invaluable for this research as it validated the CAPTIF findings and modelling. It also brought together the combined influence of loading and environment, something that is hard to simulate in the CAPTIF trials.

The LTPP data is independent of the often experienced data collection limitations associated with this type of research. The data is collected with detailed measurements; it has already contributed significantly in understanding how pavement condition changes and how this is interpreted through normal means of data collection. For example, there have been issues with defining pavement strength; this has been successfully addressed through development of structural indices (Salt et al 2010). Likewise, issues associated with high-speed rut measurement, roughness and crack detection are not an issue given that the LTPP data is 'manually' measured. Therefore quantifying how the pavements change over time, even if it is slight, can be quantified accurately with the LTPP data.

### 3.1.3 Stage 3: Refining pavement wear prediction model

The challenge always has been to link the CAPTIF and laboratory results and models with actual pavement performance. The research team had already completed research that was invaluable for addressing this issue in the project. In Henning et al (2007 CAPTIF results and LTPP data were used to develop a rutting model to forecast the rutting rate expected from low-volume road pavements plus the probability of the pavement commencing accelerated rutting (rapid failure). These models were tested on the Transport Agency state highway network and a very high correlation (up to 80%) was achieved.

These concepts were used to expand the current models for different loading characteristics. The new power rule concepts in conjunction with structural indices from Salt et al (2010) were used to characterise the pavement strength/capacity and generate performance forecasts for a variety of pavements, however a simple link with life was found to be the best relationship.

To determine damage law exponent an 'end-of-life' condition needed to be defined. The tests at CAPTIF defined the level of wear when the pavement reached its end of life as a VSD of 15mm. VSD is more conveniently measured at CAPTIF with the transverse profilometer as it is the maximum vertical difference from the start reference level of the pavement. The measurement of VSD is more stable than straight edge rutting measurements, which can be influenced by shoving on the edges of the wheel paths (figure 3.2).

**Figure 3.2 The different measures of pavement deformation**

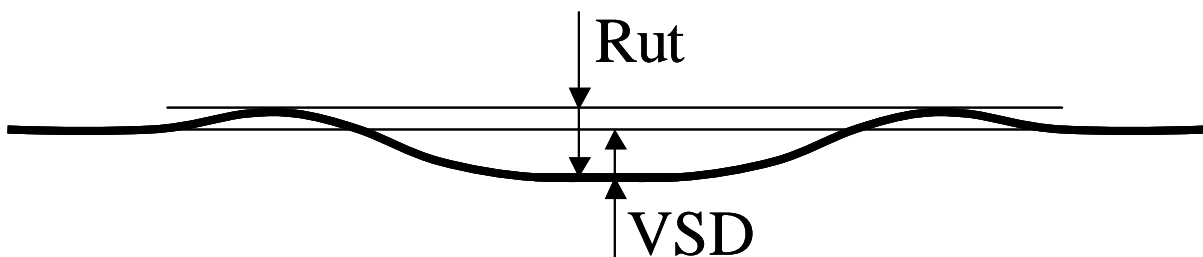
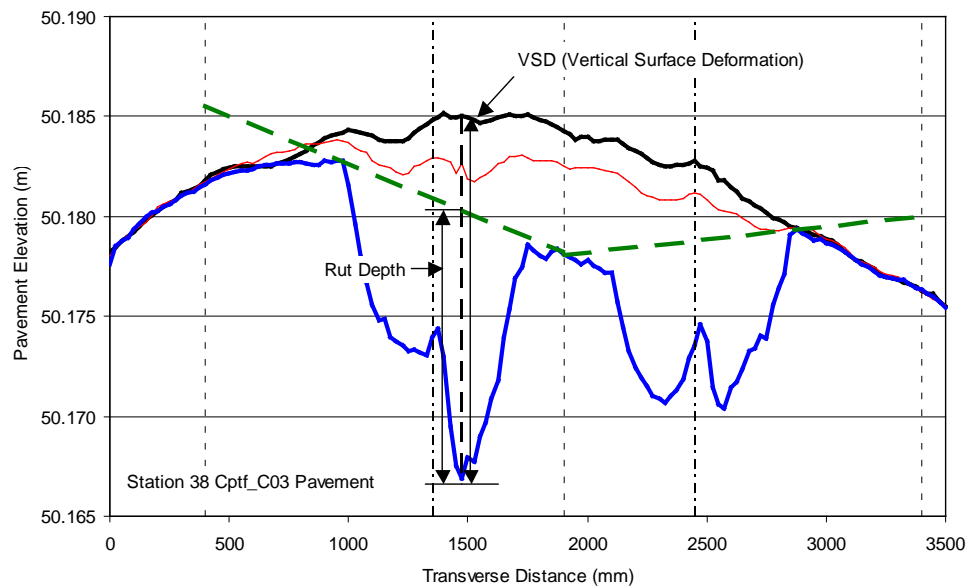
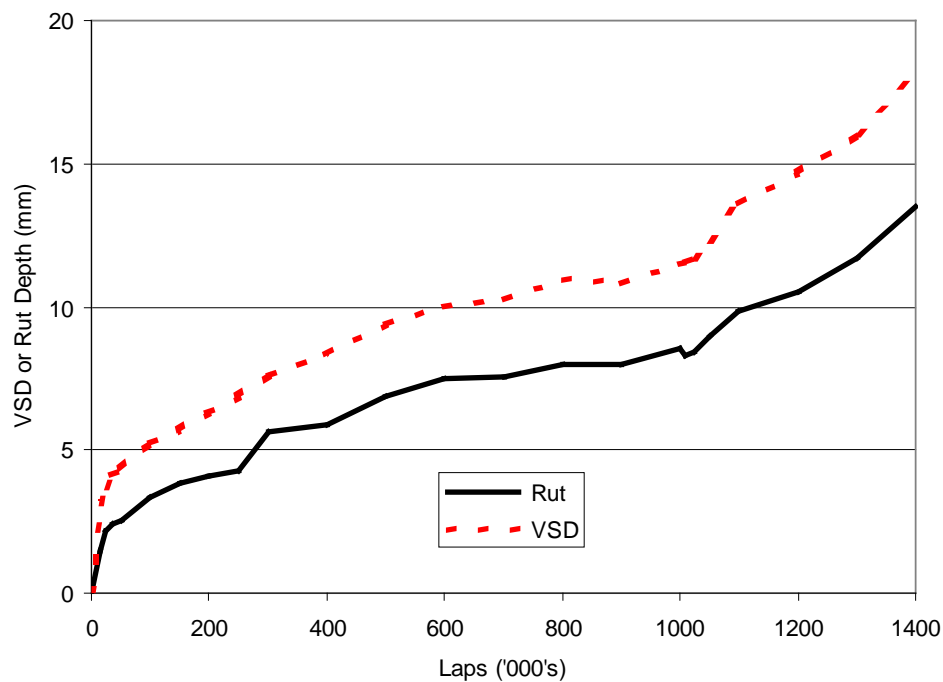


Figure 3.3 shows the comparison of measured VSD and rut depth determined mathematically from a theoretical straight edge at CAPTIF. The rut depth calculated is less than the VSD value measured at CAPTIF. This is because the central rut edges used at CAPTIF move downwards during the testing which affects the straight edge rut depth. Figure 3.4 illustrates the comparison of rutting and VSD during a project.



**Figure 3.3 Typical measurement of rut depth and VSD at CAPTIF****Figure 3.4 Comparison of rut depth and VSD measurements at CAPTIF**

As noted above the damage law exponent requires the end of a pavement's life to be defined for the tests at CAPTIF. This allows the number of wheel passes to be determined when the defined end of a pavement segment life occurs. Six factors were considered when defining the end of life for a CAPTIF pavement segment. This would be when the pavement segment had reached or exceeded a VSD value of 15mm:

- 1 How the pavement's end of life is defined for New Zealand state highways

- 2 How the CAPTIF test pavements deteriorate
- 3 The amount of extrapolation (if any) required from the CAPTIF data to determine the number of wheel passes to reach the defined end of pavement life
- 4 The different environment of CAPTIF, namely an indoor dry environment which does not suffer from water ponding in the wheel tracks and penetrating the pavement structure
- 5 Review of stations where the defined end of life was reached and exceeded
- 6 The effect of the chosen end of pavement life criteria on the damage law exponent calculated.

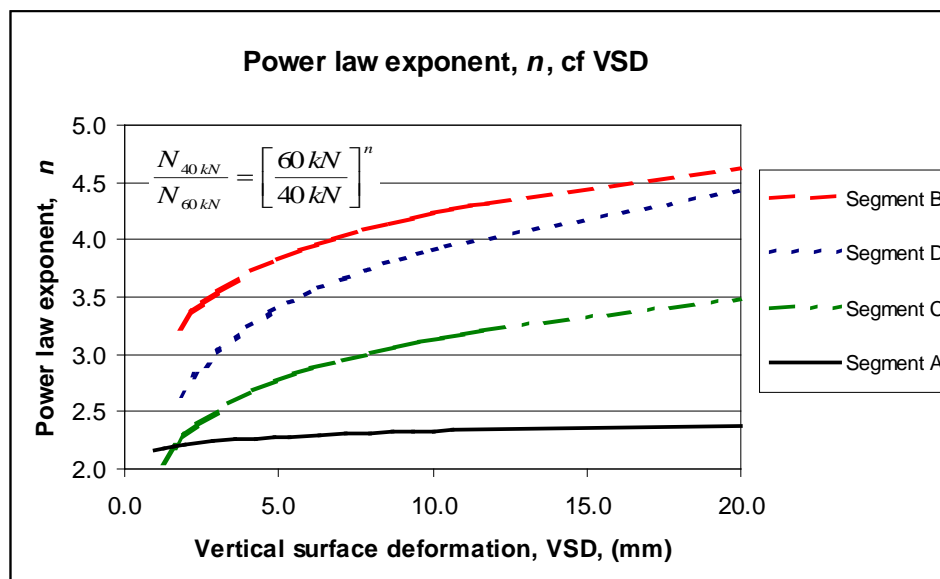
There is a range of factors governing a pavement segment's end of life on state highways in New Zealand. A rut depth of 20mm of roads in New Zealand is considered a cut-off when repairs or rehabilitation are required.

A lower value of VSD defining the pavement's end of life is considered appropriate for CAPTIF due to the indoor environment. Ruts of 10 mm or more would pond water in the field. This water would penetrate into the pavement structure and weaken the material which in turn would accelerate the rate of rutting towards failure. This would not occur for ruts at CAPTIF and the onset to failure would be much sooner. Therefore considering a rut depth of 15mm in the field would soon spell failure due to water ingress then a VSD value of 15mm at CAPTIF defining the end of life is considered appropriate.

On review of pavement segments' progression of VSD where a VSD value of 15mm was achieved it can be seen there was a slight increase in the rate of rutting after a VSD of 15mm (see dashed line in figure 3.4).

Finally, considering the damage law exponent is a relative effect of two wheel loads it was considered that the chosen VSD had a marginal effect on the resulting damage law exponent. Figure 3.5 shows the marginal effect of the chosen VSD value on damage law exponent as found in stage 3 of the mass limits project at CAPTIF (Arnold et al 2005c).

**Figure 3.5 Power law exponent,  $n$ , determined for a range of vertical surface deformation values (after Arnold et al 2005c)**



For each CAPTIF pavement segment there are at least 10 measurements of VSD at 1m stations. The number of wheel passes to reach a VSD value of 15mm was determined for each station's measurements.

This sometimes required linear extrapolation to reach a VSD of 15mm. To observe the sensitivity of the analysis to this assumption the life to achieve a 25mm VSD was also determined.

Finally, the life of the pavement segment was the 50th percentile value or when 50% of the segment has reached the failure criteria. However, for comparison, the 25th, 75th and 90th percentile values were also calculated. Other methods of defining the end of life were considered and calculated as it could be found later in the analysis that a particular method of defining life gives a better trend to predict the damage law exponent.

For managing a road network it is best practice to keep 75% of the road network below trigger levels for rutting and other distress modes. Hence, there is equally reason to choose the 75th percentile for determining the end of life of the CAPTIF segments.

## 4 Pavements tested at CAPTIF

APTs were undertaken at CAPTIF to establish basic load-related wear properties across a typical range of New Zealand local roads. The aim was to create a small high-quality dataset that could be validated with LTPP data and expanded with laboratory RLT testing and modelling. These datasets could then be used to calculate damage law exponents.

The pavements selected ranged from those with weak subgrades to those with strong subgrades. The aggregates ranged from weak to strong as well, and finally the loads tested ranged from low to high.

### 4.1 CAPTIF test details

A total of three CAPTIF tests (1, 2 and 3) were conducted where each CAPTIF test was divided into four sections (A, B, C and D). This resulted in a total of 12 pavement sections tested that were labelled 1A, 1B, 1C, 1D, 2A, 2B, 2C, 2D, 3A, 3B, 3C, 3D where the number denotes the CAPTIF test number and the following letter is the section letter. A summary of the pavements constructed is shown in the tables below.

**Table 4.1 CAPTIF tests (all pavements are surfaced with 25mm of dense AC)**

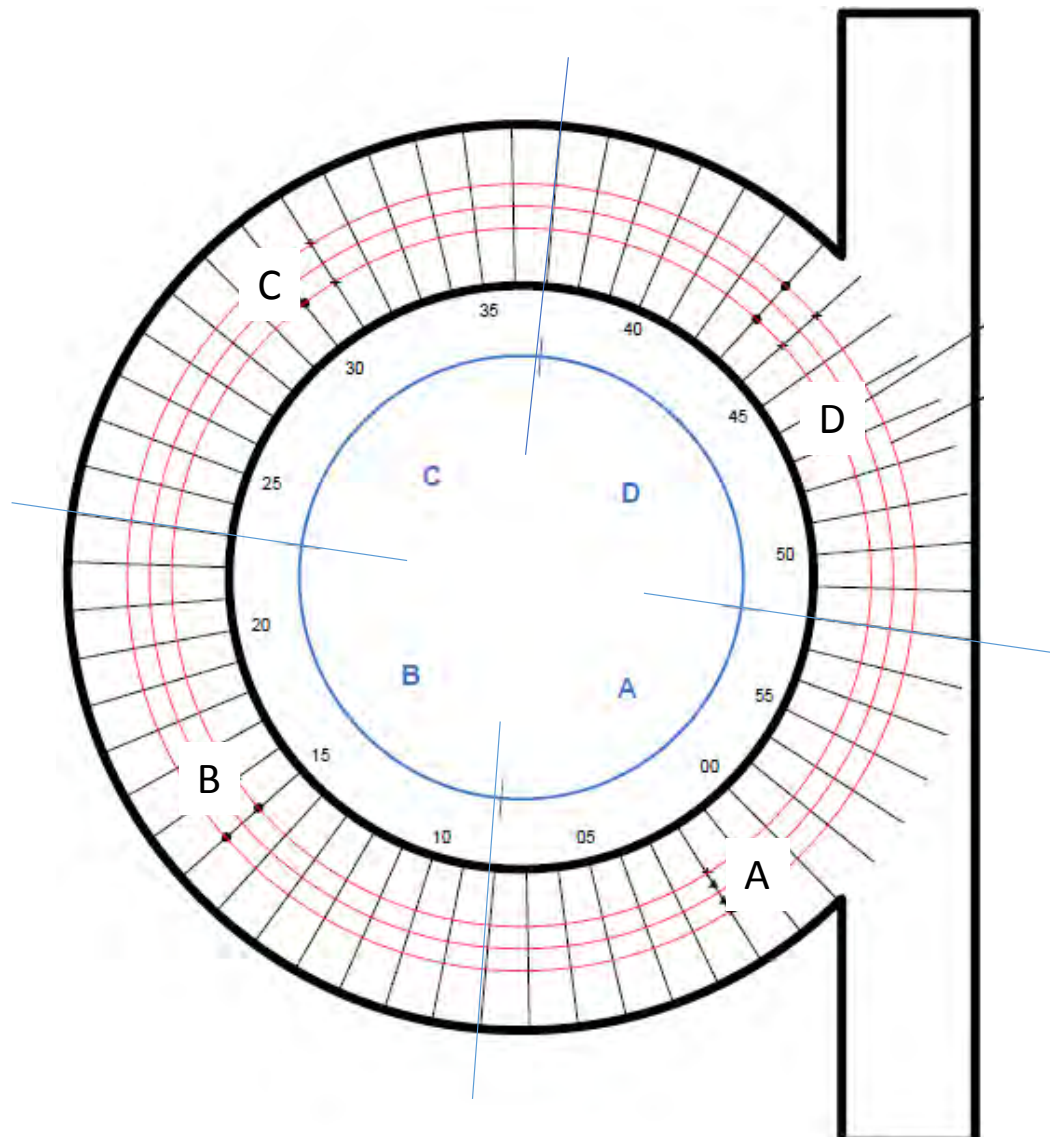
CAPTIF test construction #	Basecourse depth and type	Sub- base depth and type	Subgrade
1A	150 Taupo dacite M4	none	Todd Clay – CBR 8
1B	200 Taupo dacite M4	none	
1C	200 Canterbury M4	none	
1D	150 Canterbury M4	none	
2A	100 Uncrushed river gravel M5	600 AP5 sand (effectively a strong subgrade)	Todd Clay CBR 8
2B	200 Uncrushed river gravel M5	600 AP sand (effectively a strong subgrade)	
2C	100 Canterbury M4	600 AP5 sand (effectively a strong subgrade)	
2D	200 Canterbury M4	600 AP sand (effectively a strong subgrade)	
3A	100 Canterbury M4	200 AP5 sand	Todd Clay CBR 2
3B	200 Canterbury M4	200 AP sand	
3C	200 Uncrushed river gravel M5	200 AP5 sand	
3D	100 Uncrushed river gravel M5	200 AP sand	

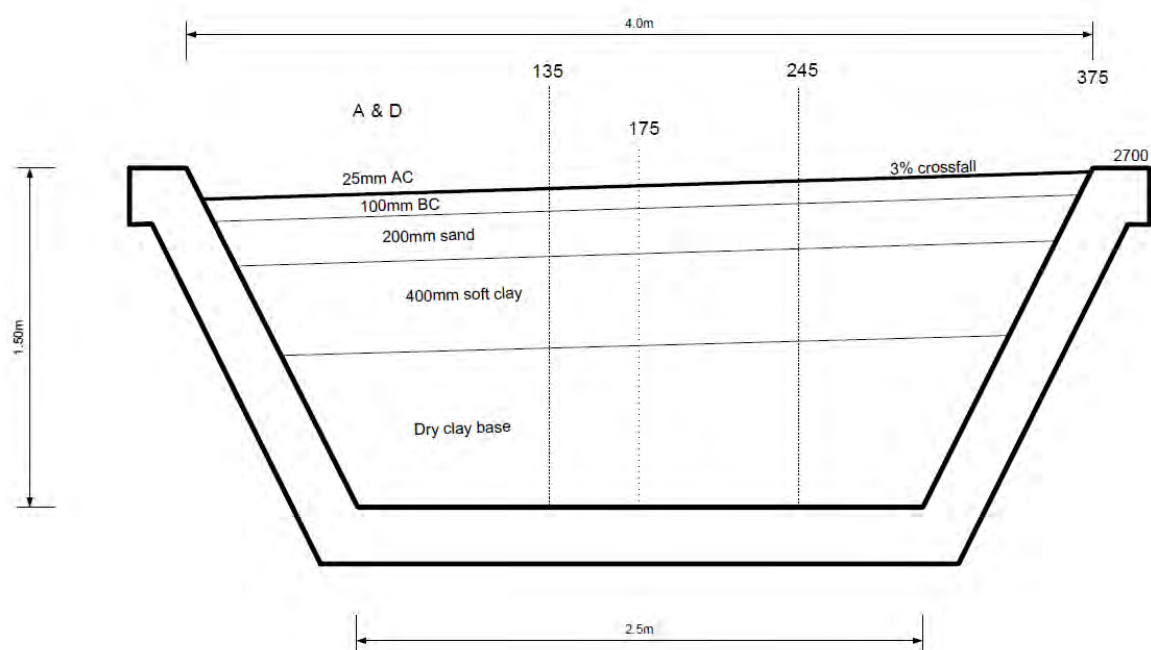
Note: Canterbury M4 = Miners AP40 M4

Uncrushed river gravel M5 = Coutts AP40 M5

The pavements were constructed in four primary segments: segment A extended from station 52 to station 8; segment B from 8 to 23; segment C from 23 to 37; segment D from 37 to 52. A 3m transition zone was allowed between materials, and a 1m transition ramp was made at subgrade level between the sections of differing thickness. A plan showing the layout of the different sections is shown in figure 4.1. An elevation showing the cross section is shown in figure 4.2.

**Figure 4.1** Plan showing the layout of the test sections for CAPTIF tests (refer to table 4.1 for pavement cross- section information)



**Figure 4.2 Example pavement cross section for section 3A**


The pavements tested can be characterised dependent on whether or not the rutting is dominated in the subgrade or aggregate layers as shown in table 4.2.

**Table 4.2 Characteristics of the CAPTIF tests**

Sections	Strong aggregate (will not rut)	Weak aggregate (will rut)
Strong subgrade (or enough cover so will not rut)	1C, 1D, 2C, 2D	1A, 1B, 2A, 2B
Weak subgrade (or insufficient cover so will rut)	3A, 3B	3C, 3D

Just after the pavements were constructed falling weight deflectometer (FWD) tests were conducted on the pavements at 1m centres in each wheel path around the CAPTIF track. Statistical results (average, median and 10th, 90th, 75th percentiles) of the D0 (maximum deflection) and D0-D200 (curvature) are listed in table 4.3.

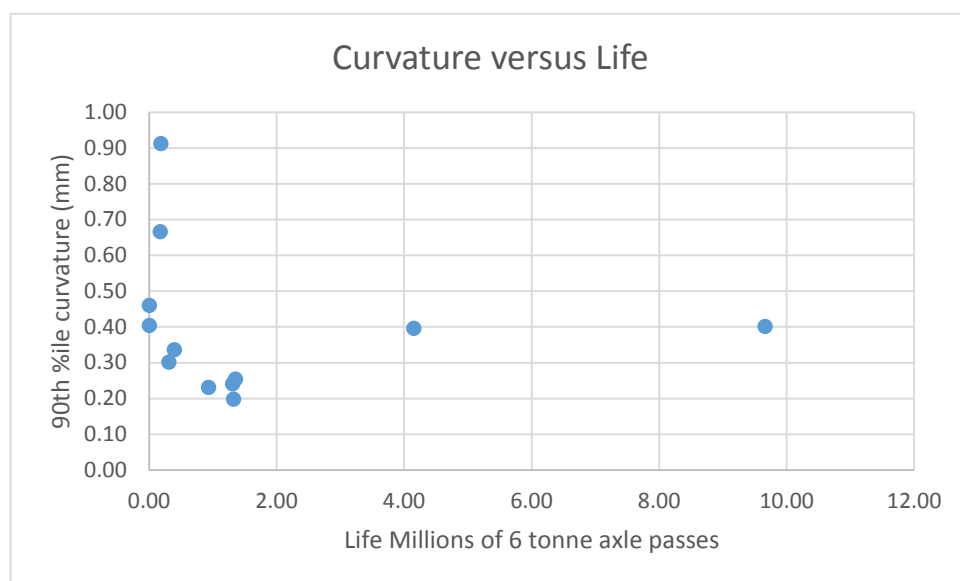
**Table 4.3 Deflection characteristics of the CAPTIF tests**

	Average		Median		10th		90th		75th		Life (millions 6 tonne passes)
	D0	D0-D200	D0	D0-D200	D0	D0-D200	D0	D0-D200	D0	D0-D200	
1A	1.79	0.73	1.77	0.71	1.56	0.59	2.01	0.91	1.89	0.83	0.18
1B	1.38	0.56	1.40	0.57	1.20	0.41	1.51	0.67	1.44	0.62	0.18
1C	0.59	0.26	0.56	0.23	0.49	0.18	0.73	0.40	0.62	0.29	9.66
1D	0.73	0.30	0.73	0.29	0.63	0.21	0.85	0.40	0.77	0.34	4.16
2A	0.59	0.21	0.59	0.21	0.54	0.18	0.65	0.24	0.62	0.22	1.31
2B	0.55	0.18	0.55	0.18	0.50	0.15	0.59	0.20	0.58	0.19	1.33

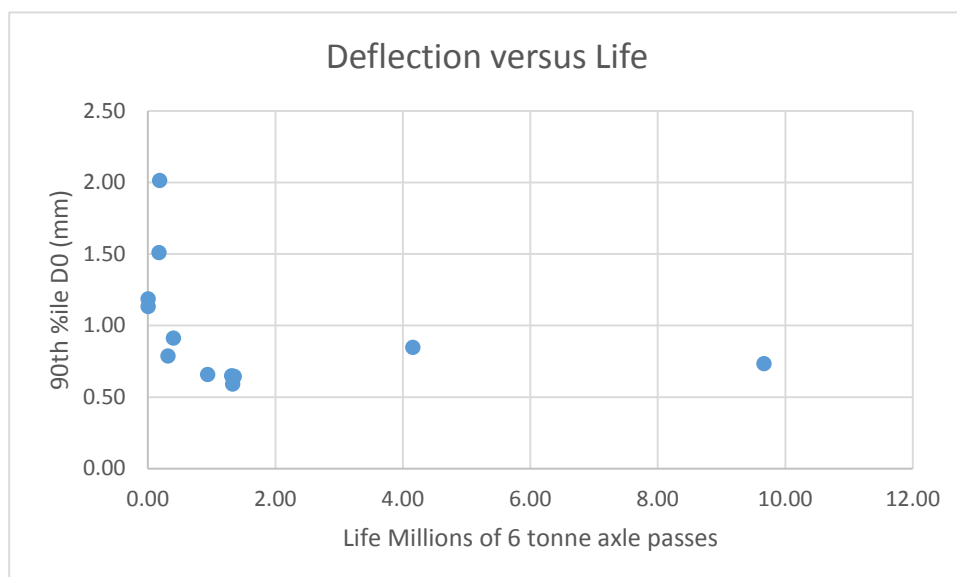
	Average		Median		10th		90th		75th		Life (millions 6 tonne passes)
	D0	D0- D200	D0	D0- D200	D0	D0- D200	D0	D0- D200	D0	D0- D200	
2C	0.61	0.20	0.60	0.20	0.57	0.17	0.66	0.23	0.65	0.22	0.94
2D	0.59	0.21	0.59	0.21	0.54	0.18	0.64	0.25	0.62	0.23	1.36
3A	0.72	0.27	0.79	0.26	0.52	0.20	0.91	0.34	0.84	0.32	0.40
3B	0.71	0.26	0.70	0.25	0.62	0.22	0.79	0.30	0.76	0.28	0.31
3C	0.97	0.34	0.95	0.35	0.86	0.30	1.13	0.40	1.05	0.38	0.003
3D	1.05	0.39	1.05	0.39	0.93	0.34	1.19	0.46	1.12	0.44	0.005

CAPTIF tests 1 and 3 show that the deflections and curvature increase when the aggregate is poor (ie dacite in test 1 and rounder river gravel in test 3); however, this same trend was not observed with CAPTIF test 2. Plots of curvature and deflection versus life (figures 4.3 and 4.4) do show a trend if the long life sections 1C and 1D and the short life sections 3C and 3D are removed.

**Figure 4.3 Curvature versus life**



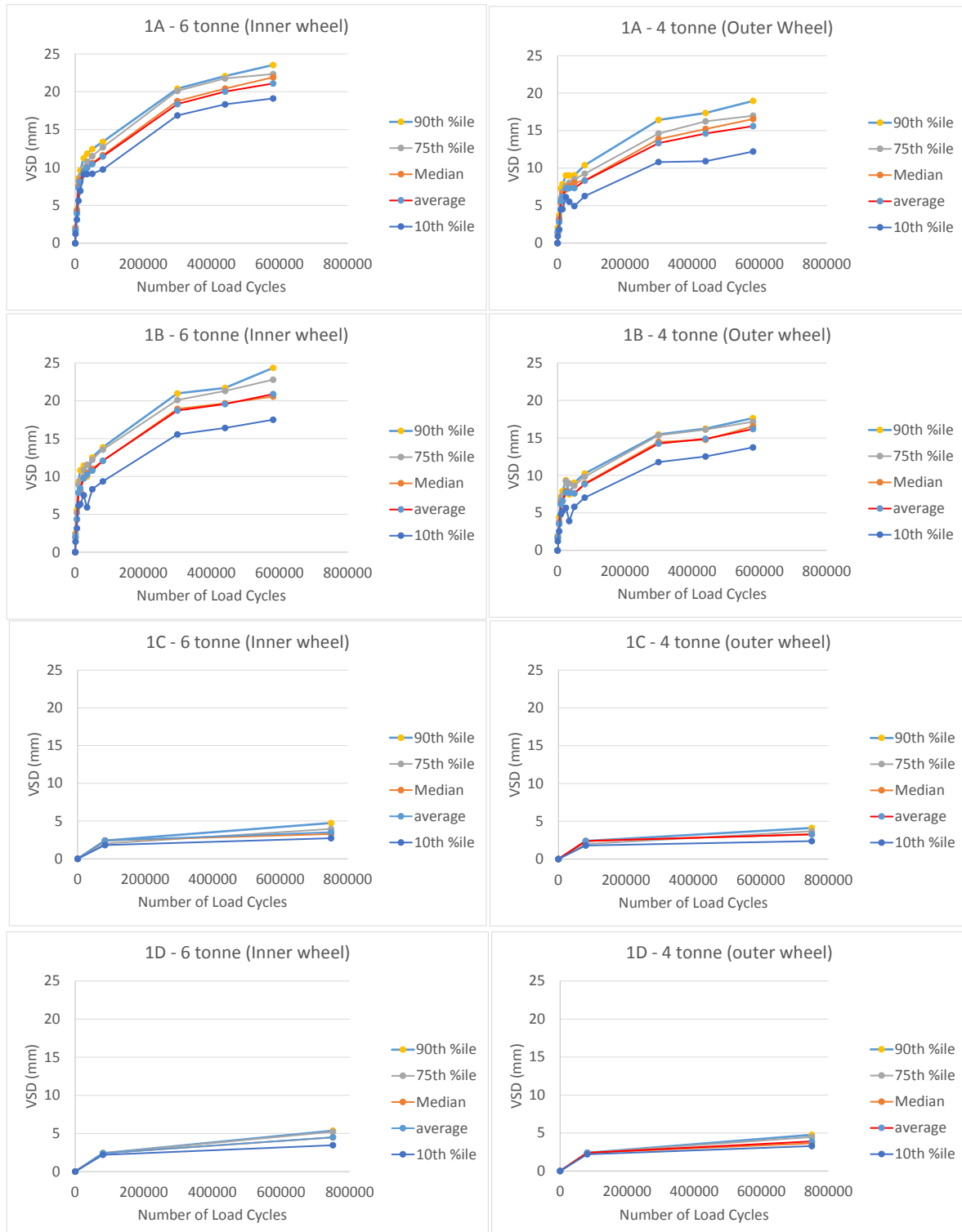
**Figure 4.4** Deflection versus life



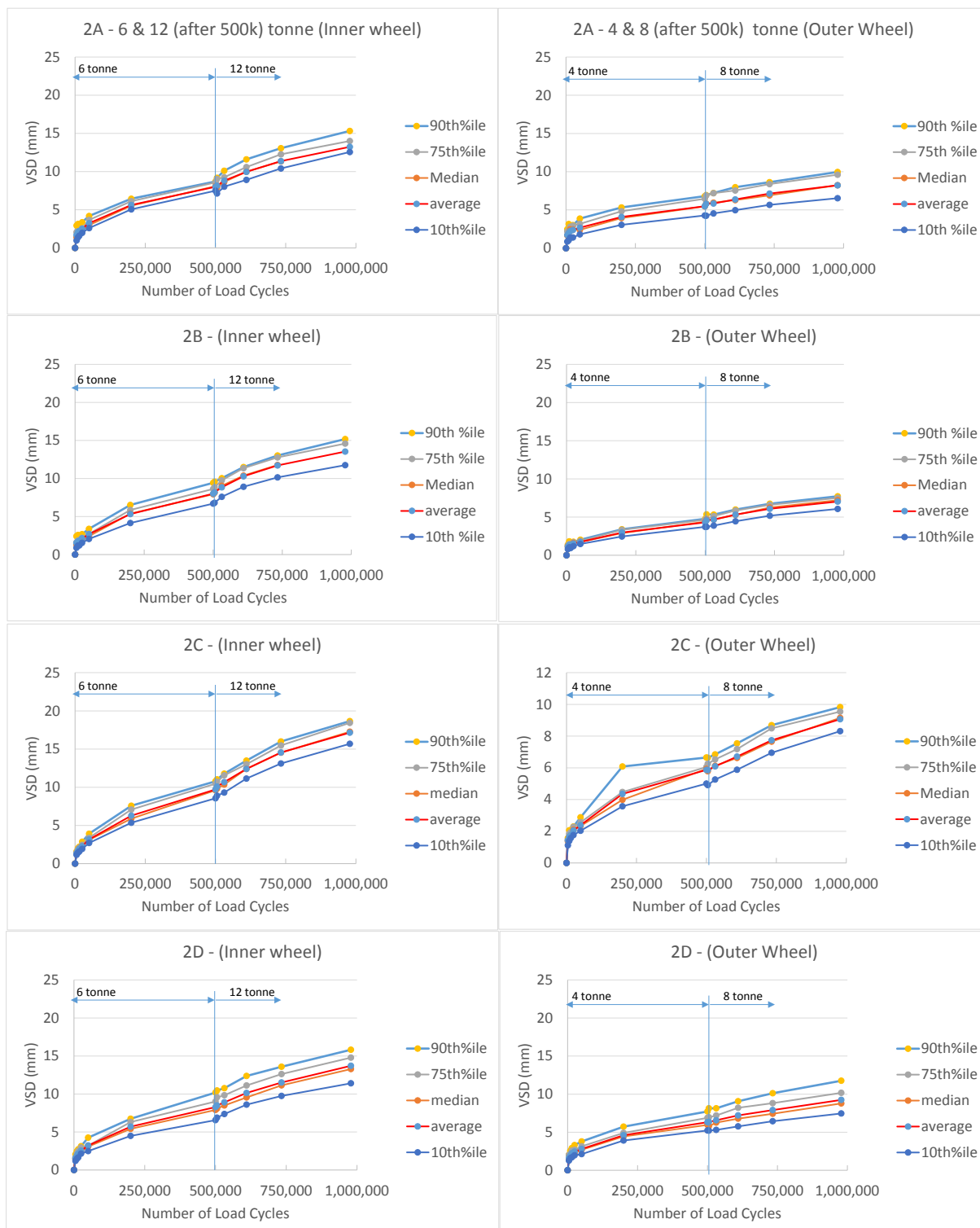
## 4.2 Rut depth measured during test

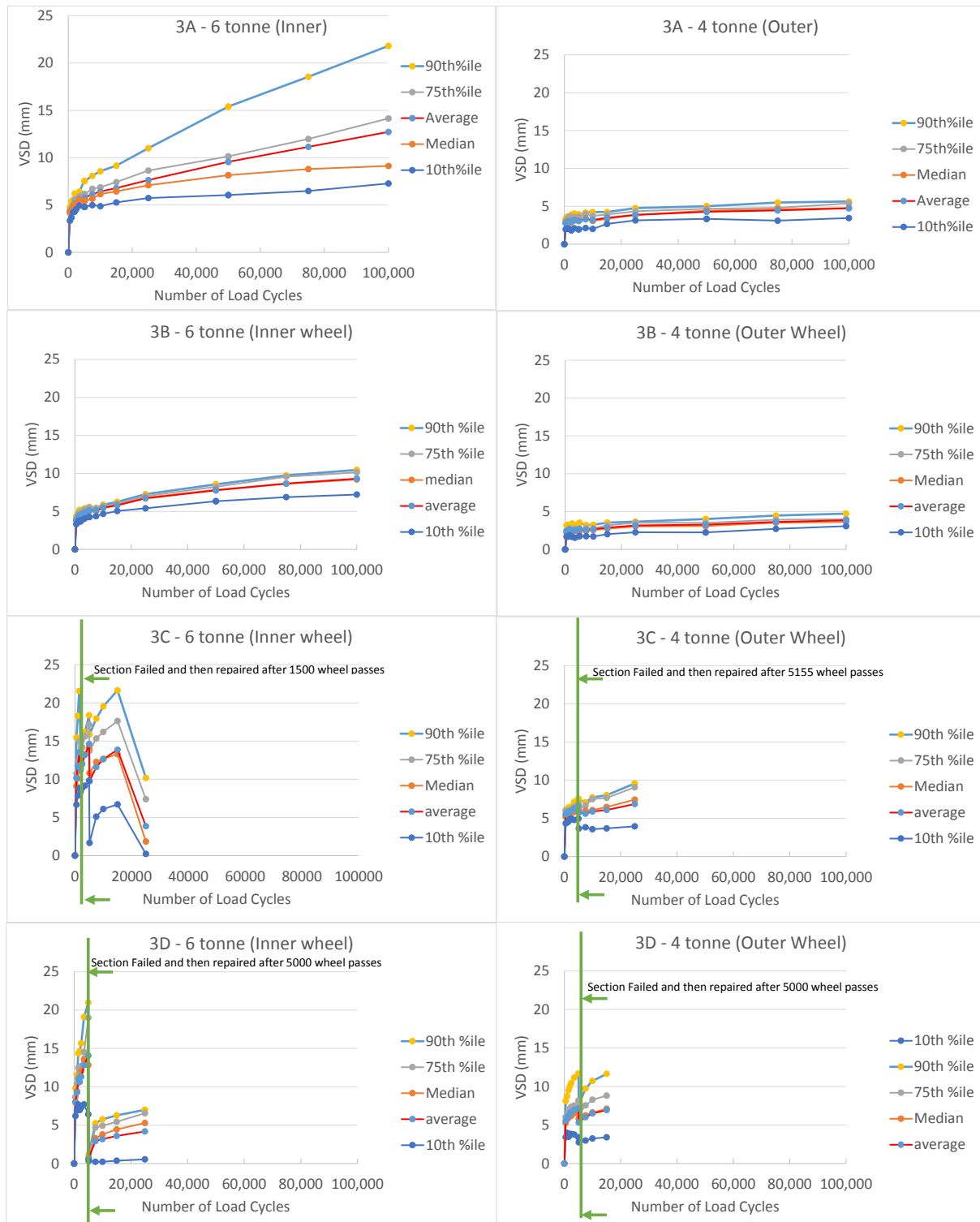
At the measurement intervals, the transverse profile of the pavement was measured at each station. For these measurements, the permanent VSD was calculated.



**Figure 4.5 VSD/rutting of pavement sections for CAPTIF test 1**

**Figure 4.6 VSD/rutting of pavement sections for CAPTIF test 2**

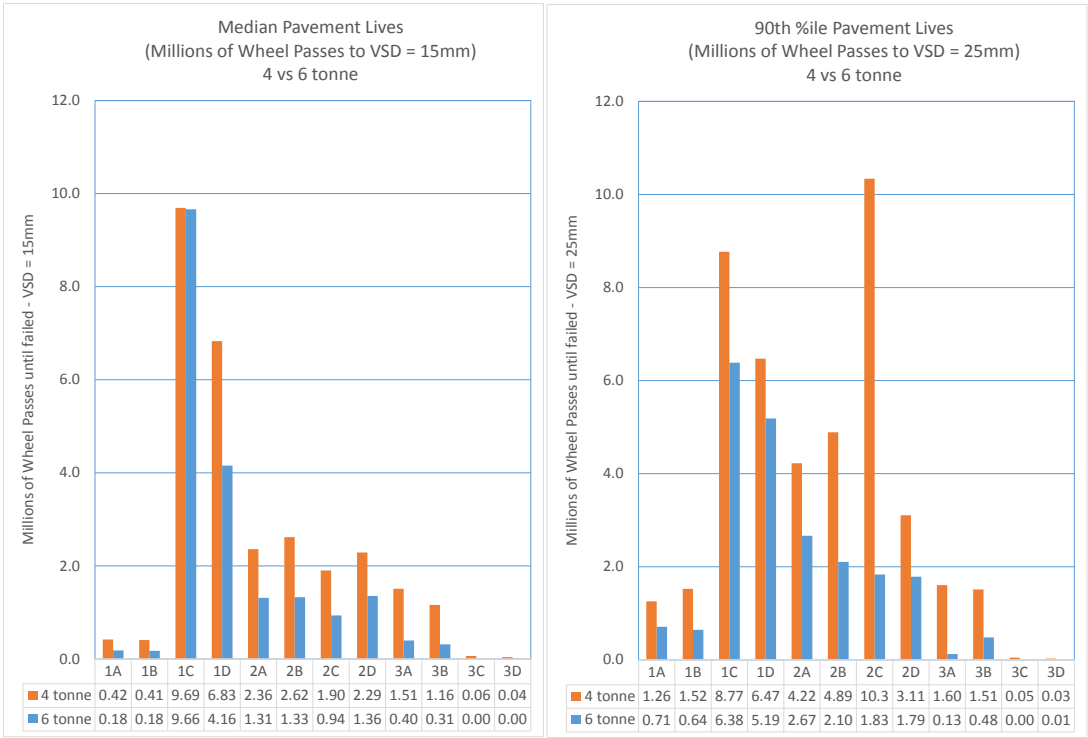


**Figure 4.7 VSD/rutting of pavement sections for CAPTIF test 3**

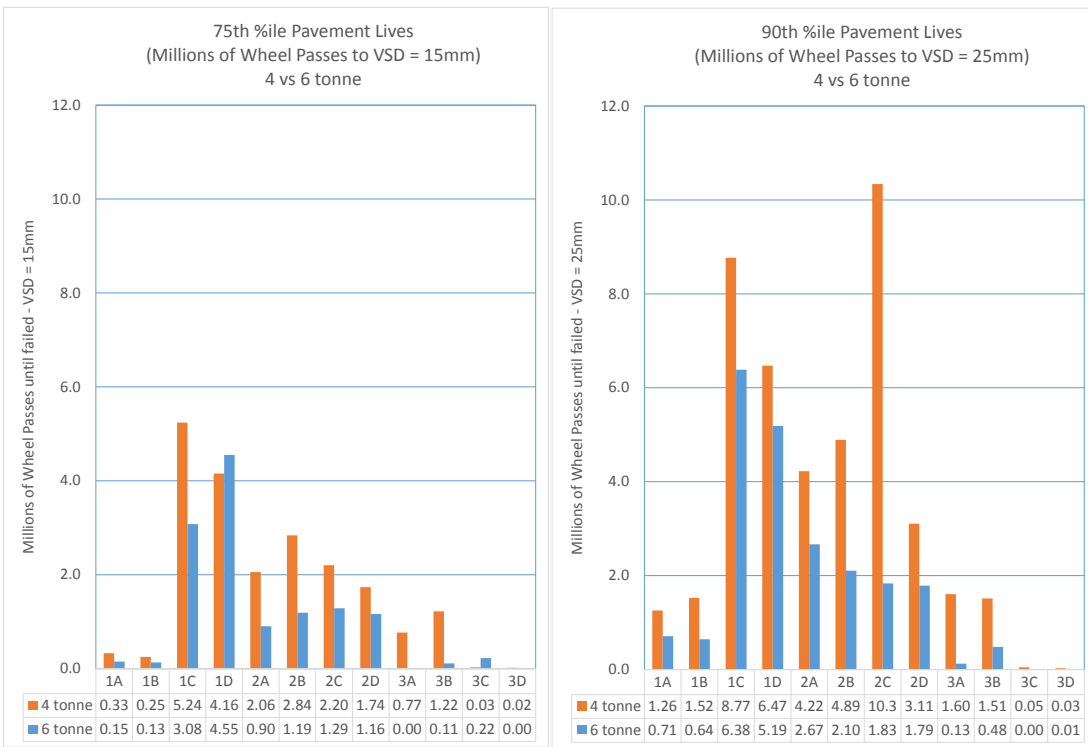
The pavement life was calculated by linear extrapolation of the VSD values to determine the number of wheel passes to reach 15mm and 25mm for pavement sections that had reached this level of rutting/VSD. When evaluating the 90th percentile life a 25mm VSD limit is more appropriate, as a higher rut depth can be tolerated for 10% of the area but for a median life the maximum VSD is 15mm. Plots of the 15mm median life and 25mm 90th percentile life are shown in figure 4.8. A plot showing the 75th percentile life

to reach a VSD of 15mm is also included for comparison (figure 4.9). It should be noted a wide range of methods to define pavement life were investigated as part of this study for determining damage law exponent before settling on the median VSD reading of 15mm.

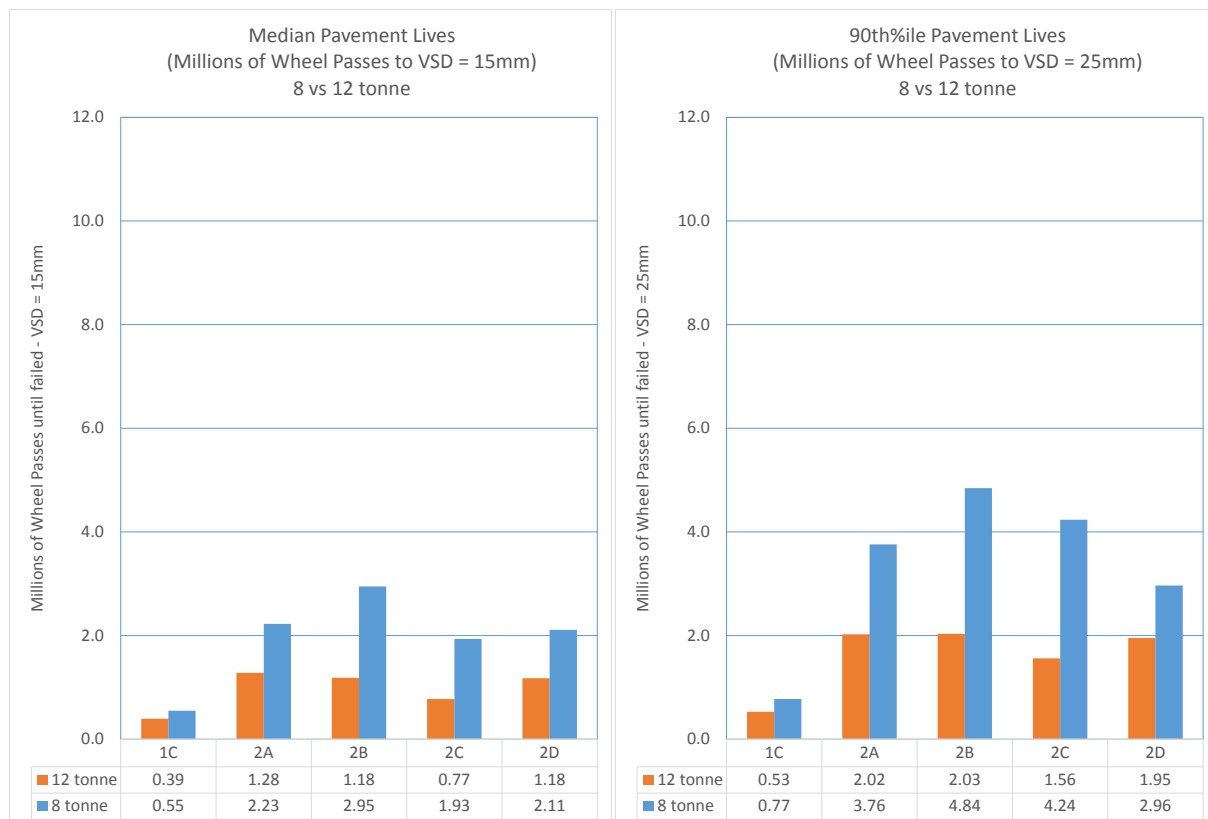
**Figure 4.8** Pavement lives calculated from VSD for 4 tonne vs 6 tonne for median and 90th percentile lives to reach 15 and 25mm respectively



**Figure 4.9** Pavement lives calculated from VSD for 4 tonne vs 6 tonne for 75th percentile and 90th percentile lives to reach 15 and 25mm respectively



**Figure 4.10 Pavement lives calculated from VSD for 8 tonne vs 12 tonne for median and 90th percentile lives to reach 15 and 25mm respectively**



### 4.3 Damage law exponents from rut depth (VSD) measurements

An analysis was undertaken on the actual rut depth (VSD) measurements from the CAPTIF test to determine the relative damage exponent between the 4 and 6 tonne axle loads for each pavement test section (CAPTIF 1, 2 and 3 with sections A, B, C and D). Generally this analysis approach may only require some data extrapolation to a failure criteria of a VSD of 15mm or 25mm. Damage law exponents were determined by pairing the lives found from 4 and 6 tonne loads as these tests were conducted at the same time. Thus the lives from the 8 and 12 tonne loads were also paired. Trying to pair the 8 tonne and 6 tonne loads with different loading histories was unsuccessful as in some cases the lives calculated from the 8 tonne load were higher than those from the 6 tonne load. This was because the 8 and 12 tonne loads were applied after 1 million load cycles of the 4 and 6 tonne loads, rather than being applied on a newly constructed pavement. Thus the trafficking by the 4 and 6 tonne loads although causing some rutting also had the effect of applying more compaction to the pavement, which made it stronger and reduced the amount of rutting from the 8 tonne wheel load.

**Figure 4.11 Range of damage law exponents calculated for each section (4 vs 6 tonnes)**

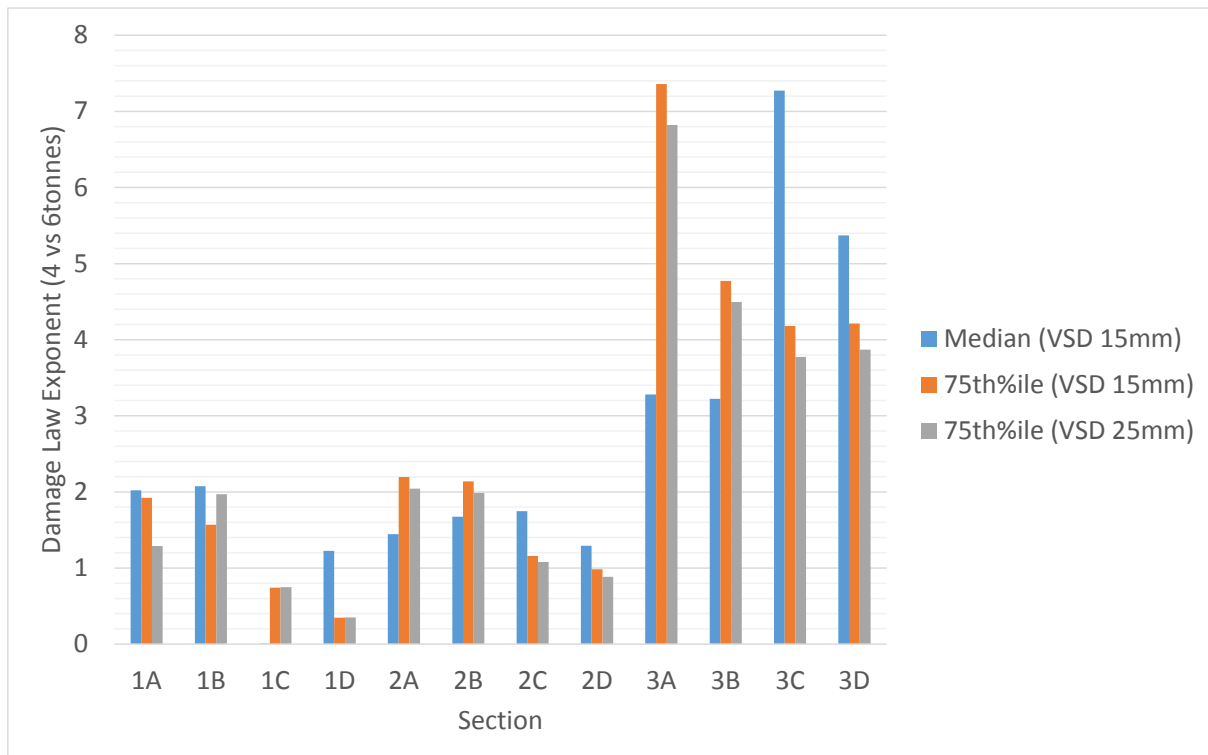
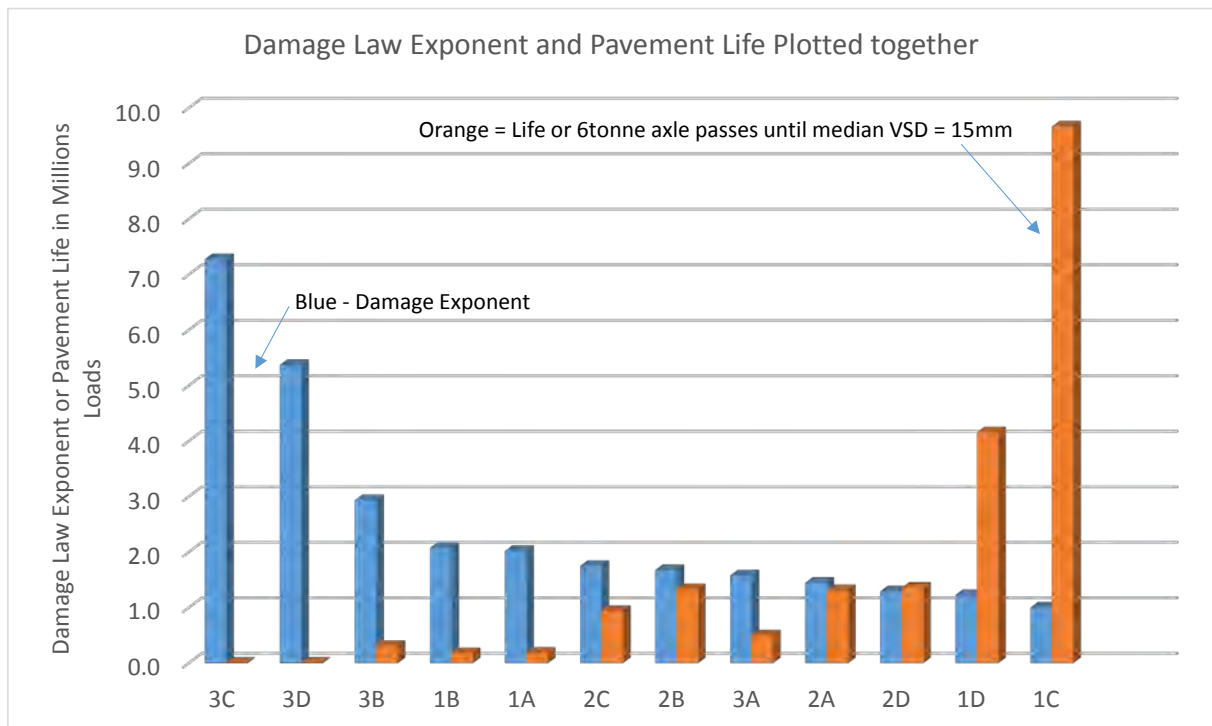
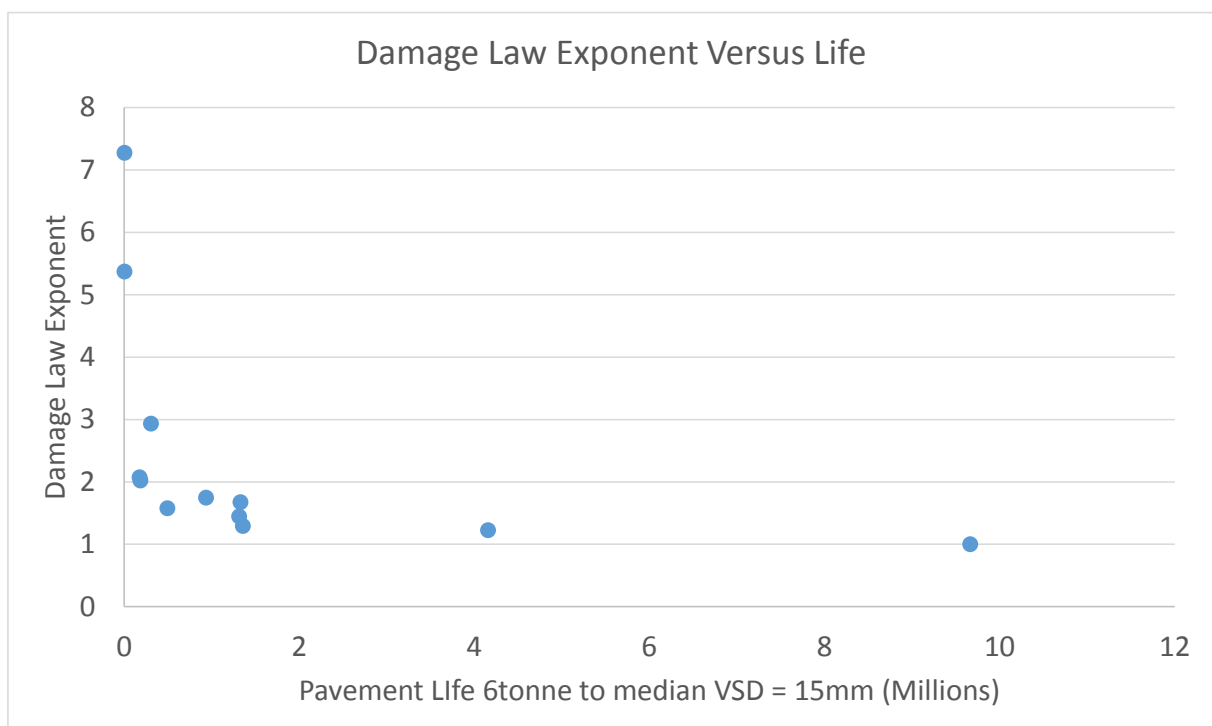


Figure 4.11 shows that the damage law exponents were from 1 to 2 for all the sections in CAPTIF tests 1 and 2. For CAPTIF test 3 the damage law exponents were all greater than 3. This trend in results was explored further and it was found the damage law exponent could be related to pavement life as shown in figures 4.12, 4.13 and 4.14.

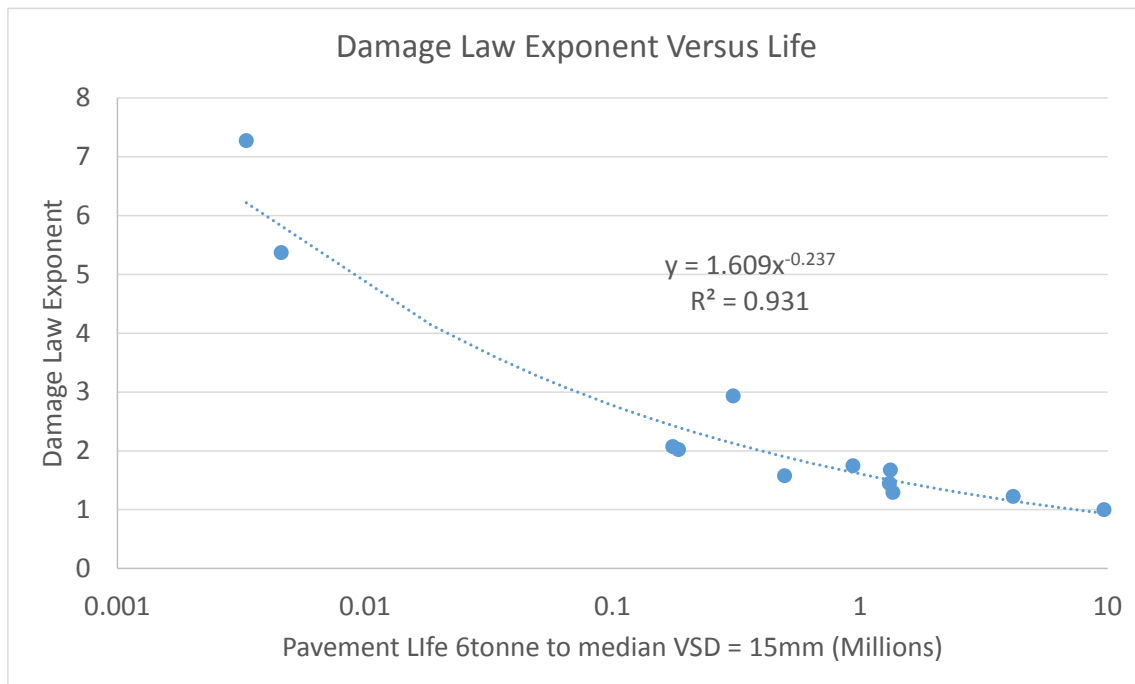
**Figure 4.12 Damage law exponents versus pavement life (defined when median VSD= 15mm) for each section (4 vs 6 tonnes)**



**Figure 4.13 Damage law exponents versus pavement life (defined when median VSD= 15mm) for each section (4 vs 6 tonnes)**

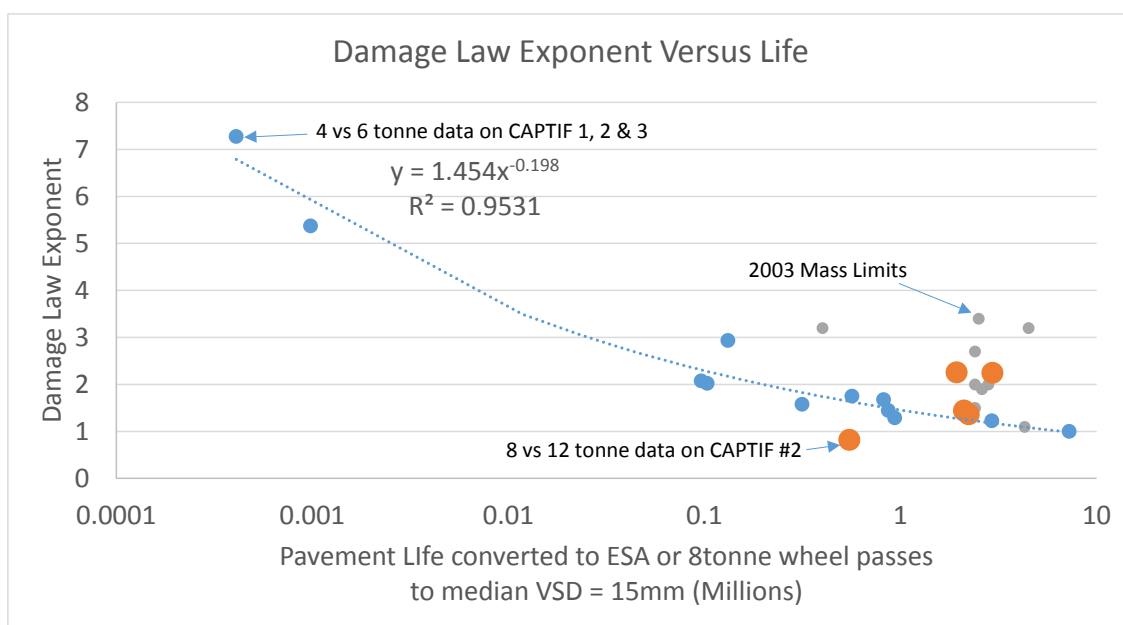


**Figure 4.14** Damage law exponents versus pavement life (defined when median VSD= 15mm) for each section (4 vs 6 tonnes)



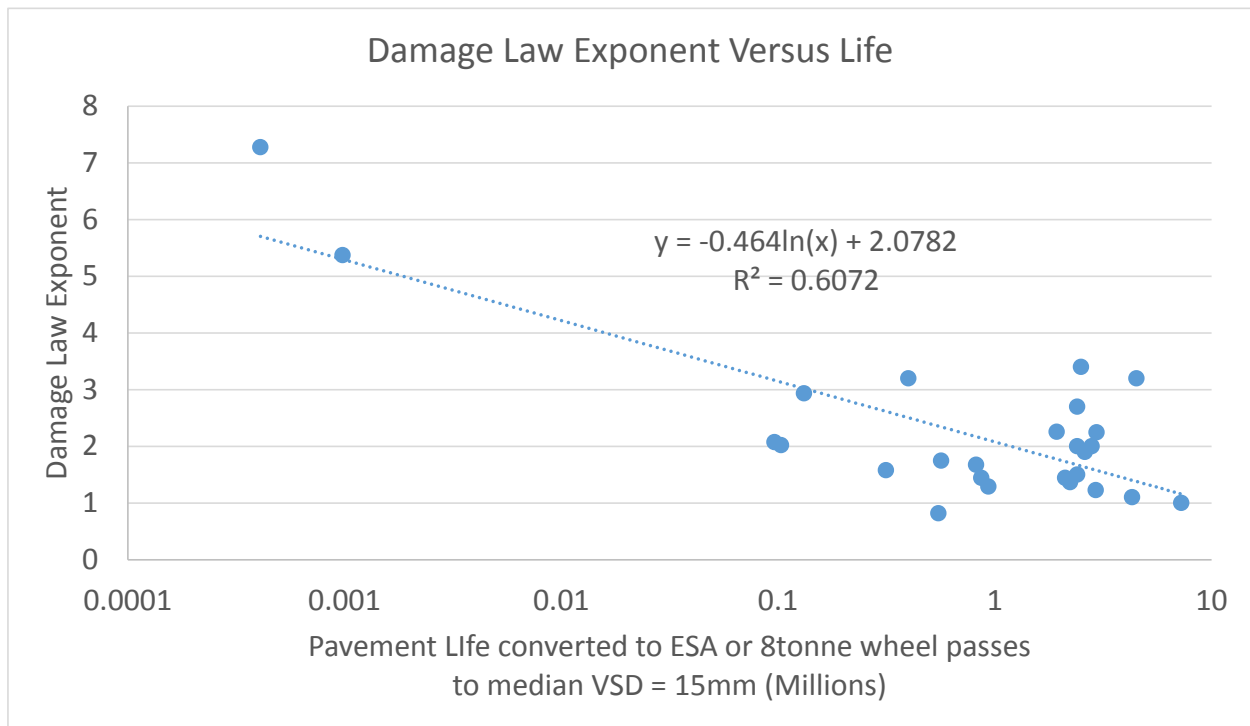
Data from the previous 2003 mass limits project (Arnold et al 2005c) was included in the life versus damage law exponent plot, as was the 8 vs 12 tonne data from test 2. Results of these inclusions of data are shown in figure 4.15. Combining all the data is shown in figure 4.16 and this does reduce the r-squared value. However it should be noted that the 2003 project used a harsher limit for life as it was for state highways, which are maintained to a higher level to support more traffic.

**Figure 4.15** Damage law exponents versus pavement life (defined when median VSD= 15mm) for each section (4 vs 6 tonnes) with 2003 mass limits data included (shown in grey) and 8 vs 12 tonne data (shown in orange)





**Figure 4.16** Damage law exponents versus pavement life (defined when median VSD= 15mm) for each section (4 vs 6 tonnes) with 2003 mass limits data and 8 vs 12 tonne data included



## 5 Rut depth modelling

Rut depth modelling has been undertaken to extend the dataset from CAPTIF. This allows damage law exponents to be explored on a wider range of materials and conditions.

### 5.1 Repeated load triaxial tests to obtain parameters for rut depth models

The RLT apparatus (figure 5.1) was used to obtain parameters for the rut depth models. The RLT applies repetitive loading on cylindrical materials for a range of specified stress conditions. The output is deformation (shortening of the cylindrical sample) versus the number of load cycles (usually 50,000) for a particular set of stress, density and moisture conditions. The method developed by Arnold (2004) for interpreting the RLT results involves relating stress to permanent deformation found from the test.

**Figure 5.1** Repeated load triaxial apparatus



A standard test for granular materials has been developed and detailed in NZTA T/15 *Specification for repeated load triaxial testing (RLT) of pavement materials* which gives stresses suitable for testing granular materials. However, these stresses are not suitable for testing subgrade soils which would fail by shear after the first load is applied. To determine appropriate stresses to test soils in a 50,000 load cycle six-stage permanent strain an identical sample is first tested in a 30-stage test of 100 cycles for each stage. The 30-stage test keeps the cell pressure constant but after each stage increases the vertical repetitive load by 10kPa. The 30-stage test is never fully completed as the soil fails at a loading stage that exceeds the strength of the soil. The load that the soil failed at could be 100kPa and thus the six-stage permanent strain test is designed so the six loading stages are less than the load where the soil will fail (in this case 100kPa). Each stage results in an increase in loading and the stage when the soil fails represents the highest loading that can be applied. Ideally at least six full stages are completed to provide enough data for the rut depth models. In this research project, many RLT tests were conducted on subgrade soils of CBR 2, 8 and 10 as used at CAPTIF to determine the necessary parameters for the rut depth model.

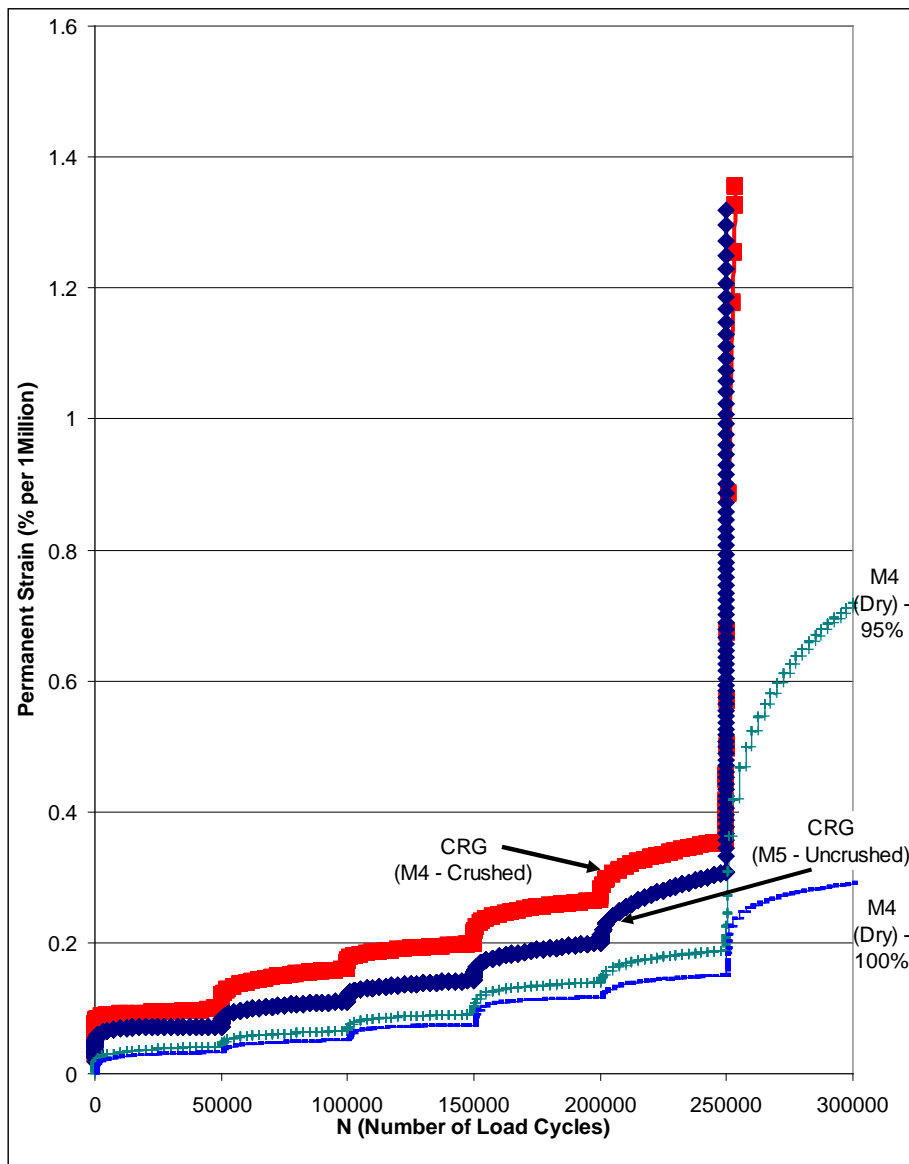
### 5.2 Repeated load triaxial test results – aggregates

Three basecourse aggregate types were used in the CAPTIF tests:

- Canterbury greywacke river gravel (CRG) crushed NZTA M4 AP40
- Tauhara (near Taupo) dacite NZTA M4 AP40
- Canterbury greywacke river gravel (CRG) uncrushed M5 AP40.

As these aggregates were used as a basecourse in the upper pavement layers the standard NZTA T15 RLT tests were conducted at the range of densities achieved in the constructed pavement. A range of dry densities and moisture contents tested on the aggregates are shown in tables 5.1, 5.2 and 5.3.

**Figure 5.2 Example RLT results on CRG crushed and uncrushed used at CAPTIF 2 test**



Results of the RLT testing showed the M5 uncrushed river gravel having the lowest initial compaction but a higher rate of permanent strain in the 5th stage.

**Table 5.1 Range of densities and moisture contents tested for the aggregates used in CAPTIF test 1**

Material reference number	CAPTIF test #	Material description	Moisture content (%)	Dry density (kg/m <sup>3</sup> )
1_M4_10th	1	Canterbury M4 – lowest (10th percentile) strength (wettest/weakest)	6	2210
1_M4_50th	1	Canterbury M4 – median 50th percentile strength (median)	6	2268
1_M4_90th	1	Canterbury M4 – highest (90th percentile) strength (driest/strongest)	4.3	2325
1_dacite_10th	1	Tauhara dacite M4 – lowest (10th percentile) strength (wettest/weakest)	7.5	1920
1_dacite_50th	1	Tauhara dacite M4 – median 50th percentile strength (median)	7.5	1925
1_dacite_90th	1	Tauhara dacite M4 – highest (90th percentile) strength (driest/strongest)	7.5	1989

**Table 5.2 Range of densities and moisture contents tested for the aggregates used in CAPTIF test 2**

Material reference number	CAPTIF test #	Material description	Moisture content (%)	Dry density (kg/m <sup>3</sup> )
2_M4_10th	2	Canterbury M4 – lowest (10th percentile) strength (wettest/weakest) (T14/793A)	4.4	2.222
2_M4_50th	2	Canterbury M4 – median 50th percentile strength (median) (T14/793B)	4.1	2.285
2_M4_90th	2	Canterbury M4 – highest (90th percentile) strength (driest/strongest) (T14/793C)	4.1	2.328
		<i>Canterbury M4 – 10 seconds per layer as per NZTA T15 (T14/3629A)</i>	4.4	2.197
2_M5_10th	2	River gravel M5 – lowest (10th percentile) strength (wettest/weakest)	4.2	2.210
2_M5_50th	2	River gravel M5 – median 50th percentile strength (median)	4.2	2.243
2_M5_90th	2	River gravel M5 – highest (90th percentile) strength (driest/strongest)	4.2	2.282
		<i>River gravel M5 – 10 seconds per layer as per NZTA T15 (T14/3630A)</i>	4.2	2.295

**Table 5.3 Range of densities and moisture contents tested for the aggregates used in CAPTIF test 3**

Material reference number	CAPTIF test #	Material description	Moisture content (%)	Dry density (kg/m <sup>3</sup> )
3_M4_10th	3	Canterbury M4 – lowest (10th percentile) strength (wettest/weakest)	4.5	2190
3_M4_50th	3	Canterbury M4 – median 50th percentile strength (median)	4.5	2228
3_M4_90th	3	Canterbury M4 – highest (90th percentile) strength (driest/strongest)	4.5	2261
		<i>Canterbury M4 – 10 seconds per layer as per NZTA T15</i>	<i>4.5</i>	<i>2147</i>
3_M5_10th	3	River gravel M5 – lowest (10th percentile) strength (wettest/weakest)	4.5	2172
3_M5_50th	3	River gravel M5 – median 50th percentile strength (median)	4.5	2211
3_M5_90th	3	River gravel M5 – highest (90th percentile) strength (driest/strongest)	4.5	2246
		<i>River gravel M5 – 10 seconds per layer as per NZTA T15</i>	<i>4.5</i>	<i>2282</i>

### 5.3 Todd clay RLT tests

RLT tests were conducted on the Todd clay at a range of dry densities and moisture contents as shown in table 5.4.

**Table 5.4 Range of densities and moisture contents tested for the Todd clay**

Material reference number	CAPTIF test #	Material description	CBR	Moisture content (%)	Dry density (kg/m <sup>3</sup> )
1_TC10th	1	Todd clay – lowest (10th percentile) strength (wettest/weakest)	7	21	1550
1_TC50th	1	Todd clay – median 50th percentile strength (median)	7.5	19	1550
1_TC90th	1	Todd clay – highest (90th percentile) strength (driest/strongest)	8	16	1550
3_TC10th	3	Todd clay – lowest (10th percentile) strength (wettest/weakest)	2	29.6	1459
3_TC50th	3	Todd clay – median 50th percentile strength (median)	3	28	1511
3_TC90th	3	Todd clay – highest (90th percentile) strength (driest/strongest)	5	26.5	1552

## 5.4 Crushed sand AP5 RLT tests

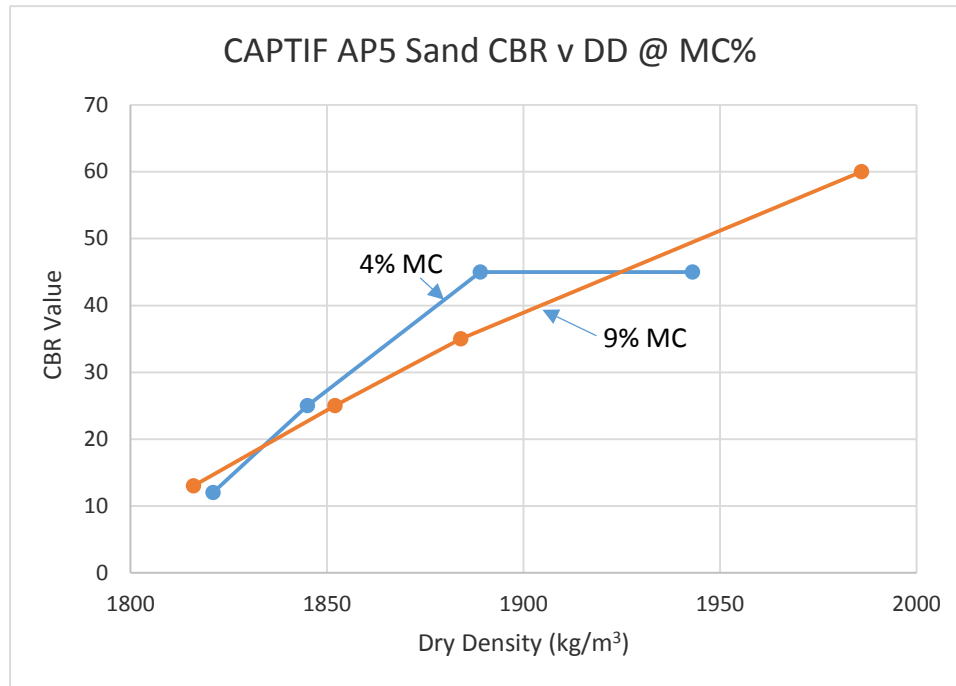
RLT and CBR tests were conducted on the sand at a range of dry densities and moisture contents as shown in tables 5.5 and 5.6 respectively. The CBR results are shown in figure 5.3 and illustrate that density controls strength for the sand.

**Table 5.5 Range of densities and moisture contents tested for the AP5 crushed sand**

Material reference number	CAPTIF test #	Material description	CBR	Moisture content (%)	Dry density (kg/m <sup>3</sup> )
2_Sand_10th	2	AP5 sand – lowest (10th percentile) strength (lowest density/weakest)	35	9	1,892
2_Sand_50th	2	AP5 sand – median 50th percentile strength (median)	45	9	1,949
2_Sand_90th	2	AP5 sand – highest (90th percentile) strength (highest density/strongest)	60	9	1,977
3_Sand_10th	3	AP5 sand – lowest (10th percentile) strength (lowest density/weakest)	13	9	1,816
3_Sand_50th	3	AP5 sand – median 50th percentile strength (median)	25	9	1,851
3_Sand_90th	3	AP5 sand – highest (90th percentile) strength (highest density/strongest)	35	9	1,884

**Table 5.6 CBR tests on a range of densities and moisture contents for the AP5 crushed sand**

CBR	Moisture content (%)	Dry density (kg/m <sup>3</sup> )
45	4	1943
12	4	1821
25	4	1845
45	4	1889
60	9	1986
13	9	1816
25	9	1852
35	9	1884

**Figure 5.3** Dry density versus CBR for the AP5 sand

## 5.5 Rut depth prediction method

### 5.5.1 Introduction

The first step in predicting rutting is to undertake multi-stage permanent strain RLT tests on the materials in the pavement including the subgrade, subbase and basecourse. This allows relationships between stress and permanent deformation to be determined for each material in the pavement. From the RLT tests, a relationship between stress and resilient modulus is also determined for use in a finite element programme to determine stresses and strains in the pavement at incremental depths. These stresses and strains are imported into a spreadsheet where the rut depth is calculated at each depth increment using the relationships to predict rutting found from RLT testing. The background to this approach can be found in Arnold (2004).

## 5.6 Predicting rut depth at CAPTIF for all sections

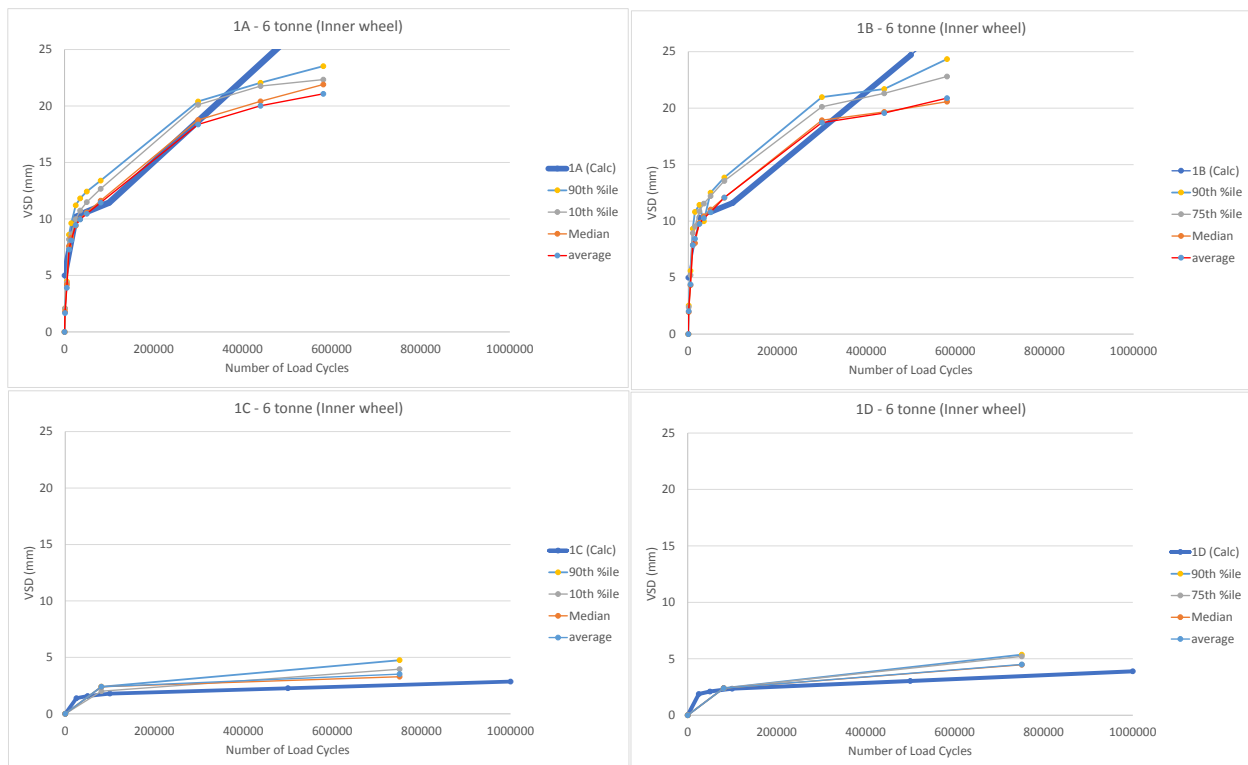
Rutting (VSD) predictions were calculated for all the CAPTIF tests. Figures 5.4 to 5.9 show the predicted rut depths compared with measured VSD (rut) depth found for all the sections. It was found that to get accurate predictions minor adjustments to the initial rut depths were generally required. The sand layer in CAPTIF test 3 caused problems for the model as it had a very low density. This resulted in early failure within a few load cycles which did not match actual performance. However, when the model for the sand layer was repeated at a higher density, as done for CAPTIF test 2, then the predictions of VSD were reasonably close to those measured. This suggested that either the compaction of the overlying aggregate and asphalt layers further compacted the sand layer or the initial density measurements were wrong.

For CAPTIF test 1 the best predictions were obtained with the RLT test on the Tauhara dacite at the lowest density (resulting in higher rutting in the aggregate) and the drier Todd clay test at 16% moisture content (resulting in less rutting in the subgrade).

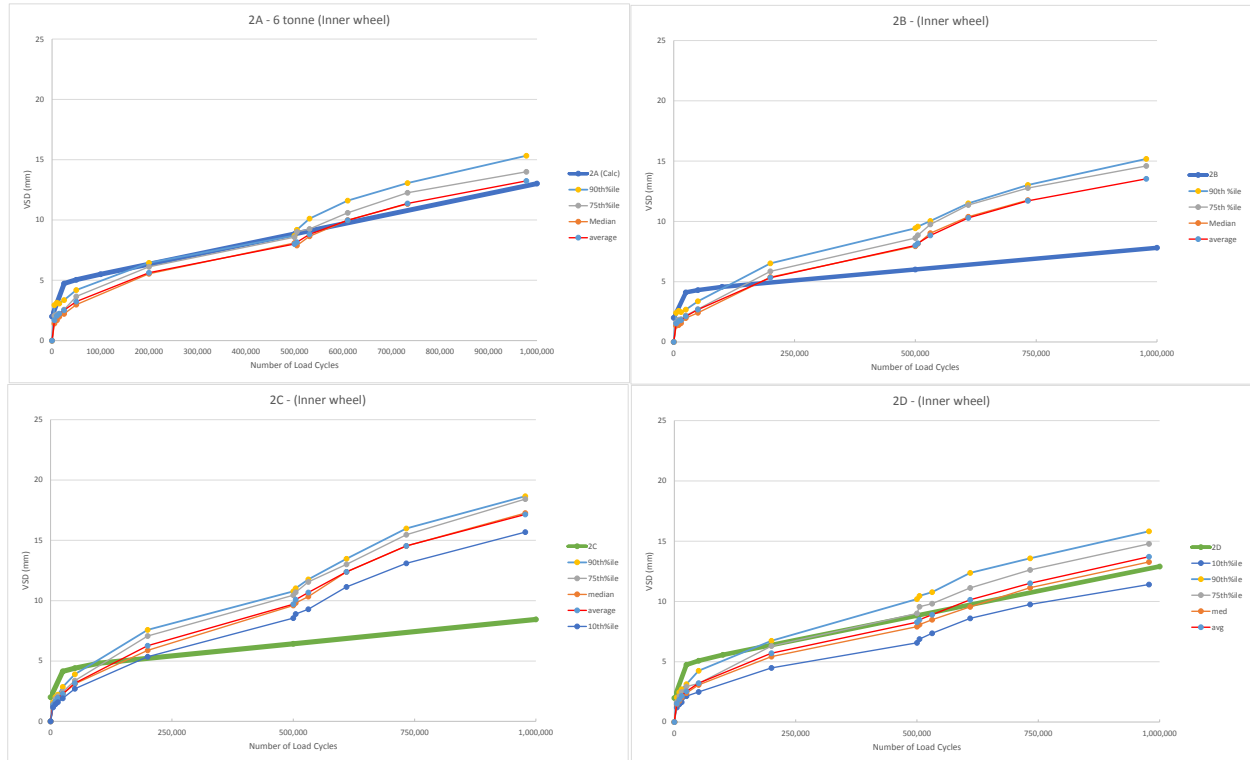
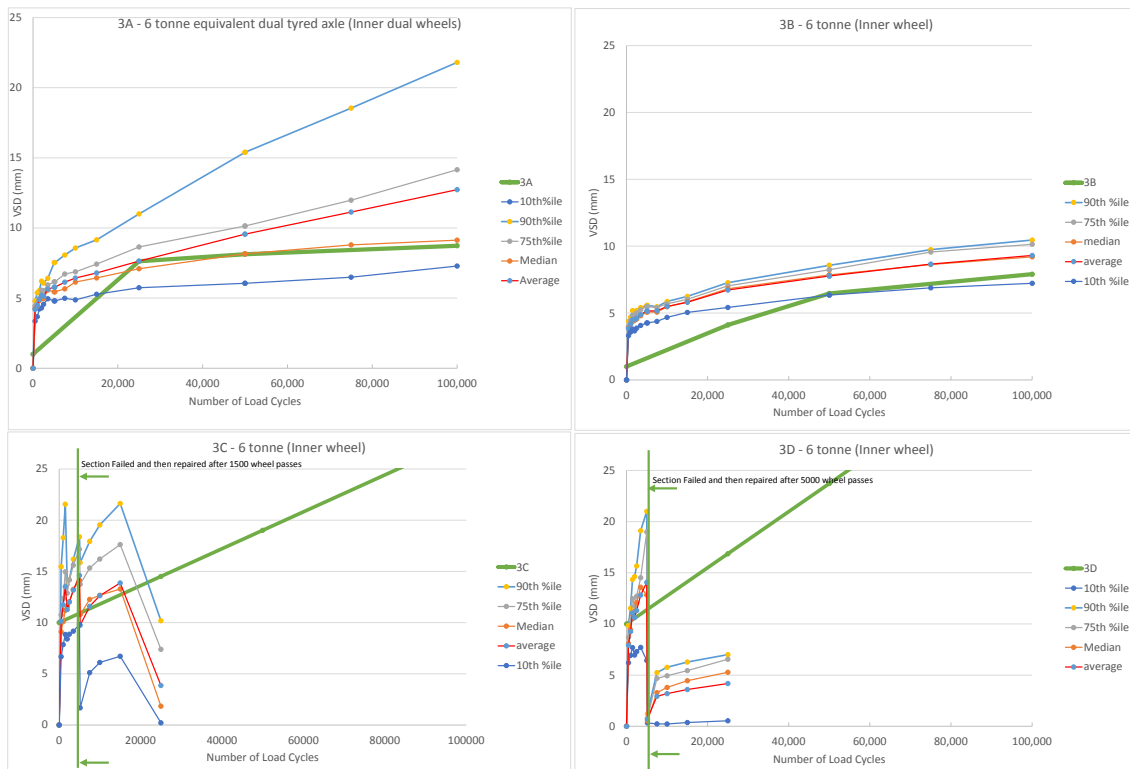
CAPTIF test sections 3C and 3D predicted rapid failure and, because of this unstable behaviour, predictions of pavement life although short were not as short as observed at the test track. As a result of this inaccuracy the calculated damage exponents from the computed life for sections 3C and 3D were later excluded as this was not typical pavement behaviour.

Poor prediction of rut depths was observed for sections 2B and 2C with the 6 tonne axle load, shown in figure 5.5. Sections 2B and 2C had a basecourse aggregate depth of 200mm while sections 2A and 2D had only 100mm of aggregate. It was expected from the modelling that sections 2B and 2C would have less rutting and a longer life as they had thicker basecourse layers. However, the actual rutting measured in the CAPTIF test was higher than modelled and was nearly identical to sections 2A and 2D. Past tests at CAPTIF have shown that basecourse depth (unless extremely shallow on weak subgrades) has little influence on life, and these pavements were effectively on a very strong sand subgrade.

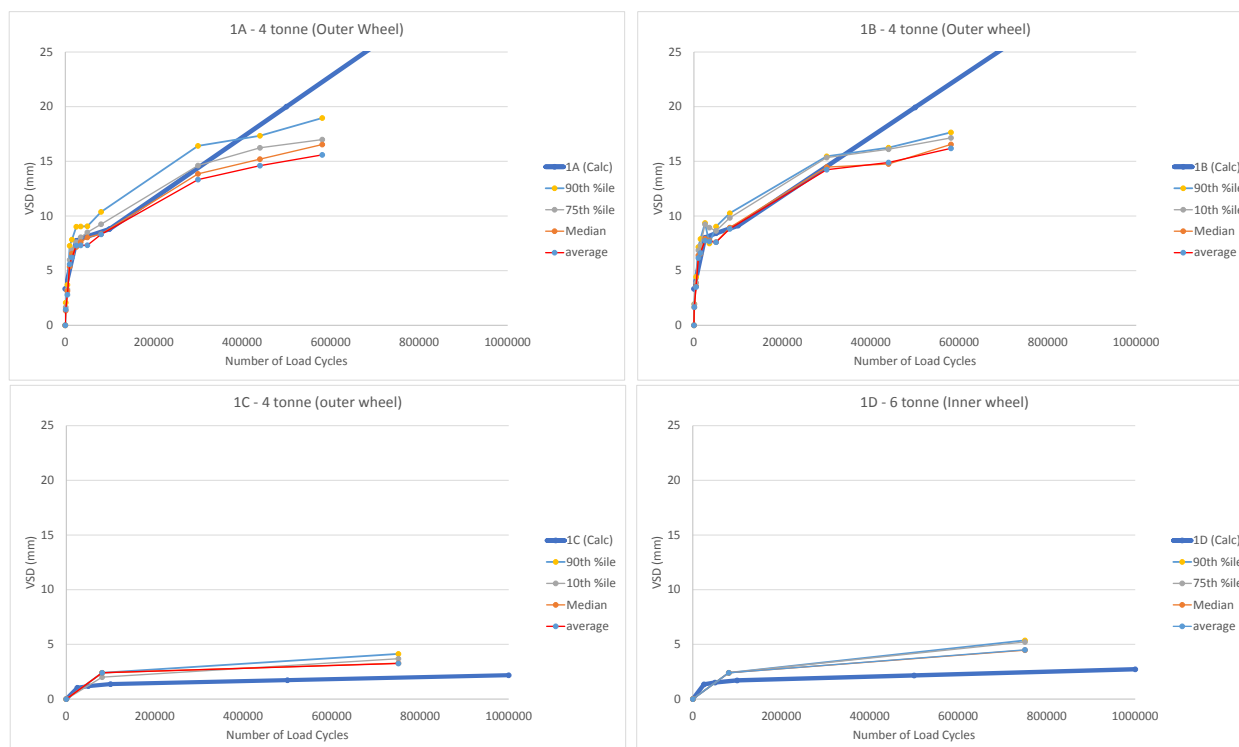
**Figure 5.4 Predicted VSD compared with measured for CAPTIF test 1 (6 tonne)**



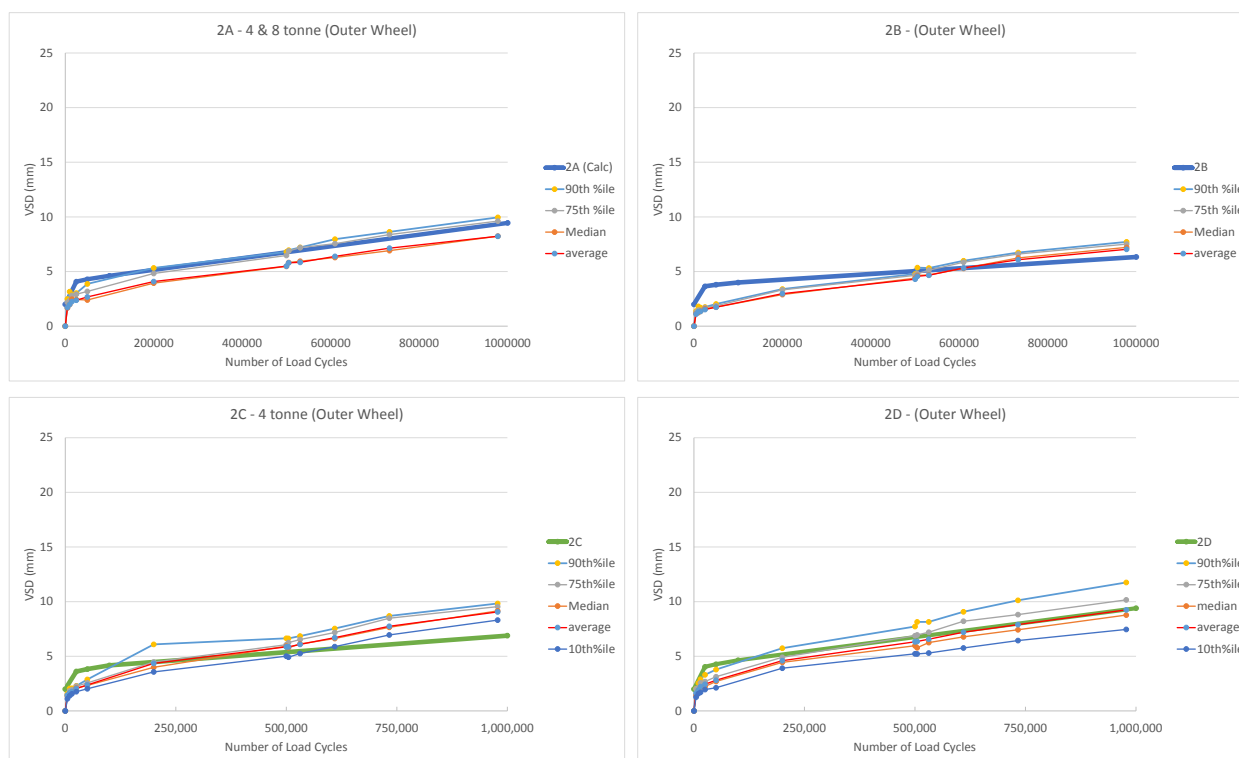


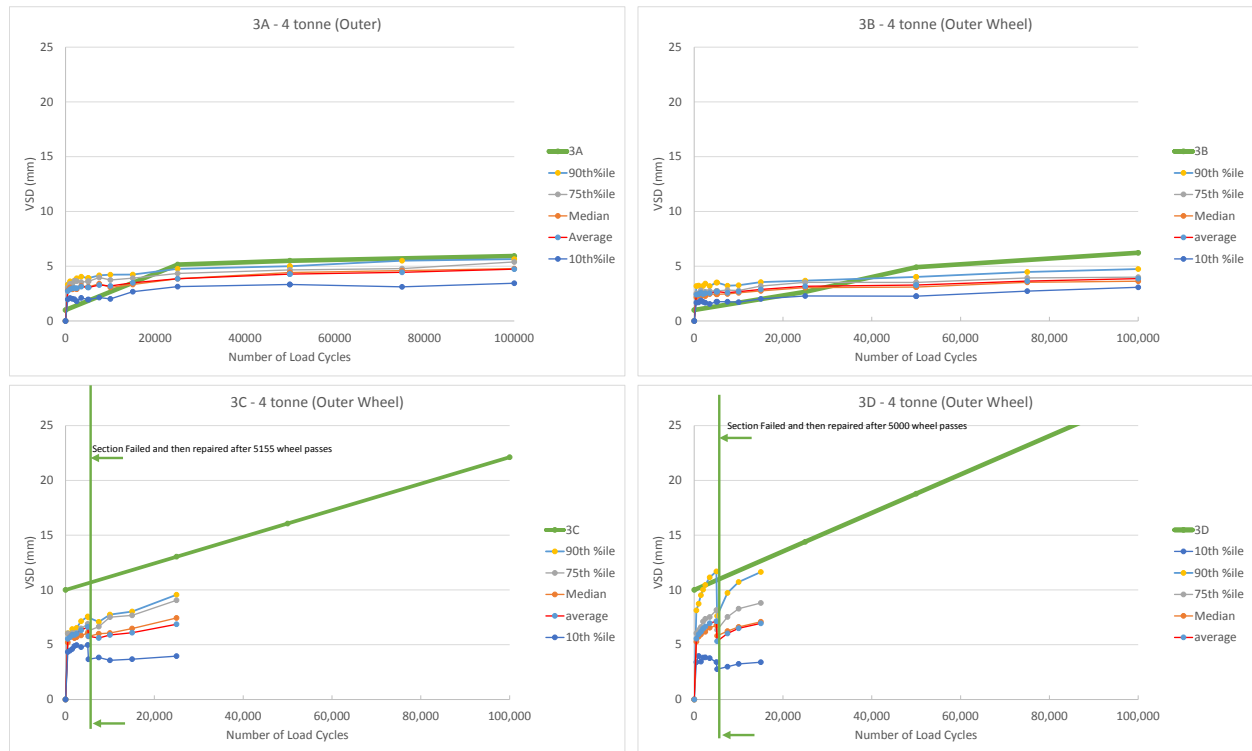
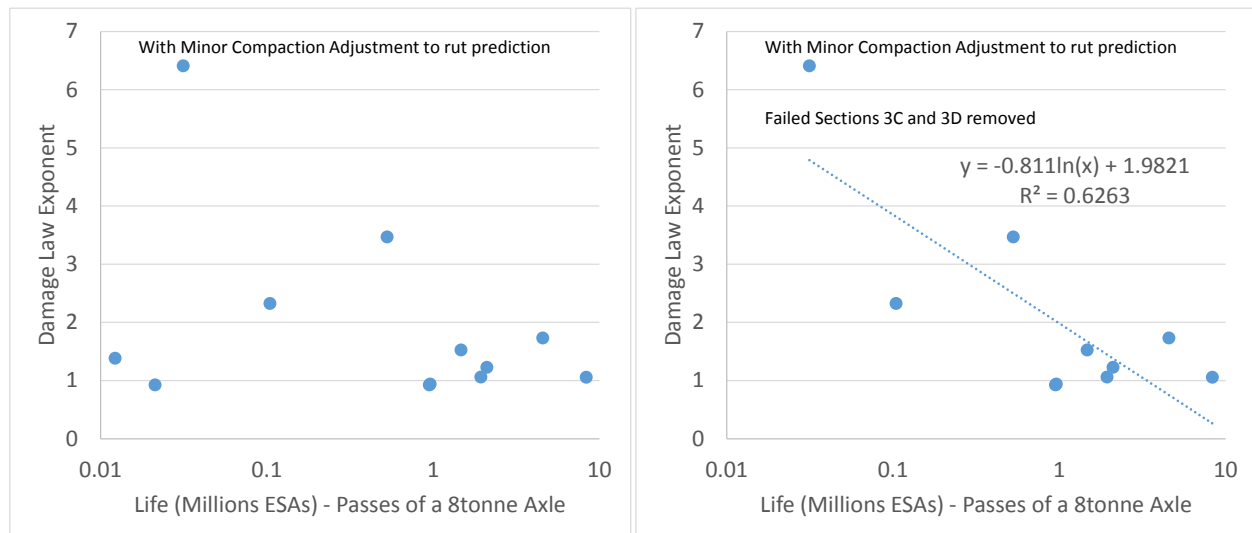
**Figure 5.5 Predicted VSD compared with measured for CAPTIF test 2 (6 tonne)****Figure 5.6 Predicted VSD compared with measured for CAPTIF test 3 (6 tonne)**

**Figure 5.7 Predicted VSD compared with measured for CAPTIF test 1 (4 tonne)**

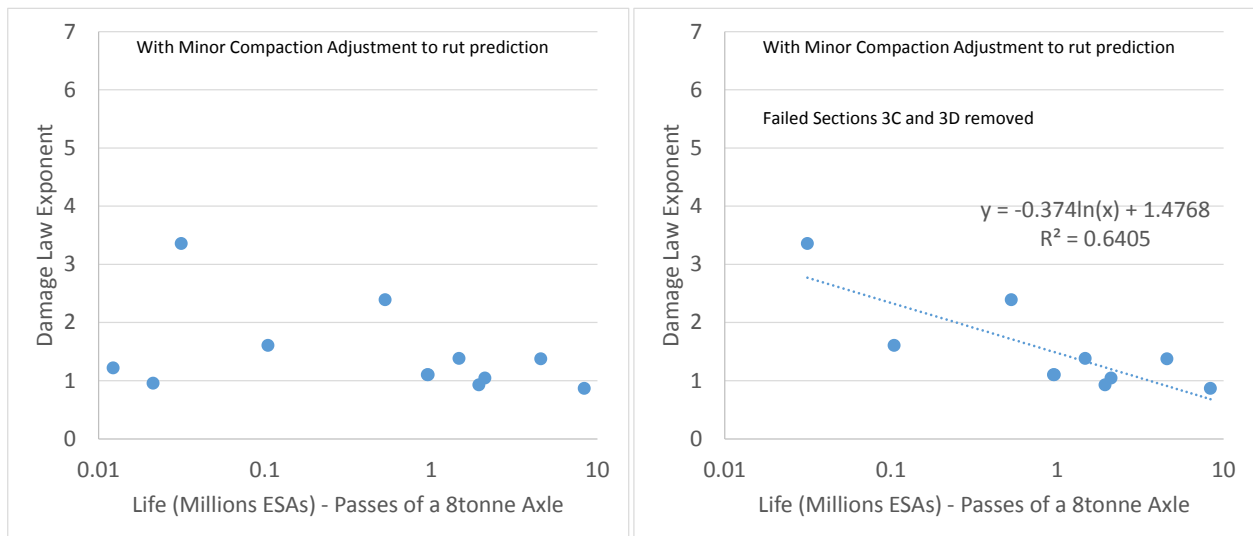


**Figure 5.8 Predicted VSD compared with measured for CAPTIF test 2 (4 tonne)**

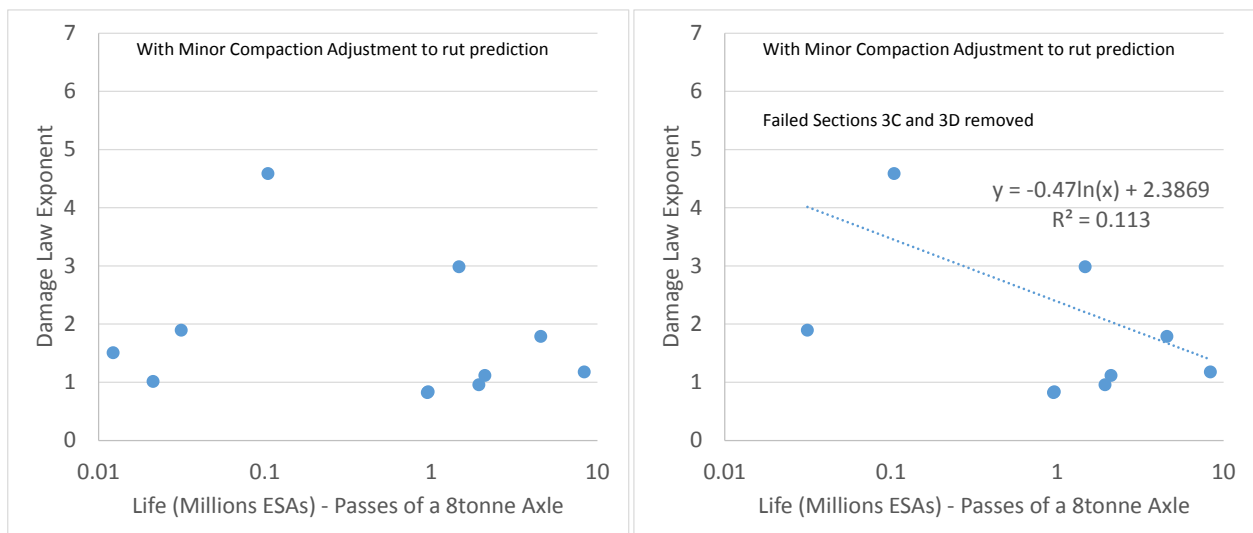


**Figure 5.9 Predicted VSD compared with measured for CAPTIF test 3 (4 tonne)****Figure 5.10 Damage law exponents calculated from predicted lives for 6 tonne wheel path (inner)**

**Figure 5.11 Damage law exponents calculated from predicted lives for 4 tonne wheel path (inner)**



**Figure 5.12 Damage law exponents calculated from predicted lives for 12 tonne wheel path (inner)**



Figures 5.10, 5.11 and 5.12 show the calculated damage exponents for each of the modelled axle weights. There is a general trend of higher damage law exponents for the 4 and 6 tonne axle loads with an  $R^2$  of around 0.6. This trend is similar to the damage law exponent calculated from actual measure VSD values at CAPTIF. However, this relationship with life is not as strong with the 12 tonne axle loads modelled to predict rutting, presumably as the model is starting to predict range C behaviour from Dawson's model.

Most of the damage exponents calculated are between 1 and 2 for the 4, 6 and 12 tonne axle loads compared with the 8 tonne axle load. One reason for this is the tyre pressures for all the axle loads are nearly the same to match the testing at CAPTIF. Tyre pressures used at CAPTIF were those recommended by the transport industry as these are commonly used on New Zealand roads. For the 4 and 6 tonne axle loads, the tyre pressures modelled and tested were 758kPa while for the 8 and 12 tonne loads the tyre pressures used were 827kPa. The effect of tyre pressure in the model was explored, in summary reducing the tyre pressure, (particularly if it is assumed, as here, that low pressures can only be run with low loads). This resulted in a longer life of the lower loaded axles causing a larger difference in lives between the high

and low axle loads and thus a higher damage exponent result, which was not unexpected. More detailed results can be found in appendix A.

The validity of the modelling approach was also explored through comparison with in situ instrumentation. This indicated the model was predicting the right amounts of relative damage in each layer, providing further confidence in its predictions. The details of the comparison can be found in appendix C.

## 5.7 Modelling typical LTPP pavements

For this study four pavements were modelled representing a range of LTPP sites. The following granular pavements with 4, 6, 8 and 12 tonne dual tyre axle loads were modelled:

- 250mm of aggregate on subgrade CBR 8
- 400mm of aggregate on subgrade CBR 8
- 400mm of aggregate on subgrade CBR 2
- 600mm of aggregate on subgrade CBR 2.

The analysis was repeated by modelling two different aggregate qualities (average and poor rut resistance as assessed by the RLT test). Eight pavement types were analysed four times with axle loads 4, 6, 8 and 12 tonnes. These pavement types were chosen because of available RLT data and to cover a range of pavements in the LTPP database (table 5.7).

The relationship between vehicle axle loadings and pavement wear on local roads

**Table 5.7 LTPP sites with similar characteristics as those modelled**

ID	cal- 34	cal- 43	cs- 14	cs- 20	cs- 21	cs- 24	cs- 42	cs- 52b
<b>Location</b>	SH45 RP 97/5.5	SH1 RP 447/2.8	SH27 50/0.77	SH31 RP 0/5.65	SH35 RP 250/10.07	SH2 544/12.52	SH73 RP91/2.7	SH83 RP 0/4.8
<b>Network</b>	West Wanganui (Taranaki)	South Canterbury	East Waikato	PSMC (001)	Gisborne	Napier	North Canterbury	Coastal Otago
<b>Strength</b>	Weak pavement	Strong pavement	Strong pavement	Strong pavement	Weak pavement	Strong pavement	Strong pavement	Weak pavement
<b>Age</b>	New pavement	Old pavement	New pavement	New pavement	Old pavement	New pavement	New pavement	Old pavement
<b>Aggregate depth</b>	400	210	400	420	100 (?)	170 (?)	310	240
<b>4 day soaked subgrade CBR</b>	8	6	7	2	2.5	1.5	3.5	1.5
<b>ESA per year</b>	210,461	118,457	255,107	190,444	57,175	133,200	17,943	80,938
<b>Rut rate (mm/1 million ESA)</b>	1.59	-0.53	1.21	0.27	3.17	2.64	8.63	1.82
<b>D0 (mm)</b>	1.24	0.62	0.66	0.69	1.08	0.63	1.46	1.05
<b>Curvature (D0-D200) (mm)</b>	0.45	0.19	0.19	0.18	0.33	0.17	0.74	0.30

Plots showing the axisymmetric finite element models of the four pavements modelled under an 8 tonne load can be found in appendix B.

Results of the modelling are summarised in table 5.8 and are damage law exponent versus life for the different axle weights modelled. These are plotted in figures 5.13 and 5.14.

**Table 5.8 FEM modelling results**

Aggregate depth	Subgrade CBR	Aggregate quality	Axle (tonnes)	Life (axle) (millions)	Life 8 tonne (million ESAs)	Exp
400	2	Poor	4	1.96	0.28	2.8
400	2	Poor	6	0.96	0.28	4.36
400	2	Poor	12	0.02	0.28	6.2
250	8	Poor	4	2.30	0.42	2.5
250	8	Poor	6	1.35	0.42	4.1
250	8	Poor	12	0.03	0.42	6.7
400	8	Poor	4	2.36	0.75	1.7
400	8	Poor	6	1.57	0.75	2.6
400	8	Poor	12	0.33	0.75	2.0
400	2	Average	4	4.14	0.88	2.2
400	2	Average	6	2.10	0.88	3.0
400	2	Average	12	0.02	0.88	9.1
250	8	Average	4	5.02	1.12	2.2
250	8	Average	6	2.72	1.12	3.1
250	8	Average	12	0.03	1.12	9.2
600	2	Poor	4	3.10	1.18	1.4
600	2	Poor	6	2.13	1.18	2.0
600	2	Poor	12	0.33	1.18	3.2
400	8	Average	4	7.03	2.91	1.3
400	8	Average	6	4.84	2.91	1.8
400	8	Average	12	1.61	2.91	1.5
600	2	Average	4	8.09	3.07	1.4
600	2	Average	6	5.17	3.07	1.8
600	2	Average	12	1.20	3.07	2.3

Figure 5.13 Damage exponent calculated from FEM for 4, 6 and 12 tonne axles

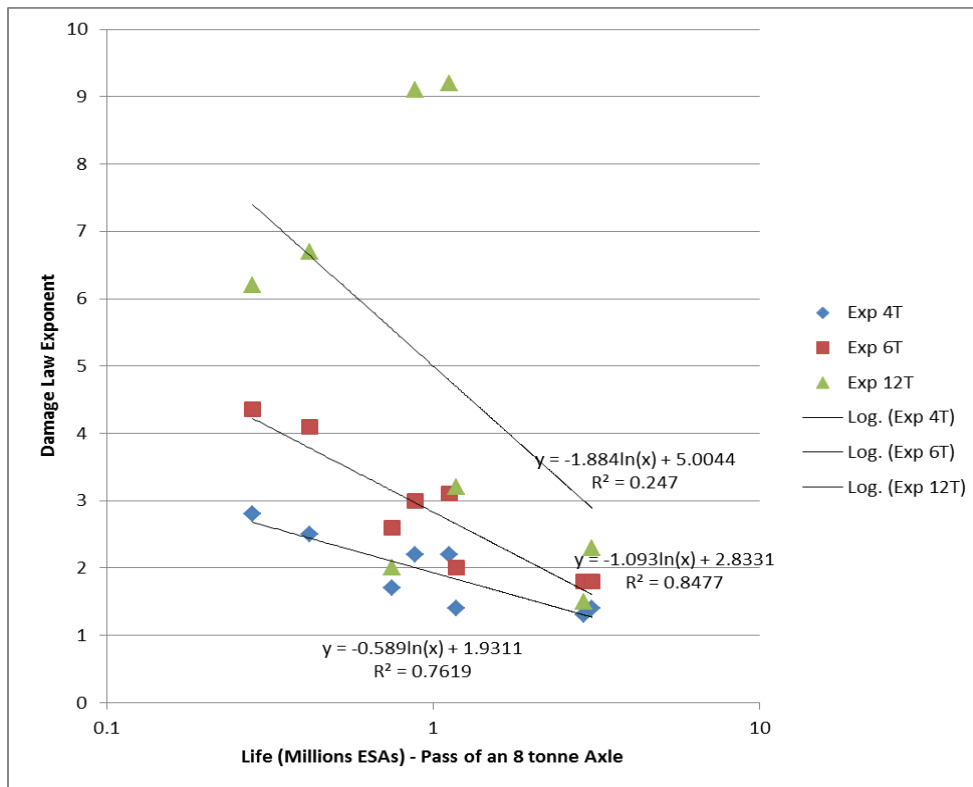
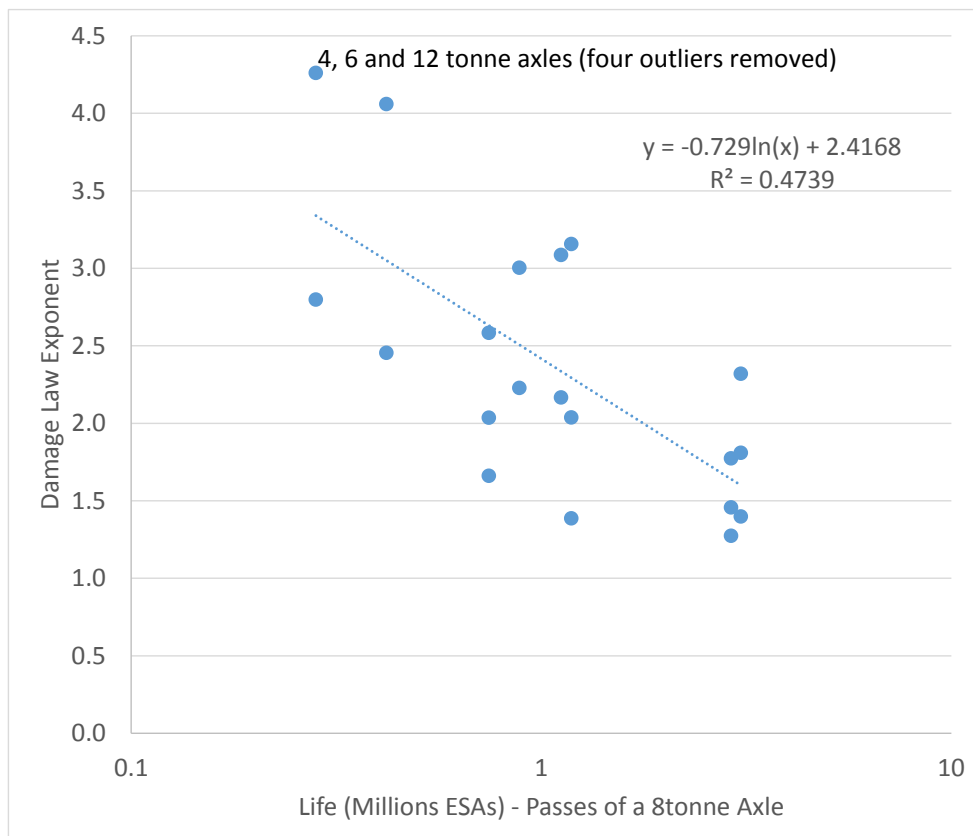


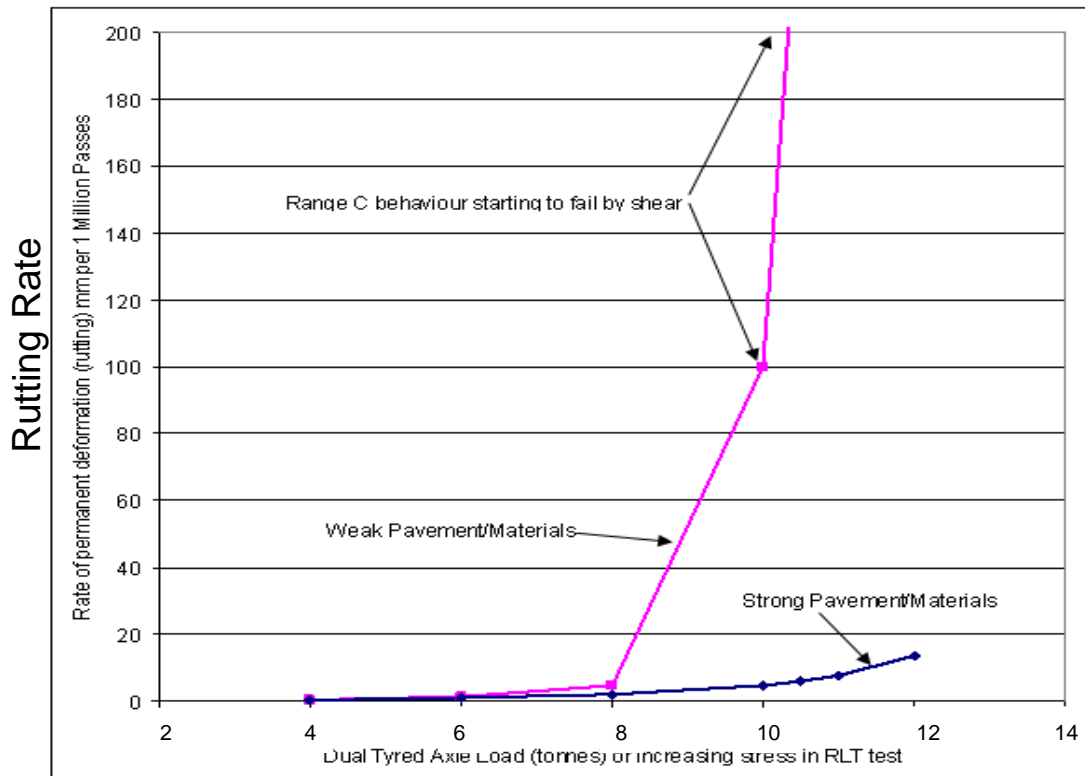
Figure 5.14 Damage exponent calculated from FEM for all axles





Reviewing the results of the calculated damage law exponent it can be seen there are good relationships between pavement life and damage law exponent for the 4 and 6 tonne axles with R-squared values greater than 0.7 (figure 5.13). However, in the 12 tonne axle damage law exponents, the relationship with pavement life was poor. This was caused by a few outliers with damage exponents greater than 6 and in one case a value of 9 was calculated (table 5.8). This can be explained by the 12 tonne load sometimes causing rapid shear failure of the aggregate or subgrade with very high rut rates. Figure 5.15 illustrates this effect and why high damage exponents can be calculated for the 12 tonne axle where there are weak pavement materials that can fail by shear. This same rapid failure may occur on weak local roads with a 12 tonne axle that simply fails the aggregate and/or subgrade in shear. Figure 5.15 shows there is a small difference between rut rates in the 4, 6 and 8 tonne load tests and thus the calculated damage exponent is low. However, for the pavement with weak materials there is a very large difference in rut rates for the 8 and 12 tonne axle loads which results in the calculation of a very high damage exponent.

**Figure 5.15** Rutting rate as a function of axle load showing effect of rapid shear failure



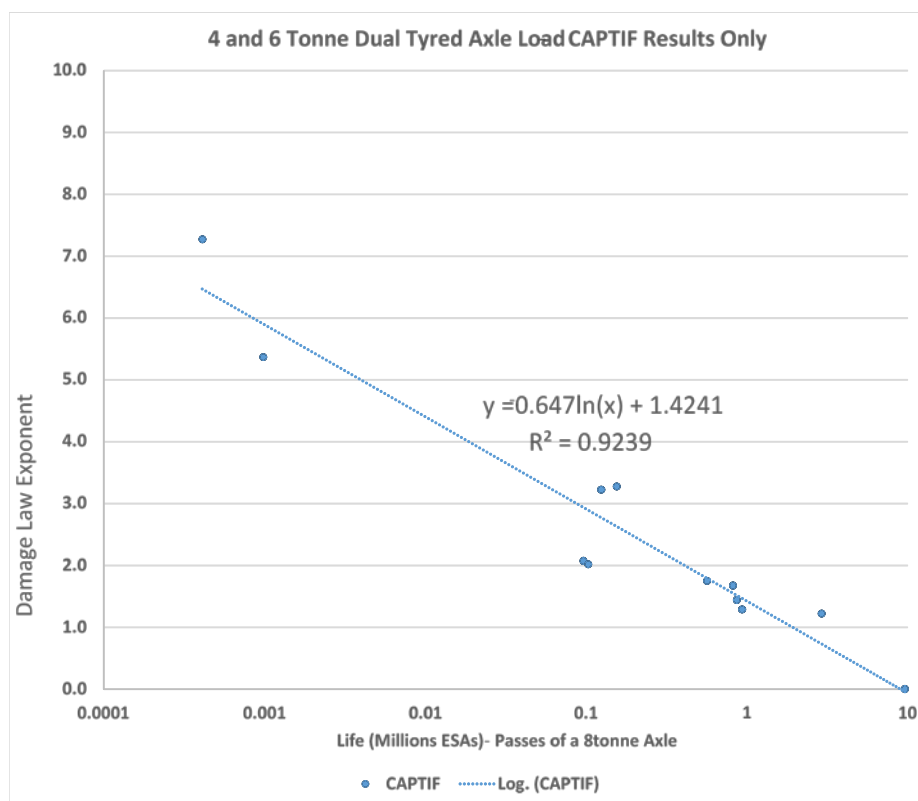
## 5.8 Combining all data

This research has involved many different CAPTIF tests and rut depth models with various pavement types, depths, materials and different wheel loads. All the results in terms of pavement life versus damage law exponent have been combined to look for trends in the data. The first plot (figure 5.16) shows the results from the CAPTIF tests for the 4 and 6 tonne axle loads. The CAPTIF results show a very good correlation with damage law exponent and life with an R-squared of 0.92. Figure 5.17 shows the results of combining all the data and the correlation of the relationship drops off significantly. Figure 5.18 presents the data with what were considered outliers removed. The outliers were:

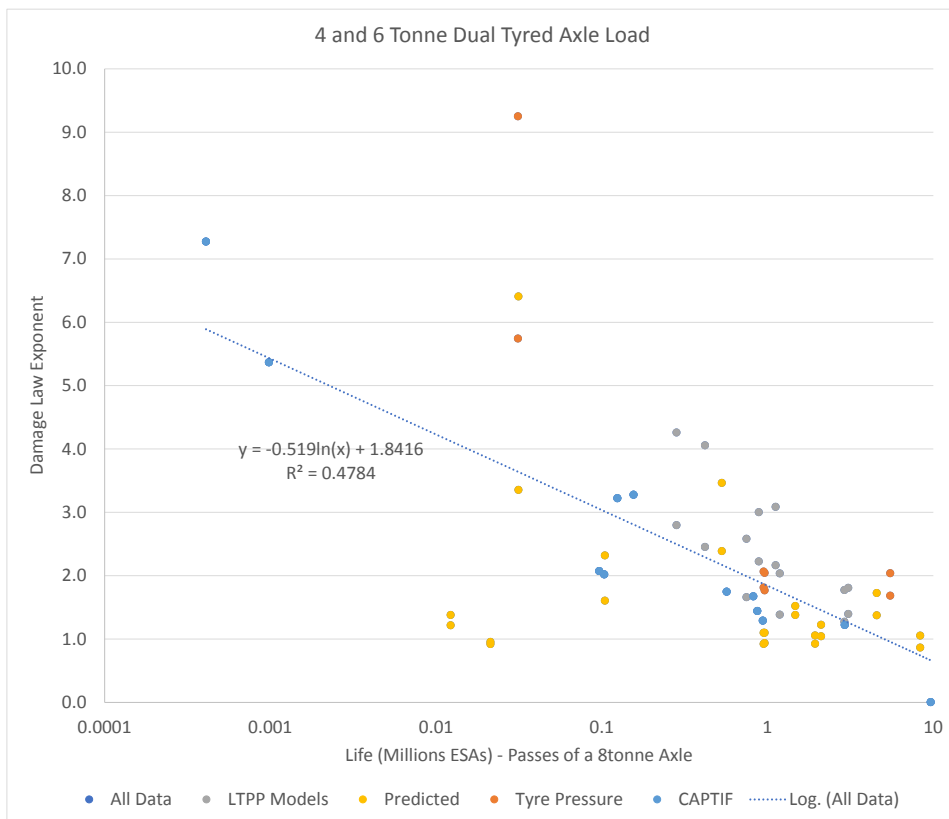
- The modelled exponents for sections 3C and 3D – these sections failed within 20,000 wheel passes and the unstable behaviour was difficult to model.
- The modelled damage exponent for reduced tyre pressure in section 1A. Reducing the tyre pressure significantly on the weak aggregate reduced the damage caused by the 4 tonne axle resulting in a larger difference in life predicted between the 8 tonne and 4 tonne axles and thus a high damage law exponent of 9. This predicted result is reasonable and expected for a reduced tyre pressure effect on weak aggregates but was removed from the dataset as trucks do not reduce their tyre pressures for lighter loads when travelling on public roads.

Combining all data with three outliers removed resulted in a reasonable correlation between life and damage law exponent with an R-squared of 0.73 (figure 5.18).

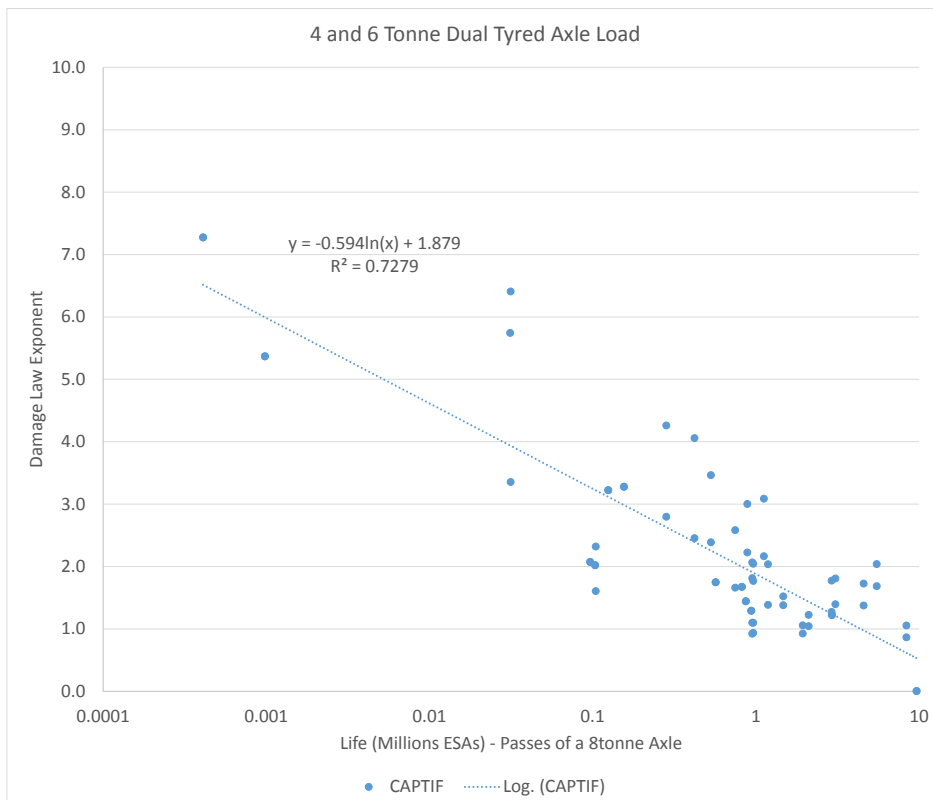
**Figure 5.16 CAPTIF results for 4 and 6 tonne dual tyred axle loads**



**Figure 5.17 Combining CAPTIF predictions calculated from rut depth models (all data included)**

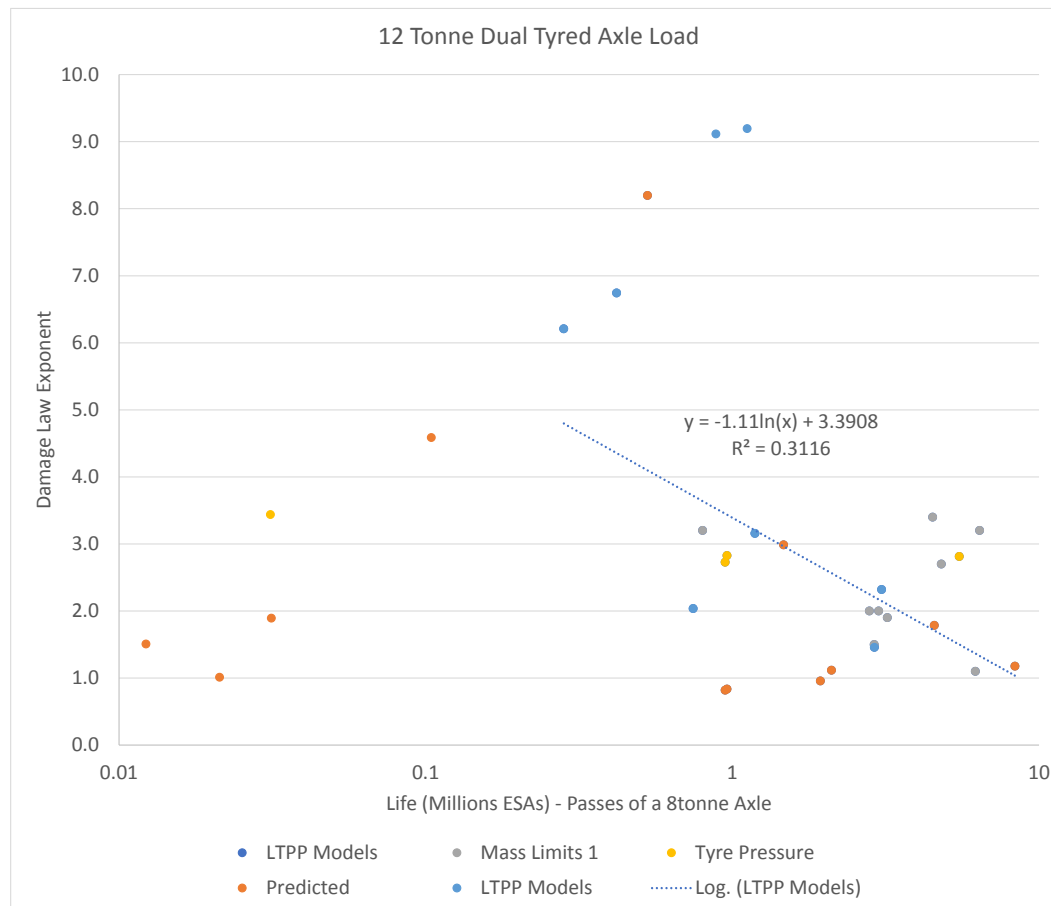


**Figure 5.18 Combining CAPTIF predictions calculated from rut depth models (outliers sections 3C and 3D data removed as failed early - 1A tyre pressure reduced data point removed also)**



The combined data set was also reviewed for the damage law exponents for the 12 tonne axle as shown in figure 5.19. Results from the 12 tonne axle loads show a poor correlation between damage law exponent and life, although an R-squared of 0.3 is obtained if 6 outlier points are removed from the regression fit. This poor result is due to the weak pavements failing in shear for either or both the 12 tonne and 8 tonne axle loads and the unstable behaviour results in an array of different damage law exponents. It also potentially explains the contradictions found in the literature.

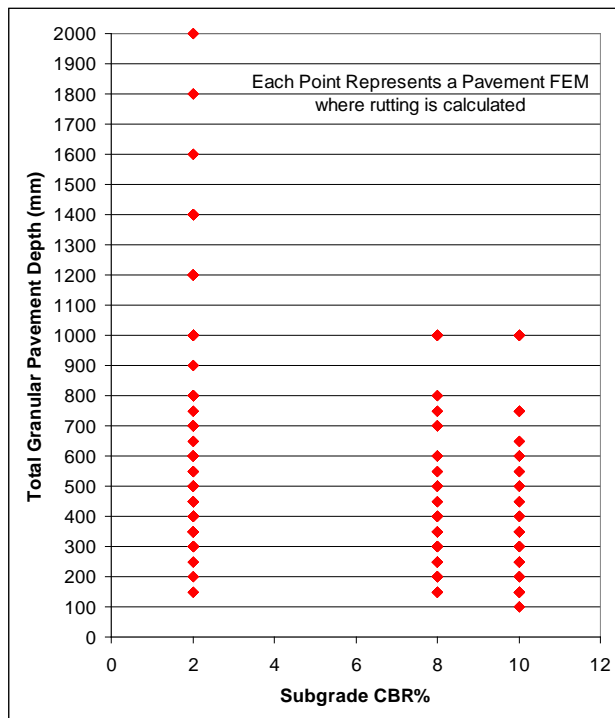
**Figure 5.19** Combining CAPTIF predictions calculated from rut depth models for 12 tonne axles



## 5.9 Recommended further rut depth modelling

In the Transport Agency research project, *Pavement thickness design charts derived from a rut depth finite element model* (Arnold and Werkmeister 2010b) a range of pavements were modelled to predict life for an 8 tonne axle load using RLT tests on different sub-base (good, average, poor), basecourse (good, average, poor) and subgrades (CBR 2, 8 and 10). The pavement depths modelled in this research are summarised in figure 5.20. To cover these variables around 500 different pavement combinations were modelled for an 8 tonne axle. It is recommended for future research that these pavement combinations be remodelled again but with different axle loads of 4, 6 and 12 tonnes to calculate the damage exponents.

**Figure 5.20** Pavement depths modelled to predict life (when rutting reaches 15mm) (Arnold and Werkmeister 2010)



## **6 Stratification framework for applying different loading exponents to a network**

### **6.1 Background**

This section documents the analysis of the LTPP database to determine factors that significantly affect pavement deterioration rates across New Zealand. The purpose of this analysis was to develop a stratification framework that could be used for classifying different road types operating under different conditions. This would allow load equivalency factors to be applied to different regions and pavement operating conditions and clearer relationships developed between traffic loading and deterioration. This stratification process considers the environmental effect in terms of drainage quality and rainfall along with urban and rural splits. By classifying the road along with information on climate, drainage, pavement depth, aggregate quality and subgrade strength it allows a more accurate estimate on pavement life in terms of the cumulative number of ESAs until the pavement fails by rutting and requires rehabilitation. The pavement life is used to determine the appropriate damage exponent and the unit cost per ESA to share in the cost to repair the road. Hence the environmental component of pavement wear is incorporated in its effect on the pavement life.

The load equivalency concept used in pavement design is based on the AASHO road tests that aimed at quantifying the load impact from different axle loads and wheel configurations. The concept is based on the premise of the comparable damage that different wheel loads cause on pavements. The road test also demonstrated that the same truck will cause a different magnitude of damage on different pavement types. Research into pavement performance modelling has also established that the deterioration rate of roads is also a function of the climate and geology of the region (Henning 2009).

### **6.2 The LTPP database**

The New Zealand LTPP programme originated in 2000 when 64 LTPP sites were established on the state highways. Two years later, the same establishment principles were used for the establishment of an additional 84 LTPP sites on local roads, with approximately half of the sections on rural roads and the remaining portion on urban roads.

#### **6.2.1 Sites established**

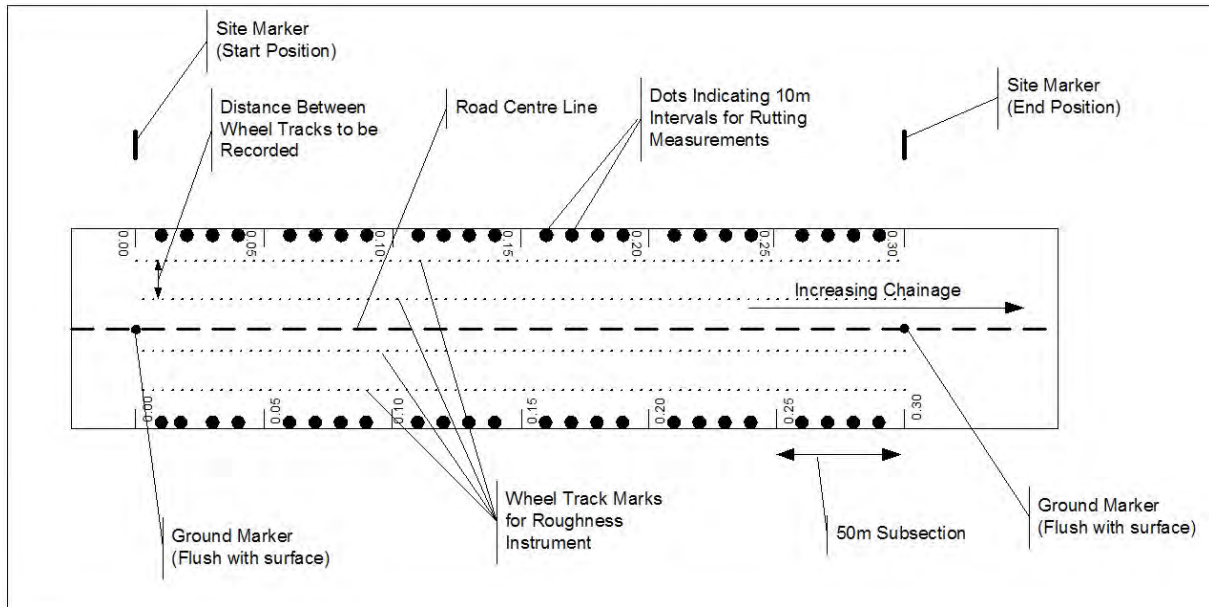
Henning (2009) describes the rationale of the site establishment in detail. In summary, the 140 sites were established across the country and covered all the expected factors that would influence the performance of the New Zealand road network including:

- climatic and soil condition
- traffic loading
- pavement types/strength
- pavement age/condition
- maintenance regime.

The LTPP sites are 300m in length with each section subdivided into 50m subsections for assessment purposes. The layout of the sites is depicted in figure 6.1. A climatic sensitivity rating was used to classify the combined impact of rainfall and soil moisture sensitivity. According to this classification, New Zealand was divided into four environmental sensitivity regions. For example, regions with a subtropical climate

and significant clayey in-situ subgrades would be classified as a sensitivity region. Typically the far northern regions of the North Island and the West Coast of the South Island were classified as high sensitivity areas. Drier regions on more stable geological formations, such as the Canterbury region in the South Islands of New Zealand would be classified as low sensitivity region.

**Figure 6.1 LTPP site layout (Henning 2009)**



## 6.2.2 Condition data

A private survey contractor has been undertaking an annual data collection on all the LTPP sites. The contract for this survey specifies the data collection accuracy and repeatability requirements. As only one contractor has been involved since the initiation of the programme, no changes were made to the methodology of data collection. An outstanding outcome from the programme so far has been the quality and subsequent usefulness of the data. The data collection includes:

- a manual assessment of all defects that involves the recording of the exact extent and dimensions of the defects
- manual measurement devices used for rutting, roughness and texture depth (refer to figure 6.2)
- traffic counts using classification loop counters
- recorded maintenance
- detailed site notes and photographs of any changes that occurred during the study period.

**Figure 6.2** Rutting measurements on the New Zealand- LTPP Programme



In addition to the above, each site is also surveyed annually using the high-speed data (HSD) collection survey as part of the Transport Agency state highway network survey processes. Four repeated runs are undertaken in both directions for each site using the HSD equipment. These parallel surveys have resulted in significant research opportunities in the data collection area.

## 6.3 Data analysis

Rutting was used as an indicator for deterioration given that it is also recognised as the primary design failing criteria for granular pavements. It is also one of the primary failure mechanisms in the field. Figure 7.3 in the next chapter shows the deterioration status of different sites on the local authority network. It is noted that rutting and roughness are the two defects most observed on deteriorating sites.

## 6.4 Results

The stratification method used for this research was clustering analysis using MATLAB. Clustering has its origin from the phycology (Tryon 1957) area and it is used to discover structures in data without needing to explain why these structures exist. It therefore sorts records into groups in a way to maximise the degree of association between the records. At the same time it aims at maximising the distance between groups. In essence it attempts to group data points together in such a way that the groups differ from each other in all respects. In the context of this research it was used to classify rut behaviour groups and then investigate the factors that are most commonly used to define these groups.

There are different techniques for undertaking cluster analysis. For this research an iterate method was used that removes one variable at a time from the clustering process in order to determine the relative contribution of each factor in contributing towards the most effective clustering. Table 6.1 illustrates the final outcome from the clustering process.

For the rut analysis, the data was clustered into four groups of different rut progression rates. For each potential value of a factor, the table depicts a probability that a variable and value combination would be used for clustering the rutting behaviour. The percentage depicted in the table gives the percentage contribution of each factor classifying the rut deterioration into one of the five cluster groups. The rural road category for example, has a 57% probability of classifying rut rate into one of the five categories. The top three factors contributing towards a rut behaviour classification were drainage condition, traffic loading and pavement strength, while the region's climate also featured strongly in the clustering results.



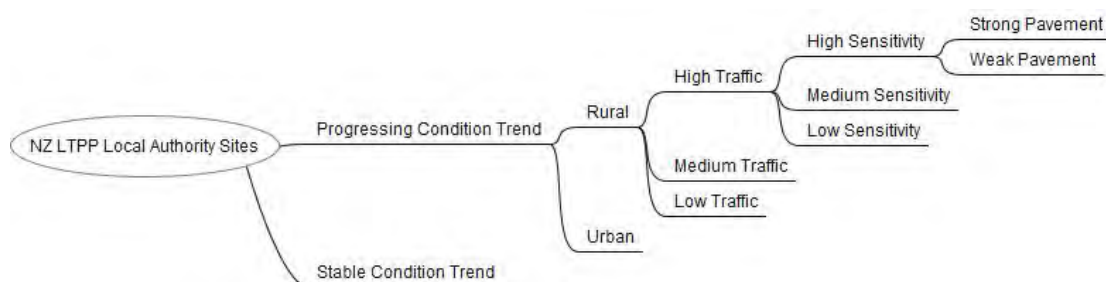
**Table 6.1 Condition trends of the LTPP sections on local roads**

Variables	Values	Probability
Drainage Condition Observed from Site Photo	Fair	71 %
Traffic Level _Low_ Daily ESA _100_ Medium _100_ Daily ESA _200_ High_ Daily ESA _200_	Low	62 %
Site Condition Trend Classification	Progressing	59 %
New Pavement Strength classification bases on total pavement depth_ SNP and D0	Strong	57 %
Road Category	Rural	57 %
Road Category	Urban	43 %
Site Condition Trend Classification	Stable	41 %
Sensitivity	Medium	41 %
New Pavement Strength classification bases on total pavement depth_ SNP and D0	Weak	35 %
Sensitivity	High	31 %
Sensitivity	Low	28 %
Avg Annual Rut Chg Rate _mm_	0.1 - 0.2	25 %
Avg Annual Rut Chg Rate _mm_	-0.1 - 0.1	25 %
Avg Annual Rut Chg Rate _mm_	0.2 - 0.7	25 %
Avg Annual Rut Chg Rate _mm_	-0.6 - -0.1	25 %
Traffic Level _Low_ Daily ESA _100_ Medium _100_ Daily ESA _200_ High_ Daily ESA _200_	High	24 %
Drainage Condition Observed from Site Photo	Poor	21 %
Traffic Level _Low_ Daily ESA _100_ Medium _100_ Daily ESA _200_ High_ Daily ESA _200_	Medium	14 %
New Pavement Strength classification bases on total pavement depth_ SNP and D0	NA	8 %
Drainage Condition Observed from Site Photo	Good	8 %

It was encouraging to note that expected results were obtained from the cluster analysis. However, it was particularly interesting to note that the drainage condition contributed significantly more to deterioration of pavements than the climatic region. This finding again confirmed views of engineers who emphasised the importance of drainage on the long-term behaviour of roads. It is important to take account of the significance drainage plays towards the performance of roads; however, it could not be used in the stratification method as drainage conditions were not a pre-determined variable.

Both traffic loading and different pavement strengths are taken into account during CAPTIF experiments. However, it is also evident that road conditions deteriorate differently in urban and rural areas. This is consistent with findings in other research (Brown et al 2010). It is thus recommended that the urban/rural split is included in the stratification framework.

Figure 6.3 illustrates the resulting stratification framework for low-volume roads. The figure was developed by applying some practical consideration to the ranking of variables from table 6.1. Variables with higher probabilities would therefore be used earlier in the stratification process. Based on this result, the recommended stratification would be urban/rural networks and then traffic volume and climatic environment. It was decided that the pavement strength classification would not make a significant difference and would also be less practical given that local authorities do not have comprehensive pavement data.

**Figure 6.3 Recommended stratification framework**

## 7 Understanding the environmental impact on road deterioration

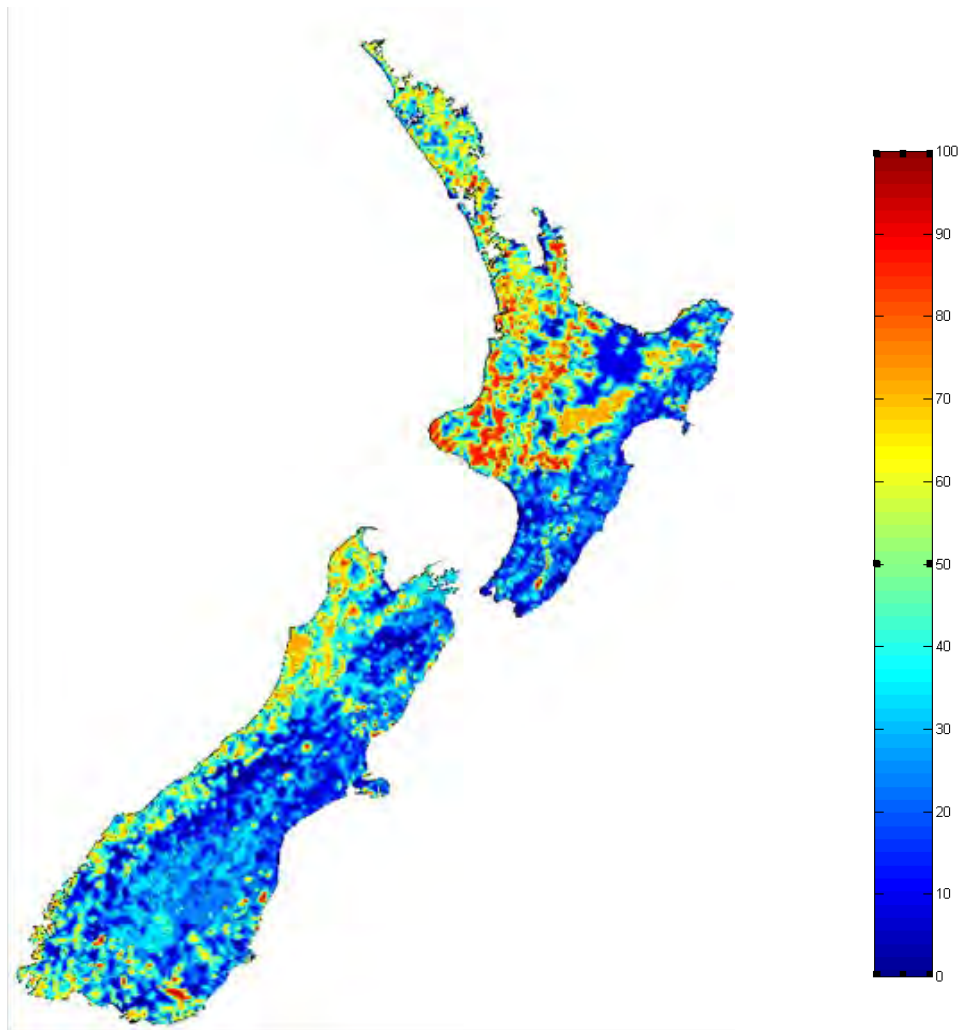
### 7.1 LTPP climatic areas

It is a well-accepted fact that roads do not deteriorate at equal rates in different climatic and geological regions. For that reason, long-term pavement performance models often include a climatic coefficient in order to take account of different climatic and geology environment (Henning et al 2004).

Earlier research work has also indicated there is a strong link between the geological makeup of an area and the climate in determining the expected behaviour of certain materials. Work completed by Weinert (1974) resulted in a moisture index that took account of a combination of climate and geology. Similar work completed by Cenek (2001) resulted in a process to classify the areas of New Zealand in terms of the sensitivity of the soils (geology) in the context of climatic conditions. This classification process was used by Henning (2009) to categorise New Zealand into different climatic zones as input into the design matrix of the New Zealand LTPP.

According to this technique, Cenek (2001) considered both the winter soil moisture or saturation levels (refer to figure 7.1) and the wet soil strength as an indicator of the susceptibility of the subgrade strength to moisture. The outcome of this relationship is graphically presented in figure 7.2 and mathematically presented as follows:

$$\frac{\text{Subgrade Strength}}{\text{Moisture Indicator}} = \left[ \log_{10} \left( \frac{\text{Wet Strength}^3}{\text{Moisture}^{0.5}} \right) \right]^{1.5} \dots\dots\dots \text{(Equation 7.1)}$$

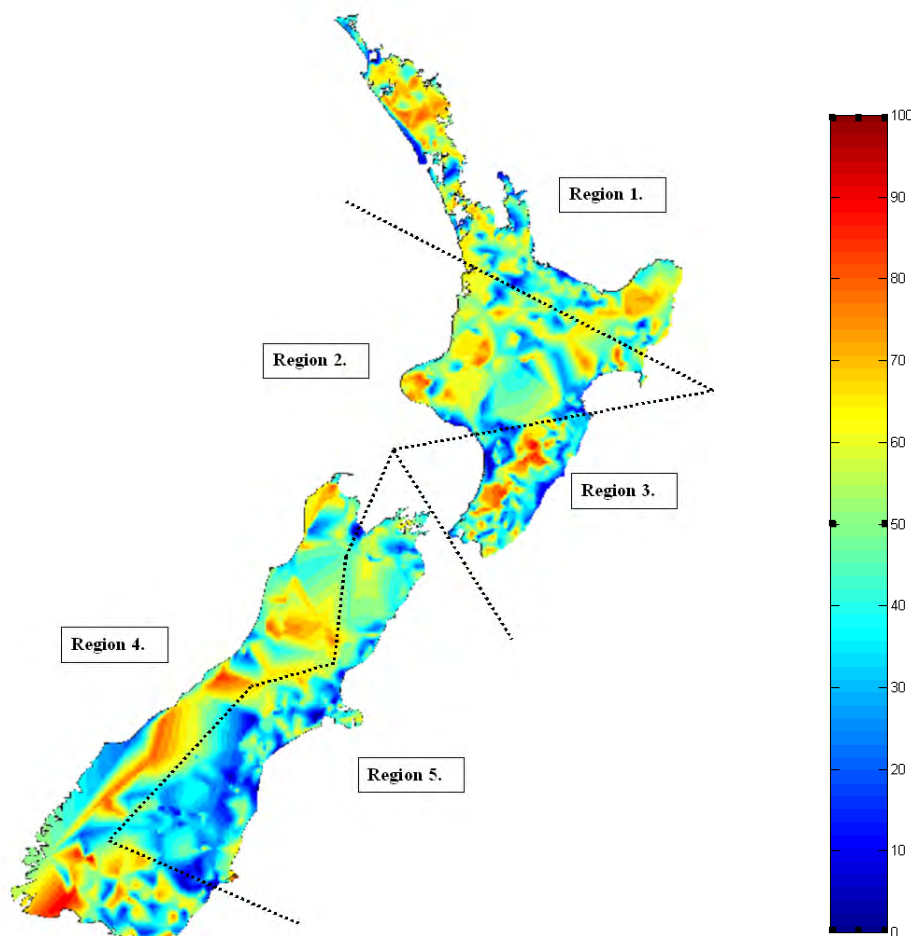
**Figure 7.1 Winter soil moisture (Cenek 2001)**

Instead of using the suggested regions in figure 7.2, Henning (2009) used the same index to classify a specific region according to the susceptibility of its subgrade strength to moisture, regardless of its geographical location. According to this method the entire country was classified according to three climatic zones. The respective regions according to this classification are depicted in table 7.1.

**Table 7.1 Climatic regions for New Zealand (based on Henning 2008)**

Sensitivity area	Calibration sections within state highway regions
High	Northland, West Waikato, Gisborne, West Coast
Moderate	Coastal Otago, Auckland, Wanganui, Taranaki, Wellington
Low	Nelson, Marlborough, Napier, East Waikato, Canterbury

**Figure 7.2 Wet strength/moisture indicator (Cenek 2001)-**



Note: Blue shading represents high wet subgrade strength/low winter moisture whereas red shading represents low wet subgrade strength/high winter moisture. Green/yellow shading represents areas where compensatory factors are in play, ie low wet subgrade strength/low winter moisture or high wet subgrade strength/high winter moisture.

The relative impact of geology and climate on the performance of a road is difficult to simulate in any accelerated experiment or laboratory test. The reality is that the impact from the climate on the aggregates could sometime take years before it starts having an impact on road performance. For the purpose of this study, the LTPP data was analysed to get an understanding of the relative performance of roads within different regions of New Zealand. This ultimately would also contribute towards a better understanding of road user induced damage of roads within the different regions.

## 7.2 Results

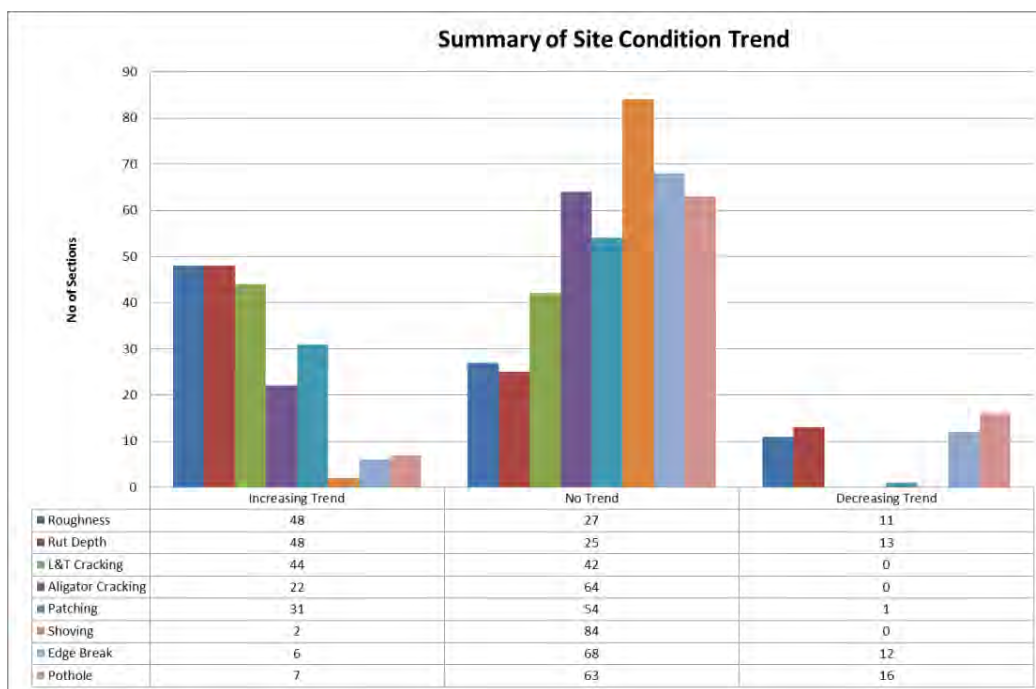
Figure 7.3 depicts changes in the defects of local council sites since 2002. It shows the number of sites that have displayed an increasing, decreasing or no trend during the past 10 years. Note that in most cases, a decreasing trend represents sections that have been rehabilitated. The changes were reported for:

- roughness (in IRI)
- wheel track rutting (in mm)
- longitudinal and transversal cracking

- alligator cracking
- patching – previously potholes or shoves
- shoving or shear failure of the roads
- edge break
- potholes.

Figure 7.3 reveals that approximately half of the sections displayed deteriorating trends. A more detailed investigation of individual sections suggested that deteriorating did not necessarily show increasing trends for all defect types. For example, if a section shows increasing rutting, it does not necessarily mean the roughness also increases. This observation further confirmed the typical behaviour of low-volume/low-strength roads that carry relatively light traffic. Another observation was that primary and secondary deterioration mechanisms differed on most sites. For example, for some sites, cracking was observed as a primary defect followed by say rutting after some years. On other sites rutting was observed a few years before cracking was first observed. It was thus clear that the failure mechanisms differ substantially between sections confirming the findings of Schlotjes et al (2011).

**Figure 7.3 Condition trends of the LTPP sections on local roads**



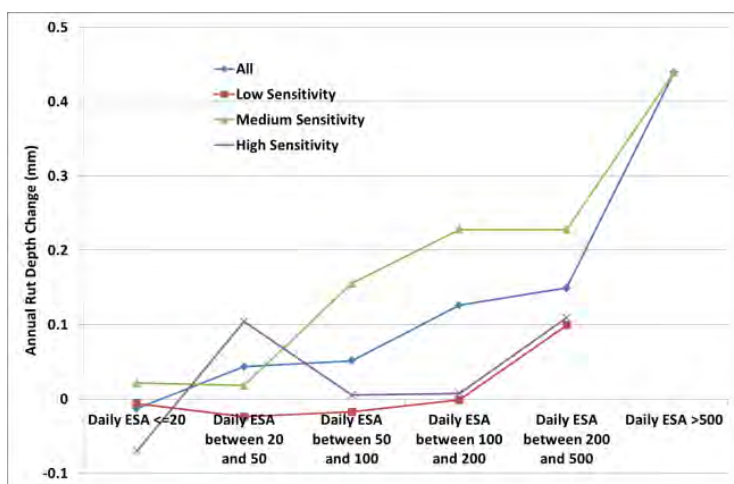
Despite the average age inferred for the pavement structures of the LTPP programme (over 15 years old), only half of the sites displayed any deterioration. For these sites, little deterioration occurred over time for the predominant period of the pavement's life. These pavements would exceed the design life of 20 to 25 years. However, at a certain point, the site starts deteriorating rapidly and would require maintenance in a short time frame. Accelerated deterioration was observed in both the LTPP programme and network data.

Figure 7.4 gives the average rut rate for different traffic volumes and climatic zones, while figure 7.5 provides the rut rate distribution between urban and rural roads. Observations from these graphs are:

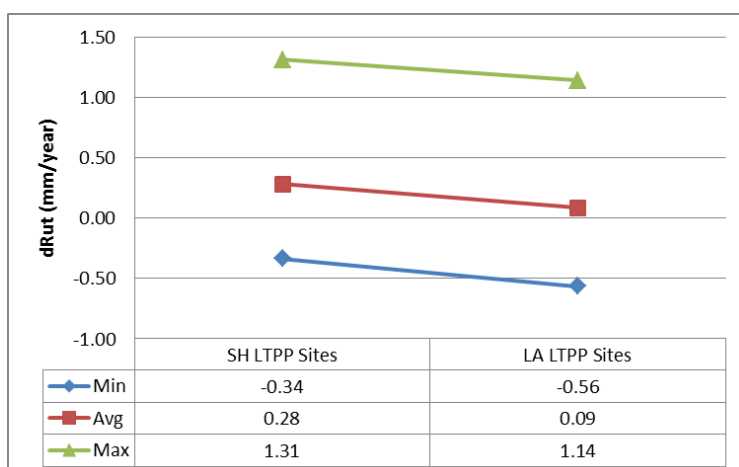
- There is a strong relationship between traffic load and rut rate.

- The climatic regions give inconsistent outcome for the rut rate, although the rut rate is the lower in the low sensitivity climatic area.
- The rut rate is higher on state highway LTPP sites than on LTPP local authority sites.

**Figure 7.4 Annual rut depth changes for traffic ranges of LTPP sites**



**Figure 7.5 Typical rut rates on New Zealand roads**



The difficulty in isolating soil moisture sensitivity effects from traffic-induced deterioration of roads results from the fact that these two impacts do not act in isolation. There is a complex interaction between the pavement, current condition, traffic volume, physical properties and the soil moisture sensitivity on the deterioration of roads. In order to isolate the impact of the soil moisture sensitivity and drainage, the relationship between the rut rate (mm/year) and the respective factors were investigated. It is acknowledged that the relationship between rut rate, traffic, climate, geology and drainage is most probably not a linear relationship. However, the aim of this part of the research only considered the relative impact, which is better understood by considering the linear trends. For comparative purposes, the current rut deterioration models for New Zealand are given by Henning (2009):

**Initial rut rate**

$$Initial\_Rut = 3.5 + e^{(2.44 - 0.55SNP)} \quad (\text{Equation 7.2})$$

**Stable phase rut rate**

$$RPR = 14.2 - 3.86 \times a_2 SNP \quad (\text{Equation 7.3})$$

**Probability of accelerated rut rate**

$$p(Rut_{accel}) = \frac{1}{1 + e^{\left\{-7.568 \times 10^{-6} * ESA + 2.434 * snp - [(4.426, 0.4744) \text{ for thickness} = (0,1)]\right\}}} \quad (\text{Equation 7.4})$$

Where:

SNP – modified structural number

ESA – equivalent standard axles

Thickness boundary – 150mm.

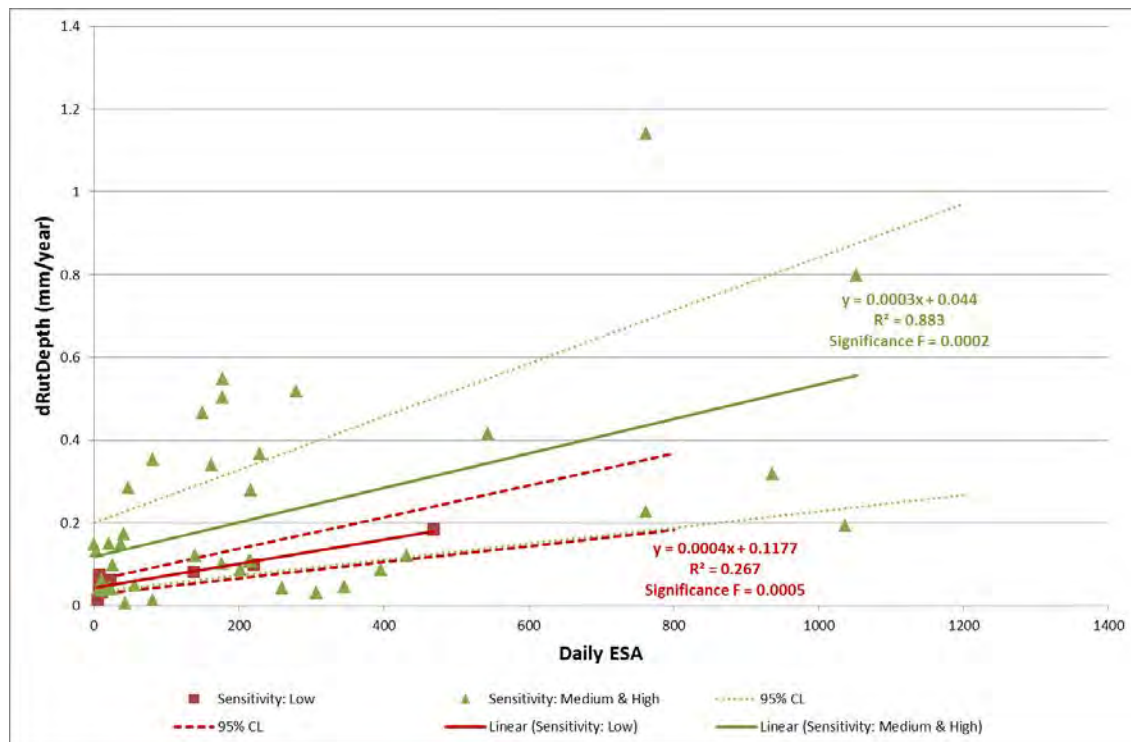
### 7.2.1 Findings for climatic regions

The climatic regional sensitivity classification has proven to be an effective measure to describe the combined impact of rainfall and soil moisture sensitivity. Many other deterioration studies have confirmed the use of these sensitivity regions to be a valid technique to stratify the country into appropriate climatic regions (Henning 2009; Brown et al 2010).

Figure 7.6 illustrates the annual change in rutting (mm/year) as a function of traffic loading and site moisture sensitivity. There is a significant variation to the trend, which is expected on the basis of findings published in Henning (2009). Most significant of these would be the stage of rutting (initial, stable or accelerated) for a particular road that would be vastly different, yet in this section, it is ignored. Also, the other variables that influence rutting, for example those contained in equation 7.4 are not considered either.

Despite the variation of the incremental rut change, it was also observed that the rate of change of rutting, as a function of traffic loading, was slightly higher for the medium and high soil moisture sensitivity areas. There was an absolute difference of approximately 0.1mm/year for the traffic ranges tested. The finding suggests that roads in more soil moisture sensitivity zones will have an increased rut rate (by approximately 0.1mm/year) compared with the more stable soil moisture sensitivity areas, for example at an ESA of 400 axles per day the rut rates are 0.16mm/year and 0.28mm/year (0.12mm difference) for the low sensitivity and medium and high sensitivity areas respectively. The expected rut rate for roads in New Zealand varies between 0.3 and 0.6mm/year. It was noted the linear relationship between the rut rate, traffic loading and climate had a moderate to strong correlation that was significant in both cases. It is also noted that a significant number of data points from the medium and high climatic area fell outside the 95% confidence level. This aspect was further investigated by considering the drainage situation for these sites and the outcome is discussed in subsequent paragraphs.

**Figure 7.6 Rut progression as a function of traffic loading and environment**



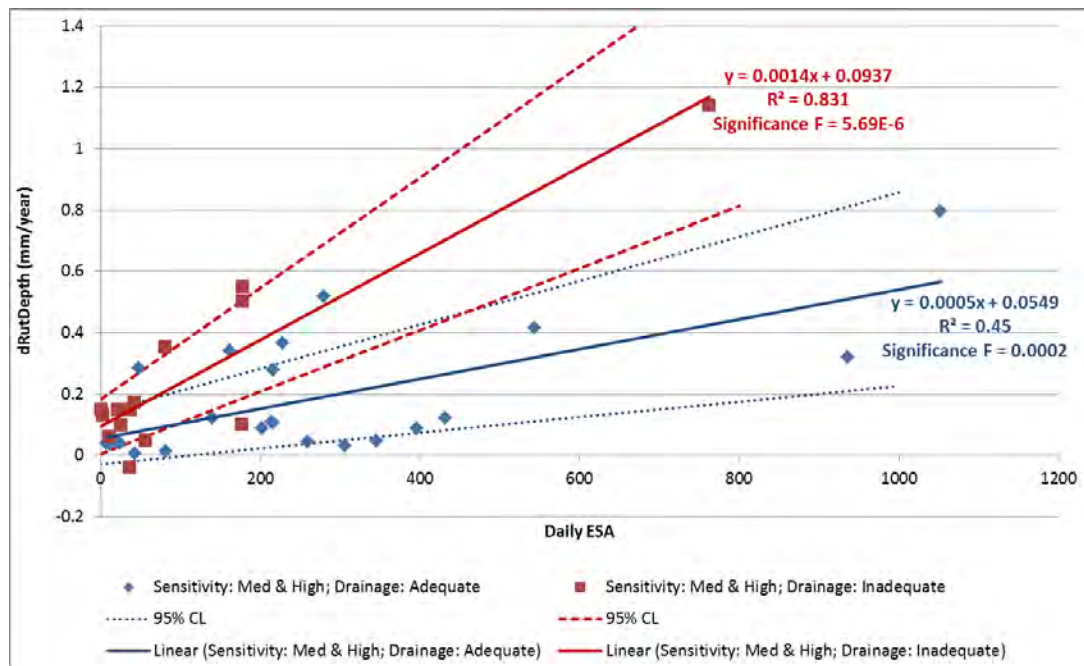
## 7.2.2 Combined impact of the environment, drainage and traffic loading

As part of the annual LTPP surveys, all assessments of sites have included a detailed rating of the drainage condition and need for drainage where it did not previously exist. The previous section has already highlighted that the drainage condition, or absence of drainage where it is needed, had a significant impact on the deterioration of a site. The remaining question was how much the drainage affected the deterioration of low-volume roads for different soil moisture sensitivity conditions.

The results revealed that for the low sensitivity sites, the drainage condition impact was much less than it was for the medium and high sensitivity sites. Figure 7.7 shows the rut rates for sites within the medium to high soil moisture sensitivity, therefore only taking the medium and high sensitivity data points from figure 7.6. Two trends were observed: one for sites having adequate drainage and the other for sites having inadequate drainage. There was a slight absolute difference between the deterioration rates at the intercept of the two drainage states. However, sites with inadequate drainage had approximately 2.5 times the deterioration rate of those sites with adequate drainage. For example, at 400 ESA per day the rut rates were 0.65 and 0.25 for inadequate and adequate drainage respectively. Again, this finding correlated well with engineering experience.

The correlation and significance of the trends were medium to high for the two expressions thus suggesting the relevance of including drainage adequacy in considering rut rate as a function of climate. When comparing the 95% confidence levels between figures 7.6 and 7.7 it can be seen that far fewer data points fell beyond the confidence levels in the latter figure thus supporting the validity of drainage adequacy as a variable for rut rate progression.



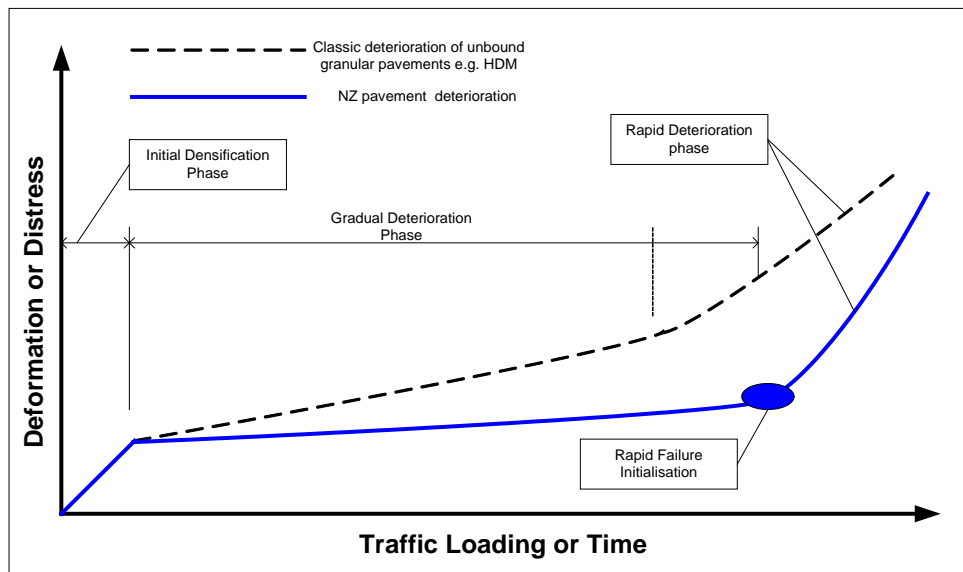
**Figure 7.7 Rut progression as a function of traffic loading, drainage condition and environment**

## 8 Benchmark CAPTIF to LTPP

### 8.1 Predicting rutting for asset management purposes

The New Zealand rut model used for asset management purposes consists of three distinct development stages as indicated in figure 8.1. The respective models are depicted in equations 7.2, 7.3 and 7.4.

**Figure 8.1 Deterioration phases for sealed granular pavements (Henning 2009)**



Given the relatively young status of the LTPP programme during the 2007 study, it was concluded that rutting models could not be based on LTPP data alone. For that reason, the LTPP data was supplemented by the CAPTIF data that resulted from the PR3-0810 fatigue CAPTIF experiment (Alabaster and Fussell 2006). Apart from developing the rutting model, a significant aim of this research was to benchmark the CAPTIF results to the LTPP and network performance. The rutting model that resulted from the research was tested to network and LTPP data as illustrated in figures 8.2 to 8.4.

Figure 8.2 compares the model outcome using both the HDM-4 and New Zealand rutting models which were developed on the basis of the CAPTIF results. The model outcomes are compared to actual initial rutting observed from the LTPP sections. It can be seen in the figure that the actual initial densification varies significantly, typically between 4mm and 10mm. Despite this variation the initiation model developed using the CAPTIF data significantly represented the actual performance in the field.

**Figure 8.2** Plot of the initial rut model developed on the CAPTIF data (plotted against LTPP observed data) (Henning 2009)

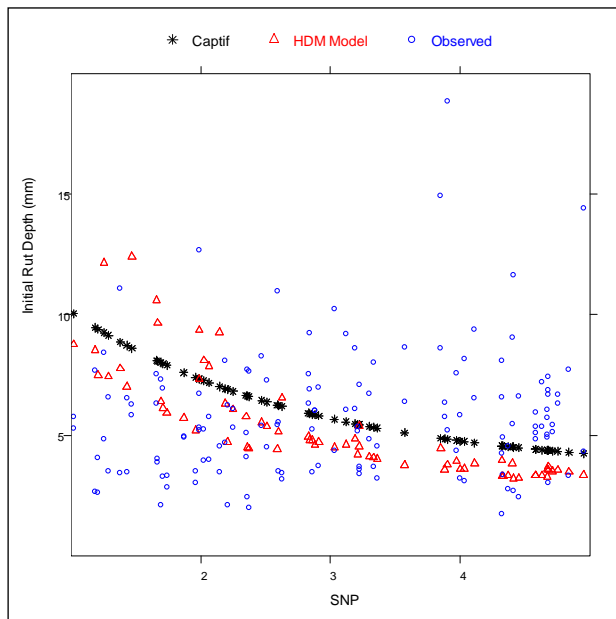
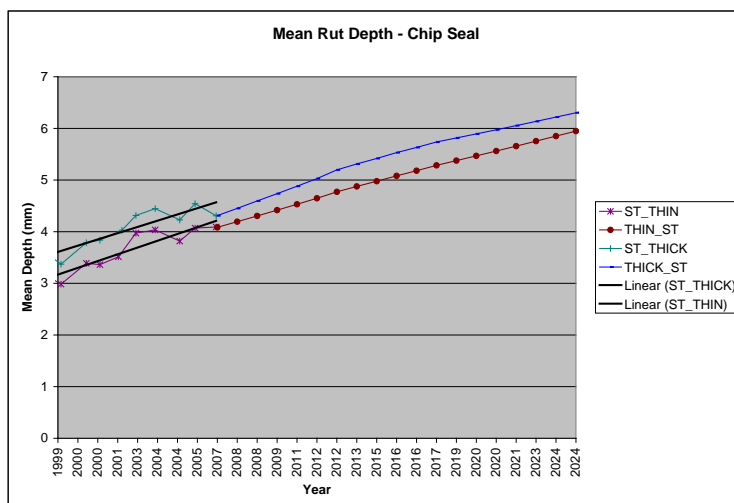


Figure 8.3 illustrates the historical rut rates on an entire network compared with the forecasted model that resulted from the CAPTIF data. There was a satisfactory comparison between the model and the network rut trends for both low (thin) and higher volume roads (thick).

**Figure 8.3** Testing the stable rut progression on a complete network dataset (Hatcher 2007)

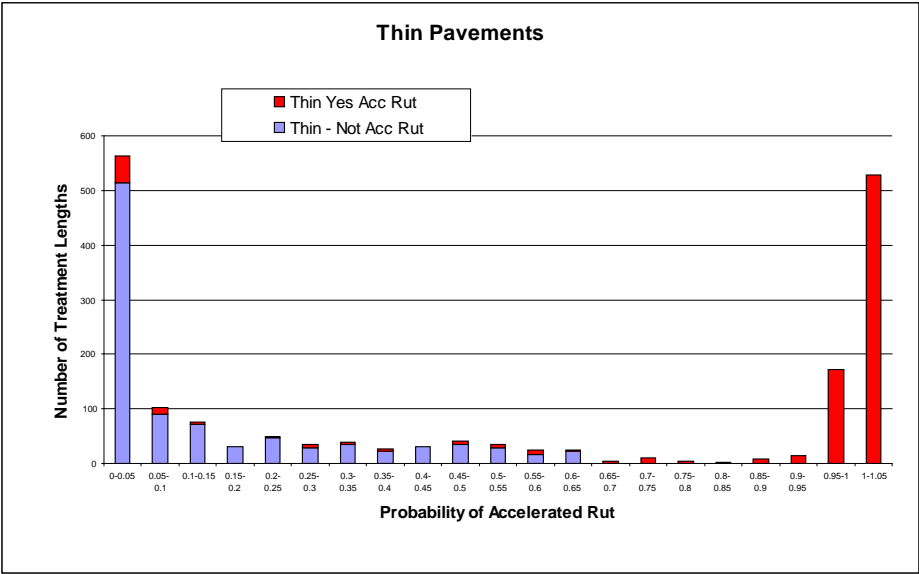


Note: ST\_THIN is thin chip seal pavements (historical rutting data)  
 THIN\_ST is thin chip seal pavements (predicted rutting data)  
 ST\_THICK is thick chip seal pavements (historical rutting data)  
 THICK\_ST is thick chip seal pavements (predicted rutting data)

Figure 8.4 shows a comparison of the predicted versus the actual initiation of accelerated rutting on a network. Note that the model (equation 7.4) forecasts the probability of a pavement going into accelerated rutting. Accelerated rut rate was defined as 1.2mm per years for a low-volume road network. From a predictive perspective accelerated rutting initiation is assumed to take place at a probability of 50%. The

figure therefore plots the forecasted ‘risk to rut’ distribution of the network. The red bars illustrate the sections that have an actual accelerated rutting rate, while the blue bars are those sections still within the limits of stable rutting. It was concluded from this comparison that the model had strong predictive power for forecasting the initiation of accelerated rutting. It was also noted that some sections with a lower probability of accelerated rutting still experienced accelerated rutting rates in the field. Closer examination indicated these sections had drainage issues.

**Figure 8.4** Comparing predicted and actual accelerated rut rate on network level (accelerated rut rate > 1.5 mm/year) (Henning 2009)



## 8.2 Benchmarking pavements

Although the previous section gave some promising results, the limitation regarding this study was that it classified the pavements according to the World Bank HDM-4 definition of modified structural number (SNP). Although this concept is well accepted in the asset management area, research has indicated that the SNP is not a comprehensive measure to classify pavements’ strength (Henning at al 2010). SNP is only an effective strength parameter if failure occurs in the subgrade. For the purpose of this research it was decided to classify pavement according to both the peak deflection and the deflection bowl curvature to obtain an estimate of subgrade protection and elastic strain occurring in the basecourse. This approach agrees with other benchmarking studies using the FWD (Horak 2008).

## 8.3 Comparison of performance between LTPP and CAPTIF

A rather coarse comparison between the rut rates of the CAPTIF trial and the LTPP sections are depicted in table 8.1. Given the data constraints it was only possible to perform a gross comparison between the two data sets (refer to the notes for the table).

**Table 8.1 Comparing rut rate of the LTPP sections with the CAPTIF sections**

	CAPTIF			LTPP		
	Do	Curv	Rut rate (mm/million ESA)	Do	Curv	Rut rate (mm/million ESA)
min	0.55	0.18	1.30	0.41	0.09	0.3
max	1.79	0.73	128.22	3.09	0.83	127.8
median	0.66	0.26	10.88	1.01	0.28	4.4

Notes:

The two outlier sections (very weak) from the CAPTIF dataset were excluded from the summary results presented in the table.

The ESA calculation for both datasets was performed using the recommended damage exponents for this research.

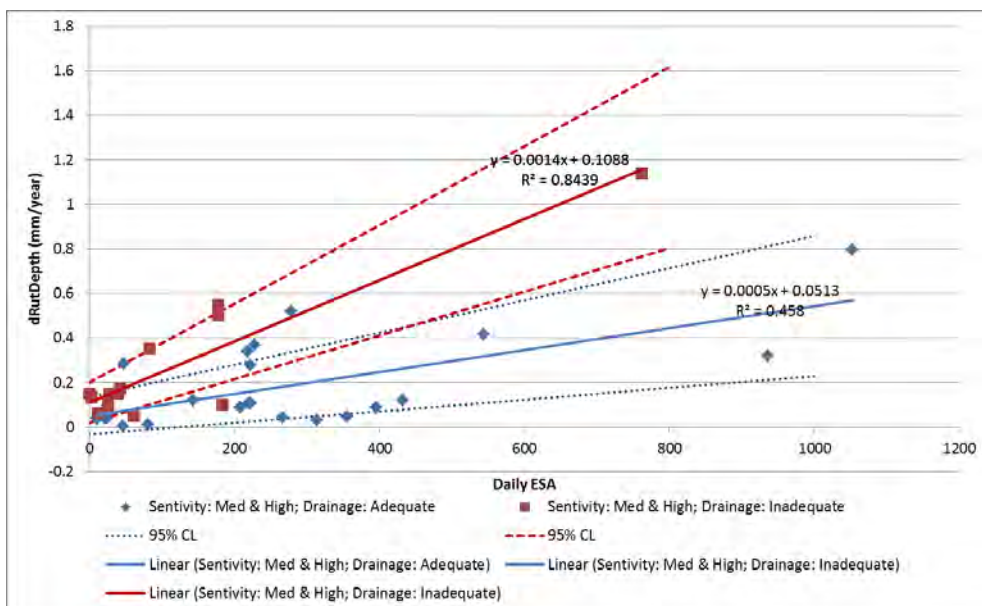
CAPTIF sections only consist of unbound granular pavement layers, while the LTPP consists of some unbound and some cemented layers (unfortunately the local authority sites do not have test pits to confirm its composition).

The CAPTIF sections were loaded with a limited transverse distribution, while the LTPP sections have no restrictions on vehicle wander.

Observations from the table are:

- As expected the pavement strength from the LTPP sites covered a much wider range compared with that of the CAPTIF dataset.
- The median rut rate on the CAPTIF experiment was higher than that of the LTPP sections, which could be explained by the difference in loading conditions between these two datasets.
- Given the significant variation in rut rates that would be observed on road networks, the comparison of the rut rate between the two dataset could be considered to correlate relatively well.

The damage exponents suggested by this research were further investigated by adjusting the cumulative loading values, which are reproduced in figure 7.7. It was hoped that adjusted traffic loading would ultimately result in a better correlation between loading and expected rut rate. The reproduced relationships are shown in figure 8.5.

**Figure 8.5 Rut progression as a function of traffic loading, drainage condition and environment – traffic loading adjusted to new damage exponents**

The comparison between figure 7.7 and figure 8.5 resulted in:

- no noticeable difference in the regression result, although the intercept in both expressions shifted slightly
- a minor improvement (0.01) in the  $R^2$  for both expressions.

For all practical purposes it is assumed there was only a minor improvement on the regression results by adopting the new damage exponent for the traffic loading on the LTPP sites. This result should be seen in the context of the LTPP data, which is not as robust as the condition data.

The comparisons in rut rate between the LTPP and CAPTIF datasets discussed in this section had a surprisingly good correlation. It is therefore safe to assume that the relative damage between the respective pavement strengths and varying wheel loads is therefore valid and could be used as a proxy for the relative damage ratio of in-service road pavements.

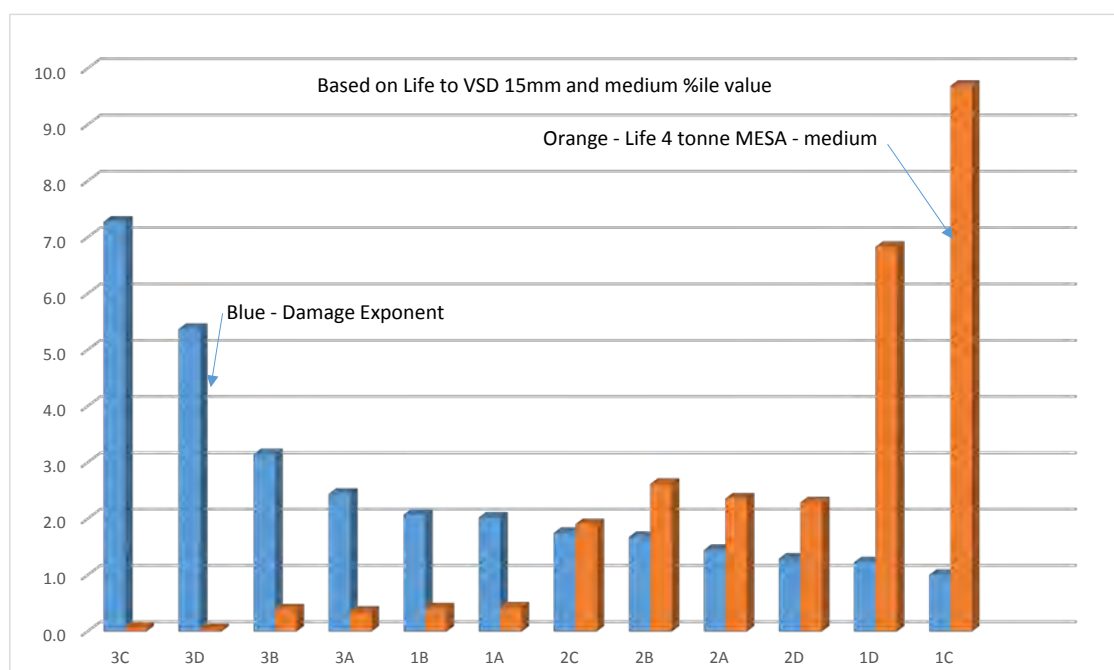
## 9 Discussion

This project undertook tests at CAPTIF to determine the damage law exponent for different axle loads, primarily 4 and 6 tonne equivalent axle loads on low strength thin surfaced unbound granular pavements. The literature showed that a range of damage law exponents have been measured from 1 to 7 with no explanation as to why. Dawson (2008) argued there was not a singular damage factor because of the differences in the materials behaviour being either being stable or in shear failure mode. The previous mass limits project (Arnold et al 2005) found a relationship between pavement strength and damage law exponent for axle loads greater than 8 tonnes with the weaker pavements resulting in the higher damage law exponent.

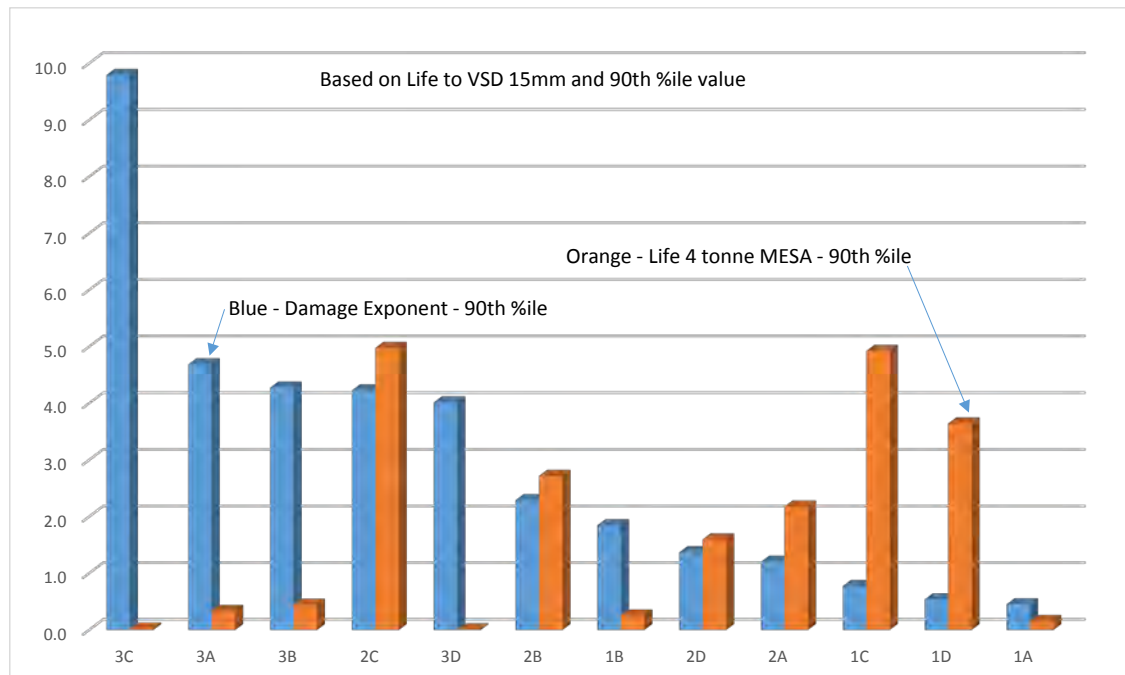
This CAPTIF study involved three separate CAPTIF tests where for each test the subgrade type and strength were the same while two different aggregate depths and two different aggregate types were used for each test. This resulted in four different test sections in three tests and thus totalling 12 pavement test sections with 12 separate results. The end of life for all the sections was when a VSD of 15mm was achieved. The VSD is a measure of rutting from a fixed datum level and ignores shoving. For those pavement test sections that did not reach the end of life, linear extrapolation was used to determine the number of cycles to reach this. Using VSD with an end of life at 15mm and linear extrapolation was the best method as proven in the earlier mass limits testing (Arnold et al 2005). Although not reported here, rutting data was looked at but discarded as no relationships or trends could be found. The data was often due to shoving on the edges of the rut, which in some cases showed the rutting reduced with increasing load cycles.

Each pavement section was 12m in length resulting in 12 measurements of rutting/VSD and thus 12 calculated lives. Therefore, a range of statistical values of lives was determined as 90th, 75th and 50th (median) percentile values. It was found that the median values of lives gave the best trends and best relationships with pavement life. Other percentile values resulted in a scatter of results and less obvious trend with pavement life as shown in figures 9.1 and 9.2, which compare pavement life and damage law exponent. The best relationship was found to be with the median values and thus the median value was used for further analysis.

**Figure 9.1 Damage exponent and life for medium values**



**Figure 9.2 Damage exponent and life for 90th %ile values**



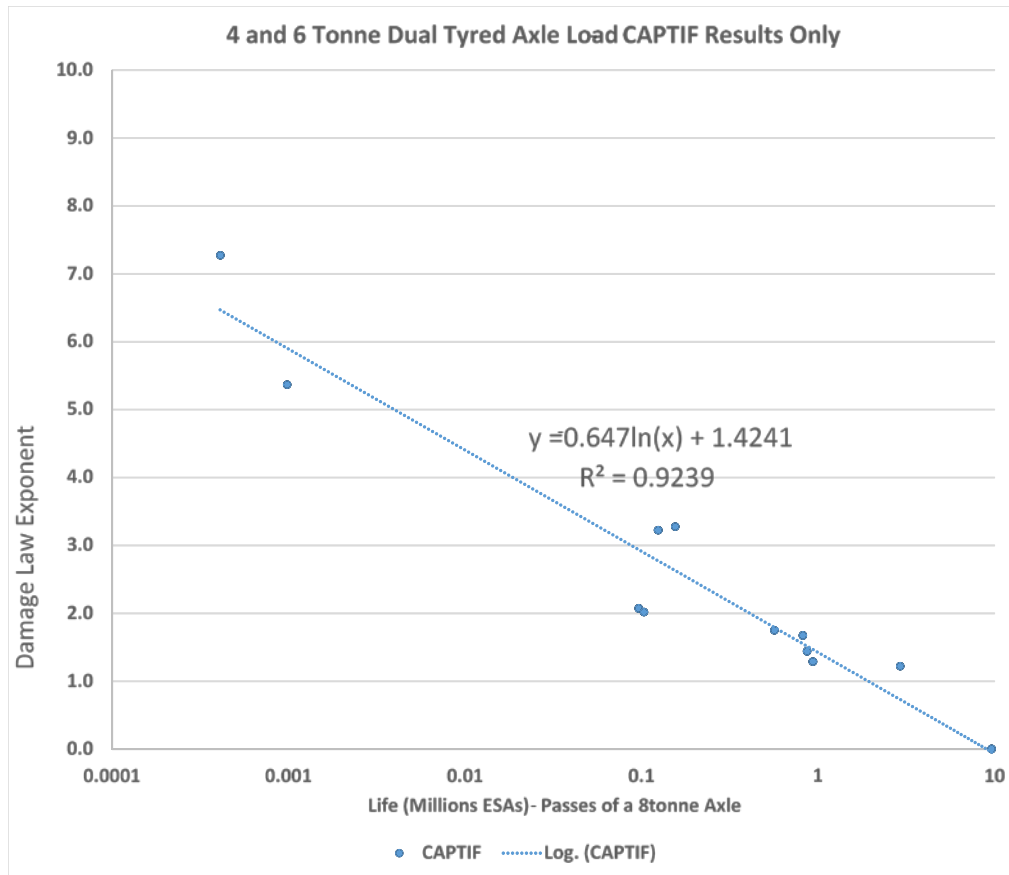
Four axle loads were studied: 4, 6, 8 and 12 tonnes, where the first pair tested was 4 versus 6 tonne axle load in the outer and inner wheel path respectively. After 1 million load cycles the plan was to follow with the 8 and 12 tonne loading on the outer and inner wheel paths. This was successful on the second CAPTIF test which used strong pavements but not on the first and third CAPTIF tests where sections had failed with the 4 and 6 tonne axle loads. However, the analysis of the damage law exponents of the 8 and 12 tonne axle loads showed no trends as a result of the testing being on pavements of the same strength (ie not enough spread of results). Also, the pre-loading of the pavement with 4 and 6 tonne loads affected the results as the rutting for the 8 tonne axle load was less than for the 6 tonne axle load. Consequently, the analysis focused on the 4 and 6 tonne axle loads and these loads were paired to determine the damage law exponent between them (ie 4 vs 6 tonnes). The 6 tonne axle was considered to be the standard axle and it was assumed the damage law exponent calculated for the 4 versus 6 tonne load was the same as what would have been calculated for a 4 versus 8 tonne axle load. For most pavements this assumption is reasonable except for pavements that show steady behaviour for loads of 6 tonnes and less but fail rapidly in shear under an 8 tonne axle load (see chapter 5, figure 5.15).

Results from the CAPTIF tests with 4 versus 6 tonne axle loads showed a clear relationship between pavement life and damage law exponent with the weak pavements resulting in a higher damage law exponent (figure 9.3). Adding the predictions from rut depth modelling of the CAPTIF pavements and a range of different pavement cross-sections that covered the LTPP sites found a good relationship between life and damage law exponent also, as shown in figure 9.4 with a R-squared of 0.72 with outliers removed. The calculated exponents for CAPTIF test 3 found that all modelled loads failed the pavement quickly with the difference in life being the same proportion as the load difference which resulted in a damage law exponent of close to 1. Results of this modelling were considered inaccurate as the weak materials exhibited unstable shear failure behaviour which is difficult to predict and thus these test results were considered acceptable outliers. The other acceptable outlier was section 1A with a 4 tonne load modelled at half the tyre pressure as the 8 tonne load which resulted in an extended predicted life of the 4 tonne axle which increased the damage law exponent to a value of 9. Even though this is a likely

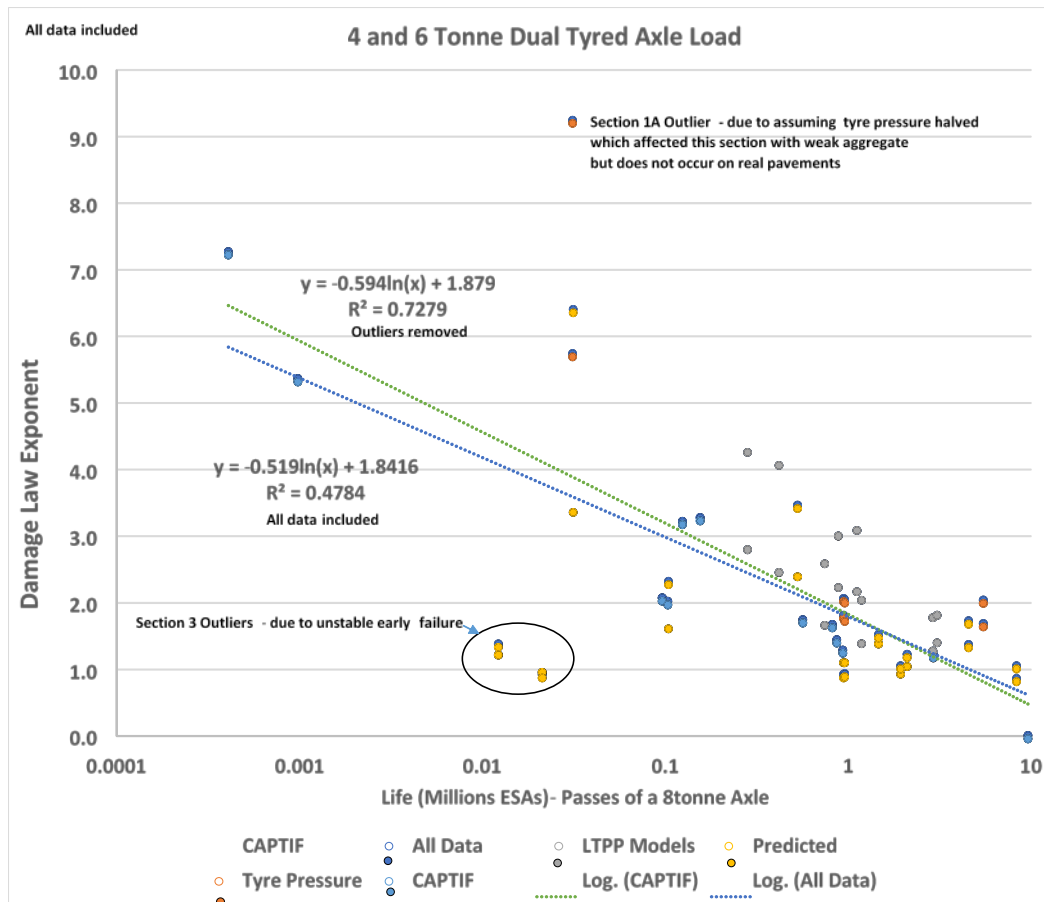


prediction for section 1A with weak aggregates, trucks do not operate on the New Zealand road network at reduced tyre pressures and thus it was considered acceptable to remove this point.

**Figure 9.3** CAPTIF results for 4 and 6 tonne dual tyred axle loads



**Figure 9.4** CAPTIF and rut depth prediction results for 4 and 6 tonne dual tyred axle loads



The reason why weak or lower life pavements have higher damage law exponents compared with strong or long life pavements is because all pavement materials have a defined shear strength, which if exceeded means the pavement will fail very quickly. Applying a load that just exceeds the shear strength of the material will fail the material within one load cycle compared with many thousands of cycles that can be applied with loads that are less than the shear strength. For strong pavement materials all the typical axle loads in this study up to 12 tonnes did not exceed the shear strength of the pavement materials and thus behaviour was stable and failure occurred in some cases after many millions of load cycles. Relative to an 8 tonne axle load that lasts for many millions of load cycles before failure on a strong pavement the damage law exponents calculated can be as low as one but typically around 2 because the relative difference in life between the 4 and 8 tonne is small on the strong pavement. In contrast with a weak pavement where the 8 tonne axle load is close to failing the pavement material in shear the 4 tonne axle load can last for hundreds of thousands of axle passes while the 8 tonne axle is close to failing the materials in shear and the pavement fails in a few thousand axle passes. As a result, the relative difference in life is large along with the calculated damage law exponent. Therefore, the damage law exponent calculated depends on whether or not the reference or axle load is close to failing the pavement by shear as illustrated in figure 5.15. In this figure, an 8 vs 6 tonne axle load on strong pavements has a rutting rate of 5mm and 1mm per million passes respectively while for a weak pavement the 8 tonne fails the material by shear and the rut rate for the 8 tonne increases to 100mm per 1 million passes while the 6 tonne has a rutting rate of 2mm per 1 million passes on the weak pavement. This large change in rutting rate on the weak pavement for an 8 tonne axle does increase the damage law exponent significantly. On another note the CAPTIF tests where the damage law exponent was calculated from pairing the lives

measured for the 4 tonne and 6 tonne loads may be inaccurate if the pavement followed the behaviour of the pavement with weak materials shown in pink in figure 5.15. This is because an 8 tonne load failed the pavement in shear while the 6 tonne load did not rapidly fail the pavement.

The comparisons in rut rate between the LTPP and CAPTIF datasets discussed in this section had a surprisingly good correlation. It is therefore safe to assume the relative damage between the respective pavement strengths and varying wheel loads are therefore valid and could be used as a proxy for the relative damage ration of in-service road pavements.

#### **9.1.1.1 Road user charges**

The results from this research project suggest lower average damage law exponents of between 2 and 1 for a majority of pavements that last at least 100,000 ESAs. Higher damage exponents of up to 7 are possible for pavements with very short lives of <25,000 ESAs. In fact, there is a relationship between pavement life and damage exponent where pavement life is the number of ESAs before pavement rehabilitation is required. However, to charge trucks a cost per kilometre to recoup enough revenue to pay for pavement rehabilitation, the damage law exponent cannot be used in isolation. The pavement life (or strength) needs to be considered also.

This research found a relationship between pavement life and damage law exponent which indicates a need for different RUC for different pavements depending on strength. However, if differential charging is considered then the amount to charge per heavy commercial vehicle (HCV) depends on the unit cost per HCV or per ESA to maintain and rehabilitate the road, together with the damage law exponent value to determine the ESA per HCV. With this method, the cost of maintaining the road is fairly divided based on the relative damage caused by HCVs. For a weak road that lasts 100,000 ESAs before rehabilitation after 50 years, for example, the unit cost to maintain the road per ESA can be determined. Another way to determine the unit cost to maintain the road per ESA per km is to simply determine the annual costs to maintain the road or network divided by the amount of ESA per year. There are more trucks to pay for the cost of a highly trafficked road and thus the unit cost per ESA is less. This difference can result in costs per ESA for a low-volume road to be approximately 30 times higher than for a highly trafficked road.

Assessing the overall cost of maintaining and rehabilitating a length of road or a network to determine an appropriate cost for HCVs should take into account environmental components like settlement and chipseal oxidation. The unanswered question is how to determine the exact environmental component of the cost for the network and once determined how is the cost fairly divided between road users. A recommended method for considering the environmental factor of costs is to include it as one of the factors in estimating pavement life. For example the life estimate for a pavement with poor drainage in a high rainfall area would be less than the life estimate (total number of ESAs to failure) for the same pavement with good drainage. Data from the LTPP sites can be used to determine the effect of environmental factors on pavement life. Changing the pavement life estimate will affect the unit charge per ESA (for pavements with poor drainage the life is less so there are fewer heavy vehicles to pay for the repair) and it will also affect the exponent value used to calculate the ESA.

## 10 Conclusions

- The literature found a range of damage law exponents from 1 to 12 is possible while Dawson (2008) concluded there can never be a single damage law exponent as the value depends on the type or behaviour the material exhibits, ie primarily either stable or shear failure.
- For the first mass limits study at CAPTIF the damage law exponent could be related to pavement structural number (SNP) where a weak pavement with a low SNP value resulted in a high damage exponent while strong pavements with a high SNP value resulted in low damage exponent value.
- The best measure of pavement life at CAPTIF is the medium life found from the number of axle passes to achieve a VSD of 15mm found from linear extrapolation of the VSD data if necessary.
- A good relationship between pavement life found at CAPTIF and damage law exponent for the 4 and 6 tonne equivalent axle loads was found as shown in figure 4.15.
- The rut depth modelling using RLT data of the aggregate and subgrade materials was proven to give reasonable predictions of VSD as measured in the CAPTIF test for example the results of CAPTIF test #1 are shown in figure 5. 4.
- Sections 1A and 1B showed significantly higher rutting than sections 1C and 1D where the only difference was aggregate type. This difference in performance was measured in the NZTA T15 RLT test and associated rut depth modelling (table 5.8).
- Rut depth modelling showed that reducing the tyre pressure by the same proportion as the tyre weight resulted in an increase in life and an increase in damage law exponent; however, actual heavy vehicles do not reduce tyre pressures for reduced loads and thus the result is of interest only and was not studied at CAPTIF.
- Modelling typical LTPP pavements with 4 and 6 tonne axle loads reached the same conclusion as at CAPTIF where there was a strong relationship between pavement life and damage law exponent even when the modelling data was combined with the CAPTIF data as shown in figure 9.4.
- The CAPTIF and rut depth modelling results for the 12 tonne axle load showed a large scatter in damage law exponents and a relationship between pavement life and damage law exponent could not be found. This was probably because the 12 tonne axle load was close to failing the aggregate or subgrade material in shear, resulting in unstable behaviour (high damage law exponents), or because for strong pavements the load was a long way from shear failure (low damage law exponents).
- The reason for high damage law exponents for weak/short-life pavements can be explained by figure 5.15, which shows rut rate versus axle load.
- Measured strains at CAPTIF correlate with the predictions from rut depth modelling in terms of which materials (aggregates, sand or subgrade) contribute the most to rutting.
- The higher volume roads will have a lower unit cost per ESA or per truck pass because there is a greater amount of traffic to share the cost of maintaining the road compared with a low-volume (short-life) road. The cost of pavement rehabilitation on a low-volume weak road is approximately 30 times higher than on a highly trafficked strong road as there are a thousand times fewer HCVs to pay for the maintenance.

# 11 Recommendations

- The main recommendation is to consider using a damage law exponent based on the data provided in figure 9.4. There is a clear trend between damage law exponent and life.
- On average, state highways with a 25-year design traffic loading of greater than 1 million ESAs should consider using a damage law exponent of approximately 2; however, designing for the heaviest commercial vehicles operating on local low-volume roads with a lower life would need to consider a damage law exponent closer to 6. With the scatter in the results it may be prudent to consider a more conservative value for routine design.
- This research has provided a relationship between damage law exponent and pavement life which together with a known heavy vehicle traffic count can be used to determine an appropriate rut wear component cost for HCVs travelling on any section of road.
- The environmental component of damage can be taken into account with the estimate of pavement life which is then used in an equation to determine the damage law exponent. For example a pavement that has poor drainage in a high rainfall area will have a lower pavement life than the same pavement in a low rainfall area with good drainage. The data from the LTPP sites can be used to determine how environmental factors affect the pavement life.
- The current RUC system could be reviewed using a weighted average approach with the six road classification types used to assist in determining an appropriate exponent to use in pavement structural/rutting wear calculations. This would not alter the total RUC required from industry but may alter the incremental cost of running lightly load truck axles vs heavy loaded axles.

## 11.1 Future research

In the Transport Agency research project, *Pavement thickness design charts derived from a rut depth finite element model* (Arnold and Werkmeister 2010b) a range of pavements were modelled to predict life for an 8 tonne axle load using RLT tests on different sub-base (good, average, poor), basecourse (good, average, poor) and subgrades (CBR 2, 8 and 10). The pavement depths modelled in this research are summarised in figure 5.20. To cover these variables around 500 different pavement combinations were modelled for an 8 tonne axle. It is recommended for future research that these pavement combinations be remodelled again but with different axle loads of 4, 6 and 12 tonnes to calculate the damage exponents.

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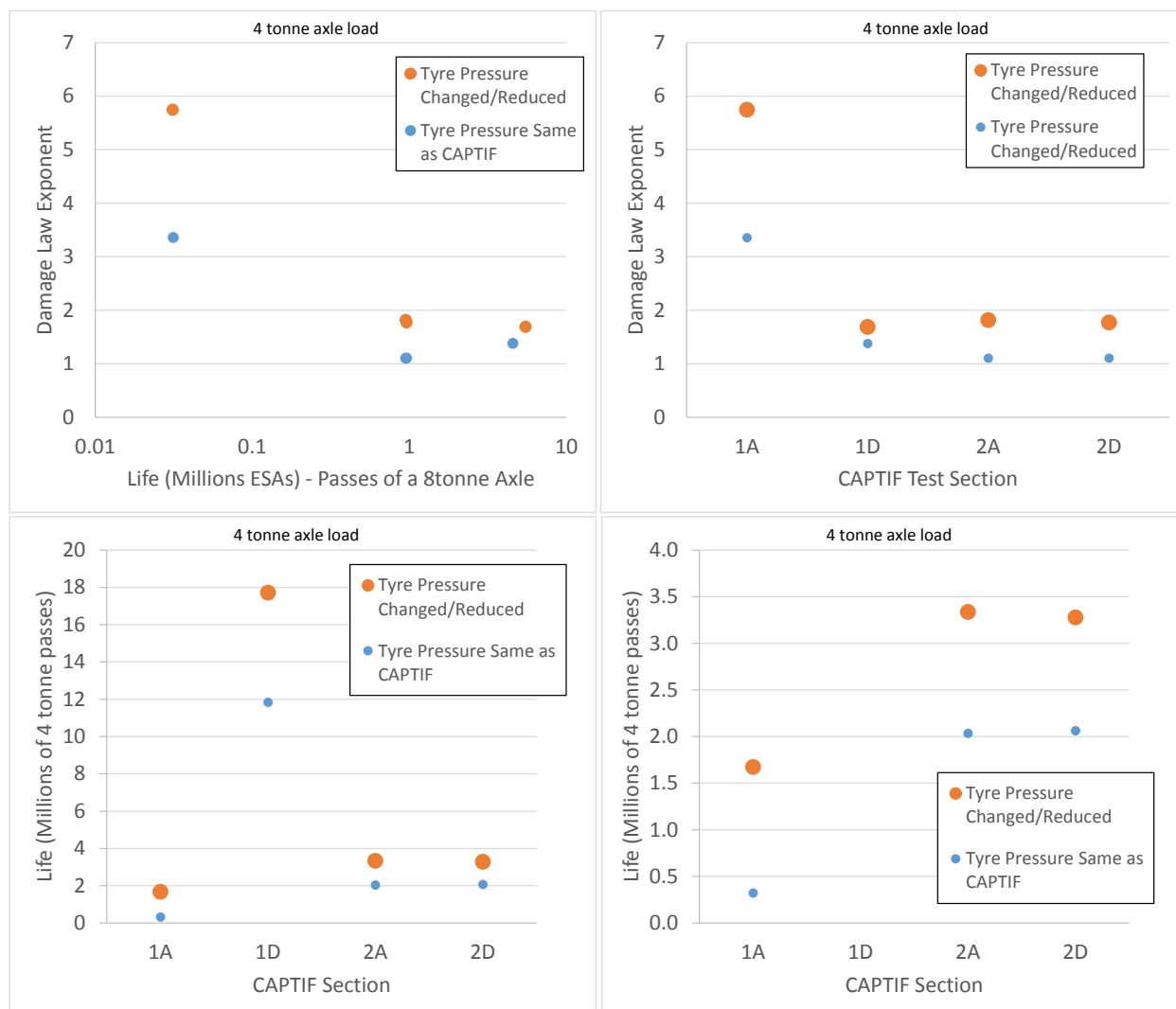
## Appendix A: Tyre pressure effect on damage exponent

The effect of tyre pressure on damage exponent was assessed by remodelling CAPTIF tests 1 and 2 using the following tyre pressures and equivalent axle loads as detailed in table A1. The results of this analysis are shown in figures A1 to A3. Results for the 12 tonne axle load show the damage law exponent increases to around 3 because of an increase in tyre pressure compared with the earlier modelling. For the lighter 4 and 6 tonne loads the life increased with reduced tyre pressures which resulted in an increase in damage law exponents from around 1 to 2. In all cases when reducing the tyre pressures the life increased around 40%. In summary, for loads less than 8 tonnes reducing the tyre pressure to the value used for the 8 tonne reference load has the effect of increasing both the life of the lower loaded axles and the damage exponent. This result is not unexpected.

**Table A.1** Tyre loads and tyre pressures modelled for CAPTIF tests 1 and 2

Equivalent dual tyred axle load (tonnes)	Equivalent single wheel load modelled (kN)	Tyre pressure (kPa)
4 tonnes	20 kN	413.5 kPa (early modelling at 758kPa)
6 tonnes	30 kN	620.25 kPa (early modelling at 758kPa)
8 tonnes	40 kN	827 kPa (same as CAPTIF test)
12 tonnes	60 kN	1240.5 kPa (early modelling at 758kPa)

**Figure A.1** Effect of changing tyre pressure when modelling 4 tonne axle



**Figure A.2** Effect of changing tyre pressure when modelling 6 tonne axle

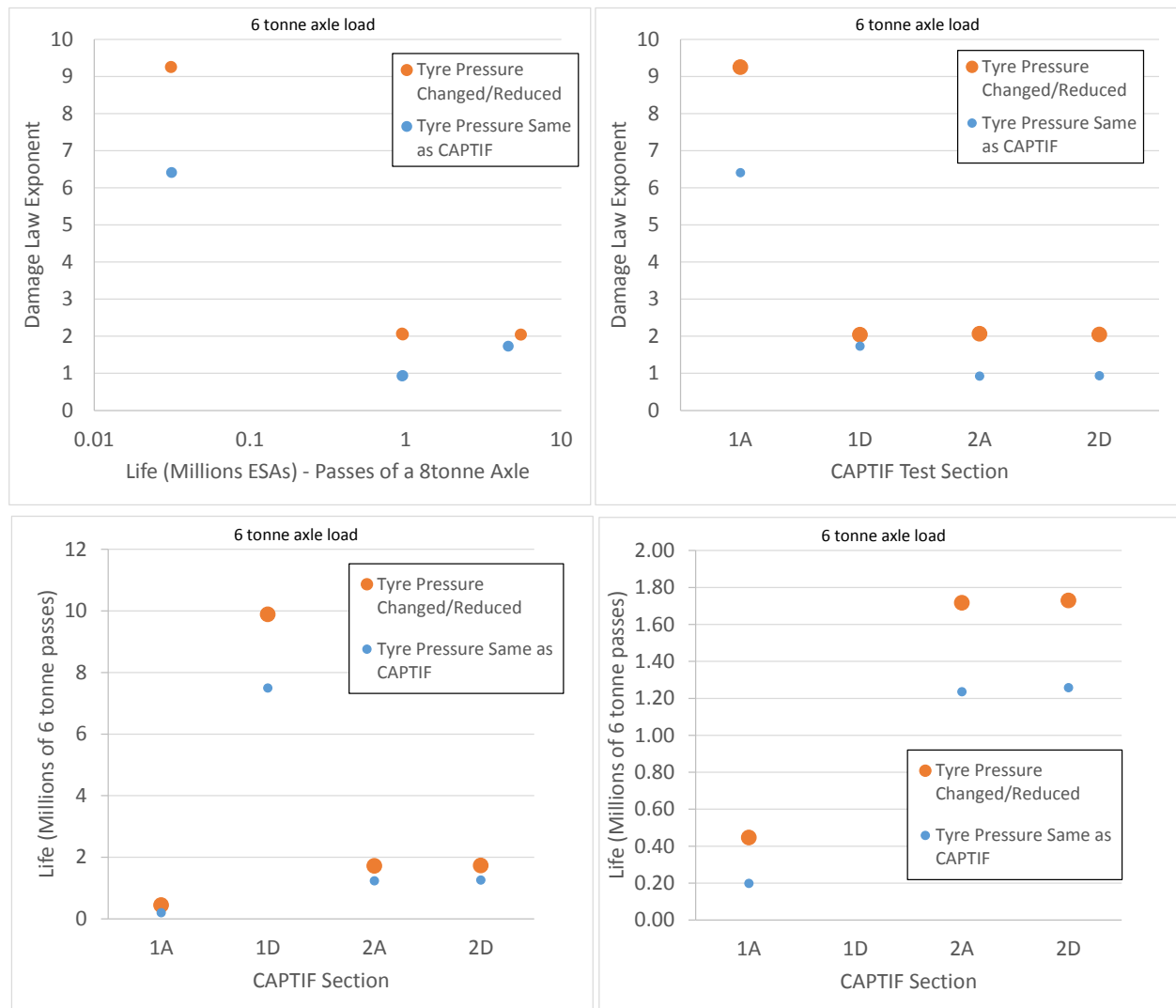
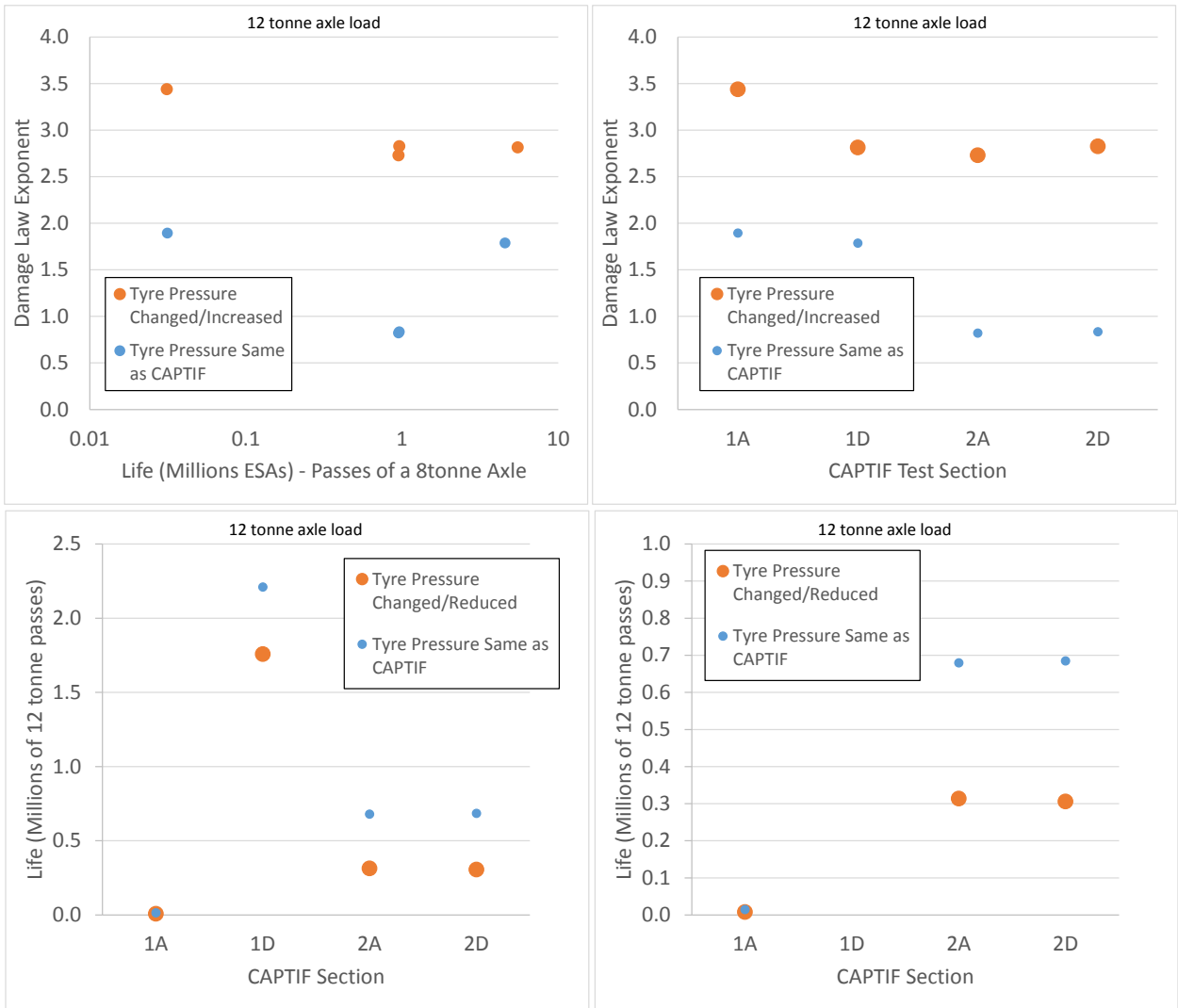
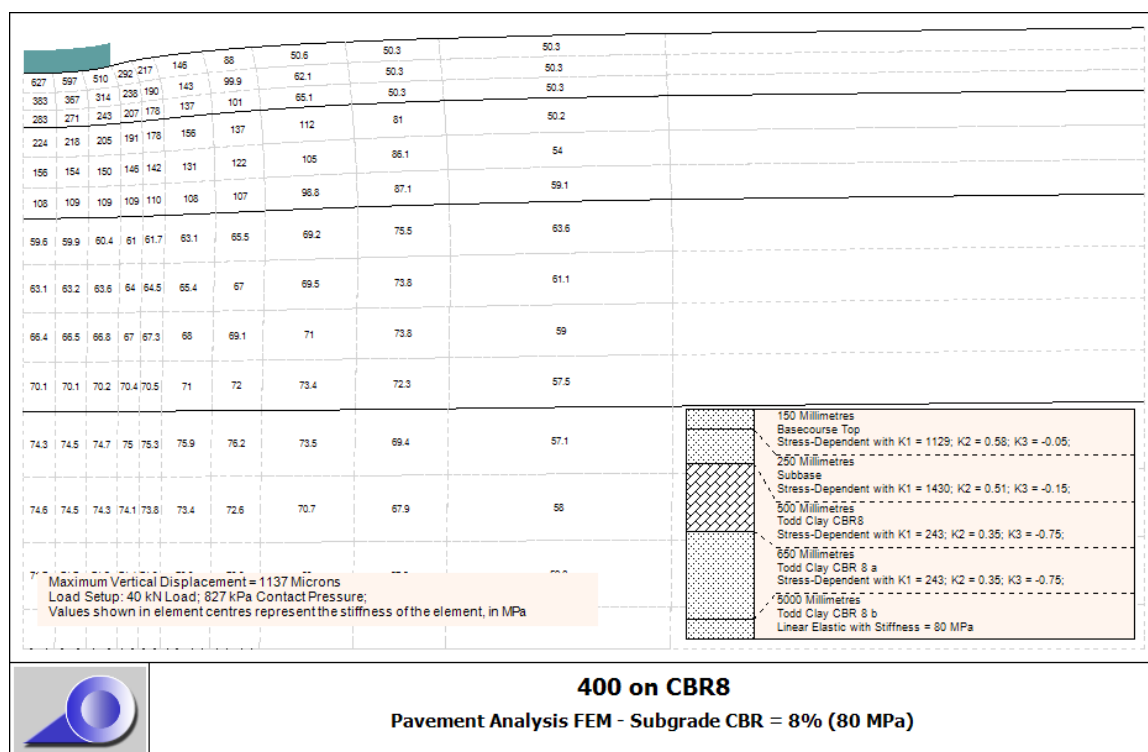


Figure A.3    Effect of changing tyre pressure when modelling 12 tonne axle



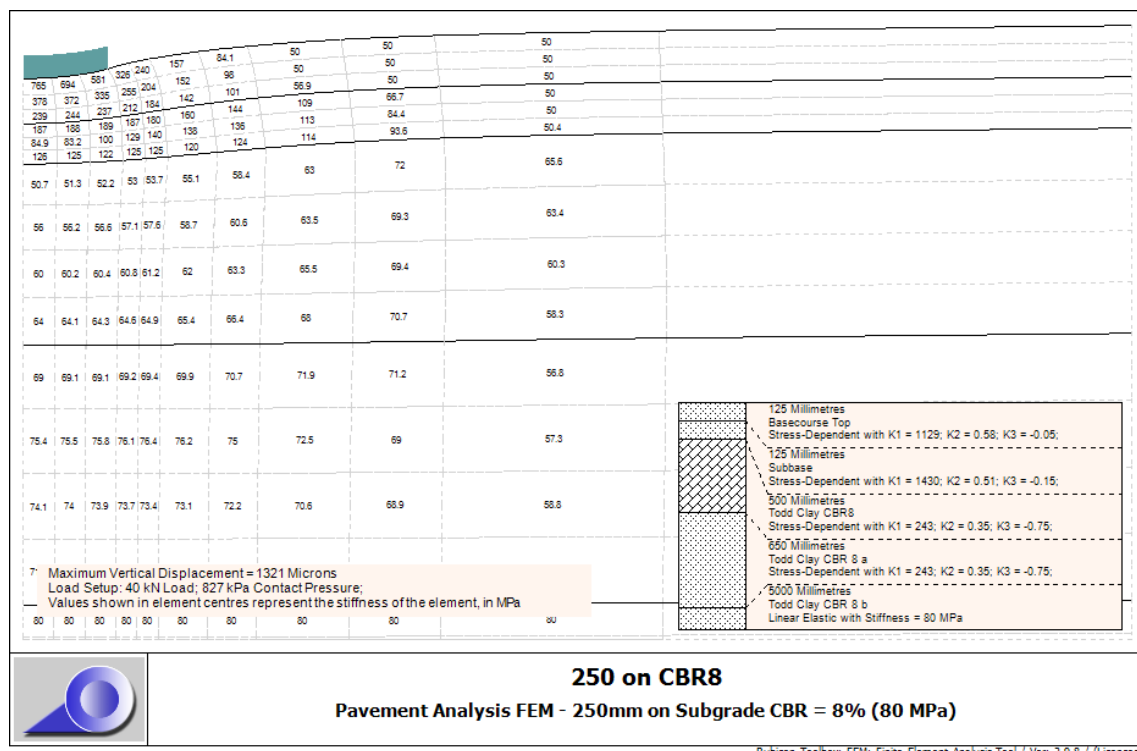
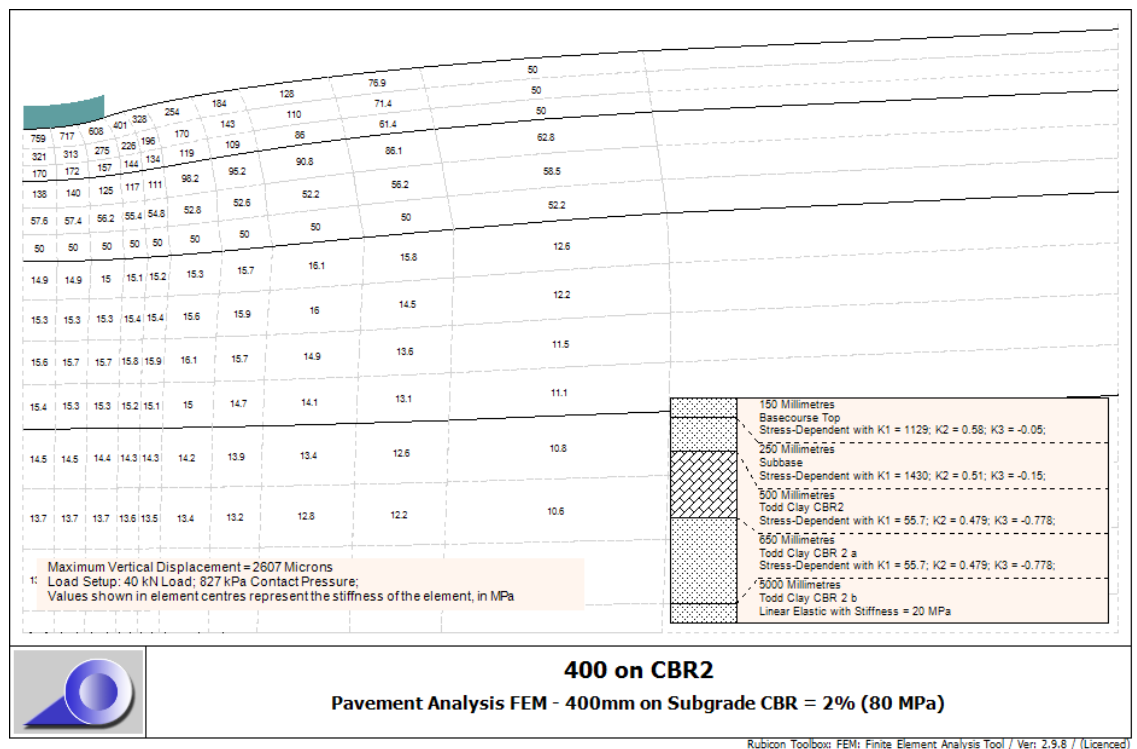
## Appendix B: Finite element analysis of LTPP pavements

Figure B.1 Finite element analysis model of 400mm aggregate on a subgrade CBR of 8%

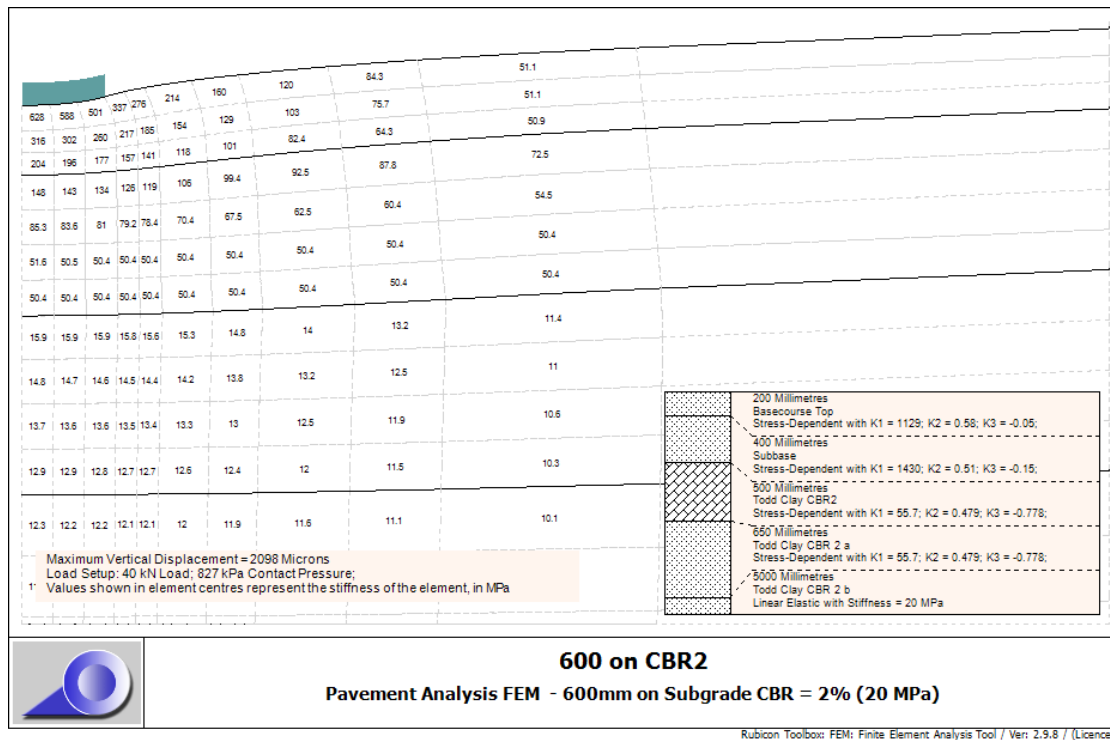


Rubicon Toolbox FEM: Finite Element Analysis Tool / Ver: 2.9.8 / (Licenced)



**Figure B.2** Finite element analysis model of 250mm aggregate on a subgrade CBR of 8%**Figure B.3** Finite element analysis model of 400mm aggregate on a subgrade CBR of 2%

**Figure B.4 Finite element analysis model of 600mm aggregate on a subgrade CBR of 2%**



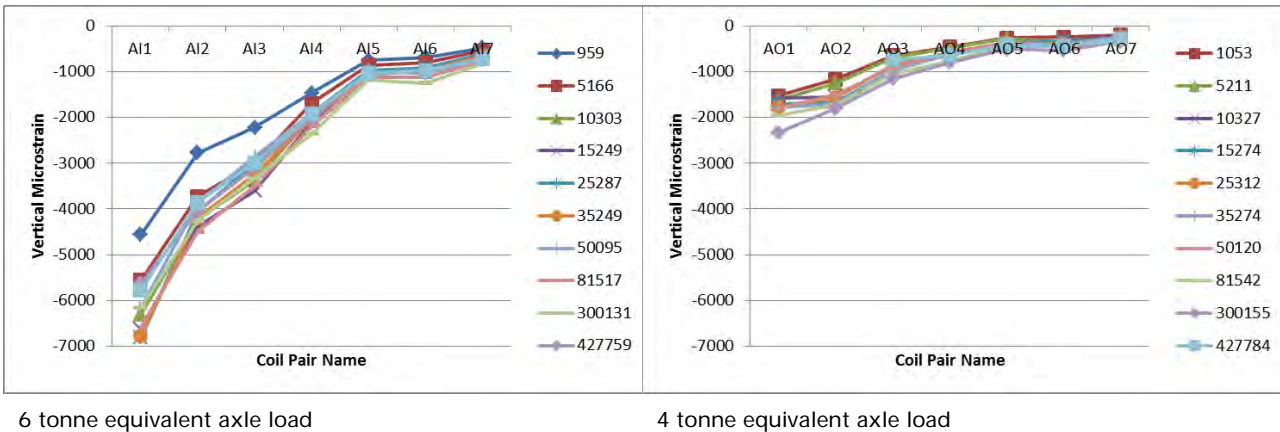
# Appendix C: Model verification with in situ instrumentation

## C1 Measured internal elastic and permanent strains – CAPTIF test 1

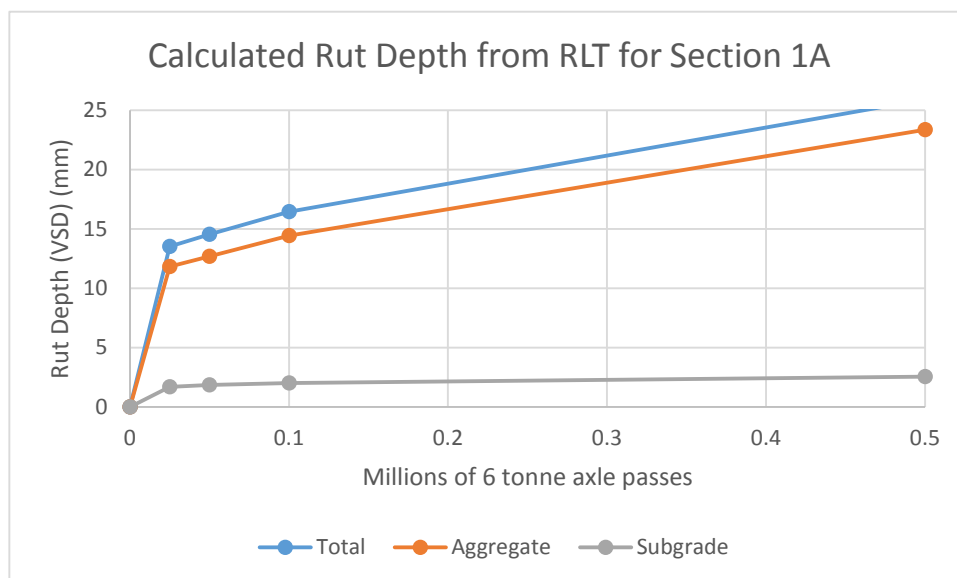
Vertical elastic strains were measured during the testing using seven pairs of CAPTIF strain coils stacked vertically in the pavement at a spacing of 75mm. These vertical strains were recorded at various load stages throughout the testing at CAPTIF.

CAPTIF test 1 section A with Taupo dacite showed very high strains as high as 7,000 micro-strains for the 6 tonne wheel path within the aggregate base layer which increased with the increasing number of load cycles (figure C1). The 4 tonne wheel path reached strains as high as 2,300 micro-strain within the base aggregate layer. The measured vertical resilient strains in the 6 tonne wheel path were approximately 3.5 times higher than in the 4 tonne wheel path. Interestingly, the long-term rutting rate for the 6 tonne wheel path measured in the CAPTIF test was 3.4 times higher than the 4 tonne wheel path resulting in a damage exponent of 3 (table C.1). Thus the measured vertical resilient strains were related to rutting measured at the surface. Further, those strains measured in the basecourse layer were unusually high which suggests a significant amount of the rutting was within the basecourse layer. This conclusion is supported by the RLT testing and associated rut depth modelling (figure C.2). Further, the VSD was calculated from the CAPTIF strain coils as shown in figure C.3 and this also shows most of the permanent strain/rutting was in the basecourse layer.

Figure C.1 Resilient vertical strains from CAPTIF strain coils for CAPTIF test 1A



**Figure C.2** Calculated rut depths from RLT for section 1A



**Table C.1** Measured pavement lives, rutting rates and damage exponents for 4 and 6 tonne loads in CAPTIF section 1A

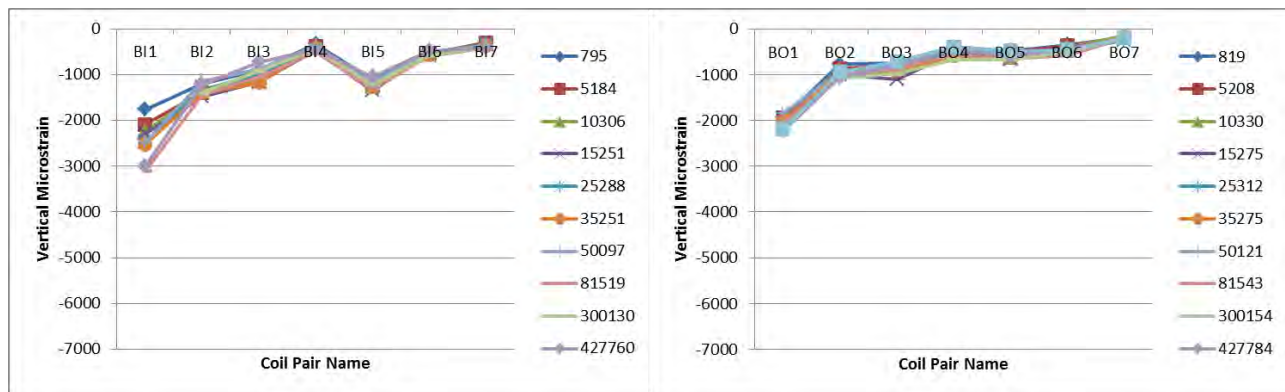
Equivalent axle load (tonnes)	Long term rate of rutting mm/1M (15mm VSD)	Life to VSD = 15mm (millions) (15mm VSD)	Damage exp (VSD 15mm)	Damage exp (slope - 15mm)
6	32.70	0.18	2.02	3.04
4	9.54	0.42	2.02	3.04

**Figure C.3** Measured vertical surface deformation/rutting from CAPTIF strain coils for CAPTIF test 1A (6 tonne axle)

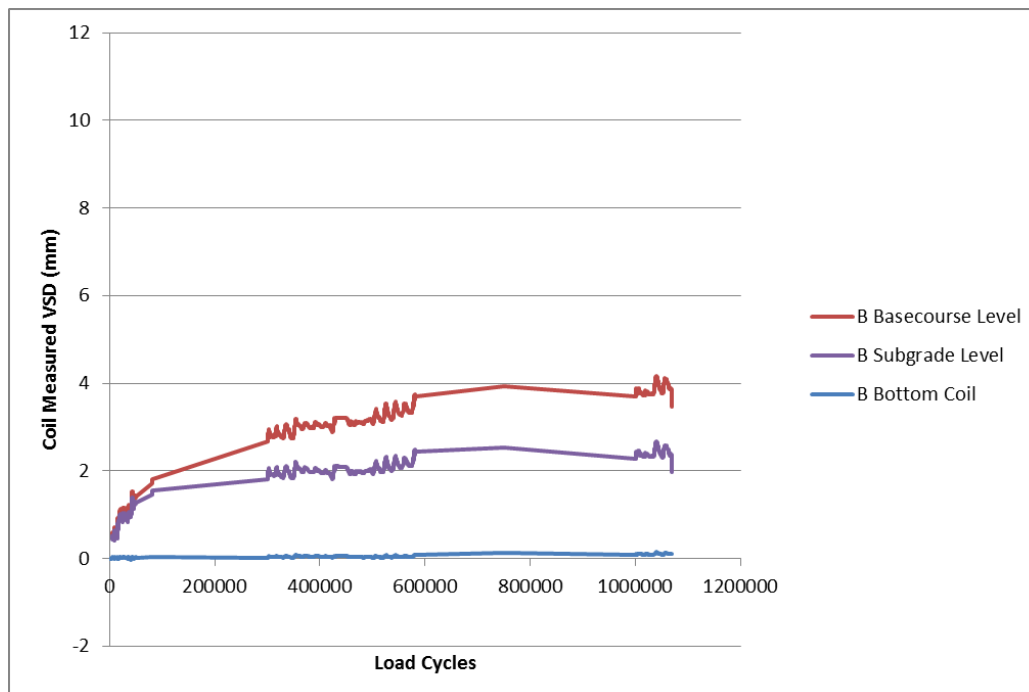


Section 1B was the same as section 1A except a greater thickness of dacite aggregate was used. Measured vertical resilient strains are shown in figure C.4 but the highest strain recorded in the dacite aggregate basecourse layer was a 3,000 micro-strain which is less than the 7,000 micro-strain recorded in section 1A. The ratio of strains for the 4 and 6 tonne axle loads was approximately 1.5 while the ratio of rutting rates was 4.25 or the ratio of lives was 2.32. Thus the differences in resilient strains did not fully explain the larger differences in measured surface rutting and lives as was the case for section 1A. Nevertheless, Section 1B shows the same trend as 1B where most of the strain and rutting was attributed to the basecourse layer as shown with the measured deformation of the CAPTIF strain coils (figure C.5).

**Figure C.4 Vertical resilient strains from CAPTIF strain coils for CAPTIF test 1B**

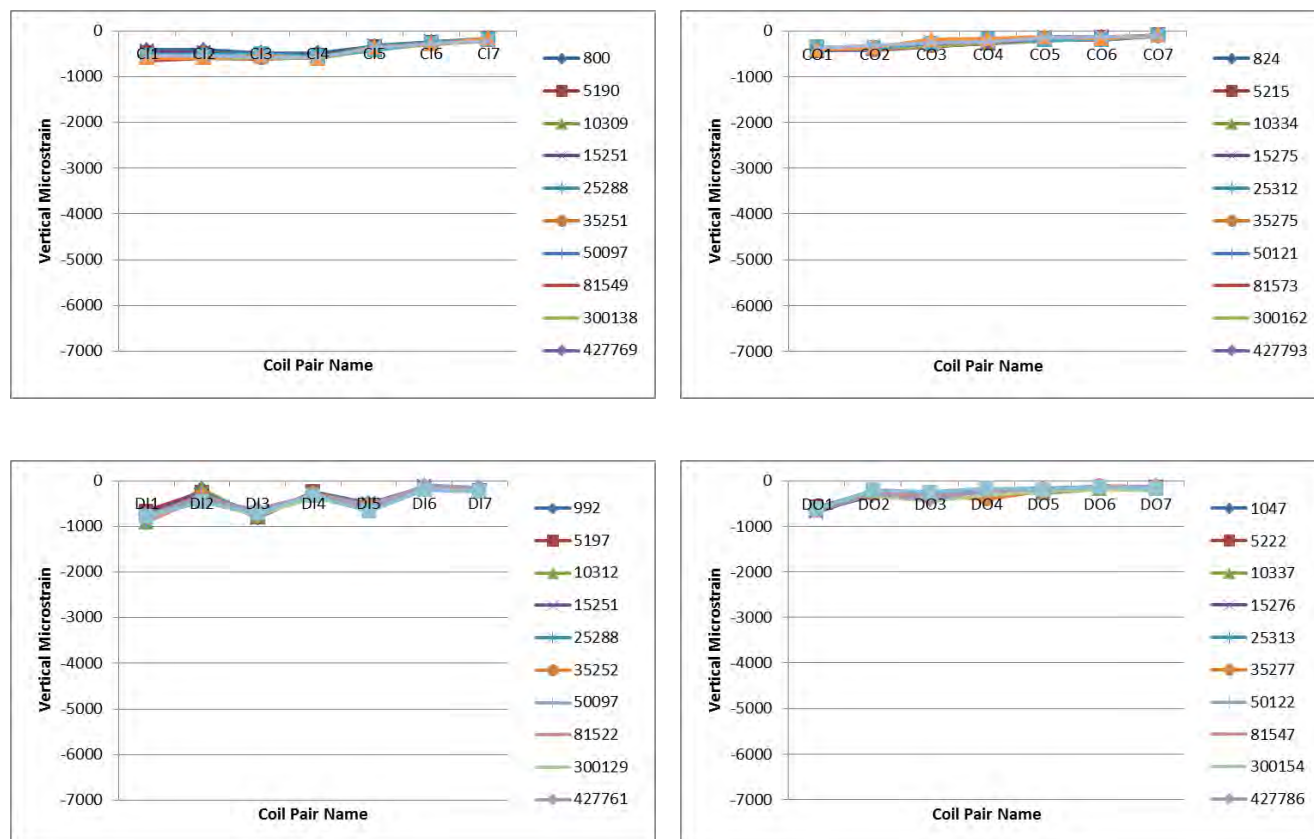


**Figure C.5 Measured vertical surface deformation/rutting from CAPTIF strain coils for CAPTIF test 1B (6 tonne axle)**

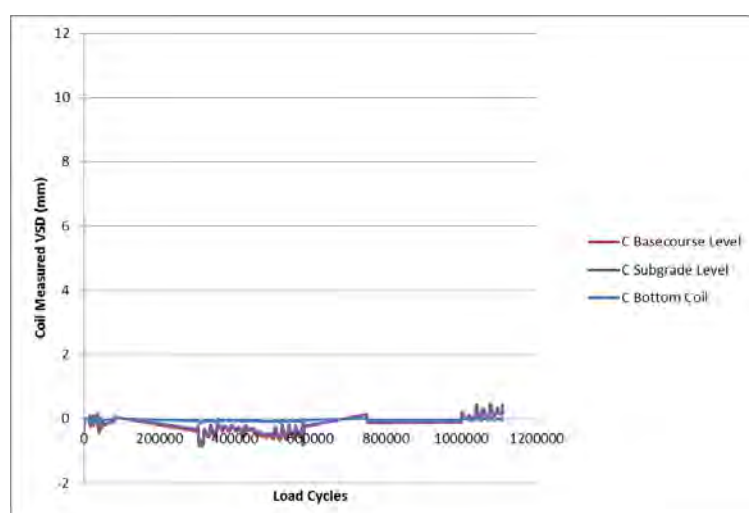


Strains measured in sections 1C and 1D were very small and as a result these sections showed very little rutting of no more than 5mm. This result was significantly different to sections 1A and 1B which used Taupo dacite aggregate. The RLT testing and associated rut depth modelling also predicted very low rutting. Figures C.6, C.7 and C.8 show very low elastic and nil permanent strains were recorded (figure C.8).

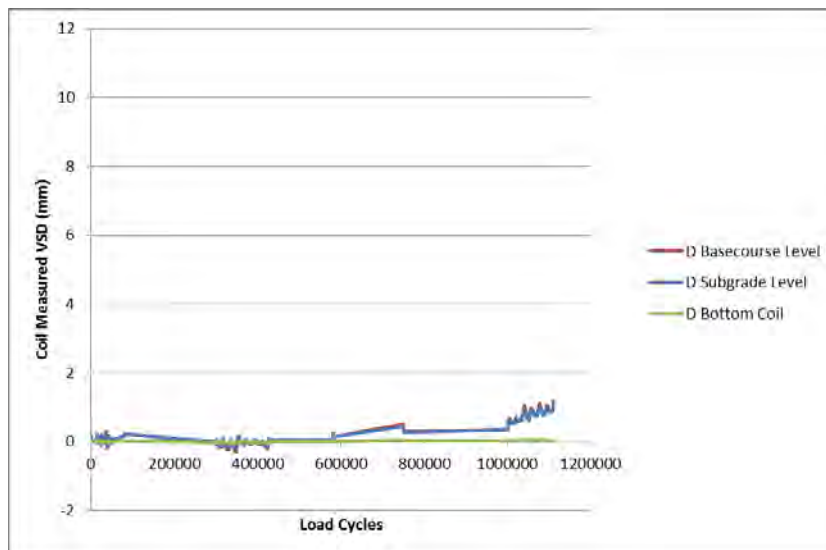
**Figure C.6 Vertical resilient axial strains measured in CAPTIF test sections 1C and 1D**



**Figure C.7 Measured vertical surface deformation/rutting from CAPTIF strain coils for CAPTIF test 1C (6 tonne axle)**



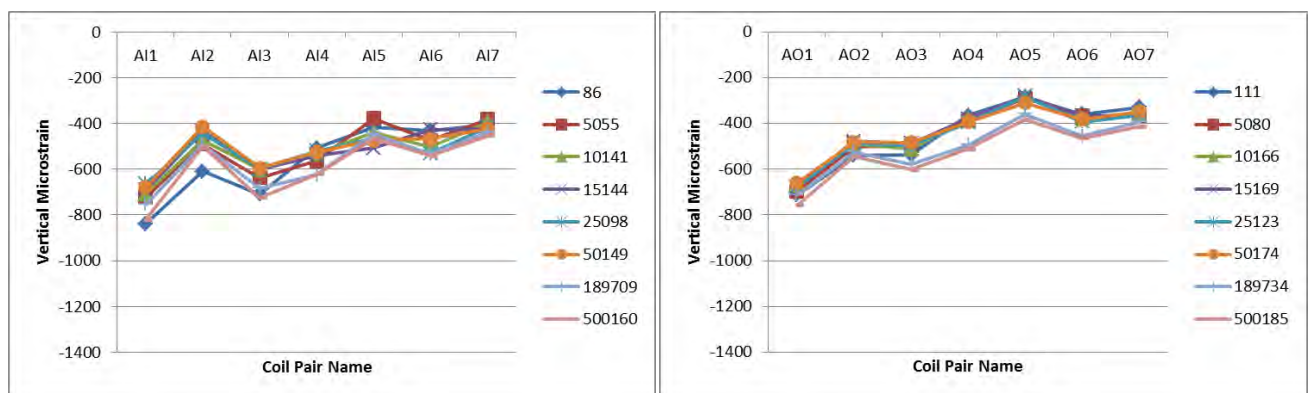
**Figure C.8 Measured vertical surface deformation/rutting from CAPTIF strain coils for CAPTIF test 1D (6 tonne axle)**



## C2 Measured internal elastic and permanent strains – CAPTIF test 2

CAPTIF test number 2 showed modest strain measurements and very little difference in measured strains between the two wheel paths with 6 and 4 tonne axle loads. This may also explain why the damage law exponents found in this test were low from 1.3 to 1.8 which indicated that the difference in rutting between the 6 and 4 tonne loads was lower than expected if using the Fourth Power Law. The permanent strains measured (figure C.10) show that half the rutting at the surface was from the sand layer while the other half was in the basecourse aggregates. Figure C.11 shows the calculated rutting and demonstrates that around 80% of the surface rutting was from the 600mm sand layer. The permanent strain plots are from single coil pairs at 75mm spacing and if the deformations in the other six coil pairs in the sand layer are added together then it is likely most of the rutting will be seen in the sand layer.

**Figure C.9 Vertical resilient axial strains measured in CAPTIF test sections 2A, 2B, 2C and 2D**





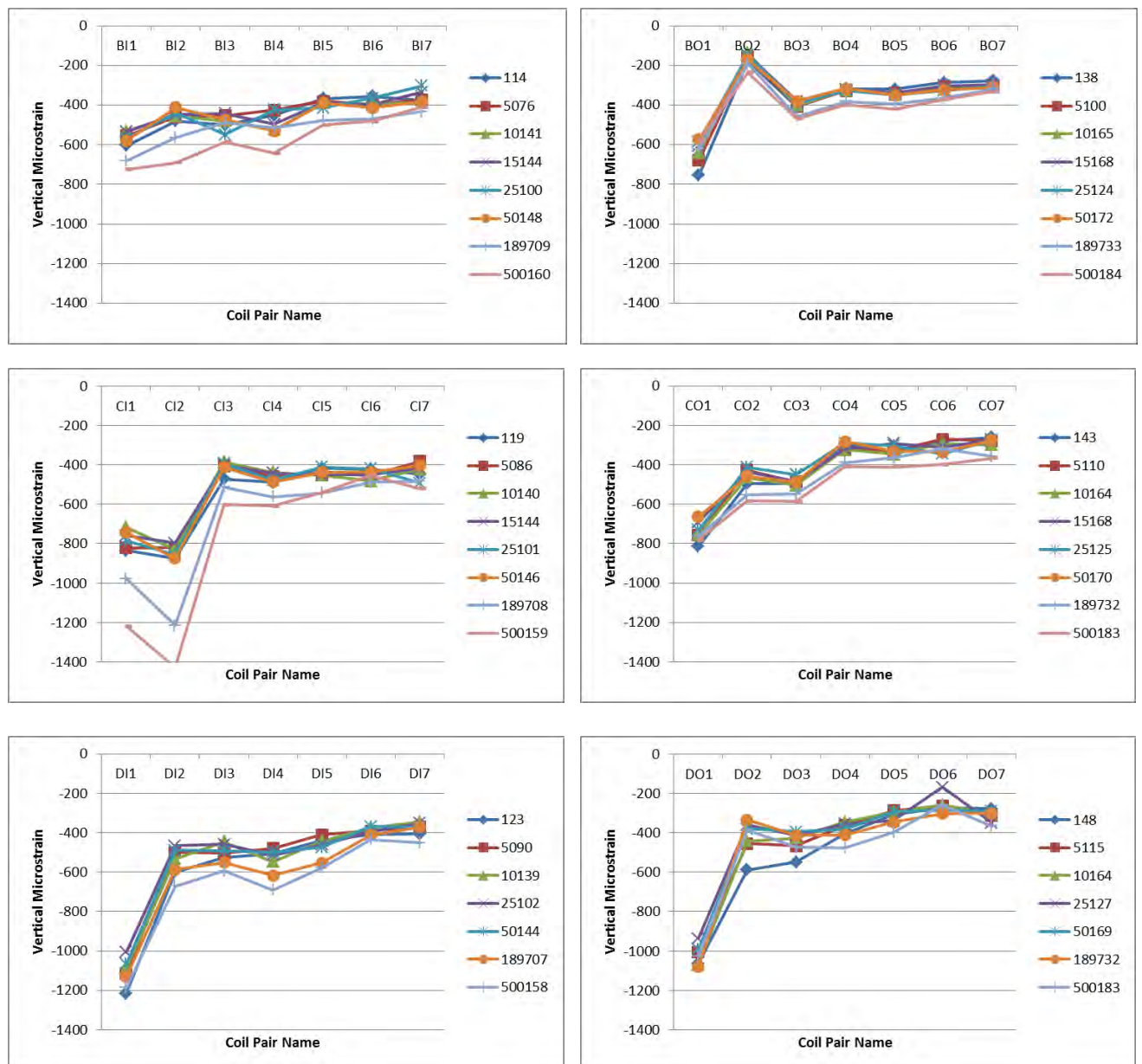
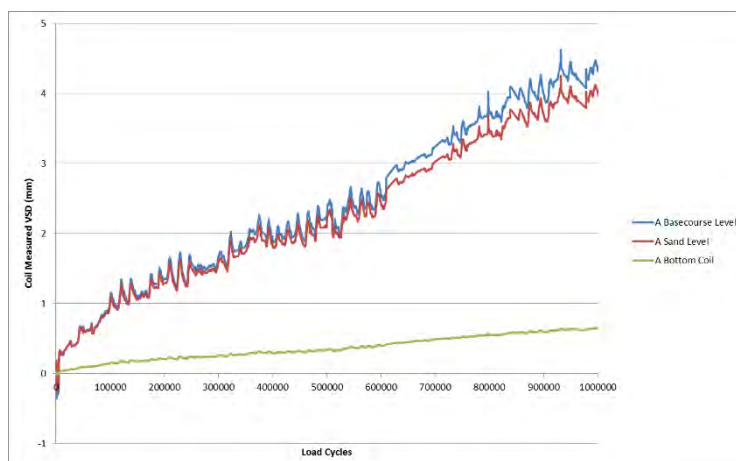


Figure C.10 Vertical deformation measured in CAPTIF strain coils for CAPTIF test sections 2A, 2B, 2C and 2D





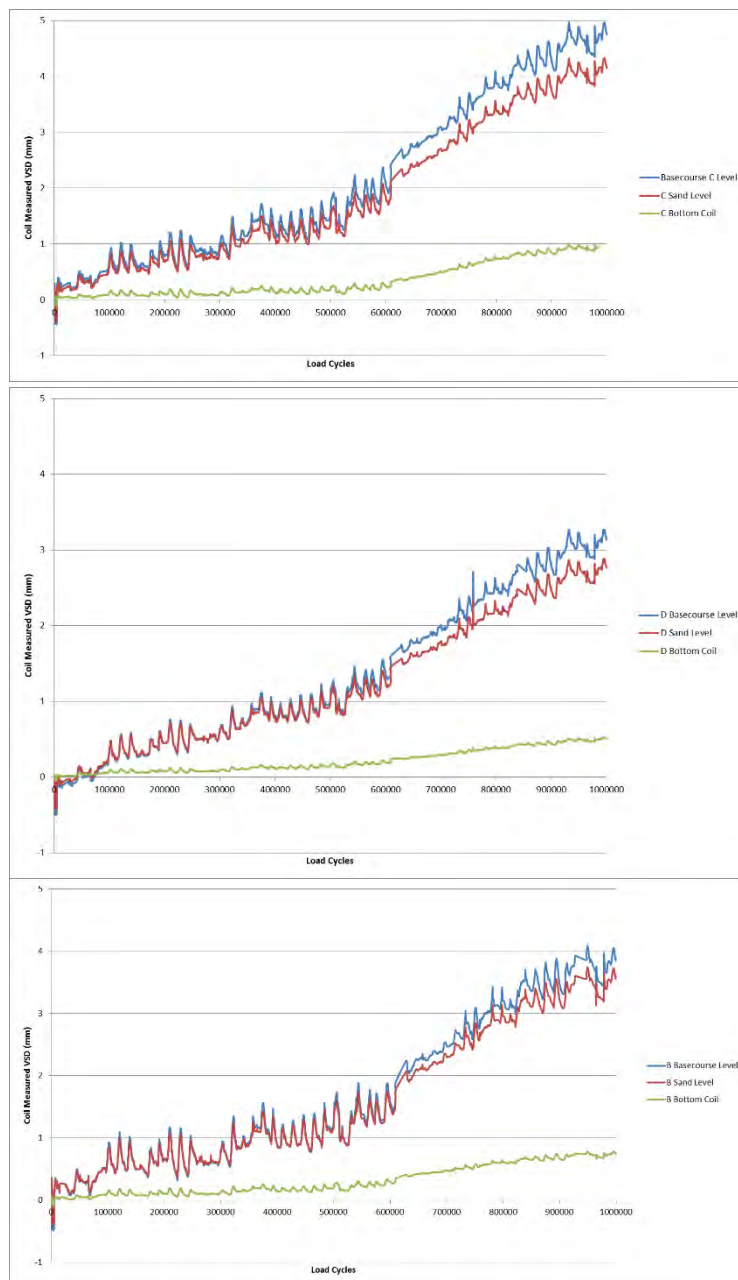
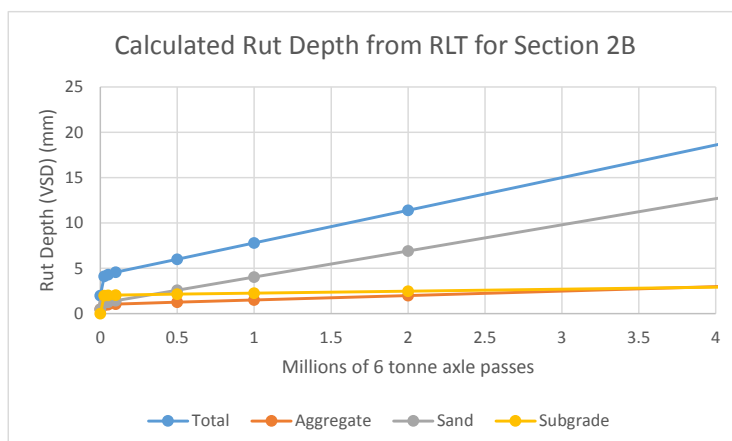


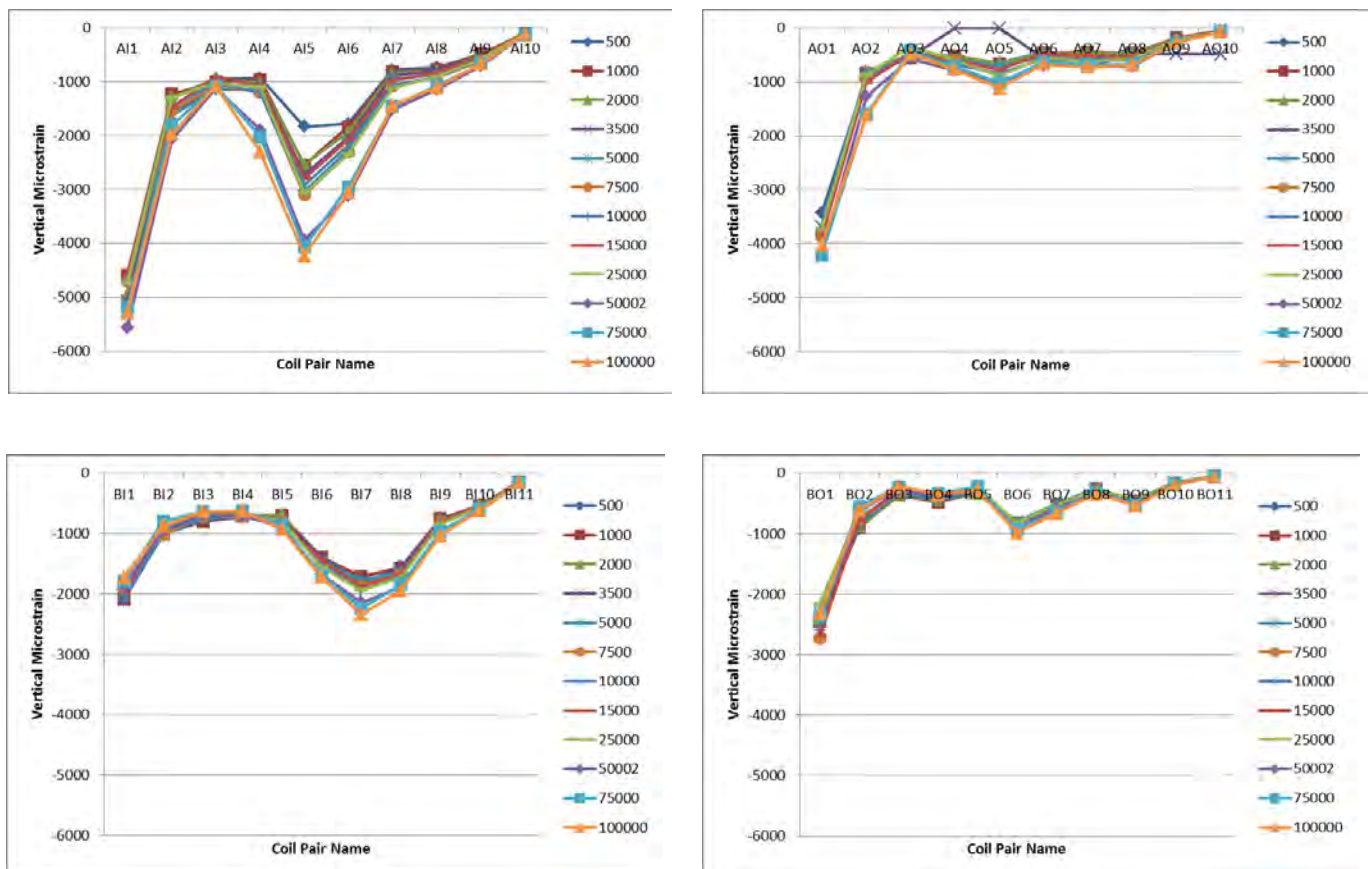
Figure C.11 Calculated rut depth from RLT tests



### C.3 Measured internal elastic and permanent strains – CAPTIF test 3

Resilient strains in CAPTIF test 3 were high at the top of the basecourse and at the top of the weak subgrade (CBR=2%). Sections 3A and 3D had the highest strains as these were the shallowest pavements with only 100mm of aggregate. Sections 3C and 3D failed very quickly which was expected as a rounded river gravel was used, but the measured resilient strains were no higher than sections 3A and 3B which used the crushed rock M4. The deformations measured in the CAPTIF strain coils (figure C.13) show that most of the rutting was due to the aggregate and sand layers despite the high resilient strains recorded in the subgrade. The RLT testing and associated modelling showed that most of the rutting was in the subgrade (figure C.14) rather than in the sand layer. However, the RLT testing and modelling of the sand layer at the actual measured density when constructed found the pavement failed quickly (within 1,000 load cycles) due to significant rutting in the sand layer. Changing the RLT test with the sand at a higher density resulted in less rutting in the sand layer and delivered a predicted surface rut depth that was closer to the measured rutting at CAPTIF. Thus it is likely that most of the deformation was in the sand layer but testing and modelling weak and unstable pavements makes it difficult to predict rut depth.

**Figure C.12 Vertical resilient axial strains measured in CAPTIF test sections 3A, 3B, 3C and 3D**



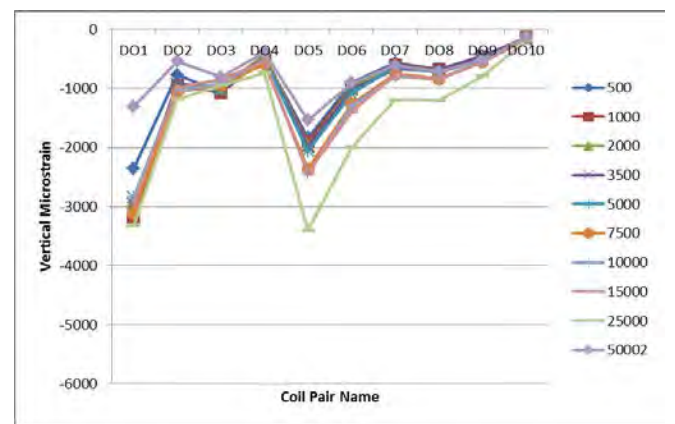
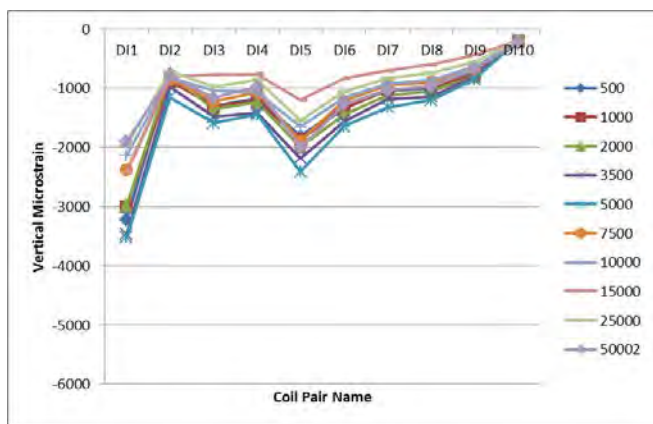
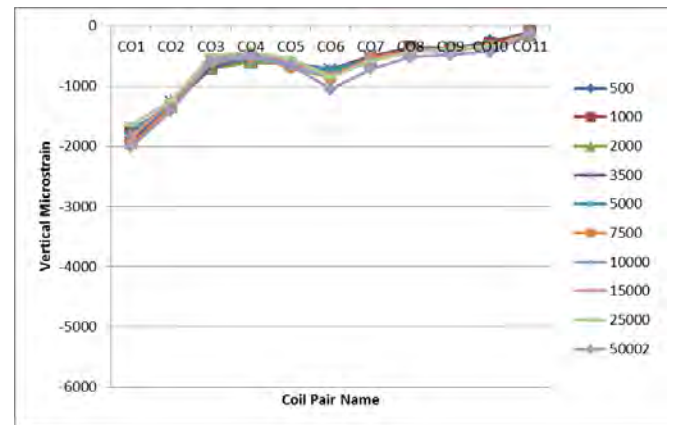
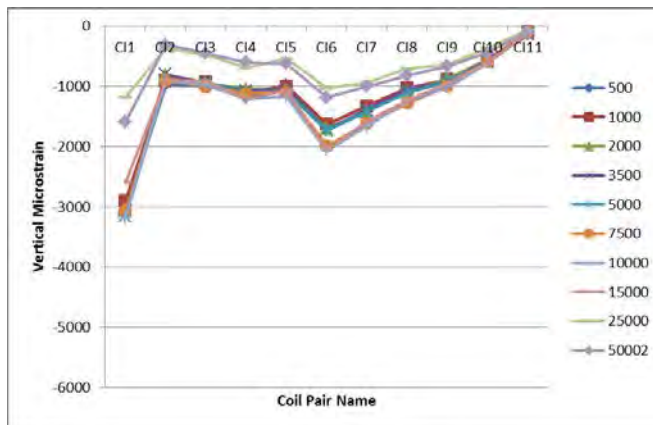
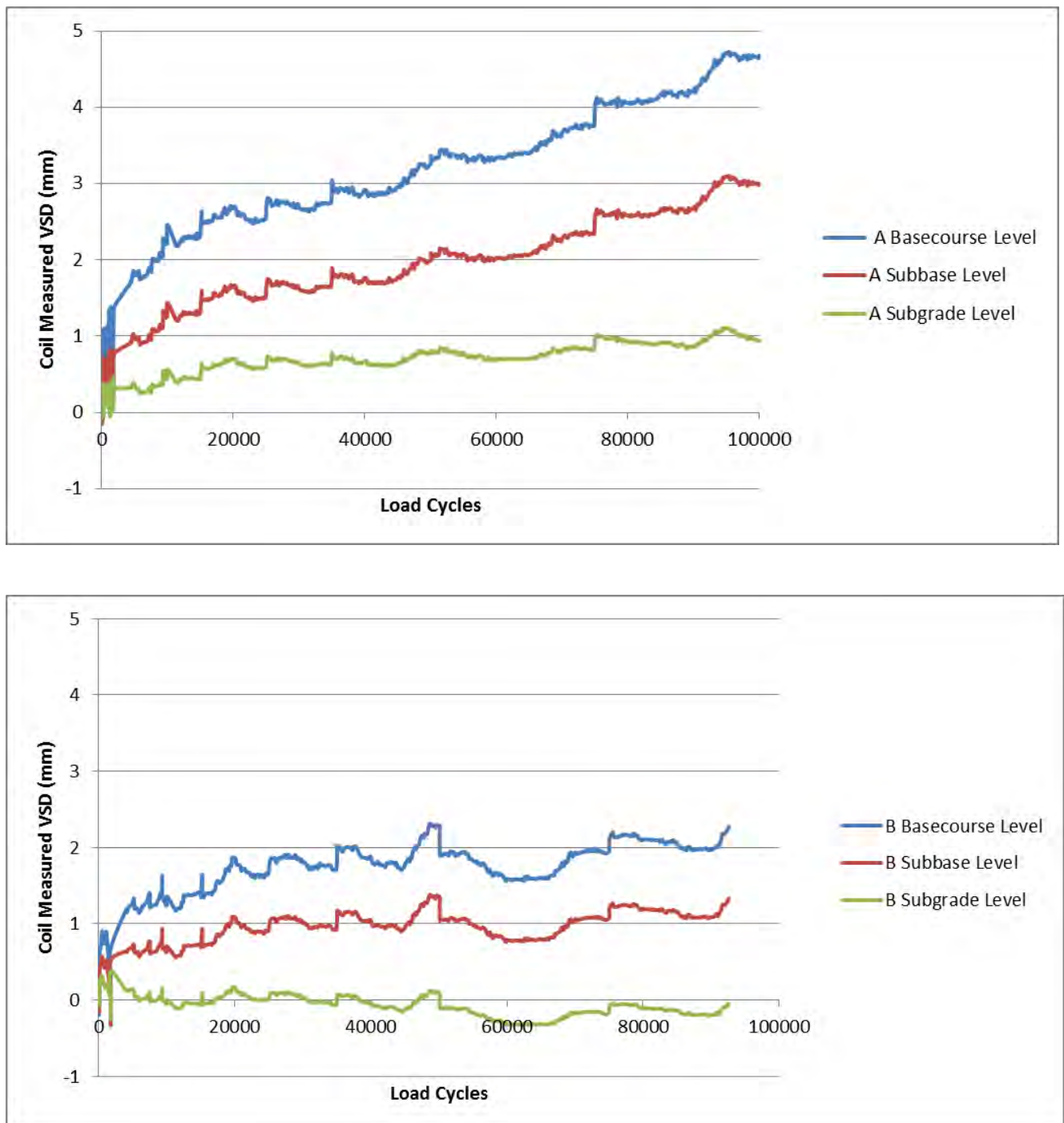


Figure C.13 Deformation measured in the CAPTIF strain coils in CAPTIF test sections 3A, 3B, 3C and 3D



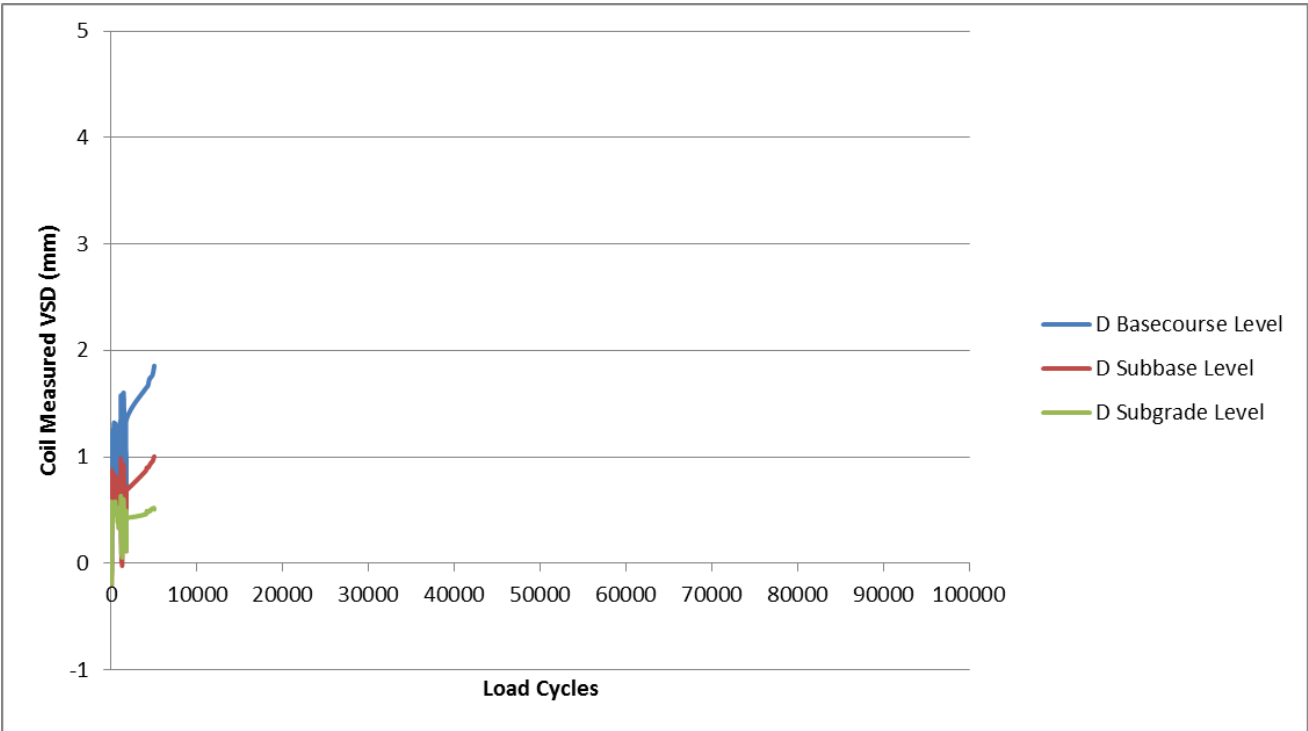
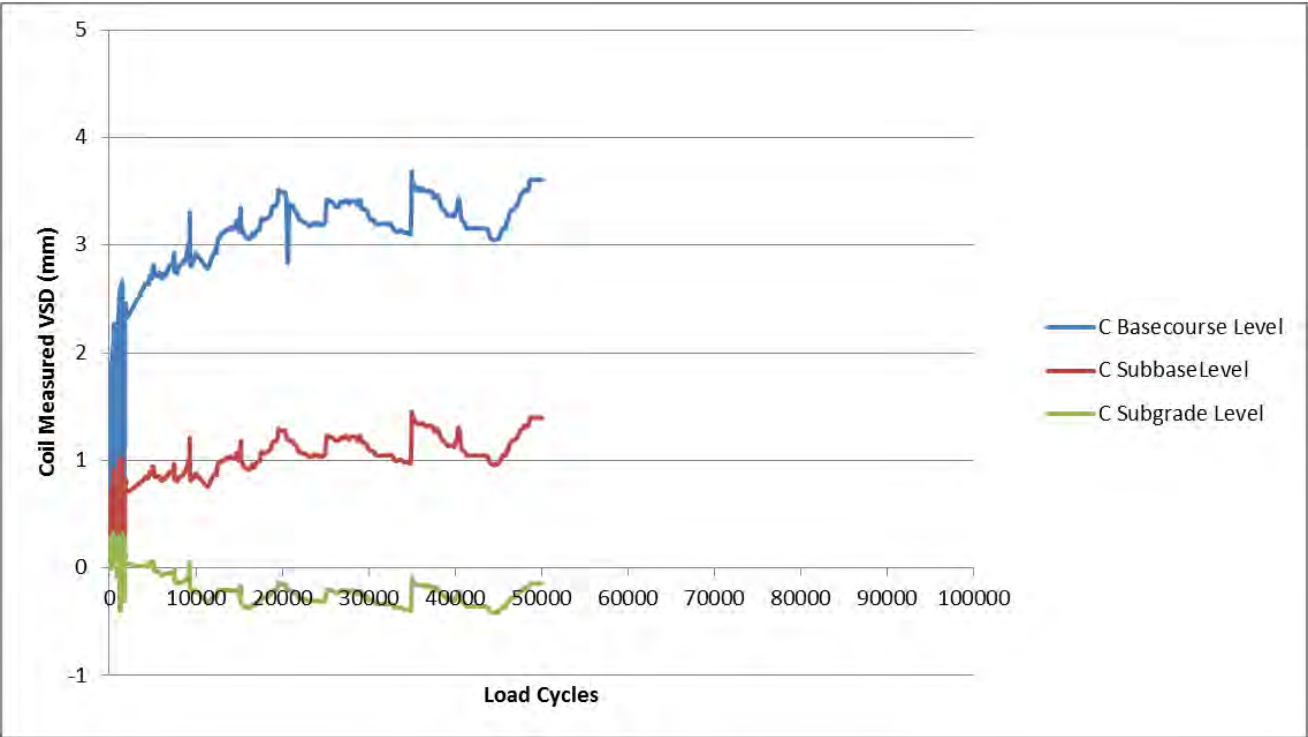


Figure C.14 Calculated rut depth from RLT testing for CAPTIF section 3B

