DO WE REALLY KNOW HOW THIN SURFACED GRANULAR PAVEMENTS BEHAVE UNDER HIGHER AXLE LOADS?

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ABSTRACT

Transit New Zealand's (Transit's) research on mass limits aims to determine the impact of higher vehicle masses on different pavement structures and materials. The findings are of considerable interest in New Zealand, given Transit's proposed changes to vehicle mass limits, and the need to understand the impacts of these changes on New Zealand's road network, and noting that Australia has proposed to adopt a significantly different model for its current performance based standards for heavy vehicles project.

The previous test undertaken at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) was to compare the impact on pavement wear between the 8.2 and 10 tonne dual tyred axle. These results were reported at the previous conference held in Delft during June 2002, where the concept of 'compaction' and 'wear' stages in pavement life was introduced. An additional accelerated pavement test was undertaken to compare the wear from a 12 tonne axle with that from an 8.2 tonne axle. The two loading units at CAPTIF were configured to be the same in all respects except load. They trafficked parallel paths on five different thin-surfaced pavements for 1 million load cycles. The pavement was extensively monitored throughout the test with both reponse and performance measurements recorded.

In drawing conclusions from the research it must be recognised that the test pavement contained a strong dry subgrade in ideal environmental conditions that are unlikely to be replicated in real life. The behaviour of New Zealand's more typical weaker or saturated subgrades was not investigated, nor was the effects on older and/or poorly maintained surfaces where moisture may be entering the base.

This research has extended the previous findings with the aim of providing models suitable for the network analysis of impacts of new heavy vehicles, and has explored the models in a simplified example of a network subjected to an increase in mass limits. The research also proposes a framework for assessing the impact of new innovative heavy vehicles on unbound granular pavements using strain measurements made within the pavement layers.

INTRODUCTION

The road transport industries in New Zealand and Australia have been lobbying for increases in the allowable mass limits for heavy vehicles on the basis of increased efficiency and benefits to the economy. Some of the proposals for increased mass limits involve increased axle mass limits, which would clearly lead to additional pavement wear. Road controlling authorities, while sharing the industry's aims for increased efficiencies in the road transport system, are concerned that the additional pavement wear generated by higher axle loads is fully reflected in road user charges, so that the standard of the network can be maintained.

The study reported here is stage 3 of a multi-stage accelerated pavement testing programme, that is investigating, via research at Transit New Zealand's (Transit's) Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF), on typical New Zealand and Australian pavement designs, the relative effect on pavement life of an increase in axle mass from 8.2 tonnes (i.e. 40kN load for a half axle with dual tyres) to 12 tonnes (i.e. 60kN load for a half axle with dual tyres). Results from stages 1 and 2 determined the research conducted in stage 3 and are reported in previous papers (de Pont et al, 2001 and 2002) and at the previous conference held in Delft in 2002 (de Pont et al, 2002). At that conference the concept of 'compaction' and 'wear' stages in pavement life was first introduced.

ACCELERATED PAVEMENT TEST

In this study, which was co-funded by Austroads (representing road agencies in Australasia) and Transit and conducted in 2002, an accelerated pavement test was undertaken at CAPTIF to compare the effect of mass on pavement wear for five different segments of pavement which are more typical of those found in New Zealand and Australia. The study is reported fully by Arnold et al (2003). The two "vehicles" at CAPTIF were configured with identical suspensions but with different axle loads. One was loaded to 40kN to simulate the current 80kN axle load limit, while the other was loaded to 60kN to simulate a possible increase to a 120kN axle load limit. The tyre inflation pressure used was 800kPa. The test was conducted with the two vehicles trafficking parallel independent wheel paths so that the relative wear generated by the two could be compared. The objective was to answer three of the questions that remained from the testing over the last two years that was funded by Transfund New Zealand (New Zealand's road funding agency).

The first question was how would low quality materials behave? The testing to date had only considered premium quality materials. The results of the Mass Limits Stage 1 testing (de Pont et al, 2001) had indicated that even proven high quality materials have considerably different performance to that assumed in New Zealand's current design methodology, which includes the fourth power law for assessing the effect of different axle loads. The preliminary results also suggested that the behaviour is very material dependent with power relationships between 2 and 9 observed.

The testing was also to determine whether the simple fourth power law type model is actually capable of predicting distress caused by higher axle loads (120 kN). This is particularly important as New Zealand's bus industry has requested an increase in axle limits to 120 kN to match those used in Europe where the majority of bus chassis are sourced. This test will allow at least a simple direct understanding of the Road User Charges (RUC) level that should be applied to such a proposal as well as providing a better understanding of how we should convert traffic spectrums to Equivalent Standard Axles (ESA's) for design.

Finally, by testing varying thicknesses of pavements, an understanding would be obtained of the behaviour of under strength pavements as well as the strong pavements that had been tested to date. Without this additional testing it would not be possible to predict the likely network needs using the current deterioration models. The calibration of such models use historical data to predict future needs.

The information obtained from the tests is required to accurately:

- quantify the benefits of allowing increased axle loads by accurately assessing the costs,
- convert traffic spectrums to ESA's for pavement design, and
- calibrate international pavement deterioration computer models such as Deighton's Total Infrastructure Management System (dTIMS), which in turn uses Highway Design and Management 4 (HDM 4) to simulate conditions and scenarios.

CAPTIF

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

At the ends of this frame, two radial arms connect to the vehicle units shown in Figure 2 below. These arms are hinged in the vertical plane so that the vehicles can be removed from the track during pavement construction, profile measurement, etc., and in the horizontal plane to allow vehicle bounce. A more detailed description of the CAPTIF and its systems is given by Pidwerbesky (1995).

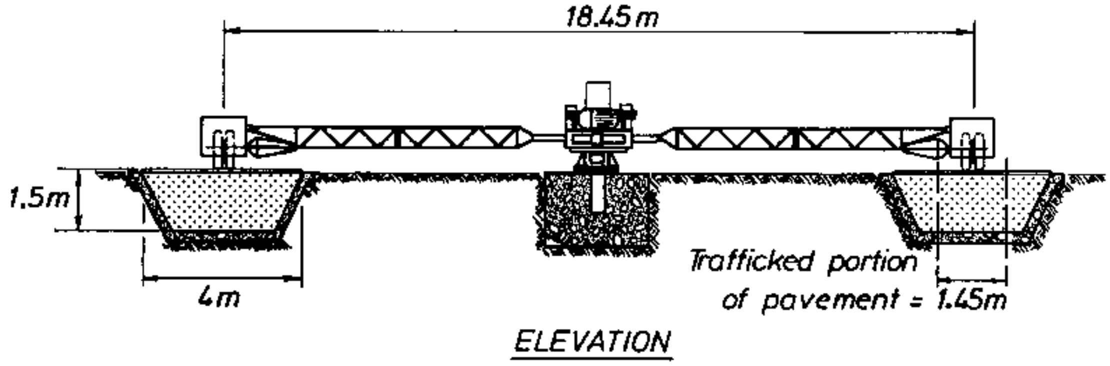

Figure 1. Elevation view of CAPTIF.

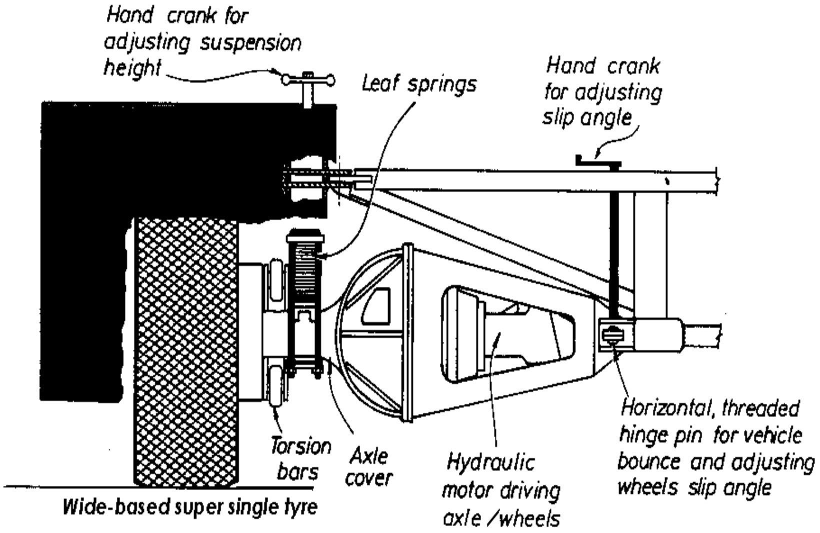

Figure 2. The CAPTIF vehicle unit.

PAVEMENT CONSTRUCTION

The pavement was constructed at CAPTIF in five segments. The five test pavements constructed comprised of three different basecourse materials (2 x premium and 1 x local crushed aggregates), all three basecourse materials were placed in thick highway strength pavements (275mm thick basecourse) and the two higher quality materials were placed in thinner pavements (225mm thick basecourse). The two high quality materials were a 20 mm maximum sized Australian rhyolite/rhyodacite, acid igneous crushed rock, and a 40 mm maximum sized New Zealand crushed alluvial greywacke gravel. The lower quality material was a 40 mm maximum sized New Zealand uncrushed alluvial greywacke gravel.

A plan showing the layout of the different segments is shown in Figure 3 below. An elevation showing the cross section of the pavement design, location of the two wheel paths and the in-situ instrumentation is shown in Figure 4 below.

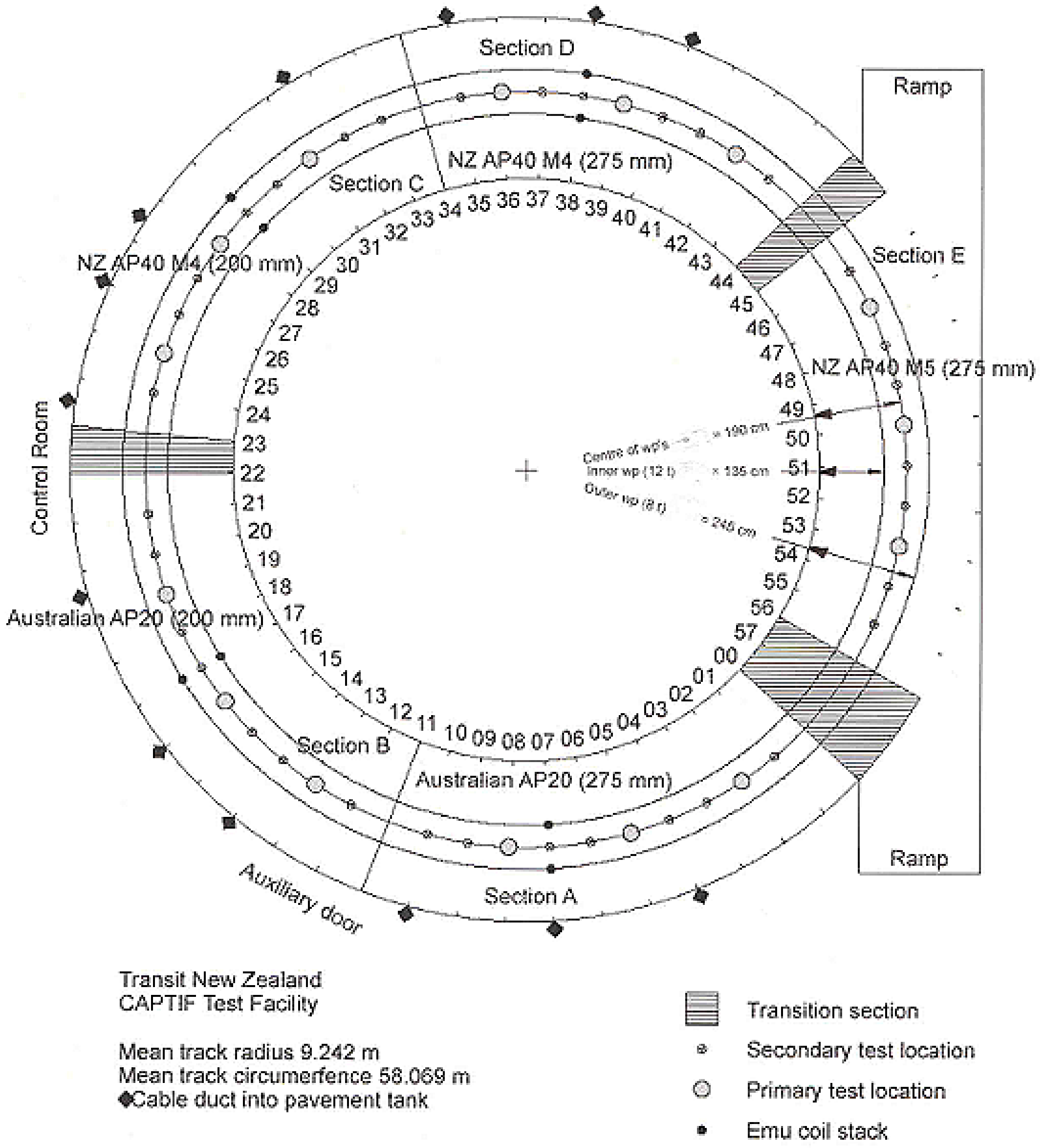

Figure 3. Plan showing the layout of the test segments.

Table 1. Details of pavement construction.

Segment	Pavement basecourse details
A	275 mm depth of 20 mm max sized Australian rhyolite/rhyodacite acid igneous crushed rock
B	200 mm depth of 20 mm max sized Australian rhyolite/rhyodacite acid igneous crushed rock
C	200 mm depth of 40 mm max sized New Zealand crushed alluvial greywacke gravel
D	275 mm depth of 40 mm max sized New Zealand crushed alluvial greywacke gravel
E	275 mm depth of 40 mm max sized New Zealand uncrushed alluvial greywacke gravel

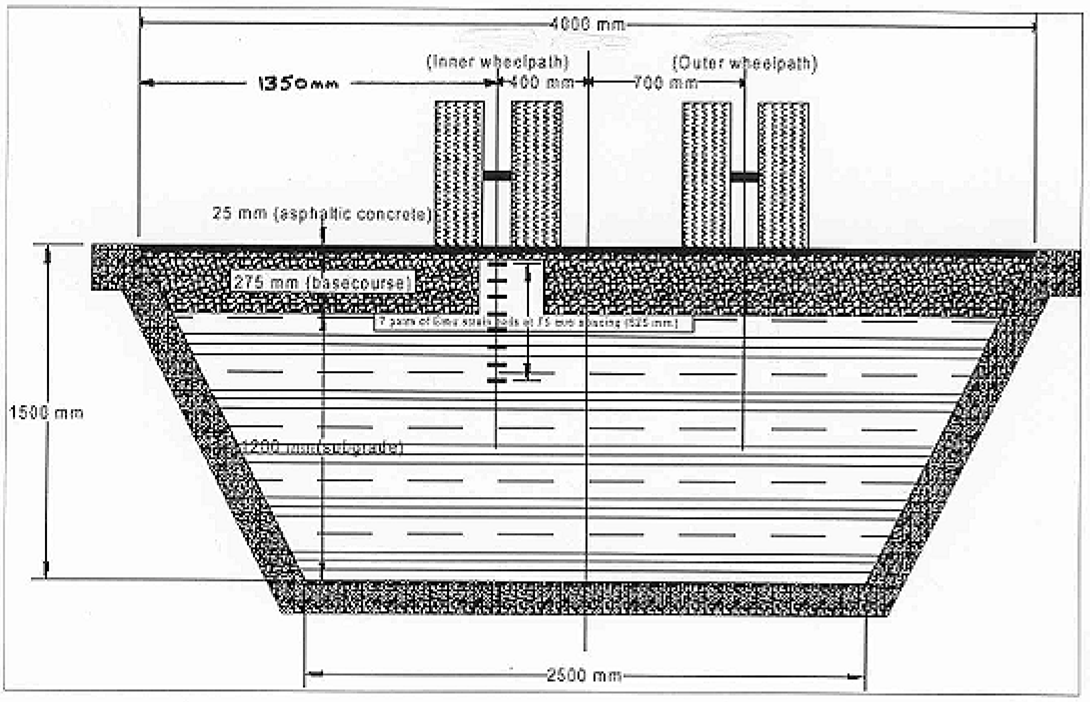

Figure 4. Pavement cross-section and wheel path locations.

PAVEMENT TEST

Testing consisted of applying loading with stops at regular intervals to collect sets of measurements. At the start of the test the number of load cycles between measurement sets is relatively small (10,000 cycles) but, as the test progresses and the rate of change of the pavement stabilises, this is increased until in mid-test 100,000 load cycles are applied between measurements. Testing proceeded until 1,000,000 load cycles had been applied. Although the design guide indicated that the as-constructed pavement should have reached the failure criteria at this point, this was not the case with average rut depths around 10mm compared to a failure criterion of 25mm. As the rate of increase of rutting was steady and slow, it did not appear that the pavement would achieve the failure criterion without applying a large number of additional cycles and thus the test was terminated.

ANALYSIS OF RESULTS

For the analysis, vertical surface deformation (VSD) was used as the main measure of pavement wear. The VSD is the difference between the pavement profile at a point in time and the as built pavement profile. VSD is directly related to rutting and the variability in VSD leads to increased roughness. As VSD in previous CAPTIF tests has been shown to be correlated to dynamic loading and to the variability in pavement structure, increased VSD results directly in increased roughness. Both roughness and rutting are key measures used by road controlling authorities to determine the need for pavement maintenance. By comparing the rate of progression of VSD under the two loading regimes, the impact of mass increases on pavement wear can be determined and the impact on road maintenance costs can be estimated. It has further been suggested (de Pont et al, 2002) that VSD may consist of two components, namely an initial compaction stage followed by an approximately linear wear related stage.

The conventional approach to comparing the wear generated by two different axle loads is the power law method. This states that the amount of pavement wear caused by one pass of an axle is proportional to some power of its axle load. The most widely used value for this power is four. Thus if a given level of wear is achieved by N_{inner} load cycles of a load P_{inner} or by N_{outer} load cycles of a load P_{outer} these are related as follows:

$$\frac{N_{outer}}{N_{inner}} = \left[\frac{P_{inner}}{P_{outer}}\right]^n \tag{1}$$

$$where \quad n \text{ is the exponent of the power law}$$

The analysis assumed that measurements of the VSD (an indication of road wear) could be used to determine possible values of the power law exponent n, a similar assumption to that made in the development of the fourth power law. Exponent values determined ranged from 2 to 4.5 for all the segments except Segment E (the low strength uncrushed gravel basecourse) where values as high as 6 were calculated during the test, but the rapid failure of both inner and outer paths lead to a final exponent of only 2.6. The scale used to plot results in Figure 5 was determined to ensure clear differences between Segments A, B, C and D.

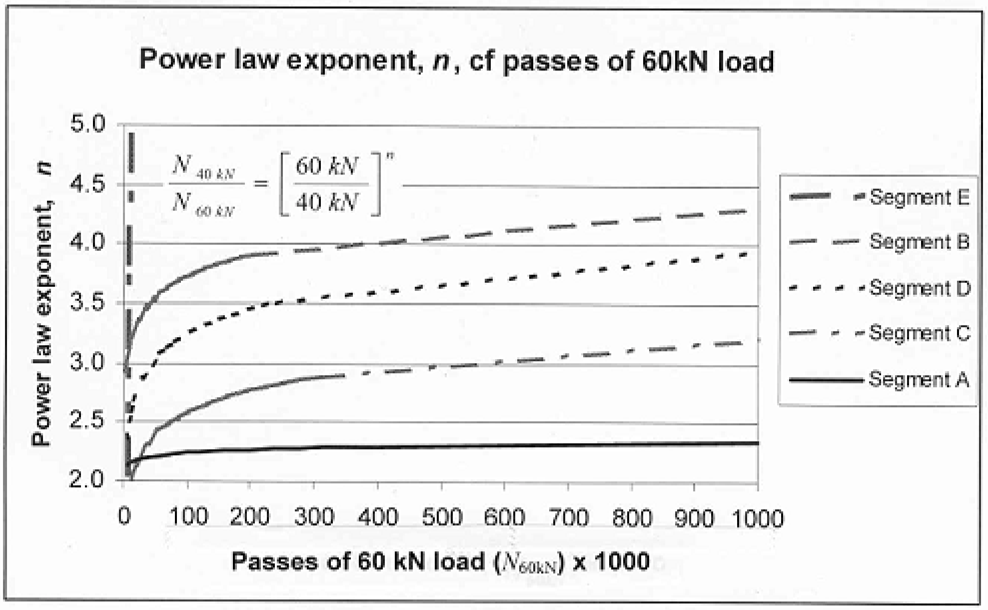

Figure 5. Power law exponent, n, determined for a range of number of passes of a 60 kN load.

The Stage 1 research found exponent values ranging from 3 to 9 (de Pont et al, 2001) using this method.

Equations by de Pont et al (2001), Wolf and Visser (1994) and Kinder and Lay (1988) that were of either an exponential or linear type function were also fitted to the measured VSD verses load application data. All the equations used to predict VSD fitted the data fairly well, probably the power law model (Kinder and Lay, 1988) gave the best fit but its use in extrapolating the results may not be appropriate as it predicts an ever decreasing rate of change in VSD with increasing load cycles.

Pidwerbesky (1996) also reported the Wolf and Visser (1994) model provided a better estimate of the permanent surface deformation in thin-surfaced unbound granular pavements. The model proposed by Wolff and Visser (1994) for deriving pavement surface deformation y is:

$$y = (a + mx)(1 - e^{-bx}) \tag{2}$$

where: a, m and b are constants and x is the number of wheel passes

Equation 2 has been modified to account for the pavement type and the load and takes the form:

$$VSD = k\left(\frac{P}{40kN}\right)^c (a + mx)\left(1 - e^{-bx}\right) \tag{3}$$

where:

VSD is vertical surface deformation (mm);
P is the half axle load in kN;
k is a constant particular to the pavement;
x is the number of wheel passes; and
c, a, m and b are constants.

As well as individual coefficients for each pavement, common coefficients for the model that provided the best fit to the Stage 3 and 1 data combined were determined. Table 2 reports the coefficients and Figure 6 compares the common coefficient model with the measured data for Stages 3 and 1.

Table 2. Common coefficients for the Wolff and Visser model.

	k	**a**	**m**	**b**	**c**	**Mean error (mm)**
Stage 3						
Segment A	1.000	1.87	0.00421	0.0691	1.31	0.64
Segment B	1.143	1.87	0.00421	0.0691	1.31	0.60
Segment C	1.097	1.87	0.00421	0.0691	1.31	0.46
Segment D	1.076	1.87	0.00421	0.0691	1.31	0.34
Stage 1						
Segment A	1.156	1.87	0.00421	0.0691	1.31	0.45
Segment B	1.056	1.87	0.00421	0.0691	1.31	0.51
Segment C	0.610	1.87	0.00421	0.0691	1.31	0.52
Segment D	0.622	1.87	0.00421	0.0691	1.31	0.38

The models that gave the best fit to the measured data are those with the lowest mean error. Mean error is the average absolute difference between calculated and measured value. As can be seen all models report a mean error less than 0.7 mm which would be considered sufficient accuracy when predicting rut depth in a pavement. In fact the Kinder-Lay, the de Pont and the Wolff and Visser models all fitted the measured data well.

The de Pont and Wolff and Visser equations were applied to an imaginary road network to predict the impact in terms of rehabilitation requirements should the axle load increase from 80kN to 120kN, these results were compared with a base power model which was simply a power model fitted to the data. Table 3 lists the assumptions made, and Table 4 shows the results of modelling.

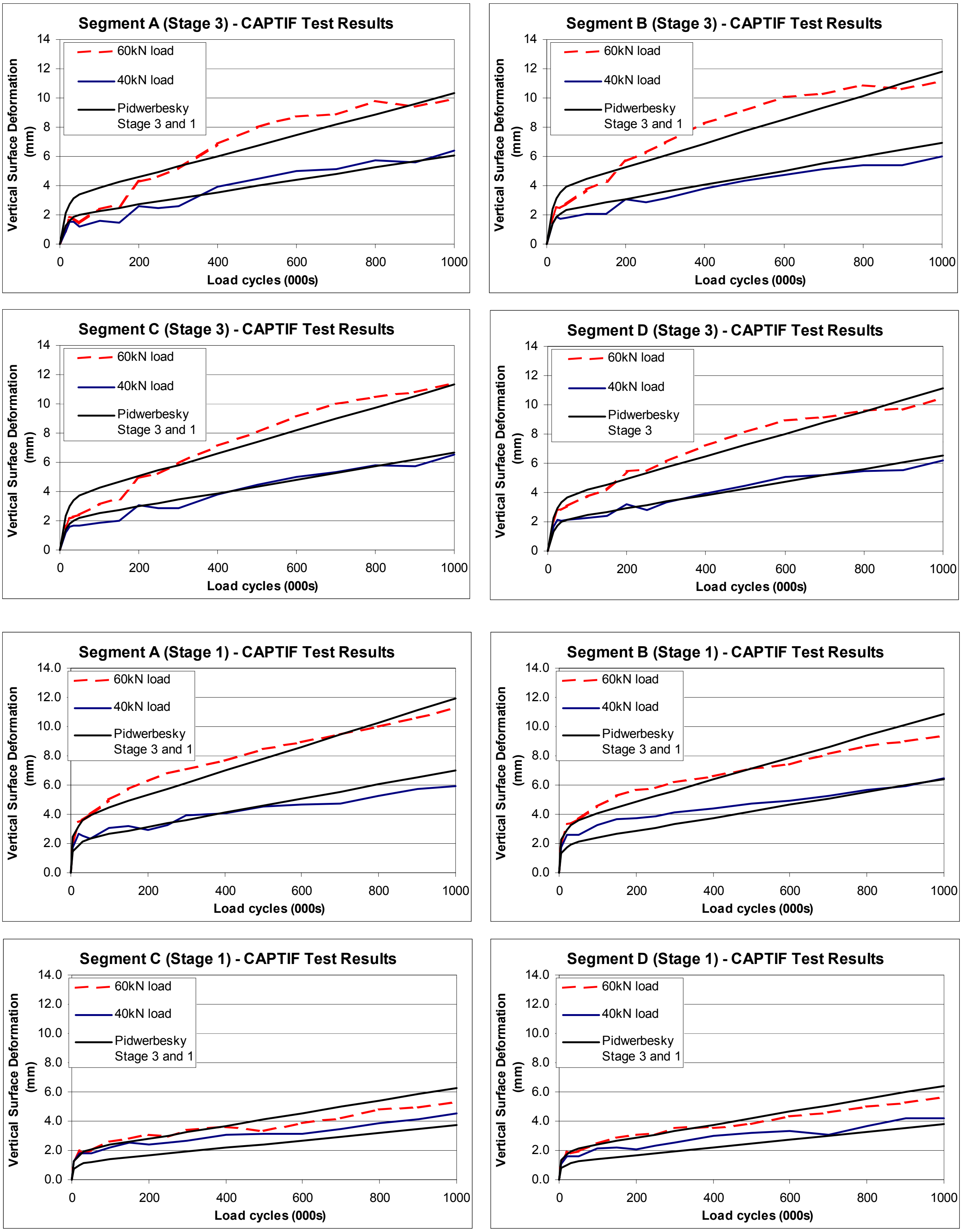

Figure 6. Common coefficient Wolff and Visser model compared to measured values.

Table 3. Assumptions used in imaginary road network for existing condition.

Item	Assumptions
No. of road sections	40
Pavement types	All 40 pavement sections are the same type as Segment C tested in this stage 3 research.
Age	Each section with a different age from 1 to 40 years.
Existing Traffic	100k passes of a 80kN axle per year.
Rehabilitation	1 section per year or pavement life = 40 years.
VSD when rehabilitation required	VSD value when rehabilitation required is the value determined for 4 million passes of a 80kN axle from either the best fit function to measured date or the VSD function being tested for Segment C.
Existing VSD	As predicted for each road section of ages 0 to 40 years with the VSD equation for 80 kN that is being tested.
New Traffic	100k passes of a 120kN axle per year.

Table 4. Average rehabilitation requirements and appropriate power law exponent predicted for an increase in axle load from 80kN to 120kN.

	Base Power model	**Best fit de Pont compaction-wear**	**Common coeff. de Pont compaction-wear**	**Best fit Wolff & Visser**	**Common coeff. Wolff & Visser**
80 kN loads					
Average rehabs per year	1	1	1	1	1
Increase to 120 kN loads					
No. rehabs in Year 1	8	5	7	6	10
Average Rehabs per year:					
Excl. year 1	3.50	1.92	1.97	1.95	1.95
Incl. year 1	3.6	2.0	2.1	2.1	2.0
Power exponent:					
Year 1:	5.1	4.0	4.8	4.4	5.7
Avg. excl. year 1:	3.1	1.6	1.7	1.7	1.7
Avg. incl. year 1:	3.2	1.7	1.8	1.9	1.7

FINDINGS FROM TESTS

The aim of this study was to compare the pavement wear generated by a 120kN axle load with that of a standard 80kN axle with a view to predicting the implications in terms of pavement damage of a change in the legal axle load limit on New Zealand highways. A pavement was tested at CAPTIF, which comprised five distinct segments; these consisted of a combination of basecourse aggregates and pavement depths. One of the vehicle units at CAPTIF was configured to generate a 40kN wheel load (equivalent to an 8.2 tonne axle) and the other was configured for a 60kN wheel load (equivalent to just over a 12 tonne axle load). The two vehicles were then used to apply 1,000,000 load cycles to parallel wheel paths on the pavement. During the testing, measurements were taken to record the pavement wear, the pavement condition, the pavement response to the vehicle loading, and the vehicle response to the pavement.

From the measurements recorded a number of important findings can be deduced as follows:

- VSD (vertical surface deformation), which is a fundamental form of pavement wear that results in both rutting and increased surface roughness, again proved to be the most useful measure for monitoring pavement wear at CAPTIF;
- The 60kN wheel load resulted in VSD values nearly twice those obtained with the 40kN wheel load in all the pavement segments;
- Segment E that used a low quality uncrushed gravel basecourse failed at 87,000 load cycles under the 60kN wheel load and at 250,000 load cycles under the 40kN load;
- A conventional power law relationship was fitted to describe the differences in VSD between the two levels of loading for each of the five pavement segments, it was found the exponent for the power law varied from 2 to 4 for Segments A, B, C and D;
- For Segment E an exponent of up to 6 was initially exhibited, but with the rapid onset of failure the exponent was reduced to a final value of 2.6;
- The value of the exponent depended on the pavement type and the value of VSD taken to be the end of pavement life;
- Reviewing the progression of VSD with load cycles shows that the pavement underwent two distinct phases of VSD. An initial period of rapid change was observed, here called compaction, followed by a period with a constant (linear) rate of change, called wear. Least squares regression can be used to fit a straight line to the linear part of the VSD versus load cycles curve. The intercept of this line with the y-axis then gives the compaction component, and the slope gives the wear;
- The compaction-wear linear relationship was modified to include a multiplier for the pavement type and the ratio of axle load to the reference load of 80kN. It was found that this relationship with common coefficients could be fitted to all the VSD data from this research and in stage 1 where a 100kN axle load was compared to the 80kN load, this relationship has the advantage of being able to predict VSD for other pavement types and axle loads;
- A power law model (Kinder & Lay, 1988) and a linear model with a blending function that models the initial progression of VSD in the compaction stage (Wolff and Visser, 1994) were also fitted to the VSD data using both best fit coefficients (where the coefficients were changed for each pavement segment) and common coefficients across the whole dataset; and
- All equations used to predict VSD fitted the data fairly well, probably the power law model (Kinder & Lay, 1988) gave the best fit but its use in extrapolating the results may not be appropriate as it predicts a ever decreasing rate of change in VSD with increasing load cycles.

APPLICATION OF FINDINGS

A selection of these equations were applied to an imaginary network to predict deterioration in terms of VSD and thus rehabilitation requirements each year should the traffic change from 100k passes per year of a 80kN axle to 100k passes per year of a 120kN axle. It was assumed that the imaginary network consisted of 40 sections and that with the 80kN axles only 1 section requires rehabilitation per year.

The results of the analysis were:

- All equations predicted in the first year from 5 to 10 sections needing rehabilitation and then after this the linear type equations (compaction-wear and Wolff and Visser) predicted on average around 2 sections per year needing rehabilitation, while the base power model predicted on average 3.6 sections per year requiring rehabilitation. Note that this knowledge must be tempered with the fact that the 120kN axle would have carried considerably more freight per axle pass;
- The power exponent that relates damage (number of rehabilitations per year) to the ratio of axle load to a reference load of 80kN raised to this power exponent ranged from 1.7 to 1.9 for the linear model and was found to be 3.2 for the base power model;
- The large number of pavement sections requiring rehabilitation in the first year after the introduction of the 120kN axle loads all required rehabilitation again when they reached the ends of their lives. Thus there will always be times when a greater than average amount of rehabilitation is required;

- The average number of rehabilitations required each year predicted by the base power model (3.6 per year) was significantly higher than the other models (2.0 per year). The base power model was the best match to the measured data. However, the large difference in results is partly due to how the VSD results are extrapolated beyond the measured data. The base model predicts an ever decreasing rate of change in VSD while the linear type models predicts a constant increase; and
- Segment E used low strength rounded aggregates that failed within 250k load cycles, for this segment exponent values as high as 6 were calculated as the test progressed but the rapid nature of the final failure reduced the exponent to 2.6 at the end of testing. This result illustrates that weaker sections in the road network which are adequate at present could fail quickly with the introduction of higher mass limits.

The result of this accelerated pavement test principally provides an indication of the performance of a relatively strong pavement, on a strong dry subgrade, in ideal dry environmental conditions. The behaviour of weaker or saturated subgrades has not been investigated, nor have the effects on older and/or poorly maintained surfaces where moisture may be entering the base. These are the more typical conditions in New Zealand.

RECOMMENDATIONS

- Further validation of the models proposed to predict VSD with load cycles based on pavement type and axle load is required;
- Pavement modelling using finite element software is to be used to predict deformation, validate findings, and extend results to pavements not tested at CAPTIF (Arnold et al, 2004);
- Equations that predict VSD with load cycles are based on measured data up to 1 million load cycles. The pavement had not reached the terminal functional condition. To be sure of the correct equation form (either a power law or linear relationship) a test that reaches terminal condition is required for both the reference wheel load of 40 kN and that of 60 kN;
- Analysis of the pavement types tested in terms of how the results affect current pavement design practices is required. For example, in this test some of the thinner pavement segments had a similar life to the thicker pavement segments;
- In light of the results of a rather simple deterioration study completed to date, the models should be used cautiously particularly when predicting the relative damage to the pavement caused by an increase in axle loads; and
- Some of the models developed that predict VSD have a multiplier depending on the pavement type. In order, for these models to be applied to other pavement types a relationship needs to be developed with a common pavement parameter like the structural number and/or measurements by Falling Weight Deflectometer (FWD, a non-destructive testing device that is used to complete structural testing for pavement rehabilitation projects).

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DISCLAIMER

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