Performance Tests for Road Aggregates and Alternative Materials

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Abbreviations

ARRB	Australian Road Research Board
CAPTIF	Canterbury Accelerated Testing Indoor Facility
CBR	California Bearing Ratio
ESA	Equivalent Standard Axles
FEM	Finite Element Modelling
LVDT	Linear Variable Differential Transformer
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
RLT	Repeated Load Triaxial

Contents

Executive summary

Surface rutting is of greatest concern for the thin-surfaced unbound granular pavements that are common in New Zealand. Hence, a key parameter that governs pavement longevity is the granular materials' resistance to rutting within the pavement. Thus, any alternative pavement materials used will be required to resist rutting from within (i.e. resistance to deformation). In existing TNZ specifications, more focus has been placed on limits determined by a range of empirical performance tests (such as grading, broken faces, crushing resistance, amount of fines, etc.) rather than a direct measure of deformation resistance. It is unlikely that waste materials or mixtures of waste and aggregate will meet the requirements of the specification and thus their use is disallowed. However, these alternative pavement materials may, in fact, be resistant to deformation and perform adequately in the pavement. An opportunity to minimise waste in landfill and reduce the consumption of raw materials may be missed.

The repeated load triaxial (RLT) apparatus (a device that applies repetitive loading to simulate vehicle loading) was investigated as a suitable test for use in basecourse specifications. The aim was to develop a test that could determine a traffic loading limit for an aggregate used in the base/top layer of the pavement. Ten different aggregates were tested using three different testing methodologies and analysis methods. The first RLT test method trialled required 4 stress stages with 50 000 load cycles applied for each stage. This method found that the testing stresses applied were too high and many early failures resulted; thus the results could not be analysed to rank the materials. The second method used an AUSTROADS method which applied three test stress stages at 10 000 load cycles per stage. However, the AUSTROADS method showed a poor fit to the estimated traffic loading limit based on anecdotal evidence of the performance of aggregates used in actual roads (R^2 of 0.1 and a mean error of 3 million ESA).

During this research project, undertaken in 2005/2006, a new six-stage RLT test was developed with the aim of introducing lower testing stresses so that most materials would survive at least four of the six stages of the test. RLT tests from subsequent research projects and commercial tests were analysed to predict rutting within a pavement profile tested at Transit New Zealand's accelerated pavement testing facility, CAPTIF. The results showed a good ranking with 13 out of 14 analyses predicting the same performance (either poor, average or good) as expected, based on actual performance at CAPTIF and in the road from anecdotal evidence. It was found that a relationship between average slope in the RLT test from 25 000 to 50 000 of all six stages could be related to a traffic loading limit, which was recommended for use in specifications.

Abstract

Aggregates used as base materials in thin-surfaced granular pavements common to New Zealand contribute at least half the wheeltrack rutting and roughness seen at the surface. Currently, no reliable cost-effective measure of an aggregate's resistance to rutting in specifications exists. Several test methods using the repeated load triaxial (RLT) apparatus were investigated for use in specifications for basecourse aggregates. Rut depth prediction methods and pavement finite modelling were applied to the RLT results to determine traffic loading limits for the aggregates tested. It was found that the average slope from the six-stage RLT test was the best predictor of traffic loading limit and this test was recommended for use in basecourse specifications.

1 Introduction

Road controlling authorities are progressively moving towards minimal use of traditional aggregates and are using alternative aggregates (including marginal and recycled materials) instead. Because of this, the need for improved, more accurate, cost-effective methods to predict the performance of alternative pavement materials is increasing. A drawback to using alternative and waste materials is their performance in the road is unknown and difficult to assess. This is of particular concern when considering the use of alternative and waste materials in New Zealand pavements, as they typically consist of a thin surface overlying unbound granular materials.

Surface rutting is of greatest concern for the thin-surfaced unbound granular pavements that are common in New Zealand. Hence, a key parameter that governs pavement longevity is the granular materials' resistance to rutting within the pavement layer. Any alternative pavement materials used will be required to resist rutting from within (i.e. resistance to deformation). In existing TNZ specifications, more focus has been placed on limits determined by a range of empirical performance tests (such as grading, broken faces, crushing resistance, amount of fines etc) than on a direct measure of deformation resistance. It is therefore unlikely that waste materials or mixtures of waste and aggregate will meet the requirements of the specification and thus their use is disallowed. However, these alternative pavement materials may, in fact, be resistant to deformation and perform adequately in the pavement. Thus an opportunity to minimise waste in landfill and reduce the consumption of raw materials is missed.

Over recent years, a disproportionate number of new pavements have failed with unacceptable rutting within the first few years of construction. A recurring theme of these failures is that all the rutting occurs within the top 200 mm of aggregate. Traffic volumes, meanwhile, are higher than ever before (in the past ten years, the volume of heavy commercial vehicles has doubled). Research at Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) has also shown up to 70% of rutting can be determined to occur in the top unbound granular layers (Arnold et al. 2001). Nearly half the rutting was attributed to the sub-base layer in the American Association of State Highway Officials road test (Benkelman 1962). Results at CAPTIF also showed that increasing the granular pavement depth did not increase the pavement life obtained (i.e. number of wheel passes required to form a 20 mm rut). However, it was shown that the type of granular material used did have an affect on pavement life. Rounded aggregates had the lowest life, while a lightly cemented recycled crushed concrete aggregate had twice the life of premium virgin aggregates.

The current specification for basecourse aggregate (TNZ M/4 (Transit 2006)), owing to its empirical/recipe approach to selecting aggregates, cannot distinguish differences in performance between aggregate types. Furthermore, it is expected that a modified aggregate with small quantities of cement or lime will provide superior performance in terms of rut resistance in wet conditions to that of traditional TNZ M/4 compliant aggregates where premature failures have occurred in the past. Anecdotal evidence of this has been found in Northland, where a modified local GAP 65 aggregate, which did not comply with TNZ M/4, was found to solve the rutting problems that were occurring with traditional M/4 aggregates. The Transfield PSMC01 contract on State Highway 3 also came to the same conclusion. Current methods of design do not recognise the superior rut resistance of a local modified material which is not affected by moisture. Currently, project specific specifications are required to use local modified aggregate. Thus, local modified aggregates that could solve the rutting problems of traditional M/4 aggregates are generally not used.

The outputs of this research project are a repeated load triaxial (RLT) test procedure and associated analysis to predict the magnitude of rutting and allowable design loading. Thiswill enable us to:

- reduce the number of early rutting failures in new pavements through the selection of aggregates based on performance testing using the RLT apparatus and associated analysis;
- develop a performance test using the RLT apparatus and associated analysis to enable a traffic loading limit to be determined for use in aggregate specifications;
- enable the use of aggregates modified with cement through calculation of their expected performance (traffic loading limit) from RLT testing; and
- enable the use of waste materials and previously discarded aggregates in appropriate traffic and environmental conditions determined from performance testing using the RLT apparatus.

2 Background

ARRB Transport Research Ltd, in collaboration with Transit New Zealand, conducted the 2004/2005 Transfund Project 930 'Predicting In-Service Performance of Alternative Pavement Materials' to develop a practical method using laboratory RLT testing for predicting the performance of alternative unbound pavement materials, including recycled materials (Vuong & Arnold 2006). Research comparing RLT predictions against rut depth measured in accelerated pavement tests (e.g. CAPTIF) has shown that this method can correctly predict the performance of unbound pavement materials. Prior to introducing a new RLT test into material specifications, field validation was required by conducting RLT tests on a range of New Zealand aggregates of known performance (anecdotal or otherwise). Hence, this research project was initiated with the following objectives:

- to trial, validate and refine the practical RLT test developed in Vuong & Arnold 2006 on materials currently used on New Zealand roads with known performance;
- to trial and validate the practical RLT test method into Transit's policy (TNZ M/22) as a means of categorising materials in terms of low, medium and high traffic, and either wet or dry conditions;
- to evaluate the RLT test method to quantify the benefits of modifying/stabilising an aggregate in terms of increasing number of wheel loads to reach a certain rut depth; and
- to implement a test procedure that allows alternative materials (which includes aggregates, marginal materials, stabilised materials, those from recycled sources etc) to be used in the pavement with appropriate limits to the level of traffic and moisture condition.

3 Repeated load triaxial testing

3.1 What is RLT testing?

The RLT apparatus applies repetitive loading on cylindrical materials for a range of specified stress conditions; the output is deformation (shortening of the cylindrical sample) versus number of load cycles (usually 50 000) for a particular set of stress conditions. Multi-stage permanent strain RLT tests are used to obtain deformation curves for a range of stress conditions to develop models for predicting rutting. Figures 3.1 and 3.2 detail the RLT setup and typical output from a multi-stage permanent strain RLT test.

Resilient modulus information can also be obtained for pavement design in CIRCLY and Finite Element Models (FEMs).

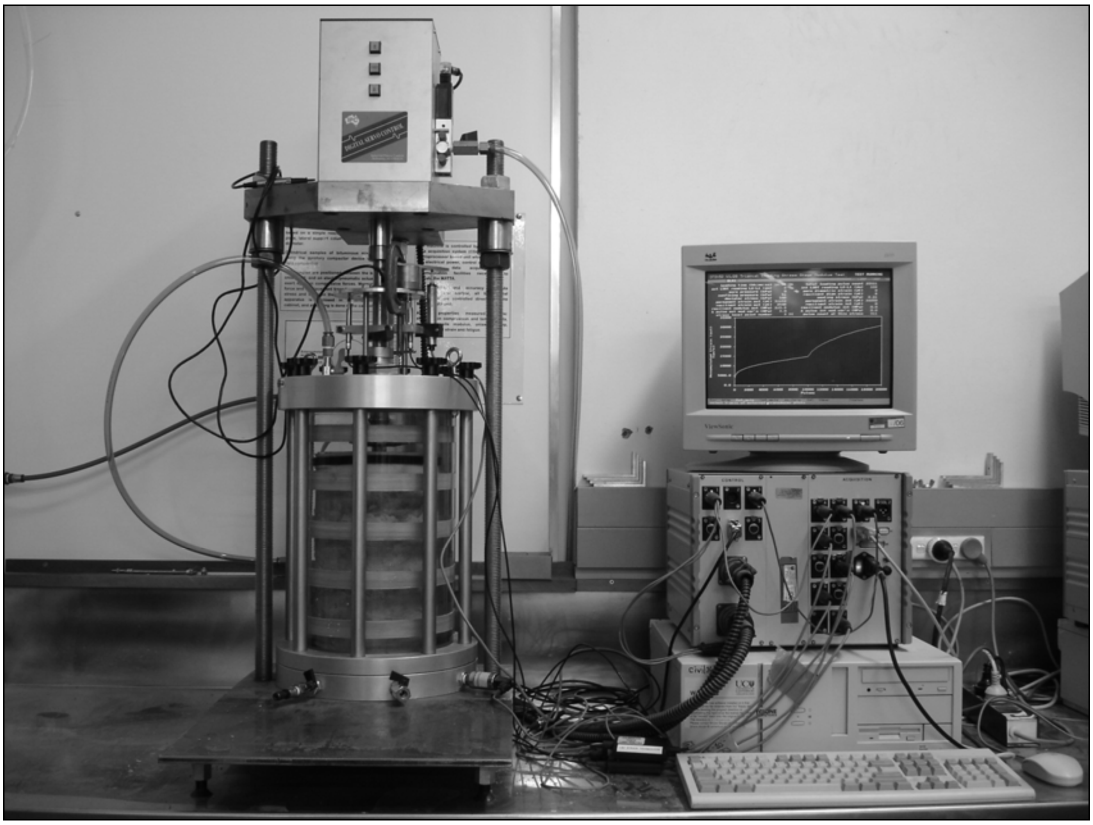

Figure 3.1 RLT apparatus and setup at the CAPTIF facility.

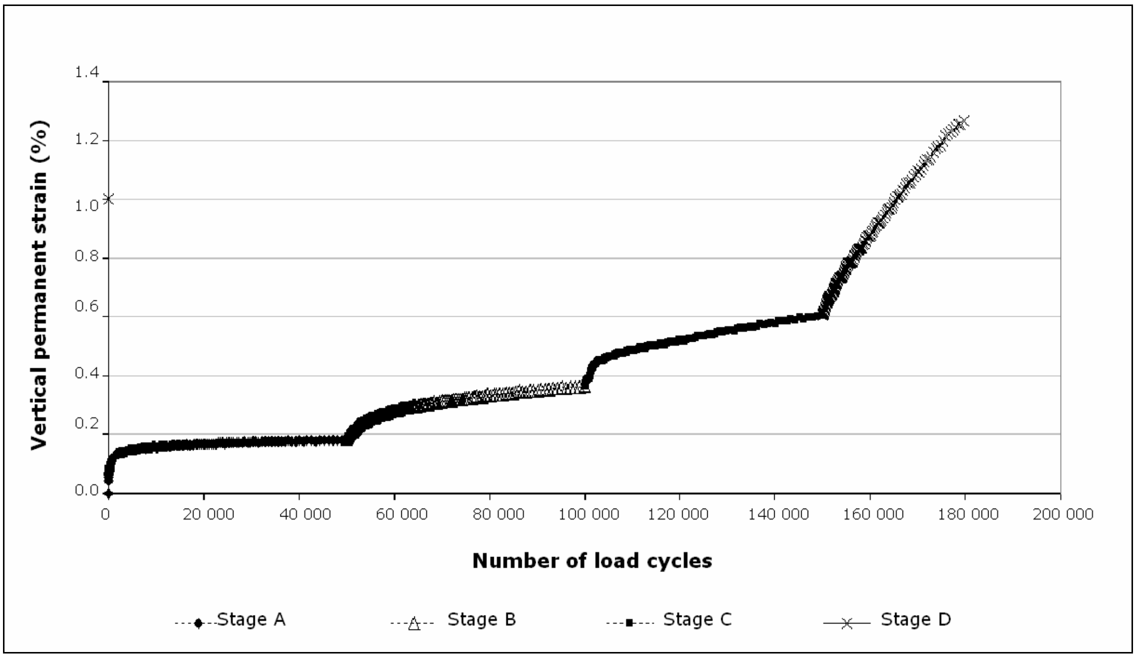

Figure 3.2 Typical output from a permanent strain RLT test.

3.2 Simplified Arnold / Nottingham test procedure and analysis

A simplified version of the method developed by Arnold (2004) is one method used in this study to interpret the results and to predict the rut depth in a pavement. It is referred to as the simplified Arnold method because only one RLT test is needed, as opposed to three RLT tests, as in the original Arnold method.

The first step is to develop a mathematical relationship between stress (both vertical and horizontal) and permanent strain rate (slope of each deformation curves (Figure 3.2 and 3.3), e.g. % deformation per million load cycles).

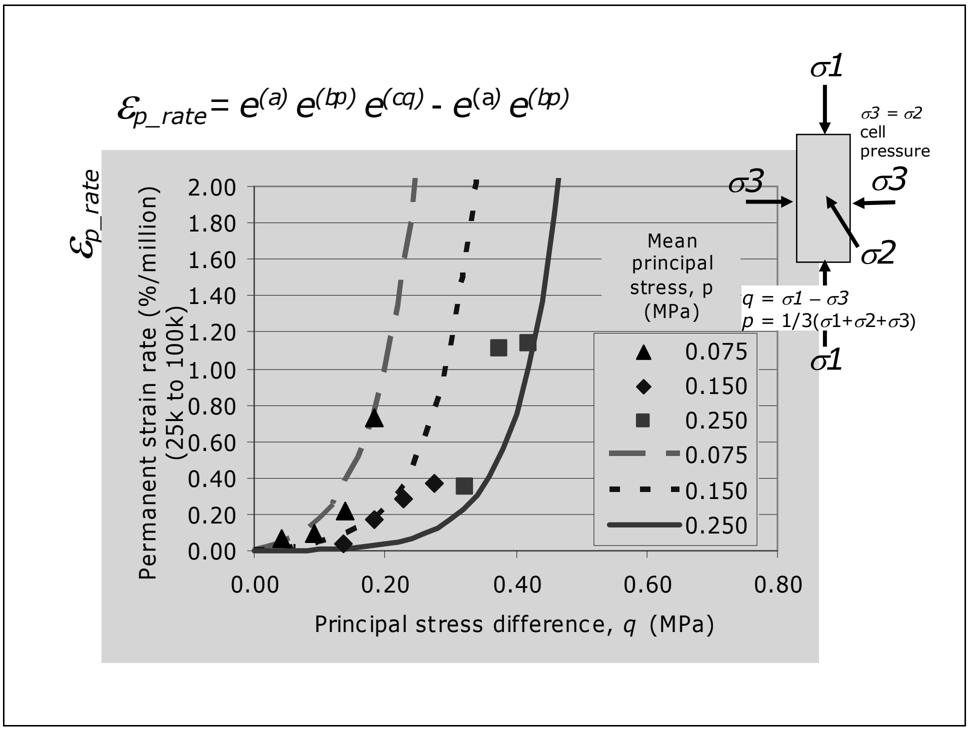

Figure 3.3 Fitting the permanent strain rate mathematical relationship to the RLT test results (shown in Figure 3.2).

Vuong & Arnold 2006 found that one RLT test at four different stress stages was required to obtain enough points for the mathematical relationship shown in Figure 3.3. This simplified Arnold RLT test procedure is detailed in Table 3.1 and was the RLT test method initially used for testing aggregates for this study. Recently, the number of stress stages tested was increased to six to include stresses at lower levels so that data from weaker aggregates could be obtained. A seventh stress stage was trialled but later discarded as it was a low stress level, causing virtually no deformation; it did, however, improve the model predictions. It was found that using the four-stage test (Table 3.1), the weaker aggregates (usually those that were saturated) would fail after the first or second stage in the initial testing regime. The new stress levels used are detailed in Table 3.2.

Table 3.1 RLT testing stresses – (RLT Test Method 1).

RLT testing stress stage	A	B	C	D
Deviator stress - q (kPa) (cyclic vertical stress)	180.0	270.0	330.0	420.0
Mean stress - p (kPa)	150.0	150.0	250.0	250.0
Cell pressure, σ3 (kPa)	90.0	60.0	140.0	110.0
Major principal vertical stress, σ1 (kPa)	270.0	330.0	470.0	530.0
Cyclic vertical loading speed	Haversine at 4 Hz			
Number of loads (N)	50 000			
Data recorded and reported electronically in Microsoft Excel	Permanent strain versus load cycles and resilient modulus versus load cycles			

p = mean principal stress (1/3*(σ1 + 2*σ3)
q = principal stress difference (σ1 - σ3)

Table 3.2 RLT testing stresses for six-stage test – (RLT Test Method 2).

RLT testing stress stage	A	B	C	D	E	F
Deviator stress - q (kPa) (cyclic vertical stress)	90.0	100.0	100.0	180.0	330.0	420.0
Mean stress - p (kPa)	150.0	100.0	75.0	150.0	250.0	250.0
Cell pressure, σ_3 (kPa)	120.0	66.7	41.7	90.0	140.0	110.0
Major principal vertical stress, σ_1 (kPa)	210.0	166.7	141.7	270.0	470.0	530.0
Cyclic vertical loading speed	Haversine at 4 Hz					
Number of loads (*N*)	50 000					
Data recorded and reported electronically in Microsoft Excel	Permanent strain versus load cycles and resilient modulus versus load cycles					

3.3 Rut depth prediction with the Arnold/Nottingham method

3.3.1 Methodology

The method used to predict rutting is reported in Arnold (2004). The following is a summary of the steps involved in order to predict the rutting of a 300 mm deep pavement at CAPTIF.

3.3.2 Step 1: extrapolation and conversion to individual results

The first step in rut depth prediction is to extrapolate the RLT results and individualise the RLT results to one test per stress stage. A power law model ($y=k1x^{k2}$) was used to extrapolate the results to 500 000 load cycles from the 50 000 load cycles. From 500 000 load cycles onwards, a linear extrapolation following the same deformation rate that was seen from 100 000 to 500 000 were used. The linear extrapolation is considered a conservative approach and follows the same trend typically found in CAPTIF tests. Another assumption used to extrapolate the results relates to adding on an incremental permanent strain value to each new stress stage, being the permanent strain value at 10 000 load cycles for the previous load cycle. Figure 3.4 illustrates a typical extrapolation method used.

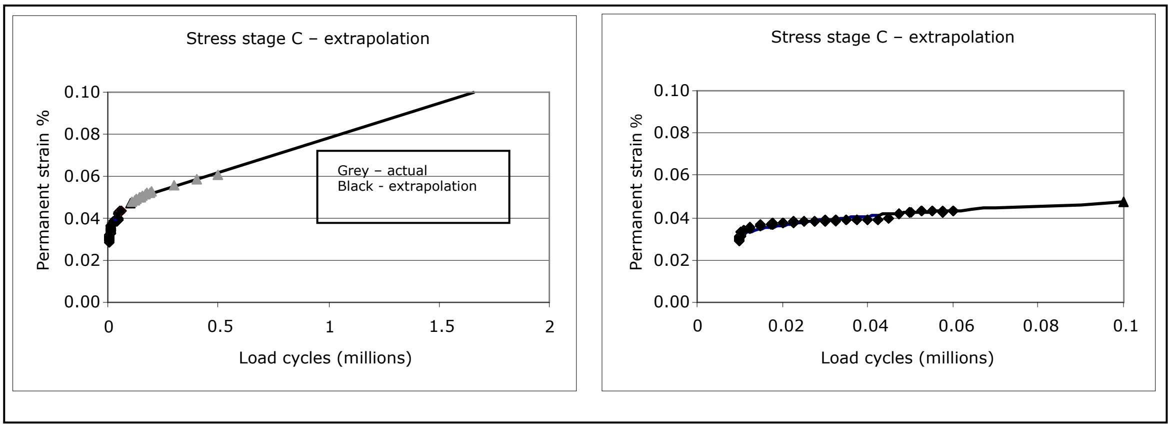

Figure 3.4 Example of the extrapolation method used with the Arnold/Nottingham method.

Assuming an additional 10 000 load cycles to achieve the initial deformation at the start of each stage was based on a simplistic approach to the Australian Road Research Board (ARRB) method (Vuong & Arnold 2006). Vuong & Arnold determined the initial load cycles based on an iteration approach to determine the value that gave the best fit using the mathematical model to extrapolate the deformations. Trialling Vuong and Arnold's approach required many iterations and was difficult to apply and hence, an assumed 10 000 load cycles were applied for the Arnold method. Furthermore, the 10 000 load cycles were based on the point where the deformation curve begins to flatten and stabilise, as observed in the RLT test (see Figure 3.5).

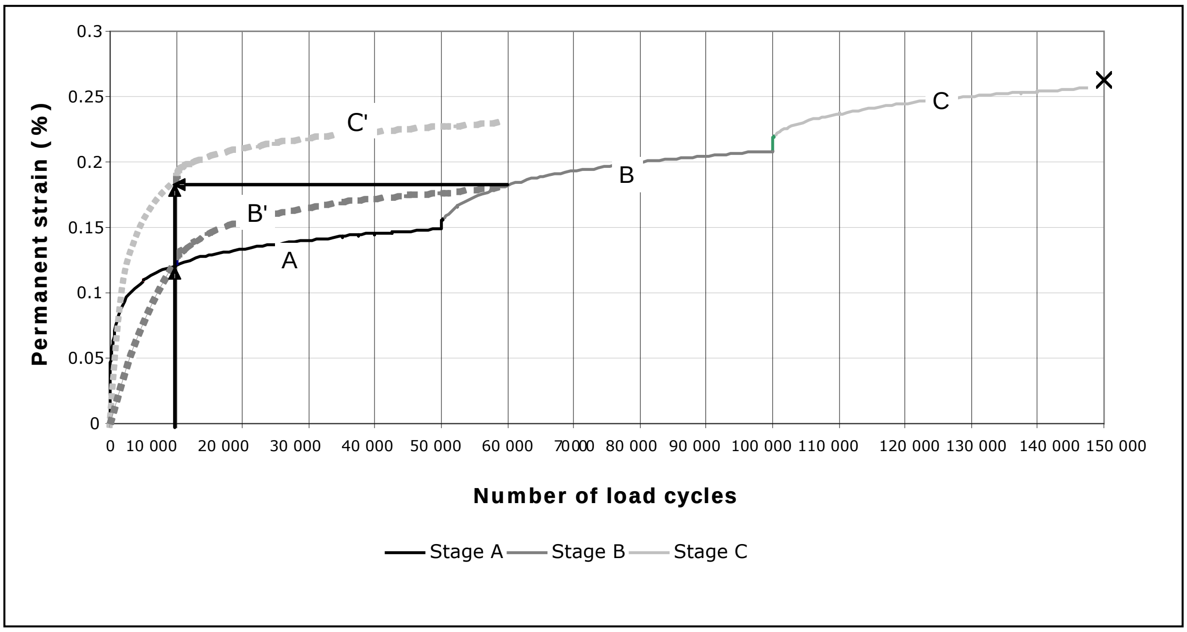

Figure 3.5 Transforming a multi-stage test result into a single stage.

Gidel et al. (2001) assumed that each new loading stage starts at nil deformation as if it were a single stage test. The only argument against this approach is the amount of initial deformation calculated is under-estimated, as found by Arnold (2004). Furthermore, Gidel et al. have not validated their approach with accelerated pavement test data.

Finally, as the initial deformations in a multi-stage test are likely to be more prone to error because of the influence of sample preparation/compaction, the Arnold method separates the initial deformation after 25 000 load cycles from the long-term rate of deformation in the calculation of rutting. It was found by doing this that the key criteria in classifying the performance of an aggregate was the long-term rate of deformation; the initial deformation showed a large scatter in the modelling and it could be ignored without affecting the overall ranking of an aggregate's performance (Arnold 2004). Also the initial amount of rutting is insignificant for design traffic loadings greater than 5 million Equivalent Standard Axles (ESA) as shown in Figure 3.6.

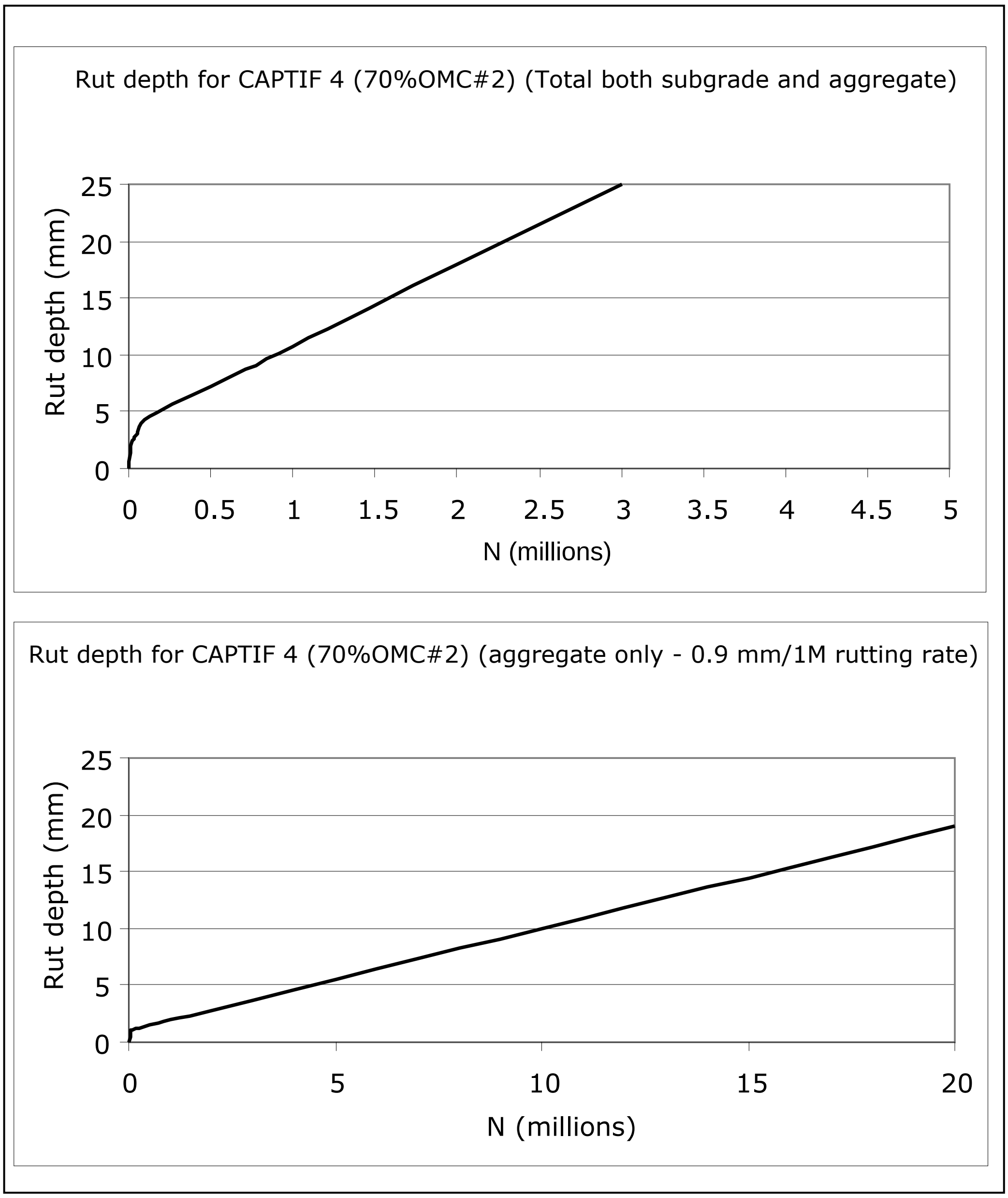

Figure 3.6 Rut depth predicted for the CAPTIF 4 (70%OMC#2) aggregate.

3.3.3 Step 2: permanent strain rates and associates stress

From the extrapolated RLT results, permanent strain rates and associated stresses are determined (Table 3.3 is an example of results from one of the aggregates tested, CAPTIF 4 70%OMC #2).

Table 3.3 Permanent strain rates and associated stresses for the RLT Test Method 2.

Stress (MPa)		Magnitude %	Slope % / million load cycles		
p	*q*	First 25k	25k to 50k	50k to 100k	100k to 500k
0.150	0.180	0.063	0.337	0.169	0.049
0.150	0.270	0.117	0.486	0.269	0.093
0.250	0.330	0.131	0.231	0.123	0.039
0.250	0.420	0.162	0.747	0.559	0.326

p = mean principal stress ($1/3*(\sigma_1 + 2*\sigma_3)$)
q = principal stress difference ($\sigma_1 - \sigma_3$)

3.3.4 Step 3: equation parameters to predict permanent strain rate from stress

Equation 1 taken from Arnold (2004) is used to determine the permanent strain rate for any stress not tested. Parameters *a*, *b* and *c* are determined by using the solver in Microsoft Excel to the actual measured and extrapolated values in the RLT test listed in Table 3.3.

$$\varepsilon_{p(rate\ or\ magn)} = e^{(a)}\, e^{(bp)}\, e^{(cq)} - e^{(a)}\, e^{(bp)}$$

$$= e^{(a)}\, e^{(bp)}\, (e^{(cq)} - 1) \qquad \text{Equation 1}$$

Where:

e = 2.718282;

$\varepsilon_{p(rate\ or\ magn)}$ = secant permanent strain rate or just permanent strain magnitude;

a, b & c = constants obtained by regression analysis fitted to the measured RLT data;

p = mean principal stress (MPa); and

q = mean principal stress difference (MPa).

An example of the using Equation 1 to fit the measured data is detailed in Figure 3.7.

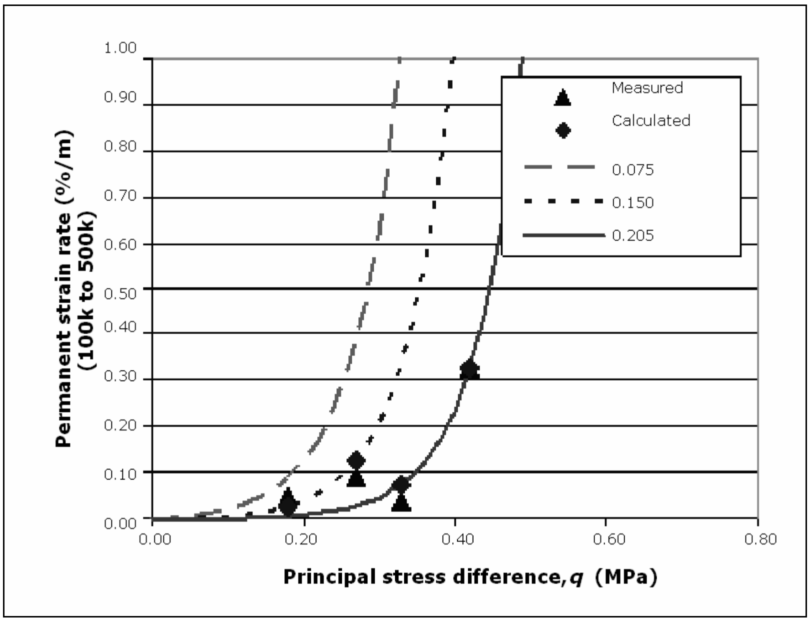

Figure 3.7 Example of fitting permanent strain rate equation to the RLT data in Table 3.3.

Parameters *a*, *b* and *c* for Equation 1 for one of the aggregates tested are listed in Table 3.4.

Table 3.4 Parameters a, b and c (Equation 1) for one of the aggregates tested (CAPTIF 4 70%OMC #2).

ε_{rate}	Model parameters (Arnold, 2004) $\varepsilon_{rate} = e(a)\, e(bp)\, e(cq) - e(a)\, e(bp)$			Mean error
	a	***b***	***c***	ε_{rate} (% / million)
ε_{mgn} (25k)	-3.195	-15.361	12.552	0.026
ε_{rate} (25k-50k)	-1.864	-15.000	12.684	0.048
ε_{rate} (50k-100k)	-2.906	-15.000	14.310	0.030
ε_{rate} (100k-500k)	-4.214	-15.000	16.293	0.022

3.3.5 Step 4: Finite element modelling to calculate stress and deformation

An axisymmetric FEM, ROSTRA, was used to calculate stresses in a typical CAPTIF pavement under a standard 8 tonne axle (40 kN). The wheel load was simulated as a circular load of 550 kPa. The pavement depth used was 300 mm of aggregate over a silty clay subgrade (CBR[1] =10) as detailed in the pavement cross-section (Figure 3.8). The FEM was validated by showing a good match with actual measured strains and surface deflections within the CAPTIF pavement.

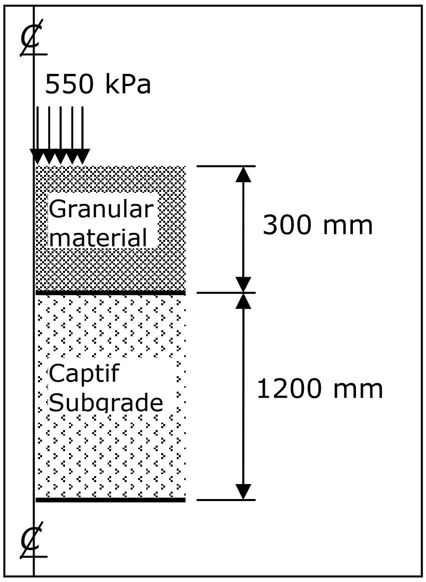

Figure 3.8 Pavement cross-section used in finite element modelling.

Elastic modulus relationships are required for input into the FEM. The relationship for the CAPTIF aggregate and the subgrade used at CAPTIF are shown in Figure 3.9. For other aggregates modelled, new elastic modulus relationships (in the form shown in Figure 3.9) are determined from the RLT test data.

[1] California Bearing Ratio

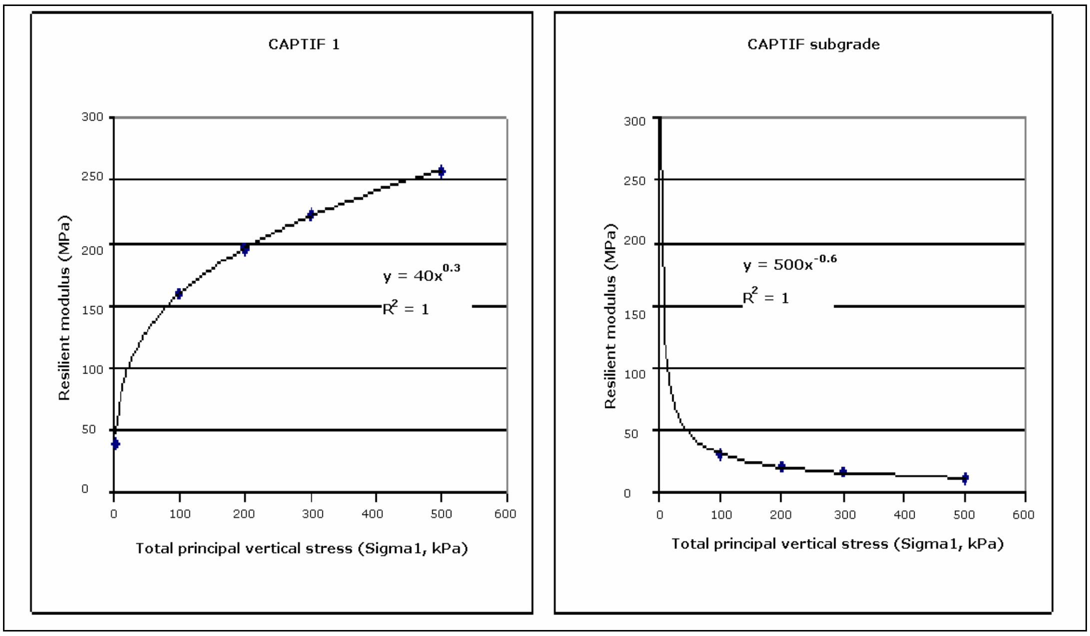

Figure 3.9 Elastic modulus relationships used in the FEM.

Running the FEM results in stresses under the wheel which are then imported into a spreadsheet to calculate the permanent strain from Equation 1 with parameters from Table 3.4. Table 3.5 details the results of the FEM and rut depth calculations for the CAPTIF aggregate 'CAPTIF 4, 70%OMC #2' and the subgrade. Figure 3.6 illustrates the result of the rut depth predictions.

Table 3.5 FEM and rut depth prediction for CAPTIF 4 aggregate 70%OMC#2 for CAPTIF cross-section.

Total pavement		Aggregate only	
Rut depth (mm) after 1 million wheel passes (ESA) (mm)	ESA to get 25 mm rut (Million ESA)	ESA to get 10 mm rut in aggregate. (Million ESA)	Long-term rate of rutting within aggregate (mm per million ESA)
Total: 7.2 Aggregate: 1.5 Subgrade: 5.7	2.99	10.00	0.9

3.3.6 Use of the analysis method

The rut depth prediction method developed by Arnold determines two key parameters for the assessment of an aggregate's performance, being the long-term rate of rutting and the number of heavy axle passes until rutting within the aggregate achieves 10 mm. These rutting performance values will be used to compare aggregates to rank their performance and for validating a more simplified analysis approach.

In the example shown in the first column of Table 3.5, the aggregate contributes to 21% of the total rut depth in the CAPTIF pavement. One of the reasons for this is because the aggregate tested in the RLT apparatus showed very low deformation and would rank as one the best quality unbound aggregates tested from New Zealand. Another reason is the difficulty in determining the initial amount of rutting/deformation from multi-staged tests, as this value needs to be assumed. Also, the RLT test will consistently give lower deformations than those which occur in a pavement because of the inability to rotate the principal stresses and cyclic confining stress caused by an arriving and passing wheel load, as detailed in Arnold's thesis (2004). Hence, because of the difference between the RLT test and real-life pavements, the analysis proposed is a tool to rank the performance of granular materials based on the long-term rate of rutting for use in material specifications rather as than an absolute predictor of pavement rut depth.

3.4 RLT testing methods used

The RLT testing methods adopted in this study are based on theapproaches developed by Transit New Zealand (TNZ M/22 (Transit 2008)), AUSTROADS (Vuong 2000, Vuong & Brimble 2000) and Nottingham University (Arnold 2004). These are summarised in Table 3.6.

Generally, the test methods have different requirements for key features, such as:

- RLT testing equipment (triaxial cell, measurement devices, software),
- sample preparation methods (e.g. dynamic and vibratory compaction), and
- testing procedures (load pulse, stress levels, number of loading cycles, drained or undrained).

In view of the great diversity of testing requirements for unbound granular materials, it is considered necessary to conduct inter-laboratory precision studies to assess the limitations of the testing method and standardise the testing requirements for practical use. Currently, only the AUSTROADS RLT testing method has been subjected to inter-laboratory precision studies (Vuong et al. 1998) for standardisation purpose, whereas other test methods have not.

Different test methods also produce different test results and require different assessment methods as discussed below.

Table 3.6 Summary of earlier RLT testing methods (from Vuong & Arnold 2006).

Features	TNZ M/22 (Appendix A)	AUSTROADS (Appendix B)	Nottingham University (Appendix C)
Material size	Maximum particle size in the range of 20–40 mm	Maximum particle size not exceeding 19 mm	Maximum particle size in the range of 20–40 mm
Sample size	150 mm diameter and 300 mm length	100 mm diameter and 200 mm length	150 mm diameter and 300 mm length
Sample preparation	Vibratory Hammer Compaction test in NZS 4402 (Standards New Zealand 1986)	Dynamic compaction methods	Vibrating compaction test method (BS 1377-4: 1990 (British Standards Institution 1990))
Target density	95% Vibratory MDD (Maximum Dry Density) (TNZ B/2:2005)	Field dry density as specified by AUSTROADS Members	Field dry density
Moisture condition	Fully saturated condition or optimum moisture content (OMC)	Field moisture content as specified by AUSTROADS Members	Field moisture content
1-DI RLT test apparatus (vertical loading pulse)	No specifications	Trapezoidal pulse with 0.2 second load and 1.8 second rest (pneumatic equipment)	Sinusoidal pulse at 5 times a second (5 Hz) (hydraulic equipment)
Triaxial cell and instrumentation	No specifications	Strict specifications of loading friction and loading piston-top cap connections when using external load cell and external displacement transducers	Using internal load cell and on-sample displacement transducers
Drainage condition	Undrained	Drained	Drained
Stress conditions for permanent strain testing	Single stage with a deviator stress of 425 kPa and a confining stress of 125 kPa	3 stages on one specimen with constant confining stress of 50 kPa and increasing deviator stresses being selected based on the vertical position of the material in the pavement (base, upper sub-base and lower sub-base)	21 stages using 3 specimens, viz. 7 stages per specimen with constant mean stresses and increasing shear stresses
Stress conditions for resilient modulus testing	As above	64 stress stages to cover stress levels at various positions in the pavement	As above
Number of specimens required	1 specimen per target density and moisture condition	1 specimen per target density and moisture condition	3 specimens per target density and moisture condition
Number of loading cycles	50 000 cycles of a specified stress level	10 000 cycles per stress stage	50 000 cycles per specified stress level
Stress and strain measurement methods	No specific measurement requirements for stress, strain and pore pressure	External load cell (for non-friction triaxial cell) and whole-sample strain measured with 2 LVDT*s mounted between loading caps	Internal load cell and on-sample strain measured at sample mid-half using studs embedded in the specimen at two opposite locations
Interpretation of test results	Trend of permanent strain rate with loading cycles	Individualisation of data for each stress stage by taking into account permanent strain developed in previous loading stages using a load equivalency rule	Individualisation of data for each stress stage by ignoring permanent strain developed in previous loading stages
Assessment criteria	Use decreasing permanent strain rate as pass-fail criterion	Simple assessment methods based on material behaviour, deformation life and design base deformation.	Compare predicted rut depth with design rut depth
Other assessment methods required	Minimum soaked CBR requirement of 80%	–	–

* Linear Variable Differential Transformer

3.5 ARRB RLT test method and analysis

3.5.1 The method

AUSTROADS/ARRB Material Assessment was developed at ARRB (Vuong 2000) in conjunction with the 2000 AUSTROADS simplified RLT test method (Vuong & Brimble 2000) so that the RLT permanent strain test results obtained with this test method can be used to predict in-service performance.

The method was developed from using a FEM computer program (VMOD-PAVE) to predict stresses in sprayed seal surfacing on granular pavements under an axle load of 40 kN on a single wheel. This enabled the representative stress levels (or design stresses) to be selected for the base, upper sub-base and lower sub-base. In addition, two other stress levels, one below and one above the design stress level, were also selected to determine the stress-dependent permanent strain characteristics for the assessment of the base deformation caused by underloading or overloading.

The AUSTROADS column in Table 3.6 details the ARRB/AUSTROADS permanent strain test while Table 3.7 lists the required values of vertical deviator stress (σ_d) and static confining stress (σ_3) for Stages 1, 2 and 3 for RLT testing of aggregates.

Table 3.7 Stress levels for ARRB / AUSTROADS permanent strain testing.

Permanent deformation stress levels						
Stress stage number	Base		Upper sub-base		Lower sub-base	
	σ_3 (kPa)	σ_d (kPa)	σ_3 (kPa)	σ_d (kPa)	σ_3 (kPa)	σ_d (kPa)
1	50	350	50	250	50	150
2	50	450	50	350	50	250
3	50	550	50	450	50	350

Figure 3.10 shows typical results obtained from permanent strain testing using the AUSTROADS standard RLT test method APRG 00/33 (Vuong & Brimble 2000) for a granular base material to be used at depth of 0–150 mm below the surface. Referring to Figure 3.10, three loading stages were applied to the single specimen compacted to specified density and moisture condition, each involving 10 000 cycles at a stress condition of specified dynamic deviator stress and static confining stress as given in Table 3.7.

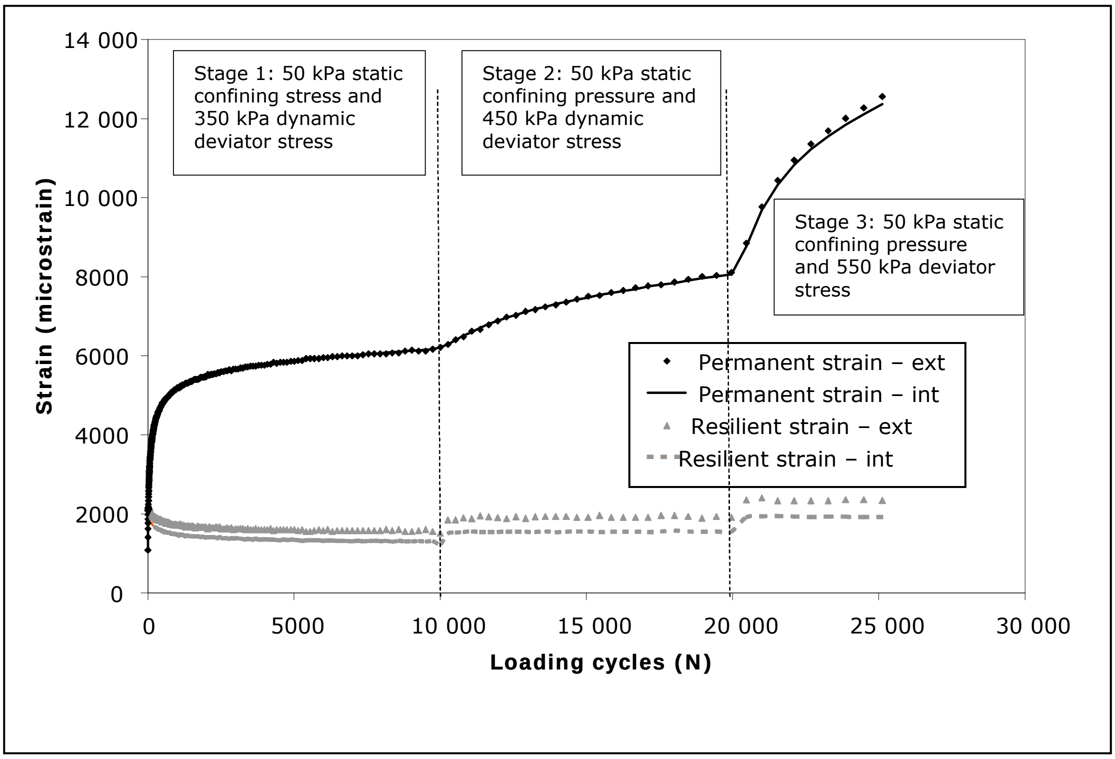

Figure 3.10 Typical results obtained from permanent deformation testing using the AUSTROADS/ARRB method.

3.5.2 ARRB/AUSTROADS simple categorisation analysis method.

The results of the three-stage permanent strain test as shown in Figure 3.10 can be used to derive the basic material behaviour, deformation life and design base deformation using three simple assessment methods (Vuong 2000) as briefly described below.

In principle, the material performance can be judged based on three basic material behaviour modes that can exhibit at a given loading stress as follows:

- Stable behaviour is defined as a decreasing permanent strain rate and a decreasing to constant resilient strain with increasing loading cycles.
- Unstable behaviour is defined as a decreasing to constant permanent strain rate and a constant to increasing resilient strain with increasing loading cycles.
- Failure behaviour is defined as a constant to increasing rate of permanent strain and an increasing resilient strain with increasing loading cycles, or when the total permanent strain reaches a nominal failure strain observed in the static triaxial shear test (say in the range of 15 000–20 000 microstrain).

Table 3.8 summarises the proposed behavioural requirements that a material should exhibit in a three-stage permanent strain test, and these tests can be used to select a base material for use in different pavement classes subjected to light, medium and heavy traffic. Referring to Table 3.8:

- For pavements subjected to light traffic (<10^6 ESA), it is considered appropriate to allow a constant deformation rate in the base layer at the design stress level under a 40 kN wheel load (Stage 2). In this case, the basecourse is considered to have passed if the results of permanent strain RLT testing show that the basecourse material exhibits stable behaviour in Stage 1 and unstable behaviour in Stage 2.

- For pavements subjected to medium traffic(10^6–10^7 ESA), where potential for higher traffic loads exists, it is considered appropriate to allow a decreasing deformation rate in the base layer at the critical stress level under a 40 kN wheel load (Stage 2). In this case, the base material is considered to have passed if the results of permanent strain RLT testing show that the base material exhibits stable behaviour in Stage 2 and may exhibit failure in Stage 3.
- For pavements subjected to heavy traffic, the stresses in the pavement should reach the stresses in Stage 3. It is considered appropriate to allow a decreasing deformation rate in the base layer at the stress level in Stage 3. In this case, the base material is considered to have passed if the results of permanent strain RLT testing show that the base material exhibits stable behaviour in Stage 2 and unstable behaviour in Stage 3.

Table 3.8 Requirements of material behaviour for granular bases (Vuong 2000).

Stage	Loading stress (kPa)		Behaviour requirements of granular bases		
	Static confining	Dynamic deviator	$<10^6$ ESA	10^6–10^7 ESA	$>10^7$ ESA
Stage 1	50	350	Stable	Stable	Stable
Stage 2*	50	450	Unstable	Stable	Stable
Stage 3	50	550	Failed	Unstable to failed	Stable to unstable

*Design stress level

A similar procedure is used for the assessment of upper sub-base and lower sub-base materials. In addition, the resilient moduli are also compared for material ranking.

This assessment method is simple and is suitable for the purpose of material ranking in specifications. However, the results in this research aim to validate the procedure with field performance data for pavement conditions in Australia and New Zealand.

3.5.3 ARRB/AUSTROADS assessment based on deformation life

Material performance can also be judged based on the number of loading cycles at a given loading stress required to reach a failure condition or deformation limit. This method involves curve fitting and extrapolation to determine the number of loading cycles required at each stage to reach a nominal permanent strain limit for failure, e.g. 15 000 microstrain. Full details of this analysis method are given in Vuong & Arnold (2006).

In this case, a curve fitting procedure is used to determine the relationship between permanent strain and loading cycles for different stress levels applied in the three-stage loading test. From these relationships, the number of loading cycles required to reach a nominal failure strain (e.g. 15 000 microstrain) can be calculated and plotted against the applied stress, as shown in Figure 3.11. Referring to Figure 3.11, the loading cycles required to reach failure at the design stress in Stage 2 (or design deformation life), and deformation lives for other stress levels outside the tested stress range (by means of extrapolation) can be used in material assessment.

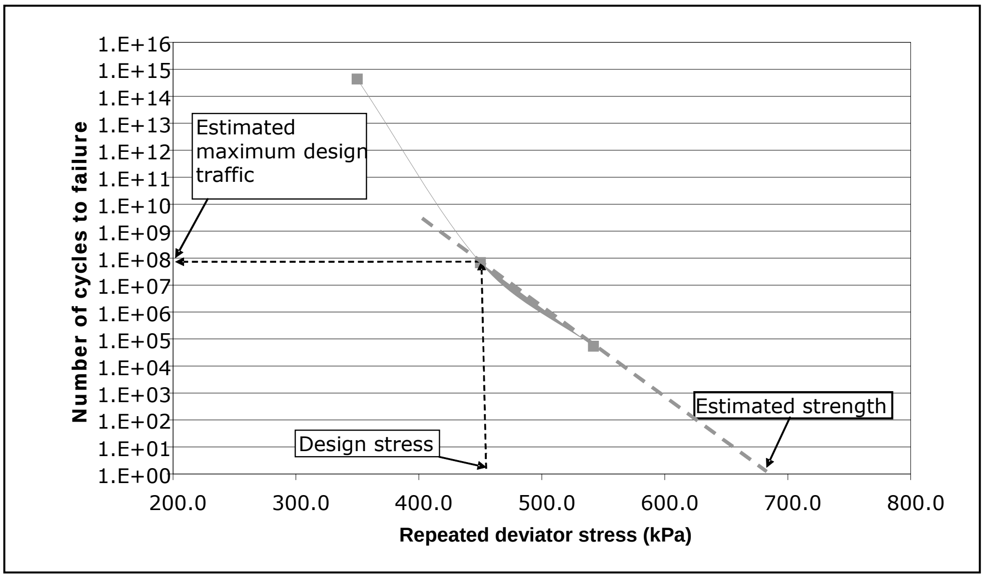

Figure 3.11 Example of the relationship between granular base deformation life and stress level.

Figure 3.12 also shows the proposed requirements of minimum deformation life (Vuong 2000), which can be used to select materials for use in different pavement classes subjected to different design lives. Referring to Figure 3.12, each line of minimum deformation life is defined by the minimum design deformation life at the critical design stress in Stage 2 and strength limits (stress that causes failure in one cycle). It was considered appropriate to use:

- the minimum design deformation life at the critical design stress as the criterion for terminal rut depths; and
- the minimum strength limit as the criterion for protection against overloading, viz. low strength base materials (minimum strength = 600 kPa) being used in low-traffic local roads (10^5 ESA) and high strength base materials (strengths >800 kPa) used in high class heavy-duty roads ($>10^7$ ESA).

In this case, the basecourse is considered to have passed for a specific pavement design life if the results of permanent strain RLT testing show that the basecourse material has greater deformation lives for the three loading stages than the required minimum deformation lives (i.e. on the right-hand side of each curve for the design life concerned).

Examples of two materials, A and B, are also shown in Figure 3.12. Material A is considered to have a better performance than Material B, as the results of the permanent strain RLT testing show that Material A produces higher deformation lives for all stress levels. In addition, based on the proposed requirements of minimum deformation life, Material A is considered suitable for pavements with a design traffic of <10^7 ESA; Material B is suitable for pavements with a design traffic of <10^6 ESA.

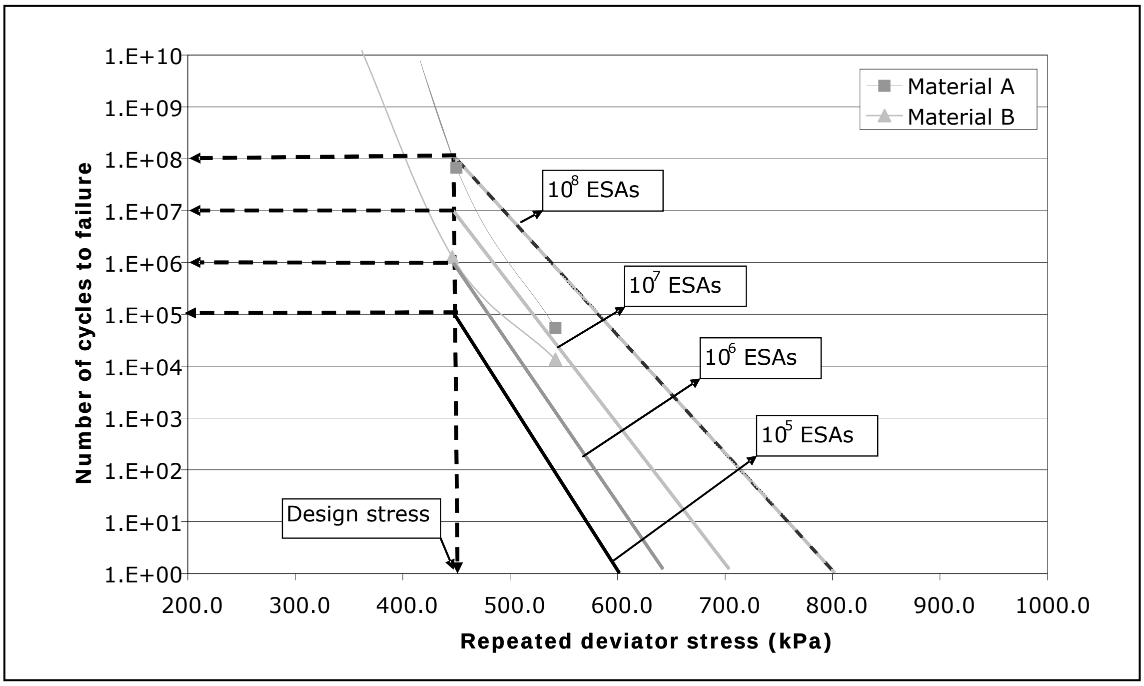

Figure 3.12 Boundaries of deformation life and strength for different pavement classes.

This method is more versatile than the assessment method based on material behaviour (see Chapter 3.5.2) as it can be used for a designated pavement design life. This assessment method is used on the RLT results for several New Zealand aggregates tested of known field performance in this study to validate the method.

4 RLT tests performed in this research

4.1 Materials tested

Both the Arnold and ARRB method were trialled on ten different basecourse aggregates sourced throughout New Zealand. The aggregates conformed to TNZ M/4 (Transit 2006) specification. The aggregates investigated came from different sources:

- Stevensons Drury Quarry;
- Pound Road Quarry as used in accelerated pavement tests at CAPTIF (Greywacke: 3 materials):
    - AP40 (CAPTIF 1),
    - AP40 + 5% silty clay fines (CAPTIF 2),
    - AP20 (CAPTIF 4);
- Poplar Lane Quarry;
- Hunua Quarry (GAP40);
- Oreti River Quarry;
- Waitakere Quarry;
- Tauhara Quarry (dacite);
- Cement modified basecourse aggregate.

Results are referred to as Materials 1, 2, 3… in a different order from the above to keep the results confidential. This is because of the possible sensitivity of the results and because further analysis and tests are yet to be conducted before any conclusions regarding the actual performance of the aggregates are obtained.

Materials that have some anecdotal evidence of field performance were selected for these tests. Field performance results of Materials 1, 2 and 4 are estimated in Table 4.1 below. Estimates of traffic loading limits were based on the assumption that a good aggregate should provide adequate performance between 8 and 15 million ESA. Poor aggregates only last up to 1 million ESA, based on materials that have been involved in early pavement failures.

Table 4.1 Material confidential ID and estimated rutting performance.

Material #	Rutting performance	Estimated traffic loading limit when wet (million ESA)	Estimated traffic loading limit when dry (million ESA)	Remarks
1	Good	8	15	CAPTIF 1 – TNZ M/4 AP40; Greywacke – material from CAPTIF tests
2	Poor when wet	1	10	CAPTIF 2 – TNZ M/4 AP40; Greywacke + 5% silty clay fines – material from CAPTIF tests
3	Good	8	15	CAPTIF 4 – TNZ M/4 AP20; Greywacke – material from CAPTIF tests
4	Good	10	15	Hard stone with good control on shape and grading at the quarry
5	Poor	1	5	Involved in early pavement rutting failure
6	Poor	1	5	Involved in early pavement rutting failure
7	Average	3	6	No reported performance
8	Poor when wet	1	8	Involved in early pavement rutting failure
9	Average	3	6	No reported performance
10	Good	10	15	Material #8 modified with cement – should give good performance

4.2 Sample preparation and target density and moisture content

Both the Arnold and ARRB RLT test methodologies were conducted on all the aggregates at a dry condition (70% OMC, referred to as *dry*) and repeated in a nearly saturated condition (100% OMC, referred to as *wet*) (Table 4.2). The target dry density was 95% of MDD (Maximum Dry Density), which is the same as the minimum allowable density in the TNZ B/2 specification for basecourse construction (Transit 2005). Despite these targets, it was often difficult to obtain the target density and moisture content. For most aggregates tested in the wet condition, the water leaked out of the specimen during RLT testing and the final moisture content was much less than the original 100% OMC used in compaction of the specimen. RLT samples are compacted for one minute using a vibrating hammer complying with NZS 4402: Test 4.1.3 (Standards New Zealand 1986) with a surcharge weight of 25 kg in 5 layers in a split mould 150 mm diameter and 300mm in length. Figure 4.1 shows a picture of the method of compaction used.

Table 4.2 RLT Target density and moisture content values.

Repeated load triaxial testing			
RLT Test per aggregate:		Dry	Wet
Target OMC:	% of OMC[a]	70	100
Target degree of compaction:	% of MDD[b]	95	95

a OMC = Optimum moisture content (NZS 4402: Test 4.1.3).
b MDD = Maximum dry density from Vibrating Hammer Compaction Test (NZS 4402: Test4.1.3)

Figure 4.1 Equipment used for compacting RLT test specimens.

Tests were only conducted once in the RLT apparatus because of time and funding constraints. However, the repeatability of the RLT test results should be the subject of a separate study.

4.3 Results: RLT Arnold Test Method 1 (four stages)

Figure 4.2 shows a typical result of the development of axial plastic strain versus number of number of load cycles for a basecourse aggregate applying the multi-stage RLT loading regime according to the Arnold (2004) RLT Test Method 1 (see Table 3.1).

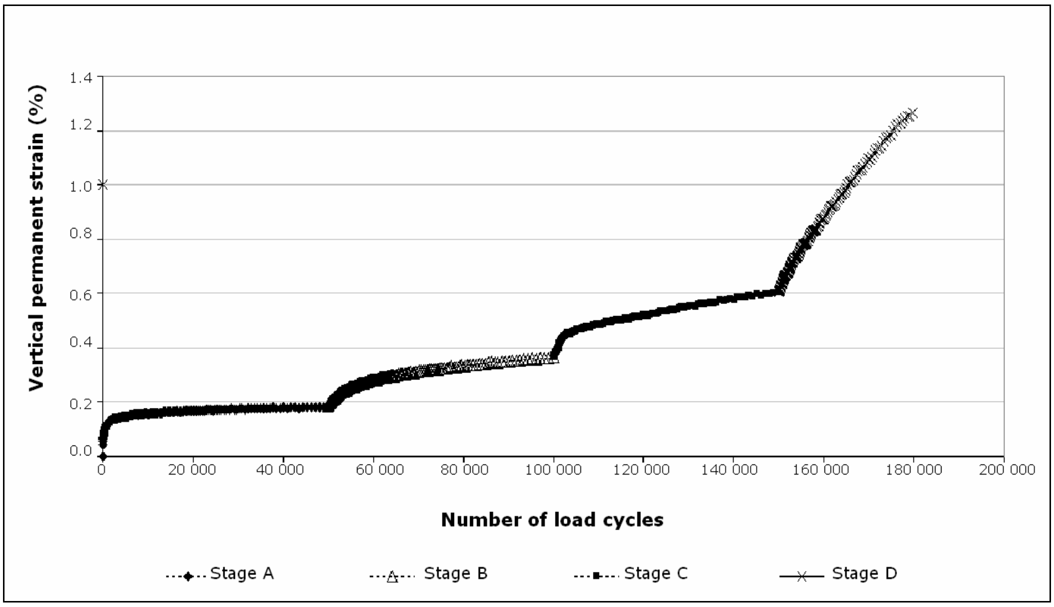

Figure 4.2 Permanent strain versus number of number of load cycles using RLT Test Method 1.

All results of RLT Test Method 1 for all the aggregates tested are shown in Appendix A.

An RLT test was repeated for Material 5, which failed in Stage 1 of RLT Test Method. To ensure survival of the material, the stress levels for each RLT test were reduced as detailed in Table 4.3. The results of the new reduced stress levels for Material 5 are shown in Figure 4.3.

Table 4.3 Reduced stress levels used for a repeat RLT test of Material # 5.

RLT testing stress stage	A	B	C	D	E
Deviator stress - q (kPa) (cyclic vertical stress)	91	119	154	140	180
Mean stress - p (kPa)	53	67	95	242	140
Cell pressure, σ_3 (kPa)	23	27	44	195	80
Major principal vertical stress, σ_1 (kPa)	114	146	198	335	260
Cyclic vertical loading speed	Haversine at 4 Hz				
Number of loads (N)	50 000				
Data recorded and reported electronically in Microsoft Excel	Permanent strain versus load cycles, and resilient modulus versus load cycles				

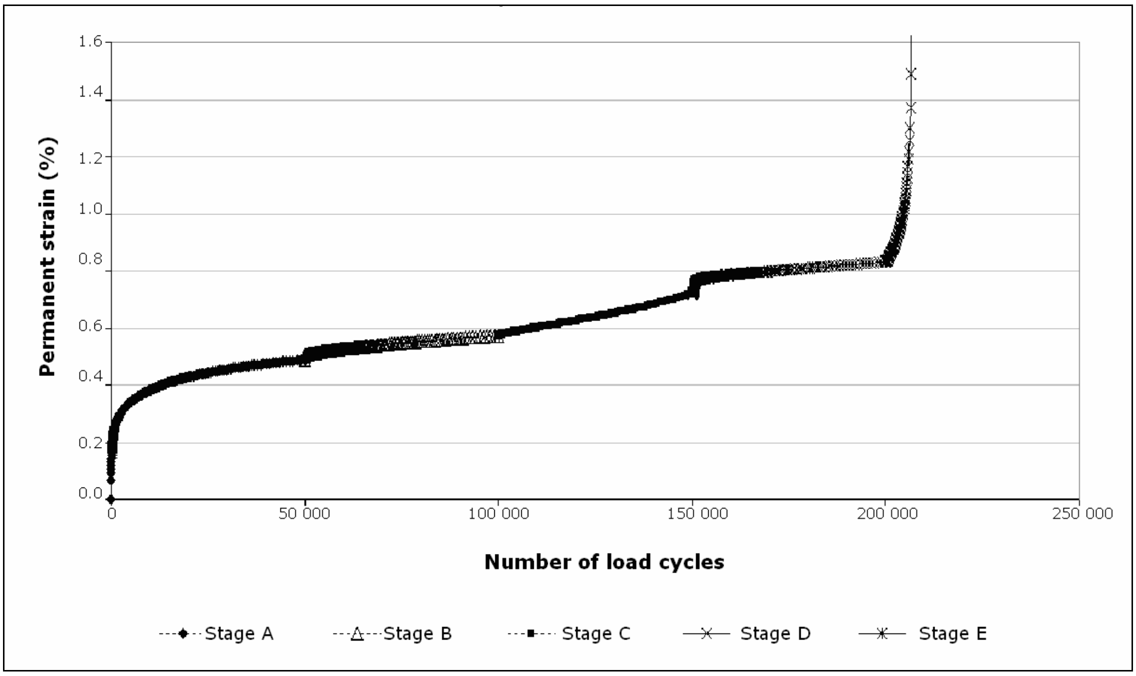

Figure 4.3 Test 5c - Material 5 (reduced stress levels);Dry: 93% MDD, 68% OMC.

4.4 Results: ARRB/AUSTROADS method

Opus Central Laboratories in Lower Hutt tested Materials 4 to 10 using the proposed ARRB/AUSTROADS three-stage RLT test (detailed in Chapter 3.5). The raw results of this testing are shown in Appendix B with an example shown in Figure 4.4. Both permanent and resilient strains from the RLT tests are used in the assessment procedure.

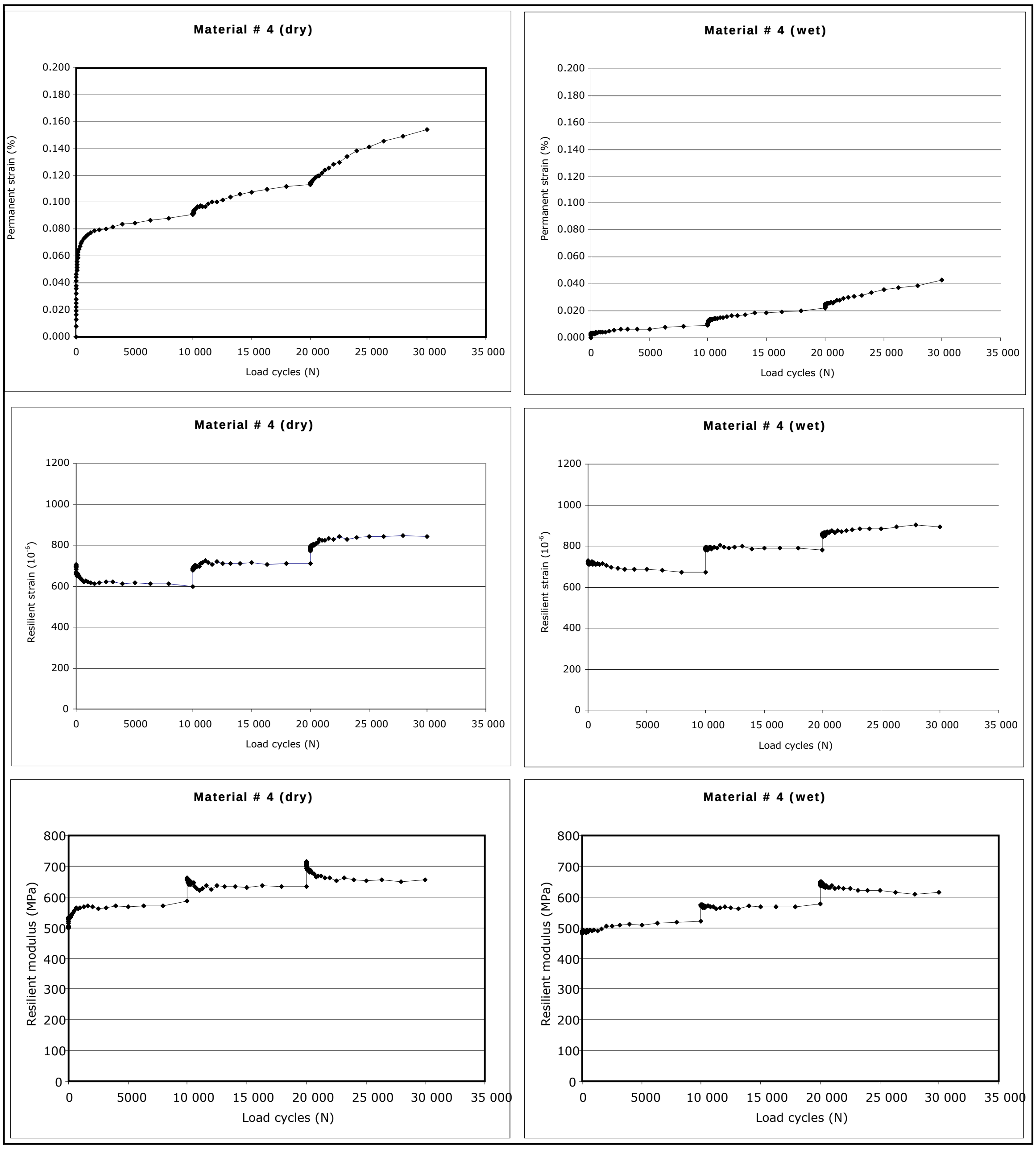

Figure 4.4 Example of RLT test results using the ARRB/AUSTROADS method.

5 Analysis of the RLT tests

5.1 Arnold / Nottingham method

Results of the RLT testing reported in Chapter 4.4 were analysed to predict rutting in a granular pavement used at CAPTIF using the methodology detailed in Chapter 3.4.

The first step in this analysis involved extrapolating the RLT results by treating each stress stage individually. This curve fitting and extrapolation process yielded permanent strain rates as detailed in Table 5.1. It should be noted that this table only reports those test results that survived at least the first three stages, as otherwise we would have insufficient data to fit the model for calculating permanent strain.

Table 5.1 Extrapolated permanent strain rate values in % / million.

Material	Stress stage*															
	A				B				C				D			
	25k	25k to 50k	50k to 100k	100k to 500k	25k	25k to 50k	50k to 100k	100k to 500k	25k	25k to 50k	50k to 100k	100k to 500k	25k	25k to 50k	50k to 100k	100k to 500k
1a	0.064	0.192	0.096	0.028	0.145	1.534	0.782	0.234	0.140	0.163	0.090	0.031	0.171	1.242	0.878	0.463
2c	0.136	0.552	0.277	0.081	0.146	0.936	0.473	0.140	0.166	2.355	1.709	0.944	0.147	0.906	0.515	0.185
3a	0.087	0.276	0.138	0.040	0.127	0.611	0.341	0.119	0.153	0.187	0.097	0.030	0.194	1.073	0.613	0.223
3b	0.036	0.374	0.187	0.054	0.155	2.594	1.550	0.608	0.147	0.563	0.319	0.114	2.142	7.236	3.924	1.305
4a	0.037	0.082	0.041	0.012	0.067	0.377	0.189	0.055	0.072	0.058	0.032	0.011	0.087	0.334	0.208	0.088
4b	0.065	0.138	0.069	0.020	0.139	0.815	0.411	0.121	0.143	0.054	0.030	0.010	0.165	0.413	0.256	0.107
7b	0.111	0.269	0.135	0.039	0.277	1.628	0.830	0.249	0.771	7.611	5.045	2.372	–	–	–	–
8a	0.185	1.237	0.622	0.182	0.409	3.836	2.014	0.634	0.416	0.647	0.370	0.135	0.463	2.963	1.912	0.858
8b	0.189	0.770	0.386	0.113	0.330	4.015	2.115	0.670	0.319	0.825	0.460	0.191	0.405	5.621	3.526	1.505
9a	0.172	0.438	0.219	0.064	0.272	2.081	1.071	0.326	0.392	3.018	1.590	0.5040	0.757	15.854	10.579	5.035
9b	0.204	0.539	0.270	0.079	0.325	2.302	1.189	0.364	0.375	1.797	0.926	0.282	0.830	24.148	18.260	10.867
10a	0.035	0.105	0.053	0.015	0.082	0.117	0.058	0.017	0.088	0.084	0.051	0.021	0.097	0.151	0.087	0.032

*See Table 3.1 for specification of the stress stages.

Permanent strain rate data were used to determine the parameters for the model (Equation 1) for calculating permanent strain from stress. Equation parameters determined are listed in Table 5.2. Graphic representations are the best way to demonstrate this extrapolation and to show how well Equation 1 (which relates stress and permanent deformation) fits the permanent strain data; an example is shown in

Figures 5.1 and 5.2, while the full results are detailed in Appendix C. It can be seen that, generally, Equation 1 fits the data well, although some errors have appeared with materials that failed or nearly failed with high deformations. Specimens near failure are unstable and it is expected that the fit would be poor (Arnold 2004).

Rut depth predictions are shown in Table 5.3 using the Arnold/Nottingham method (Arnold 2004) for a CAPTIF pavement 300 mm deep (see Chapter 3.3 for a description). The best performing material is the cement modified material (Material 10), while the worst are those materials with no result – they did not complete the test – and Material 2, to which fines were deliberately added. Rutting in Material 1 is higher than expected, as this performs well during CAPTIF tests. Material 1 was consequently retested at a later date using new improved testing stresses (Table 3.1), where it was predicted to be a good material with a long-term rutting rate of 0.9 mm/million ESA.

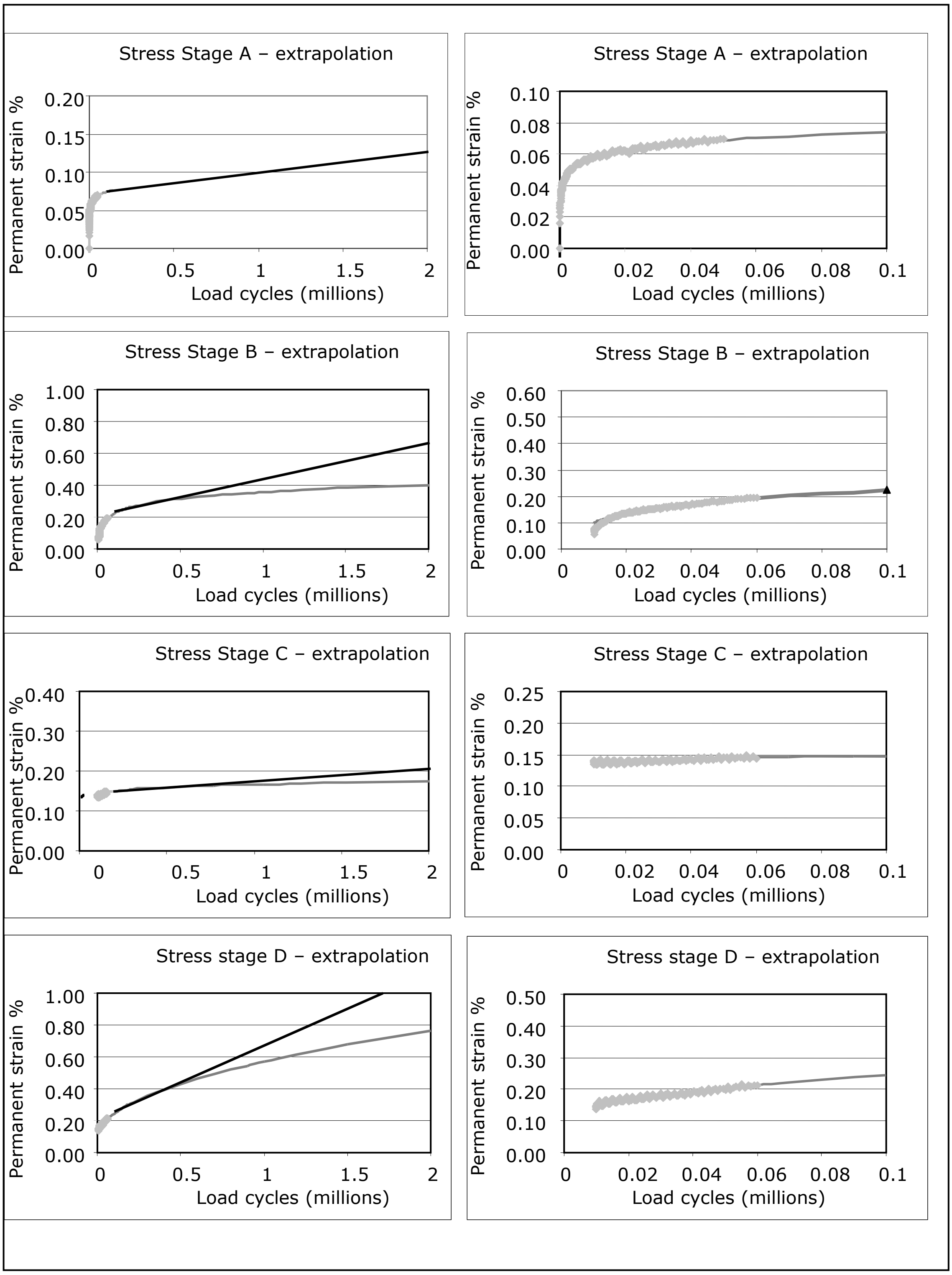

Figure 5.1 Extrapolation of RLT results for Test 1a using the Arnold method.

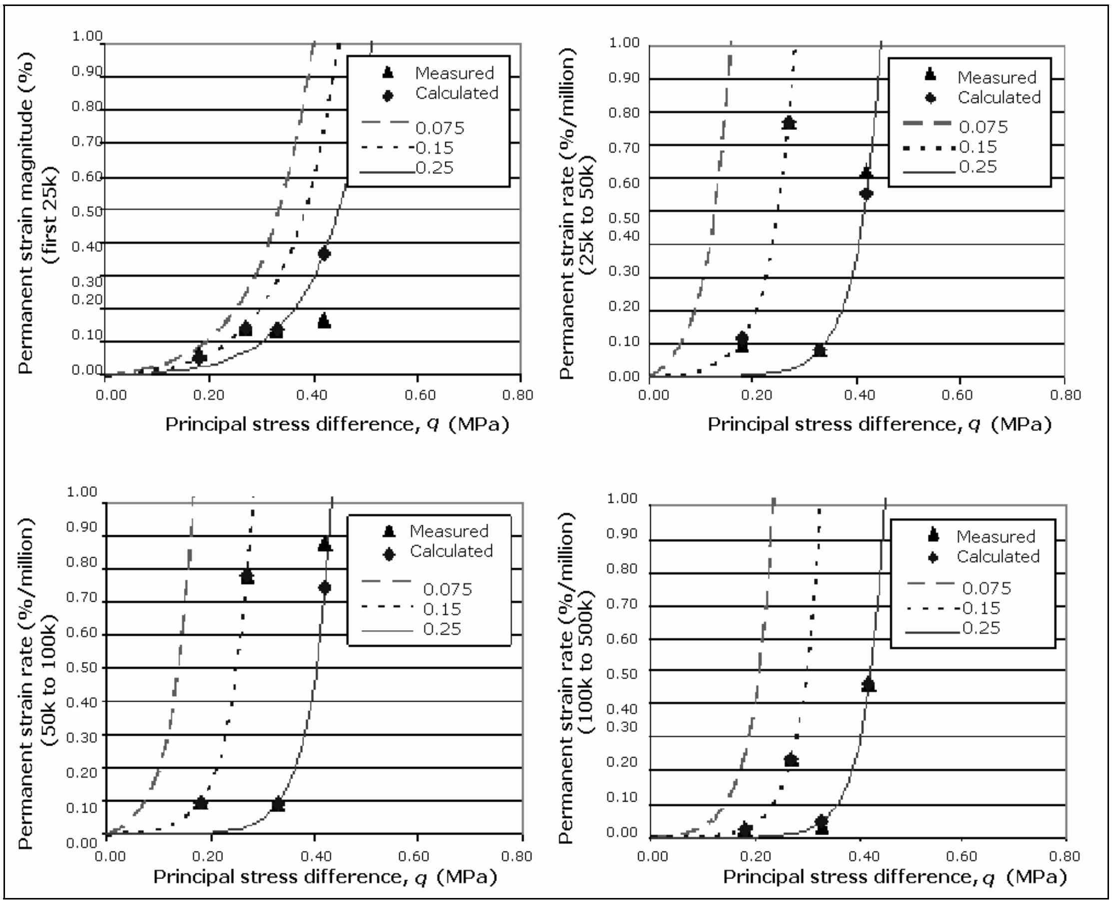

Figure 5.2 Quality of fit between measured / extrapolated permanent strain rates and Equation 1.

Material	Equation parameters															
	25k % permanent strain rate				25k to 50k %/million permanent strain rate				50k to 100k %/million permanent strain rate				100k to 500k %/million permanent strain rate			
	a	b	c	% mean error	a	b	c	% mean error	a	b	c	% mean error	a	b	c	% mean error
1a	-3.672	-7.000	10.554	0.003	-0.019	-35.248	21.259	0.044	-1.200	-35.087	23.034	0.035	-3.607	-31.021	25.218	0.005
2c	-0.001	-10.000	2.322	0.095	-2.280	-10.000	25.529	0.271	-1.103	-10.000	17.303	0.343	-3.041	-10.000	25.688	0.143
3a	-2.317	-15.140	12.842	0.049	-0.005	-17.239	8.199	0.006	-2.680	-15.000	14.356	0.034	-3.639	-15.000	14.024	0.010
3b	-4.769	-10.659	16.728	0.001	0.325	-30.214	21.531	0.027	-1.378	-29.804	23.290	0.000	-2.160	-30.656	23.195	0.005
4a	-3.280	-15.000	10.691	0.014	-3.397	-15.000	19.318	0.057	-1.824	-15.000	9.241	0.019	-4.475	-15.000	14.263.	0.005
4b	-2.913	-12.853	10816	0.021	-0.025	-35.000	18.801	0.008	-0.903	-30.000	16.762	0.011	-3.540	-25.000	17.986	0.012
7b	-2.002	-14.777	11.055	0.166	-2.583	-15.000	19.721	0.000	-2.583	-15.000	19.721	0.474	-5.464	-3.056	16.821	0.120
8a	-2.916	-5.245	10.780	0.022	-0.161	-30.377	23.080	0.206	0.000	-26.856	18.106	0.062	-1.563	-24.581	18.655	0.038
8b	-1.199	-4.686	4.626	0.008	-8×10^{5}	-20.773	17.024	0.150	-3.6×10^{6}	-26.763	18.236	0.024	-2.017	-23.056	19.329	0.001
9a	-3.681	-5.000	11.744	0.034	-2.354	-5.000	14.294	0.027	-2.993	-5.000	14.289	0.024	-4.035	-5.000	13.967	0.015
9b	-2.125	-15.059	12.210	0.082	-0.075	-9.424	8.949	0.141	-2.018	-11.281	14.457	1.863	-2.470	-20.000	16.558	0.000
10a	-2.764	-9.478	7.065	0.010	-1.798	-6.870	5.880	0.000	-1.941	-3.557	2.770	0.000	-4.179	-2.939	5.318	12.561

Table 5.2 Equation parameters for permanent strain rate calculation

Table 5.3 Rut depth prediction using the Arnold / Nottingham method.

Material	% OMC	% MDD	Test	Total pavement	Aggregate only		Ranking
				ESA to get 25 mm rut (millions)	ESA to get 10 mm rut in aggregate (millions)	Long-term rate of rutting within aggregate (mm/million ESA)	
1	65	94	1a	1.26	0.58	7.0	8
2	68	94	2c	0.09	0.06	419.4	12
3	63	96	3a	2.77	8.38	0.9	3
3	85	96	3b	0.73	0.33	19.7	11
4	71	92	4a	3.53	24.27	0.4	2
4	95	92	4b	2.17	2.42	2.0	4
7	56	93	7b	1.65	1.26	6.1	7
8	72	90	8a	0.74	0.19	14.1	10
8	80	94	8b	1.07	0.52	10.5	9
9	70	90	9a	2.78	3.82	2.2	5
9	82	89	9b	1.99	1.95	3.9	6
10	66	90	10a	3.58	100.27	0.1	1

5.2 The simplified ARRB method

Chapter 3.5 describes the ARRB method of assessment, which assesses permanent deformation and resilient strains at each of the three loading stages as shown in Figures 5.2–5.2. The permanent deformation and associated resilient strain at each stage is judged as being either:

- **Stable behaviour,** defined as a decreasing permanent strain rate and a decreasing to constant resilient strain with increasing loading cycles;
- **Unstable behaviour,** defined as a decreasing to constant permanent strain rate and a constant to increasing resilient strain with increasing loading cycles; or
- **Failure,** defined as a constant to increasing rate of permanent strain and an increasing resilient strain with increasing loading cycles, or when the total permanent strain reaches a nominal failure strain, as observed in a static triaxial shear test (e.g. in the range of 1.5–2.0%).

Based on these criteria and the results shown in the Appendices, a permanent deformation behaviour category has been applied to each loading stage for each material tested. Actual permanent strain rates and resilient strain rates were assessed as either decreasing or increasing with the aid of a spreadsheet. From this analysis, a traffic loading limit based on the ARRB assessment method detailed in Table 3.7 has been calculated and reported in Table 5.4.

Table 5.4 RLT results for the simplified ARRB/AUSTROADS assessment method.

Material	Loading stage (static confining = 50 kPa)			Traffic limit	Expected performance*
	1 (350 kPa)	2 (450 kPa)	3 (550 kPa)		
4 (dry)	Stable	Unstable	Stable	$<10^6$ ESA	Good
4 (wet)	Stable	Stable	Stable	$>10^7$ ESA	Good
5 (dry)	Unstable	Unstable	Unstable	Not suitable for a base layer	Poor
5 (wet)	Stable	Stable	Unstable	10^6–10^7	Poor
6 (dry)	Stable	Unstable	Unstable	$<10^6$ ESA	Poor
6 (wet)	Stable	Stable	Stable	$>10^7$ ESA	Poor
7 (dry)	Stable	Unstable	Unstable	$<10^6$ ESA	Average
7 (wet)	Unstable	Stable	Unstable	Not suitable for a base layer	Average
8 (dry)	Stable	Stable	Unstable	10^6–10^7	Average
8 (wet)	Unstable	Unstable	Stable	Not suitable for a base layer	Poor
9 (dry)	Stable	Unstable	Unstable	$<10^6$ ESA	Average
9 (wet)	Unstable	Unstable	Stable	Not suitable for a base layer	Average
10 (dry)	Stable	Stable	Unstable	10^6–10^7	Good
10 (wet)	Stable	Stable	Stable	$>10^7$ ESA	Good

* (refer to Table 4.1)

The results from Table 5.4 appear to be an inaccurate reflection of the expected performance of the basecourses. Some anomalies also appear in the results: for example, Materials 5, 6 and 10 perform better with a higher traffic loading limit when wet. Another anomaly is that Materials 8 and 9, when wet, were unstable in the first two lower stress stages but were stable in the final (third) stage.

5.3 The complex ARRB method

The complex ARRB method requires curve fitting and extrapolation of the RLT permanent strain data to a failure criterion of 1.5% as described in Chapter 3.5.3. Criteria for the curve fitting are described in Appendix A and these were applied to the RLT data. These criteria were formulated into Microsoft Excel to minimise the curve fitting errors. It was found that many iterations were required in order to satisfy the criteria given in Appendix A. Figures 5.3 and 5.4 show the results of fitting curves to the RLT data (Figures 3.6 and 3.10). The extrapolations of those curves to obtain traffic loading limits at the failure criterion of 1.5% are shown in Table 5.3, and Figures 5.5 and 5.6. Data in Figures 5.3–5.6 have been taken from Vuong & Arnold 2006.

Table 5.5 Traffic loading limits and strength values from extrapolation of RLT results.

Material	Load cycles to reach failure							
	1 (350 kPa)	R	2 (450 kPa)	R	3 (550 kPa)	R	Strength (kPa)	R
4 (dry)	3.9E+24	1	1.3E+20	1	1.7E+12	6	705	14
4 (wet)	2.6E+13	9	2.5E+11	9	9.1E+09	9	1241	6
5 (dry)	6.4E+07	11	5.6E+06	11	3.0E+05	11	981	11
5 (wet)	2.1E+05	13	2.3E+04	13	4.8E+03	13	1090	8
6 (dry)	1.2E+07	12	1.7E+05	12	1.5E+04	12	941	12
6 (wet)	8.2E+04	14	6.8E+03	14	1.6E+03	14	1054	9
7 (dry)	2.4E+19	5	4.5E+17	4	4.3E+15	3	1324	5
7 (wet)	5.4E+23	2	1.0E+20	2	3.2E+14	5	814	13
8 (dry)	1.8E+18	6	7.0E+16	5	5.3E+15	2	1951	1
8 (wet)	1.7E+17	7	9.6E+13	7	1.2E+12	7	1178	7
9 (dry)	3.4E+19	4	1.8E+16	6	6.0E+14	4	1543	3
9 (wet)	1.9E+13	10	8.3E+10	10	1.0E+09	10	1023	10
10 (dry)	5.2E+20	3	1.5E+19	3	6.3E+17	1	1830	2
10 (wet)	9.3E+13	8	3.1E+11	8	1.5E+10	8	1329	4

Notes to Table 5.5:
a Permanent strain = 1.5%
b R = Ranking; 1 being the best or highest load cycles or highest strength

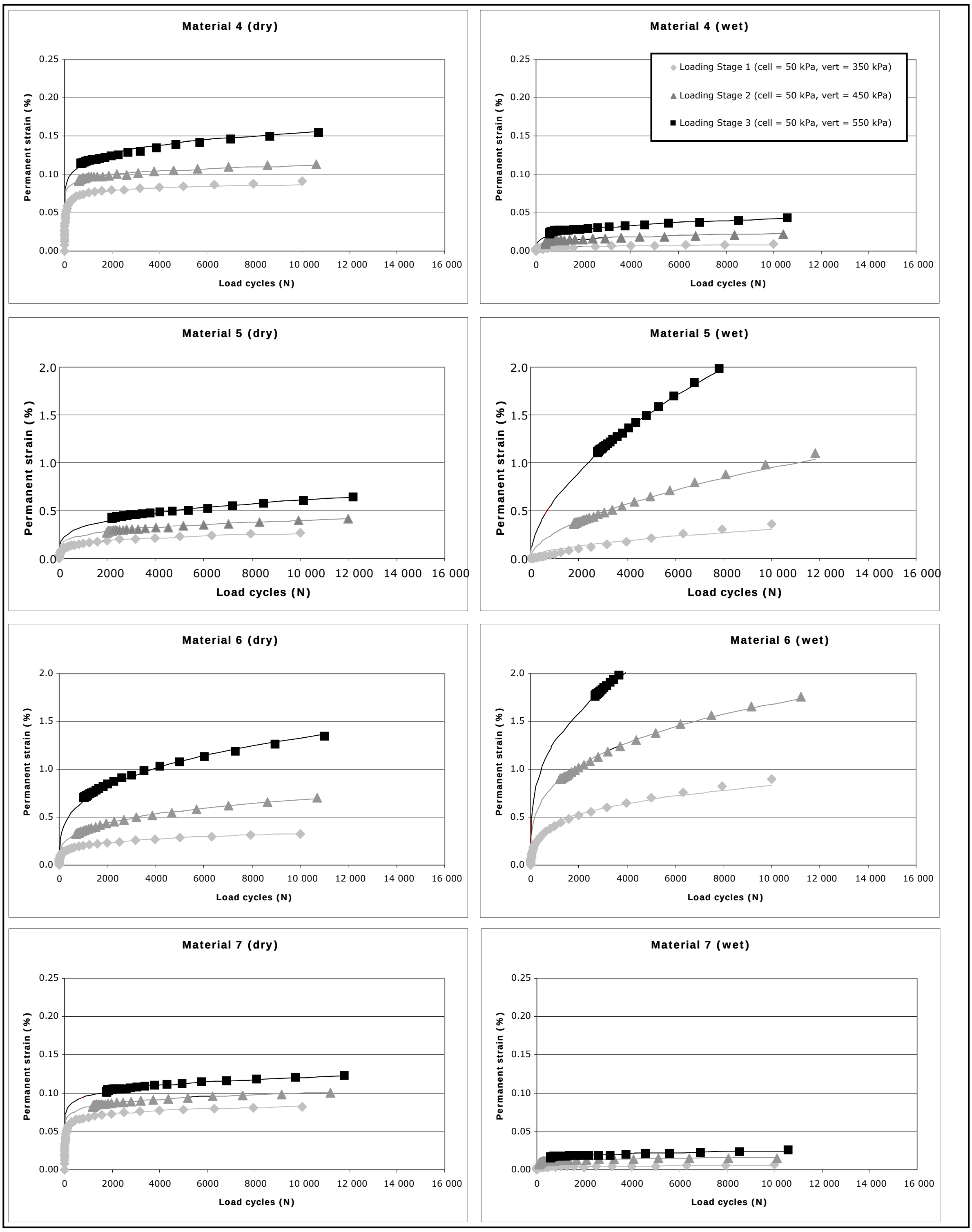

Figure 5.3 Curve fitting RLT data for Materials 4–7 using the complex ARRB method.

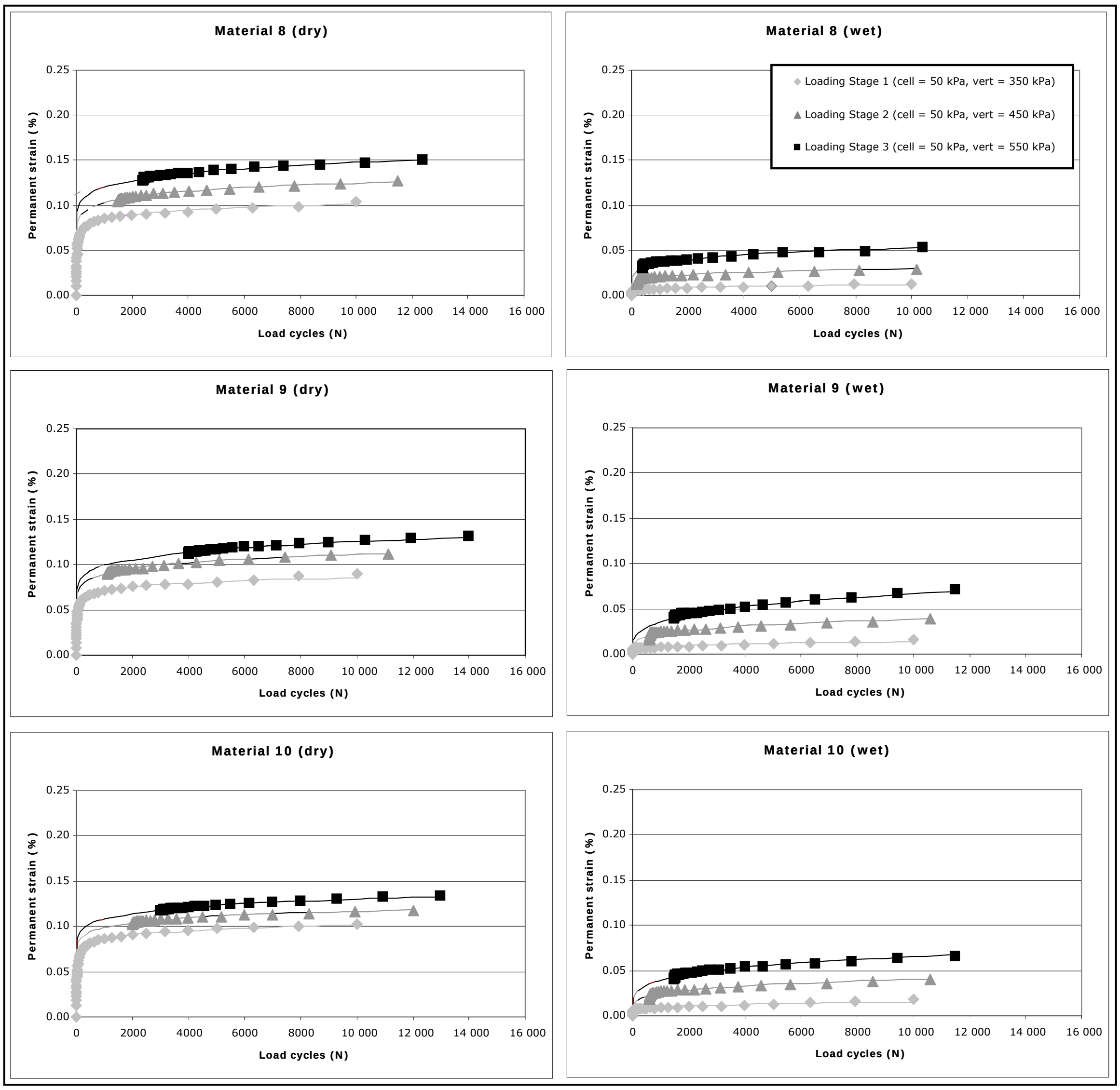

Figure 5.4 Curve fitting RLT data for Materials 8–10 using the complex ARRB method.

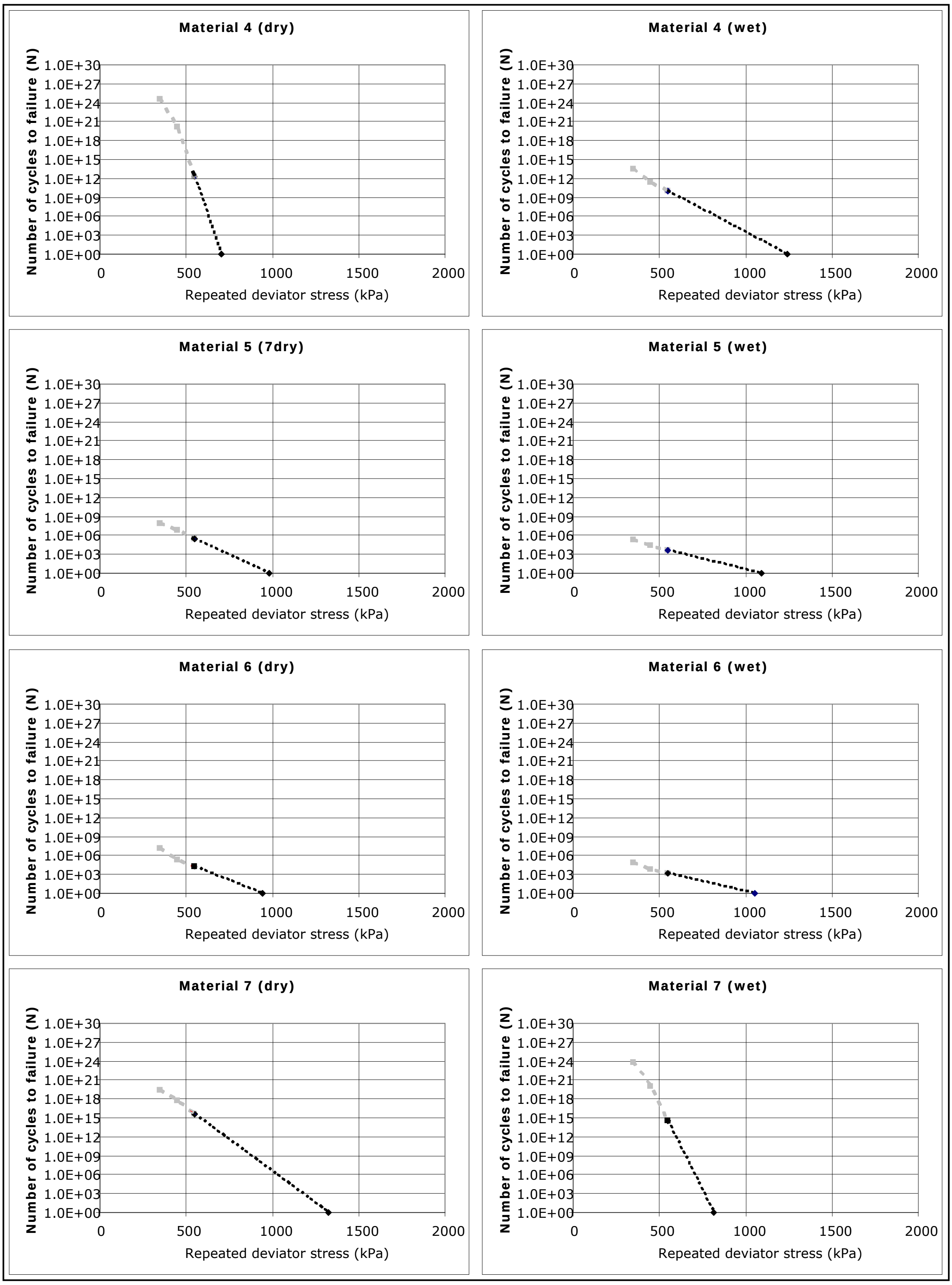

Figure 5.5: Traffic loading limits to reach failure criteria of 1.5 % for Materials 4–7 using the complex ARRB method (Appendix A).

Note: grey squares represent points derived from RLT test results (Figures 5.3 and 5.4) while the black diamond and dashed line represent the extrapolation.

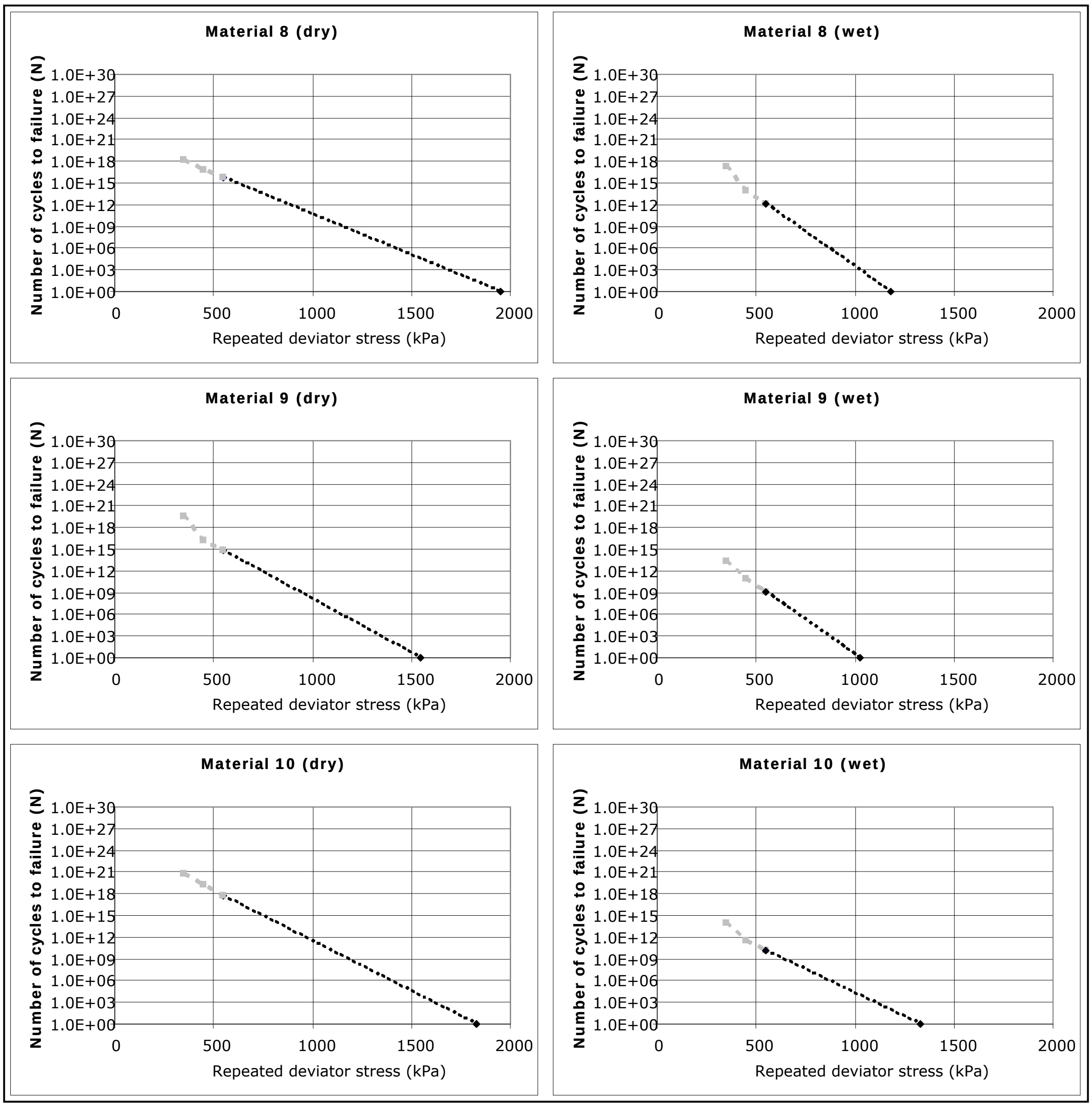

Figure 5.6 Traffic loading limits to reach failure criteria of 1.5% for Materials 8–10 using the complex ARRB method (Appendix A).

Note: grey squares represent points derived from RLT test results (Figures 5.3 and 5.4) while the black diamond and dashed line represent the extrapolation.

The strength value (i.e. deviator stress needed to cause failure at one load cycle, Figures 5.5 and 5.6) was compared with estimated Traffic Loading Limit (Table 4.1) as plotted in Figure 5.7. These results show that the strength value is a poor predictor of performance with a R^2 of 0.1 and a mean error of 3.4 million ESA around the best fit line (Figure 5.7). This was especially the case for Material 5, which had the highest deformations in the RLT test but had an average 'strength value' of around 1000 kPa.

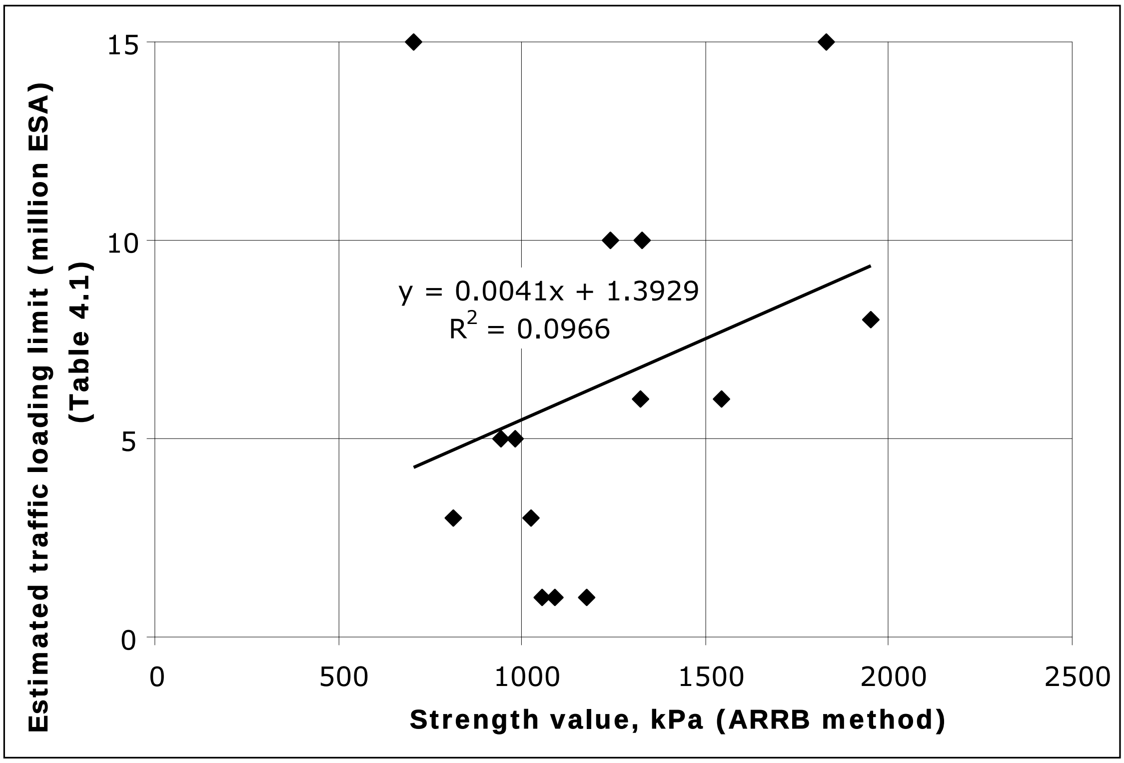

Figure 5.7 Comparison of the strength value using the ARRB method and the estimated traffic loading limit (given in Table 4.1).

5.4 Comparison

The three methods used for RLT testing and analysis were compared to the anecdotal evidence of expected material performance. Table 5.6 shows how the materials' performance was ranked according to the three methods. Materials that failed during the RLT testing using the Arnold method were ranked on how quickly they failed in the observed RLT results (Figure 4.3). Ranking of the expected performance was based on knowledge of pavements that have and have not failed in the past using the aggregate in question. In ranking the expected performance, it was assumed that a saturated aggregate will perform worse than a dry aggregate.

Table 5.6 Material performance rankings using the three methods of RLT analysis and testing, and observed performance.

Material	Arnold (Table 5.3)	Complex ARRB (Stage 3 – Table 5.5)	Simplified ARRB (Table 5.4)	Ranking based on performance*
4 (dry)	3	6	7	3
4 (wet)	4	9	2	4
5 (dry)	13	11	14	13
5 (wet)	14	13	6	14
6 (dry)	11	12	10	11
6 (wet)	12	14	3	12
7 (dry)	9	3	9	7
7 (wet)	10	5	12	8
8 (dry)	7	2	5	9
8 (wet)	8	7	13	10
9 (dry)	5	4	8	5
9 (wet)	6	10	11	6
10 (dry)	1	1	4	1
10 (wet)	2	8	1	2

* 1 = best; 14 = worst

Table 5.6 shows that the Arnold method matches the expected field performance more closely, while the ARRB methods are poor predictors of performance. A simpler method of ranking a material's performance was also applied based on traffic loading limits, shown in Table 5.7. This simple method of ranking shows an improvement to the ARRB methods but the Arnold method is still the best.

Table 5.7 Material performance grouping using the three methods of RLT analysis and testing and observed performance.

Material	Method			
	Arnold	Stage 3: complex ARRB	Simplified ARRB	Ranking based on performance
4 (dry)	Good	Good	Poor	Good
4 (wet)	Average	Good	Good	Average
5 (dry)	Poor	Poor	Poor	Poor
5 (wet)	Poor	Poor	Average	Poor
6 (dry)	Poor	Poor	Poor	Poor
6 (wet)	Poor	Poor	Good	Poor
7 (dry)	Average	Good	Poor	Average
7 (wet)	Poor	Good	Poor	Poor
8 (dry)	Poor	Good	Average	Average
8 (wet)	Poor	Good	Poor	Poor
9 (dry)	Average	Good	Poor	Average
9 (wet)	Average	Good	Poor	Average
10 (dry)	Not tested	Good	Average	Good
10 (wet)	Good	Good	Good	Good

Good = N $>10^7$ ESA
Average = $10^6<$ N $<10^7$
Poor = N $<10^6$

6 Development of a new simple RLT test method and analysis

6.1 Introduction

Since the RLT testing for this research, many other tests have been conducted for private companies and Transit New Zealand regions. The aim of these tests was to determine if the aggregates tested in the RLT test carried any risks of early permanent failure. For this commercial testing, it was important that enough information was obtained from the RLT test for predicting performance. However, results from the four-stage Arnold method (Table 3.1) in this research showed a large number of early failures. Therefore, more testing stages at lower stress levels were added to the test to develop a six-stage RLT test (Table 3.2). The same method of rut depth prediction was used for the six-stage RLT tests. A better fit to the model relating permanent strain rate to stress was possible with the new six-stage RLT test, compared with the four-stage RLT test. Furthermore, fewer early failures occurred, which is probably because the lower stress stages condition the sample in preparation for the higher stress levels.

This chapter details the new six-stage RLT test procedure and compares the Arnold/ Nottingham rut depth prediction method using finite element modelling with parameters from the RLT test such as slope and magnitude. The aim is to develop a simple assessment method of the RLT test results that could be put into specifications.

6.2 Six-stage RLT test

The new six-stage testing stresses (Table 3.2) chosen for the RLT test were based on covering the full spectra of stresses within a pavement. Actual stresses within the pavement are complicated by the fact that unbound granular materials cannot sustain tensile horizontal forces and will yield under compactive forces. This yielding deforms the aggregate both downwards and horizontally. The horizontal movement is resisted by the surrounding compacted aggregate and, therefore, residual horizontal confining stresses are built up.

The coefficient of lateral earth pressure at rest is considered an appropriate value to use for estimating the amount of residual stresses present in a granular material located in a pavement. For granular pavement materials, the cohesion may be nil and the friction angle is 50 degrees (Arnold 2004). This results in the coefficient of lateral earth pressure at rest having a value of 0.23 (Equation 2). Granular materials are located relatively near the surface, e.g. 200 mm deep. Based on a density of 2400 kg/m^3, the overburden stress would be 5 kPa. Thus, the horizontal residual stress at rest works out to be approximately 1.2 kPa. However, adding 550 kPa (for example) vertical stress from compaction and initial traffic loading, the horizontal residual stress at rest is approximately 128 kPa.

$$K_0 = 1 - \sin\varphi' \qquad \text{Equation 2}$$

An FEM, ROSTRA, was used to calculate stresses directly under a circular load of 4.1 tonnes (i.e. half the standard axle load of 8.2 tonnes)

Another factor in determining the testing stresses was the proximity to the shear failure line. Figure 6.1 shows the approximate position of the shear failure lines along with the stress conditions for the various RLT methodologies discussed in this report. Applying a 128 kPa residual stress results in a stress distribution directly under the wheel of an 8.2 tonne dual-tyred axle in line with the new six-stage stress path RLT test.

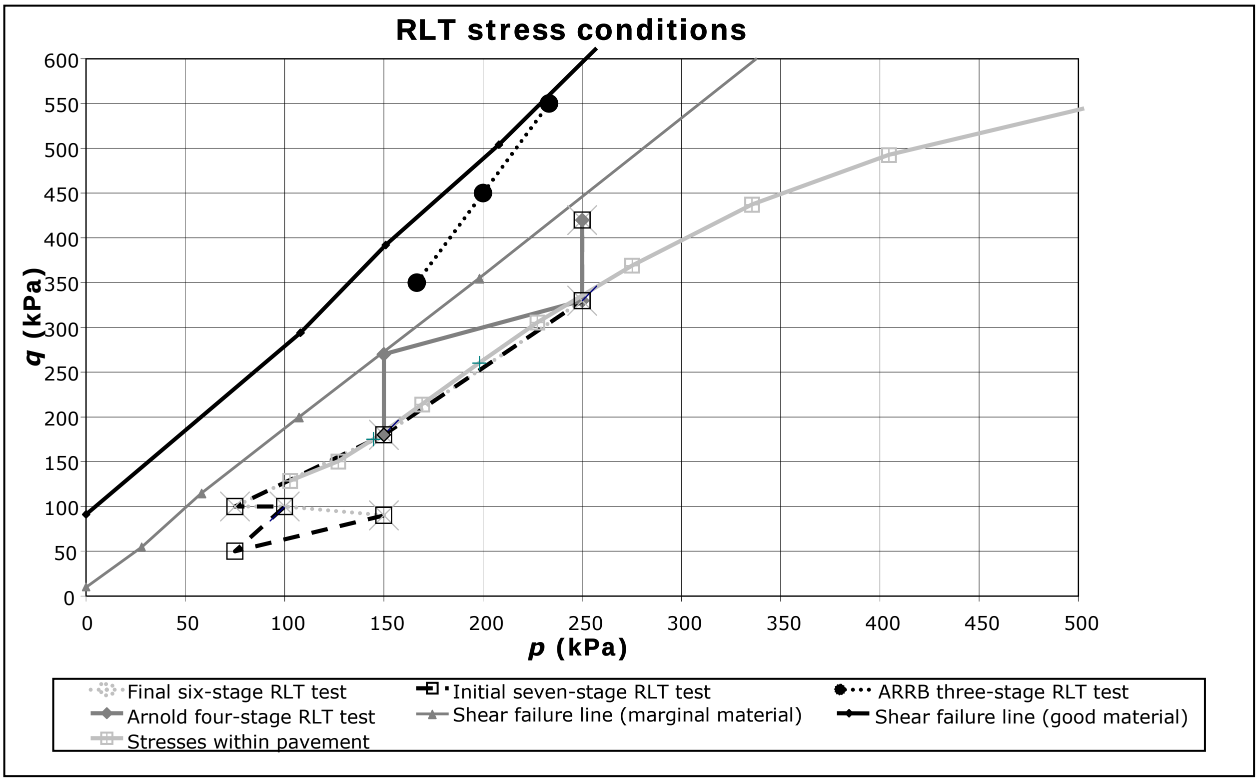

Figure 6.1 Stress conditions for the RLT tests.

6.3 Results of the new six-stage RLT test

Many materials have since been tested using the new six-stage RLT test on a commercial basis. As they are commercially funded tests, their identity is kept anonymous. Also, the performance in the field is not always known. Despite this, the results are useful in terms of comparing the predictions using the Arnold/Nottingham method and finite element modelling of a CAPTIF pavement with simple parameters from the test such as the slope from 25k to 50k in each stage. If an appropriate relationship can be found between predictions and a simple parameter then this can be used in a specification. Tests are labelled as Materials 11 to 23 to remain anonymous.

Two values were obtained from the RLT test at each loading stage; these are the secant permanent strain rate from 25k to 50k cycles, and the magnitude of permanent strain at 25k load cycles. The loading count was zeroed for each stage but the permanent strain magnitude remained cumulative, as shown in Figures 6.2 and 6.3. Table 6.1 lists the results of 23 RLT tests using the six-stage RLT test. Results show the predictions from the Arnold/Nottingham rut depth modelling along with the basic parameters obtained from the RLT test of total permanent strain at 25k (i.e. all stages summed) and the average slope from 25k to 50k for each stage. In addition, the slope of the last stage was included and a value considering both cumulative permanent strain at 25k and the average slope. The aim was to determine a simple parameter from the RLT test that could be used to predict the rutting within a granular material or a traffic loading limit.

The aggregate rutting rates and each RLT test result parameter were plotted to find relationships and how well they fitted. Traffic loading limits to achieve a 10 mm rut within the aggregate were also plotted against the various RLT parameters. Results are shown in Table 6.1. Interestingly, the use of the cumulative permanent strain value at 25k did not help the predictions and it was found the average slope was the best parameter to use. A relationship showing the mean traffic loading limit (N) with an average slope and traffic loading limit was found as shown in Figure 6.6 and Equation 3.

$$N = 19.5e^{(-1.3Savg)} \qquad \text{Equation 3}$$

Where:
N = Traffic loading limit in millions of ESA over a 25-year design period;
e = natural logarithm number 2.71;
S_{avg} = Average slope from 25k to 50k load cycles from the six-stage RLT test in units of %/million.

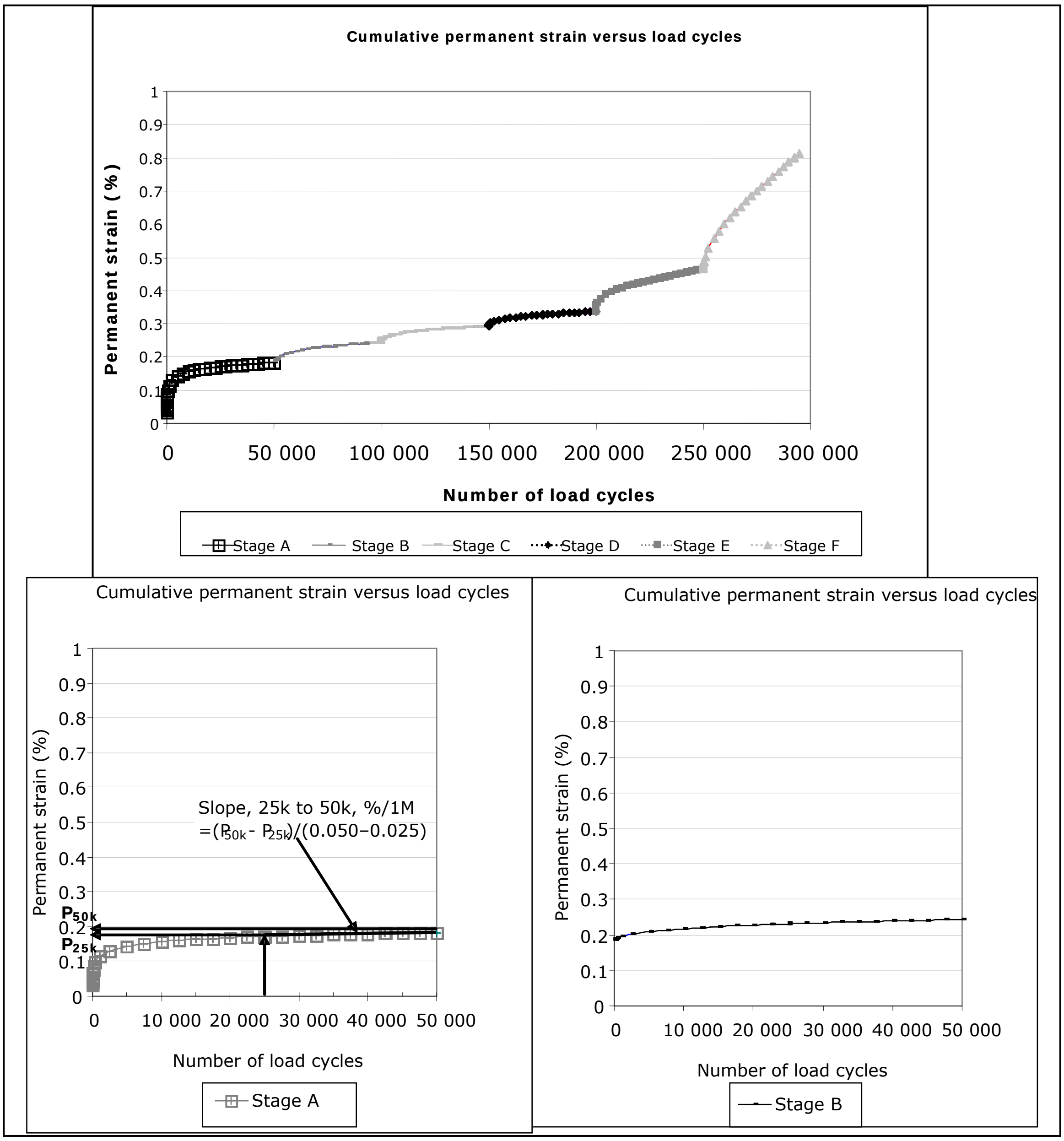

Figure 6.2 Determining slope from 25k to 50k and deformation at 25k from RLT test results for Stages A and B.

Note: the equation in the Stage A diagram m was included to show how the slope was calculated.

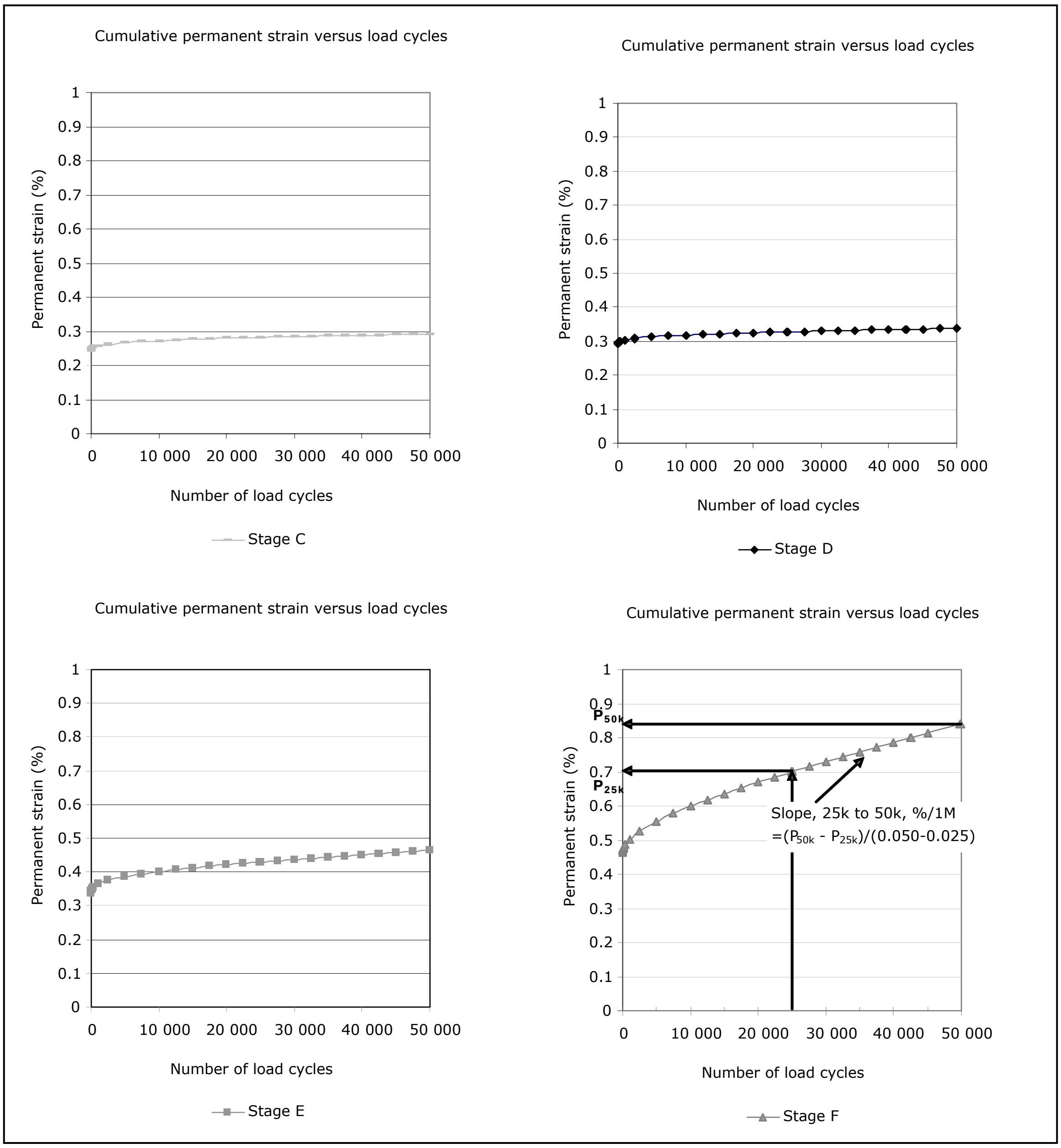

Figure 6.3 Determining slope from 25k to 50k and deformation at 25k from RLT test results for Stages C to E.

Note: the equation in the Stage F diagram was included to show how the slope was calculated.

Table 6.1 RLT test results along with associated rut depth predictions.

Test	CAPTIF pavement, 300 mm aggregate over 10 CBR subgrade			Resilient modulus (MPA) (top of pavement)	Cumulative total first 25k	Average slope 25k to 50k	Last stage slope	Cum 25k +average Slope
	Total pavement	Aggregate only						
	ESA to get 25 mm rut (million ESA)	ESA to get 10 mm rut in aggregate. (million ESA)	Long-term rate of rutting within aggregate (mm per million ESA)					
1	1.10	0.63	10.2	263	0.70	2.21	8.76	2.90
2	3.42	28.60	0.3	760	0.75	0.19	0.47	0.94
3	3.01	10.01	0.9	362	0.23	0.59	2.40	0.82
4	3.00	8.38	1.0	745	0.31	0.20	0.37	0.51
5	3.13	13.56	0.7	475	0.53	1.14	5.37	1.67
6	3.51	22.06	0.4	580	0.27	0.24	0.52	0.51
7	3.33	24.66	0.4	672	0.65	0.18	0.43	0.83
8	0.02	0.01	20.5	259	2.25	6.22	14.76	8.46
9	1.81	1.90	4.1	262	0.58	1.46	5.71	2.04
10	3.37	24.66	0.4	913	0.24	0.09	0.17	0.33
11	0.95	0.60	17.8	238	0.63	2.62	11.61	3.25
12	3.16	12.06	0.7	730	0.66	0.19	0.37	0.85
13	3.19	10.41	0.7	882	1.87	0.59	1.16	2.46
14	2.74	5.02	1.8	874	1.21	0.29	0.52	1.49
15	3.18	10.36	0.7	882	1.87	0.59	1.16	2.46
16	2.71	5.50	1.3	708	0.91	0.39	0.89	1.30
17	3.49	37.16	0.3	741	0.60	0.21	0.42	0.81
18	1.80	1.78	4.0	248	0.84	2.72	12.17	3.57
19	2.78	6.61	1.1	708	1.04	0.25	0.54	1.29
20	0.30	0.02	28.7	239	0.79	3.53	18.26	4.31
21	3.23	11.68	0.8	786	0.63	0.25	0.89	0.88
22	2.07	2.93	2.6	254	0.66	1.48	5.01	2.15
23	3.05	12.94	0.6	761	0.75	0.22	0.43	0.98

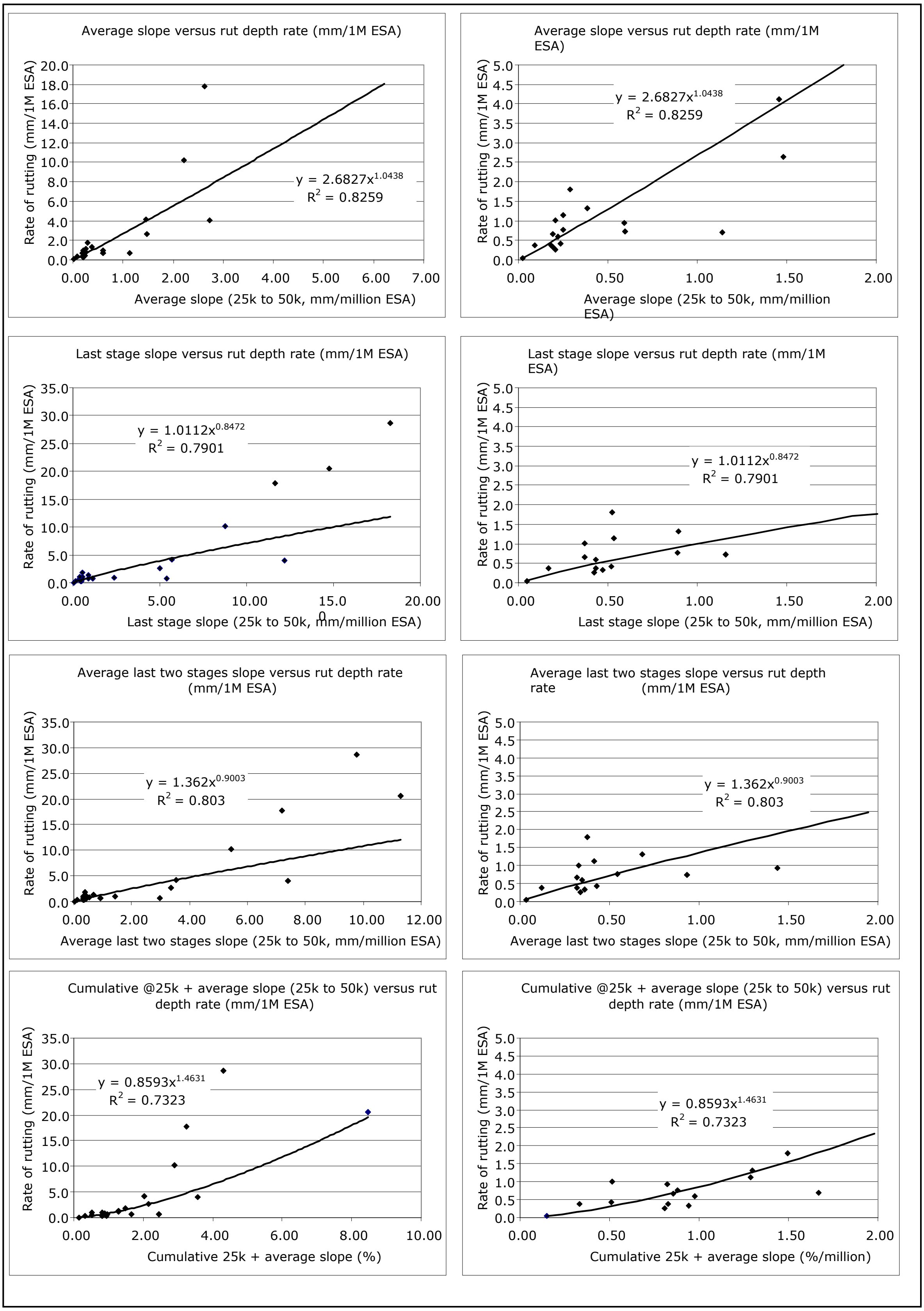

Figure 6.4 Relationships between slope and cumulative permanent strain from RLT tests with rate of rutting predictions from modelling.

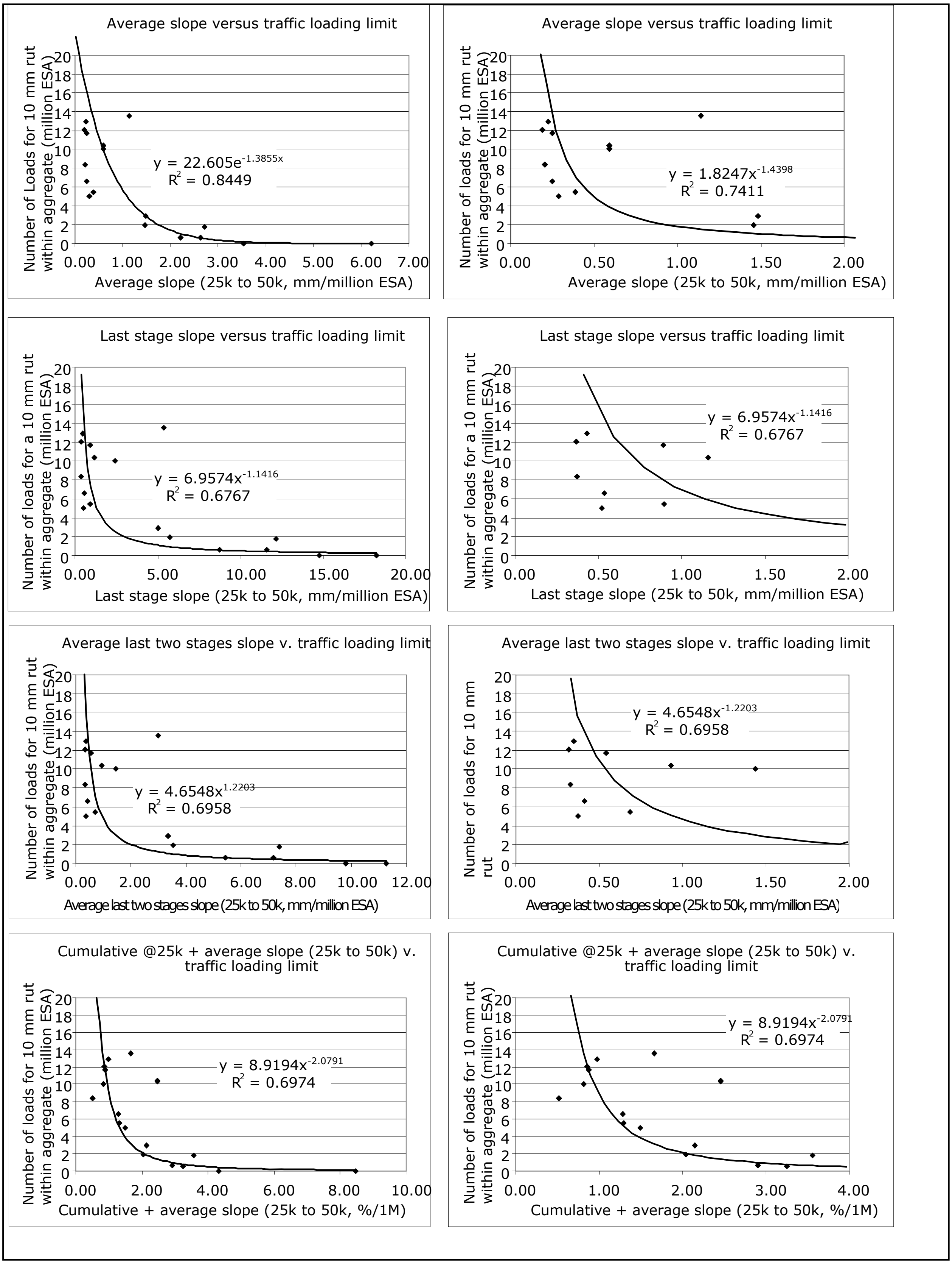

Figure 6.5 Relationships between slope and cumulative permanent strain from RLT tests with traffic loading limit from modelling.

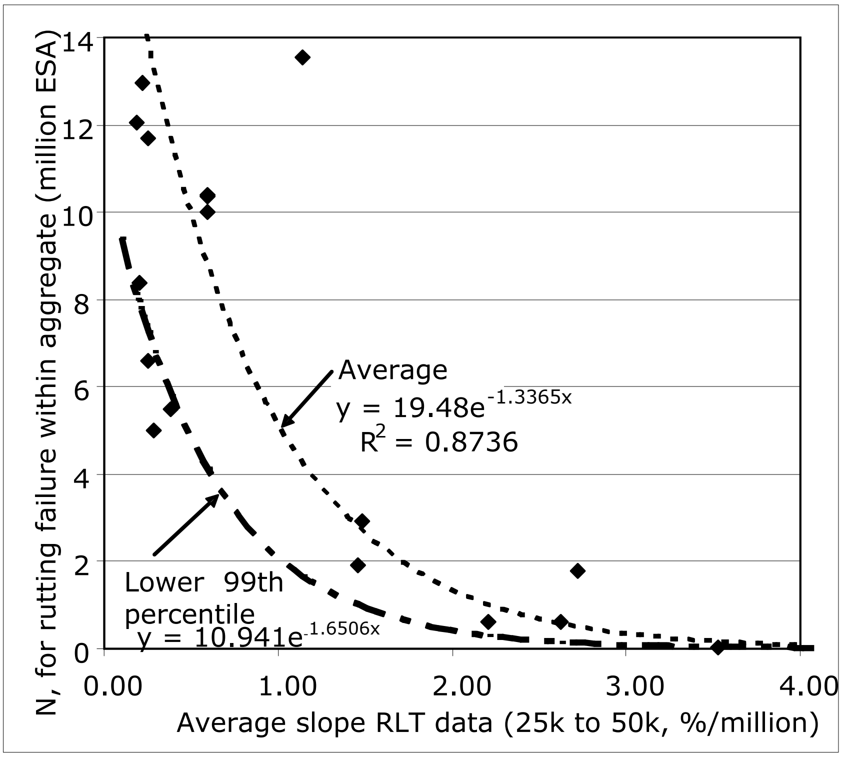

Figure 6.6 Relationships between average slope from six-stage RLT tests with traffic loading limit from modelling.

Further trend analysis was undertaken by considering both the mean and standard deviation of the slopes in the six-stage RLT test. Results of this analysis did not improve the correlation, as shown in Figure 6.7. The reason for this is that the data points with a low slope (on the left-hand side in Figure 6.7) all had a low standard deviation which, when added, did not shift the points to the right far enough to influence the correlation. Therefore, only the average slope should be used in order to keep the analysis simple.

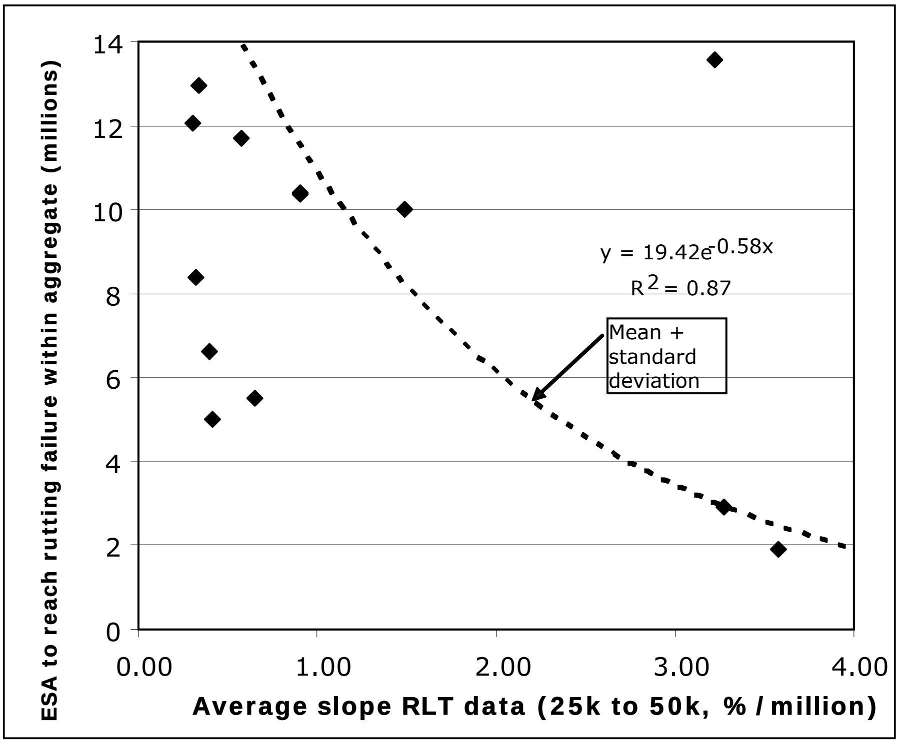

Figure 6.7 Relationships between average slope plus standard deviation in the six-stage RLT tests v. traffic loading limit predicted by modelling.

6.4 Material classification from RLT tests

In considering an appropriate classification system, we should be careful not to exclude existing aggregates in use where there have been no reported incidences of failures. At this preliminary stage, limits should not be introduced that require all highly trafficked roads to use a modified/cemented aggregate. Although technically all natural aggregates rut internally, for the best quality aggregates, this rutting will be managed by resurfacing. Therefore, a classification system is developed based on the relationship between average RLT slope for the six-stage RLT test (Table 3.2) and traffic loading limit based on the average values (Figure 6..6). The equation to use for classification in terms of design traffic loading limit is:

$$N = 19.5e^{(-1.34S_{avg})} \qquad \text{Equation 3}$$

Where:

N = Traffic loading limit in millions of ESA over a 25-year design period,

e = natural logarithm number 2.718282,

S_{avg} = Average slope from 25k to 50k load cycles of all the six-stages in the RLT test in units of %/million.

Equation 3 yielded the following average slope ranges in relation to traffic loading limits (Table 6.2). These traffic loading limits will apply for both dry (70% OMC) and wet (100% OMC) conditions. Generally, however, a wet test result would be required for the highest trafficked roads to ensure the least risk of early failure.

Table 6.2 Aggregate traffic loading limits from average slope in a six-stage RLT test .

Traffic loading limit (ESA) over a period of 25 years*	Maximum average slope from 25k to 50k of all six stages in the RLT test (%/million)
0.5	2.7
1	2.2
2	1.7
3	1.4
4	1.2
5	1.0
6	0.9
7	0.8
8	0.7
9	0.6
10	0.5
15	0.2

*The traffic loading limit can be based on the number of years until pavement smoothing will be applied for asphalt surfaced roads, based on the life of the surfacing (e.g. 10 years).

Applying Equation 3 and Table 6.2 to actual RLT test results for various modified and unmodified aggregates tested yield the results shown in Table 6.3. It can be seen that those aggregates with an average slope of less than 0.5%/million are typically the cement modified materials, while good quality Canterbury greywackes and Auckland basalts without cement have high predicted lives with average RLT slopes of less than 0.5%/million.

Table 6.3 Traffic loading limits calculated using Equation 3 and average slope from the six-stage RLT test.

Material description*	RLT average slope 25k to 50k (%/million)	Million ESA (Equation 3)
2% Cement treated basecourse	0.09	17.2
CAPTIF 1: TNZ M/4 AP40 course grading greywacke - dry	0.18	15.3
2% Cement modified (7 days' hydration in stockpile)	0.19	15.1
CAPTIF 1: TNZ M/4 AP40 course grading greywacke - wet	0.19	15.0
TNZ M/4 AP40 Basalt - dry	0.20	14.8
CAPTIF 1: TNZ M/4 AP40 fine grading greywacke - dry	0.21	14.8
TNZ M/4 AP40 Basalt - dry	0.22	14.5
4% Cement modified (7 days' hydration in stockpile)	0.24	14.2
TNZ M/4 AP40 basalt - wet	0.25	14.0
CAPTIF 1 + 10% crushed glass - wet	0.25	13.9
CAPTIF 1 + 20% crushed glass - wet	0.29	13.2
TNZ M/4 AP40 Basalt - wet	0.39	11.6
2% Cement modified (7 days' hydration in stockpile)	0.59	8.8
CAPTIF 1: TNZ M/4 AP40 fine grading greywacke - wet	0.59	8.8
2% Cement modified (7 days' hydration in stockpile)	1.14	4.2
TNZ M/4 AP40 invovled in early failures	1.46	2.8
TNZ M/4 AP40 involved in early failures	1.48	2.7
TNZ M/4 AP40 involved in early failures	2.21	1.0
TNZ M/4 AP40 involved in early failures	2.62	0.6
TNZ M/4 AP40 involved in early failures	2.72	0.5
TNZ M/4 AP40 involved in early failures	3.53	0.2
TNZ M/4 AP40 involved in early failures	6.22	0.0

* The aim is to keep the source confidential

7 Discussion and conclusions

7.1 Discussion

The classification procedure developed was based on rut depth prediction using the Arnold/Nottingham method (Arnold 2004) that was validated against CAPTIF tests. Many assumptions have been made in this procedure and these have been applied consistently to all the six-stage RLT tests conducted so that the materials' predicted performances can be compared. This consistent process has meant the materials tested could be ranked, and if an assumption is changed in the rut depth prediction method then it is expected that a similar ranking would result. For specification purposes, a simplified means of RLT test method analysis was developed. It was found that average slope gave the best predictor of traffic loading limits and that the cumulative permanent strain achieved in the first 25 000 load cycles did not improve the result. This is a good result as, usually, the first 25 000 load cycles in each stage are prone to differences in compaction and sample preparation. Testing results will be more repeatable by using the slope in the later part of the test from 25 000–50 000 load cycles.

The rut depth prediction method developed by Arnold/Nottingham should not altogether be excluded now that a simple method of analysis has been developed. Using the rut depth prediction method may achieve a higher life than that approximated using the average slope only, as the full shape of the curve is considered in extrapolation.

A soundness check on Table 6.2 as detailed in Table 6.3 shows that good quality M/4 basecourses, which are the cement modified aggregates, basalts and Canterbury greywackes, have design traffic loading limits greater than eight million ESA. This gives confidence in the limits chosen, as they will not rule out the use of these good quality aggregates on highly trafficked roads.

7.2 Conclusions

This research project undertook many RLT tests on a range of aggregates used in New Zealand on an initial four-stage RLT test and using the AUSTROADS method. Further RLT tests were undertaken using a new six-stage RLT test funded commercially and for other research projects. It was found that the new six-stage RLT test was the best predictor of pavement rutting and is recommended for use in specifications for determining a traffic loading limit based on average slope. The AUSTROADS test method did not rank the materials' performances as expected and was therefore not regarded as suitable. The initial four-stage RLT test was also considered unsuitable because it failed good quality aggregates that were expected to pass early in the testing process.

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