The effect of grading on the performance of basecourse aggregate

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Acknowledgements

Abbreviations and acronyms

CAPTIF	Canterbury Accelerated Pavement Testing Facility
DGABC	Dense-graded aggregate basecourse
ESA	Equivalent standard axle
MDD	Maximum dry density
NJDOT	New Jersey Department of Transport
OMC	Optimum moisture content
PSD	Particle size distribution
RLT	Repeated load triaxial
RLTT	Repeated load triaxial testing
UGMs	Unbound granular materials

Contents

Executive summary

This research project included a literature review, repeated load triaxial (RLT) tests and rut depth modelling to predict the rutting performance primarily for one basecourse aggregate at different gradings. Limited RLT results for another aggregate were added to supplement the results of the main test. The objective was to determine the extent to which the grading could be changed without affecting the rut depth performance found from RLT testing. Based on this knowledge, grading envelopes were established around the gradings of samples tested in the RLT apparatus where gradings within this envelope would give similar results in the RLT test. The grading envelope found from this project is very tight and it is likely that a wider grading envelope would be required for specifications. Wider grading envelopes can be established by quarry owners by undertaking several RLT tests to cover the full range of gradings expected to occur when developing an alternative in TNZ M/4. From a literature review and RLT test the following conclusions were found:

- Talbot's exponent n-value was a convenient way to describe the grading of an aggregate, where n-values >0.5 are a coarse grading while n-values <0.5 are considered fine.

- Current Transit New Zealand specification for basecourse aggregate has a Talbot's n-value varying from 0.4 to 0.6, while the majority of overseas roading authorities allow finer gradings with a Talbot's n-value of 0.3 to 0.6.

- Previous research has revealed that fine gradings have a lower strength when wet compared with coarse gradings, while the opposite is the case when dry and/or when a modifier like cement is added (ie fine gradings are best).

- The literature reported that finer gradings could be constructed with less segregation and lower total voids to minimise any further densification after opening a pavement to traffic.

- Vibrating hammer compaction tests for determining target densities for RLT testing indicated that the highest density occurred with an n-value of around 0.5 which is the same as the theoretical maximum density curve.

- RLT testing on a TNZ M/4 basecourse from Pounds Road, Christchurch at gradings with Talbot's n-values of 0.3, 0.4, 0.55 and 0.8 showed that the best performance (least rutting) in wet conditions was obtained with the coarse grading with an n-value of 0.8. The best performance in dry test conditions was obtained at a finer grading with an n-value of 0.3.
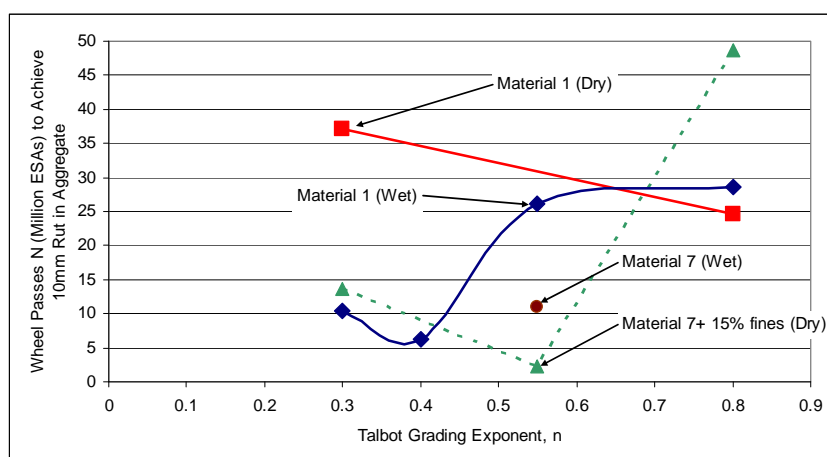
Figure E1.   Effect of Talbot grading exponent *n* (equation 1) on rutting performance for materials #1 and #7.

- The RLT test results showed that Talbot's *n*-values can vary by ±13% without greatly affecting the resulting rutting performance.

- Based on varying the *n*-value by ±13% and limiting the amount of fines new grading envelopes were recommended. These grading envelopes are about half the variation allowed for in the TNZ M/4 and RTA NSW 3051 specifications.

The result from this research project was a grading envelope around the grading of a material tested in the RLT apparatus where the results from the RLT tests and resultant rut depth prediction were nearly the same. This grading envelope can be used for establishing RLT testing criteria for determining grading envelopes suitable for production control for alternative materials approved for use as TNZ M/4 RLT regional variants.

Table E1.   Recommended grading envelope variations around a measured particle size distribution (PSD) for the RLT sample.

Sieve sizes	Acceptable variation around measured particle size distribution (PSD).
	%
37.50	-
19.00	± 5
9.50	± 5
4.75	± 4
2.36	± 3
1.18	the > of ± 14% of the measured PSD or ± 2
0.600	the > of ± 14% of the measured PSD or ± 1
0.300	the > of ± 14% of the measured PSD or ± 1
0.150	the > of ± 14% of the measured PSD or ± 1
0.075	the > of ± 14% of the measured PSD or ± 1

Table E1 shows two methods for calculating the envelope around the fine fraction. The first is ± 14% of the measured PSD and is suitable when the fine fraction is greater than 20 to 30% as the variation in grading is 2.8% and 4.2% respectively, which is a relatively

small variation around an already large quantity of fines. The second approach is limiting the grading envelope of the fines to an absolute value of ± 2% or ± 1% which is suitable for coarse materials where the fine fraction is less than 5% to nil. This is because performance is not affected by changes in fines of ± 1 or 2%, while keeping with a percentage variation around the PSD would result in an envelope of ± 0.3% which is unnecessarily tight, hence the smallest envelope is limited to ± 1%.

This research was based on limited testing by varying only the grading envelope while the plasticity of fines, shape control, moisture content and density would also have a large influence on RLT performance. Further testing would be required to consider the effect of these and in particular to establish more parameters around the quantity and quality (type) of fines.

Abstract

A laboratory study was undertaken to determine the effect of grading/particle size distribution on permanent deformation in multi-stage repeated load triaxial (RLT) tests. Results showed the coarse gradings with a Talbot's exponent n-value of 0.8 had the least amount of permanent deformation for high-moisture contents near saturation. Finer gradings with an n-value of 0.3 had the least deformation in all of the tests in dry conditions at less than 70% of optimum moisture content. Similar performance in terms of permanent deformation was obtained if variations of the n-value were less than 13%. Based on limiting the variations in n-value to ±13% new grading envelopes for use in the new Transit New Zealand specification for RLT testing were proposed.

1.   Introduction

Transit New Zealand has recently undertaken a research project, Performance Tests for Road Aggregates and Alternative Materials, funded by Land Transport New Zealand. The objective was to develop a practical repeated load triaxial (RLT) test that will enable pavement materials to be categorised according to their suitability as base materials for high, medium or low traffic situations in either dry or saturated conditions. This will ensure that pavement design life can be met; a range of materials including marginal materials (previously discarded) can be used in appropriate locations (ie low traffic and dry environments); and alternative materials (eg industrial by-products such as melter slag) and waste materials (eg glass, recycled crushed concrete) can be assessed and used appropriately according to their level of rut resistance in wet and dry conditions. This project has undertaken RLT tests on six different aggregate sources in New Zealand known to provide poor, average and good performance. As a result of these RLT tests, the aggregates will be boxed into various categories of low, medium and high traffic for both wet and dry conditions. Results of this categorisation will be compared with actual performance on the road (based on local knowledge) and at the Canterbury Accelerated Pavement Testing Facility (CAPTIF) to validate and refine the method of performance assessment.

To enable a base material that had passed the proposed RLT test method to be used in road construction a grading envelope was required to allow for material variations in the field while not compromising performance/rut resistance. It was not known to what extent slight grading variations in material used in the field compared with that used in the RLT apparatus would affect performance/rut resistance. There was a risk of early rutting failure due to the material delivered on site having a different grading from the one tested and approved for use in the RLT apparatus. Tests at a range of gradings expected to occur in the field needed to be conducted. The number of tests would depend on what could be practically controlled in production and the grading envelopes that could be established around one RLT test result from the above research project.

The project discussed in this report was undertaken to repeat the RLT tests conducted on two aggregates in the Performance Tests for Road Aggregates and Alternative Materials project at two gradings either side of the one originally tested. The objective was to determine the maximum variation in grading where changes in strength/rut resistance would still be acceptable for the material's classified use. Results in terms of predicted rut depth were determined and compared with the original grading. Interpolation was used to determine the grading boundary where the material would still be acceptable for use, either in low, medium or high traffic in wet or dry conditions. A methodology for determining the grading limit boundaries was determined as a result of this research and included in the material specification.

2.   Literature review

2.1   Introduction

A literature search was undertaken, using the resources of the Engineering Library at the University of Canterbury, on the effect of aggregate grading on performance/rutting as measured in road trials and/or the laboratory. The internet and overseas contacts were also used to determine aggregate grading envelopes used throughout the world for basecourse.

2.2   History

The Romans can be considered as pioneers in road construction. They established a vast network of roads throughout Europe primarily for military purposes. The layout seldom varied; two trenches were dug 5 m apart to act as drainage ditches and the soil between was excavated down to a firm foundation on which a multilayer granular base was laid using materials locally available (Croney and Croney 1991). The thickness of pavement was modified to take account of the strength of the soil foundation. In most cases the pavements built by Romans in other parts of Europe were surfaced with flat quarried stone to maintain the common appearance familiar to visitors from Rome.

After the Romans, roads were needed to transport surplus crops to barter with other villages. This led to the development of small market towns surrounded by satellite village communities. Long-distance coach travel became important with the growing wealth of the 17th and 18th centuries. This necessitated more permanent roads between towns. Roads were either a bed of comparatively large pitched stone with dry fines added to fill the gaps, as preferred by Thomas Telford, or were constructed with smaller angular material watered to assist granular interlock, as preferred by John Louden Macadam (Lay 1984).

The modern pavement of today is derived from macadam with a base of small angular rock material (granular) with a uniform grading overlying the soil foundation. But it is unlike macadam as it is often surfaced with a smooth waterproof bituminous layer. In New Zealand the surfacing is thin, so that the granular material with a uniform grading (a range of aggregate sizes from large to small in the right proportions to form a tightly packed dense material) in the base layer provides the necessary structural strength and resistance to shear/rutting failures from wheel loads.

2.3   Theory

Grading or gradation is the particle size distribution of the soil/aggregate or granular material. A dense grading is one in which each set of particle sizes fits snugly into the space left between the particles of the next largest size. A uniformly graded aggregate is one with a predominance of single-sized particles like stones used in chip seals. Gap or skip-graded aggregates have at least one size range of particles missing. An open-graded mix is one that contains only small amounts of fine aggregate. A dense grading is generally targeted by

most road controlling authorities for aggregate base materials due to its strength and stability from interlocking particles and fewer voids. Having fewer voids means less further densification can occur. In addition to a dense grading the aggregate sizes need to be derived from crushing to enable a strong interlock with the angular/crushed faces. Rounded stones do not interlock and will quickly deform under a wheel.

To ensure a dense grading, road controlling authorities specify grading envelope curves in their aggregate base specifications. Particle size distribution curves or grading curves are produced by plotting particle diameter in millimetres on a negative log scale against the percentage by mass of material smaller than the diameter. Following this definition the particle size distribution is expressed by Talbot's grading curve represented by the value of the exponent *n* in equation 1.

$$p = 100 \left(\frac{d}{D} \right)^n$$ Equation 1

Where p = percent passing sieve size d
    D = maximum particle size and
    n is an integer which commonly has a range between 0.3 (fine grading) and 0.6 (coarse grading).

In most specifications *n* is found to be between 0.3 (fine grading) and 0.5 (coarse grading) with values of 0.45 to 0.5 indicating better compactibility (Lay 1984). However, an experienced New Zealand engineer (Bartley 2007) is of the opinion that better compactibility is achieved with *n*-values of 0.35 as the fine-graded materials can pack down more readily to form a dense grading. A grading with *n* = 0.5 is historically known as Fuller's maximum density curve, although the actual maximum density may occur at *n*-values less than 0.5 for some aggregates. Aggregates containing mainly coarse particles have a low *n*-value, commonly known as coarse graded. Coarse-graded aggregates are difficult to handle, compact and unable to achieve their potential strength. However, coarse-graded aggregates from crushed rock of high crushing strength, if compacted properly, can result in strong interlock due to stone-on-stone contact and as they are lacking in fines they are permeable and less susceptible to failure caused by water.

A material with excessive small particles (low *n*) that are plastic clay materials will have low strength, be very sensitive to moisture and will have a surface which is slippery when wet, and dusty when dry. However, it will be easy to handle and compact as well as impermeable. It is common to add fine material (fines) from crushing or other rock sources that are non plastic and thus not sensitive to moisture to crushed stone in order to meet grading requirements and permit easier compaction.

Another important factor in aggregate gradings is the coefficient of uniformity of soil (Krebs and Walker 1971, cited in Lay 1984). This is defined as d_{60}/d_{10} where d_n is the diameter at which *n*% by mass of the soil is passed during sieving. A value of about 5 is the border between a good and bad grading. However, this measure was reported by the

authors (Krebs and Walker 1971, cited in Lay 1984) as not always 'full-proof' as a gap-graded mix can have a deceptively good coefficient of uniformity. The d_{10} diameter fraction has a major influence on permeability, cohesion and moisture sensitivity. A large amount of fines when wet can become a lubricant between the larger crushed stones and reduce the shear strength of the material.

2.4   International specifications

A range of unbound road aggregates base specifications used on major highways throughout the developed world were reviewed. Relevant specifications were found from the following countries: Germany (ZTVT-StB 95) (Bundesministerium für Verkehr 1995); Sweden; South Africa's G3 aggregate (COLTO 1989); Canada (City of Winnipeg 2006); State of Victoria, Australia (VicRoads 1998); South Australia – class 3 quarried pavement material (Transport SA 2007); Western Australia (City of Geraldton 2004); Sub-base specification, United Kingdom (Highways Agency 2004); Chile; and Finland. The upper and lower grading envelopes are plotted in Figures 2.1 and 2.2.
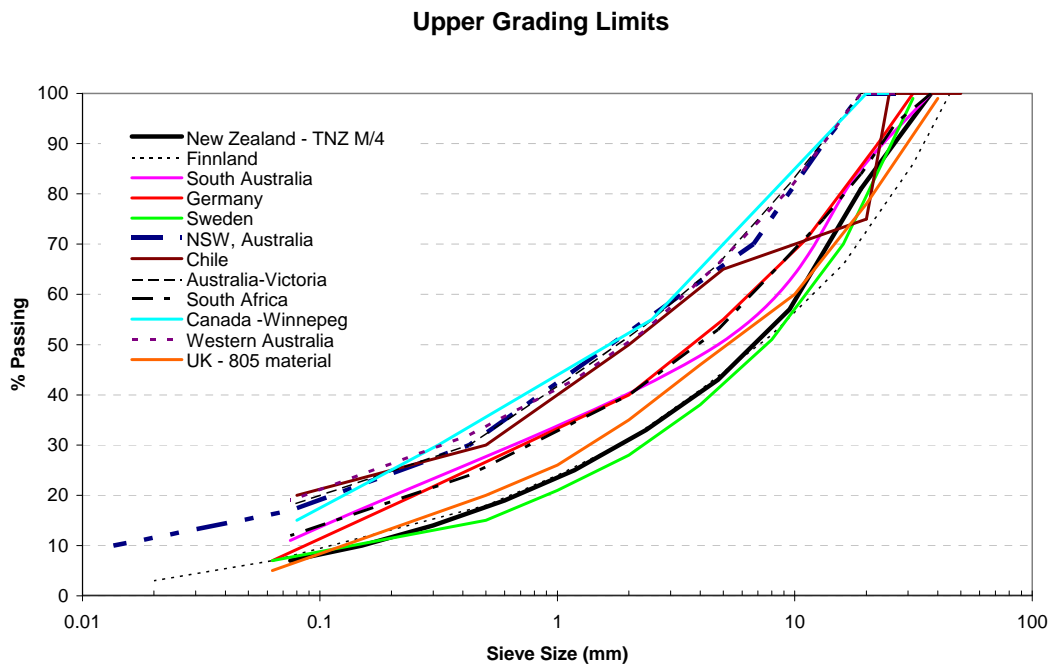
Upper Grading Limits

Figure 2.1   Upper grading limits for basecourse aggregate specifications.
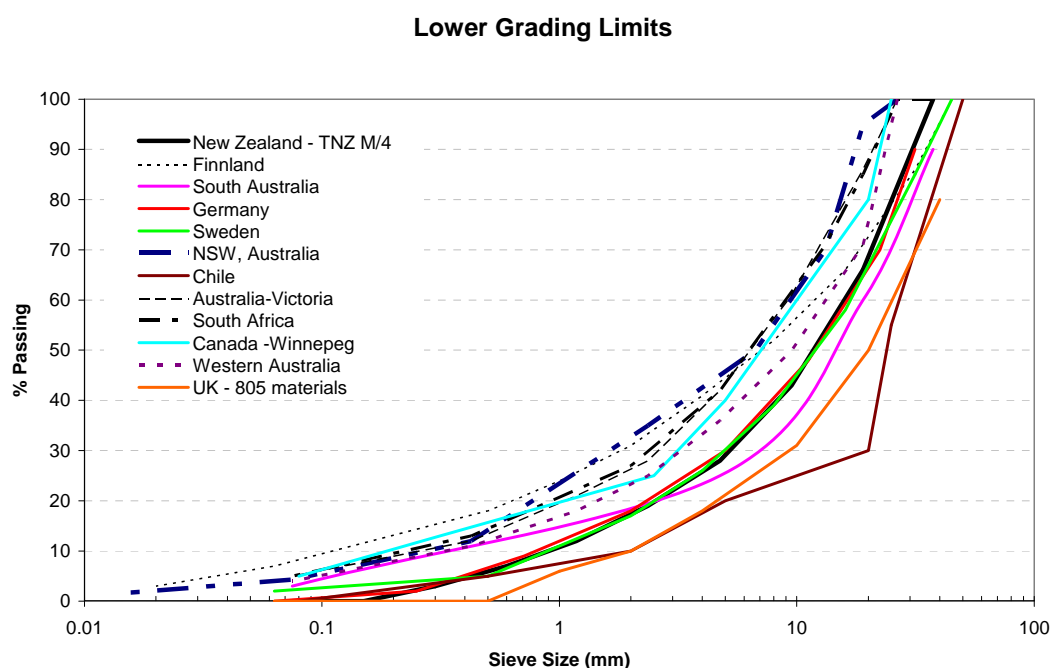
Lower Grading Limits

Figure 2.2   Lower grading limits for basecourse aggregate specifications.

Talbot's exponent *n* (equation 1) was estimated for the various aggregate gradings throughout the world and these are summarised in Table 2.1. This shows the New Zealand TNZ M/4 basecourse has an *n*-value ranging from 0.4 to 0.6 while the other gradings generally fall between 0.3 and 0.6 with a few exceptions. It should be noted when reviewing Table 2.1 that often there is not a very good fit between actual grading limits and there is a curve defined by equation 1.

Table 2.1   Talbot's exponent *n* (equation 1) values for selection of aggregates throughout the world.

	Talbot's exponent *n*-values	
	Lower limits (coarse side)	**Upper limits (fine side)**
New Zealand	0.61	0.4
Finland	0.38	0.38
Germany	0.58	0.33
South Australia	0.61	0.37
Sweden	0.55	0.47
United Kingdom	0.68	0.35
Chile	0.95	0.3
Canada	0.58	0.3
Australia–Victoria	0.5	0.3
Australia–Western Australia	0.57	0.3
South Africa	0.5	0.31
RTA NSW, Australia	0.5	0.32

2.5　　Effect of grading on aggregate permanent deformation

Thom and Brown (1988) studied the behaviour of crushed limestone material at different gradings and arrived at the conclusion that the resistance to permanent deformation decreased with increasing fines content. This could be explained by the assumption that the entire fines fraction does not necessarily fit into the pore spaces between the large particles. Therefore a skeleton of larger particles in full contact does not exist. The fines act as a lubricant between the stones. As a result the resistance against permanent deformation and stiffness decreases with increasing fines content.

Tampere University in Finland (Kolisoja 1998) found from large-scale dynamic triaxial tests that if the grading was changed so the relative density increased, then resistance to permanent deformation would rise. Significantly higher permanent strains could be expected for aggregates containing extremely high fines content (eg d < 0.074 mm > 15 %) or at a low content of fines.

Belt et al. (1997) undertook permanent strain RLT tests on a range of aggregate gradings varying the Talbot's exponent *n*-value (equation 1 and Figure 2.3). Results are shown in Figure 2.4 where the lowest permanent deformation was found at a fine grading of *n*=0.4, which was finer than the theoretical highest density grading of *n* = 0.5.
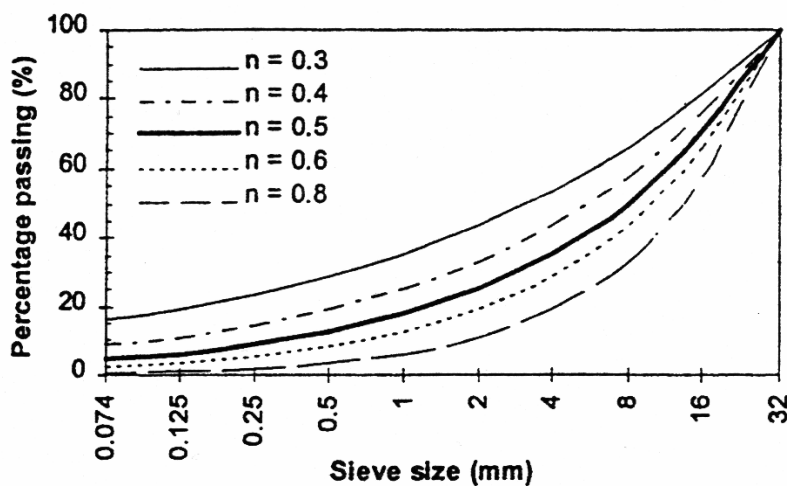

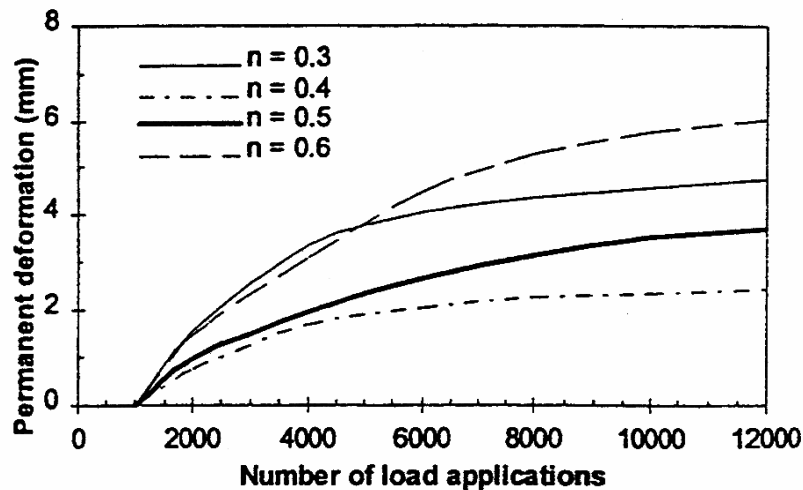**Figure 2.3　　Grading curves of aggregates tested by Belt (1997).**

Figure 2.4   Effect of grain size distribution on the aggregate's susceptibility to permanent deformations (Belt 1997).

Van Niekerk (2002) recognised in RLT tests that unbound granular materials (UGMs) with a more balanced grading performed better than the more uniformly graded materials. It is worth emphasising that the experimental observations on the effects of the gradings presented above are mainly based on the results of test series performed with RLT equipment. The RLT test applies user-defined loads and stresses at a particular sample moisture content and density. For example, the RLT test does not take into account the effects of freeze thaw where it has been found a coarser aggregate lacking in fines performs best. Furthermore, dilatant behaviour of UGMs will cause a discrepancy between experimental observations using the RLT test and the behaviour in the real pavement structure. Dilatant behaviour allows heavily stressed elements to build up an increasing confinement and thus reduce permanent deformation. Van Niekerk (2002) recognised that finer gradings showed less resilient dilatant behaviour (particles could resiliently rearrange under loading) than a coarse skeleton structure (particles would resiliently wedge sideways under loading).

The New Jersey Department of Transport (NJDOT) undertook a project on the performance of aggregates (base and sub-base materials) with a focus on gradation ranges (Bennert and Maher 2003). Resilient modulus and permanent strain RLT tests were conducted and analysed using the materials with different gradations. It was found that different materials behaved differently in regard to the change in the grading. Permanent strain RLT tests were conducted on three different aggregates at four different gradings (high end or coarse side; middle range; low end or fine side; and natural or as received). Permanent strain testing was for 100,000 cycles of a 310 kPa haversine pulse with a 103 kPa confining pressure on a sample compacted using a heavy hammer at its respective maximum dry density and optimum moisture contents. Figures 2.5 and 2.6 show the permanent strain-values at the end of the RLT test. Grading limits in New Jersey are very wide and based on their limits (Tables 2.2 and 2.3), the *n*-values were calculated for the different gradings tested, although the natural gradings were not reported.

Table 2.2   NJDOT grading curves for DGABC (dense-graded aggregate basecourse) (modified from Bennert and Maher 2003).

	NJDOT – DGABC - Percent passing		
	High end or coarse side	Middle range	Low end or fine side
Estimated *n*-value (equation 1):	0.83	0.51	0.31
Sieve size (mm)			
25.4	100%	100%	100%
19.1	55%	72.5%	90%
4.76	25%	42.5%	60%
0.3	5%	15%	25%
0.074	3%	7.5%	12%

Table 2.3   NJDOT grading curves for I-3 (sub-base aggregate) (modified from Bennert and Maher 2003).

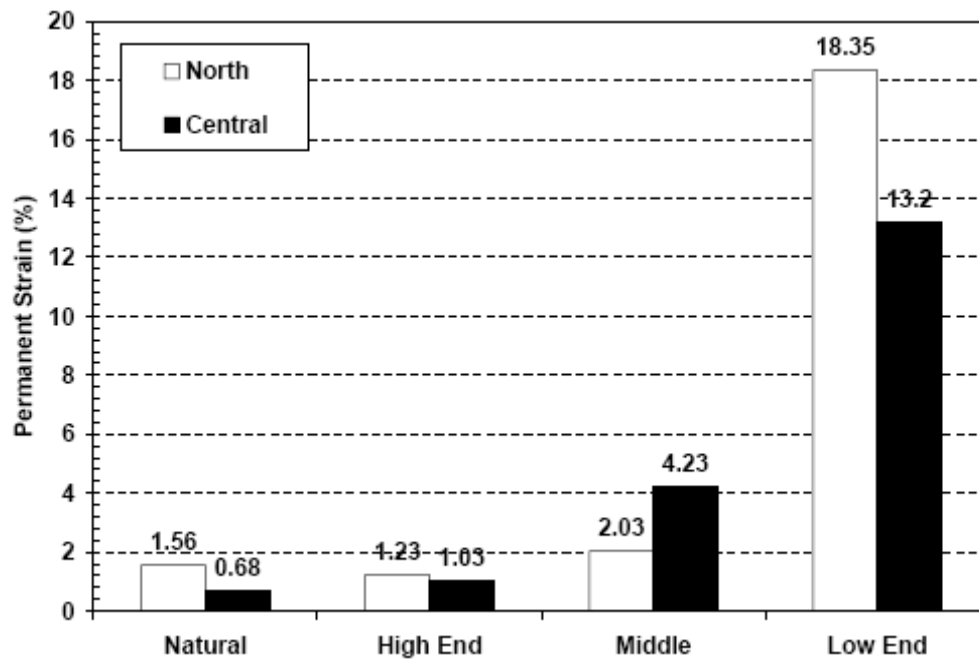|	NJDOT – I-3 - Percent passing		
	Low end or coarse side	Middle range	High end or fine side
Estimated *n*-value (equation 1):	0.61	0.28	0.39
	Note: Grading was a very poor fit to equation 1 and therefore n-*values may be misleading.*		
Sieve size (mm)			
101.6 mm	100%	100%	100%
19.1	60%	80%	100%
4.76	30%	65%	100%
0.3	5%	20%	35%
0.074	0%	4%	8%

Figure 2.5 – RLT permanent strain result for NJDOT DGABC (Table 2.2) (source Bennert and Maher 2003).
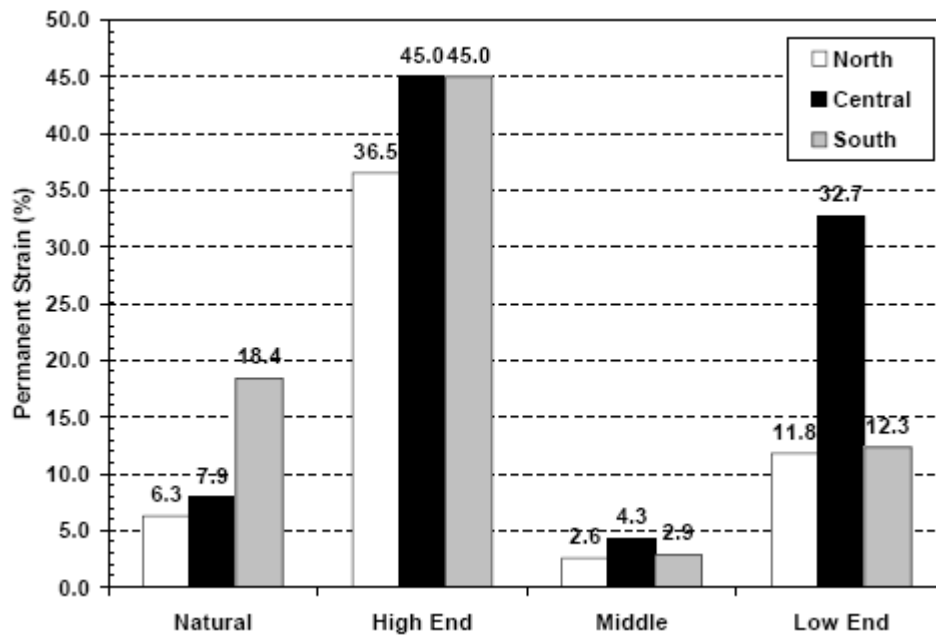
Figure 2.6   RLT permanent strain result for NJDOT I-3 (Table 3.3) (source Bennert and Maher 2003).

The RLT study by Bennert and Maher (2003) showed that the DGABC gradings at the high end performed best, with the middle grading showing adequate performance, while at the fine end very high deformations occurred. Results for the sub-base showed the grading in the middle of the envelope was best for all three aggregates. This result is not surprising given the large range in particle size distributions between the coarse and fine materials.

Thom and Brown (1988) conducted RLT permanent strain tests on crushed dolomitic limestone at a range of gradings and compaction level. They found when uncompacted, the specimens with uniform grading resulted in the least permanent strain (Figure 2.7).
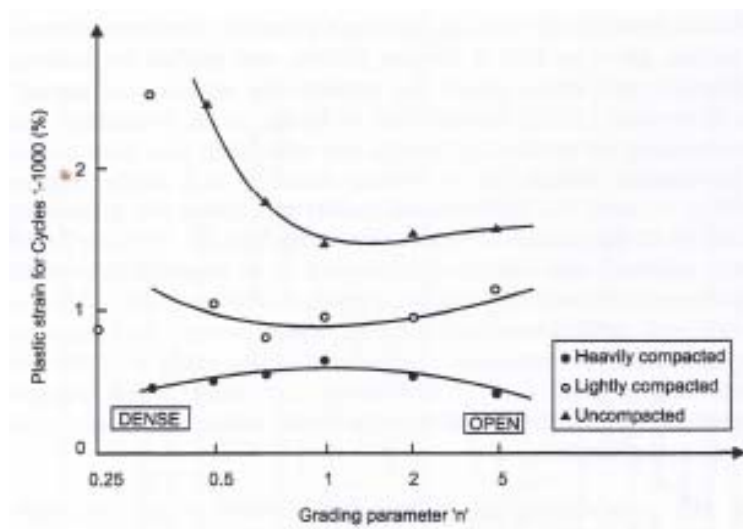

Figure 2.7   Effect of grading and compaction on plastic strain (Thom and Brown 1988).

The effect of increasing fines content generally increases the magnitude of deformation in RLT permanent strain tests (Barksdale 1972, 1991; Thom and Brown 1988). Dodds et al. (1999) confirmed this result as the material with 10% fines added showed the highest deformations. Pore water pressure development was observed by Dodds et al. (1999) to be higher for materials with higher fines content. Pore water pressure reduced the effective stress and thus reduced the beneficial level of confinement.

Thompson and Smith (1990) looked at the resilient modulus of seven Illinois granular materials, including two crushed stones, two crushed gravels, two gravels and a partially (30–35%) crushed gravel. The partially crushed gravel and one material from each of the other types conformed to one gradation specification, while the remaining three materials conformed to another. The two gradation specifications differed only in the percentage passing the 75 μm sieve. The one with the lower fines content required 0–4% passing 75 μm whereas the higher fines content required 4–8% passing 75 μm. The materials were tested at or near the maximum dry density, which varied from 1962 kg/m^3 (low fines content crushed stone) to 2300 kg/m^3 (high fines content crushed stone) and the optimum moisture content, which varied from a low of 4% (low fines content crushed stone) to a high of 9% (partially crushed stone, high fines content). At a bulk stress of 138 kPa (20 psi), their resilient modulus results ranged from a high value of 244 MPa for the crushed stone having the higher fines content to a low of 134 MPa for the partially crushed gravel, which also had the higher fines content. Looking at the pairs of similar material, in all cases, the material with the higher fines content had a slightly higher modulus. How much of the difference is attributable to the different fines content, and how much is attributable to different moisture and density conditions cannot be discerned from the data.

Along with the quantity of fines the type of fines has a significant affect on rut depth performance. Theyse (2002) found that increasing the non-plastic fines content up to 9% in a crushed rock base increased the cohesion and shear strength. However, replacing these fines with plastic/clay fines had the opposite effect and significantly reduced the cohesion and shear strength.

Arnold (2004) found in his doctorate studies that resilient modulus was higher for an aggregate with 10% clay fines added at 70% of optimum moisture content. However, results from the multi-stage permanent deformation RLT test and associated rut depth modelling clearly showed the aggregate with 10% clay fines resulted in the highest rut depth.

Walaa and El Hussein (2004) studied the effects of clay fines content and moisture content on resilient modulus and permanent strain measured in RLT tests. At moisture contents dry of optimum the resilient modulus increased while the permanent strain decreased (Figures 2.8 and 2.9). Although for moisture contents greater than optimum Walaa and El Hussein found that higher fines contents caused greater permanent strain (Figure 2.10).
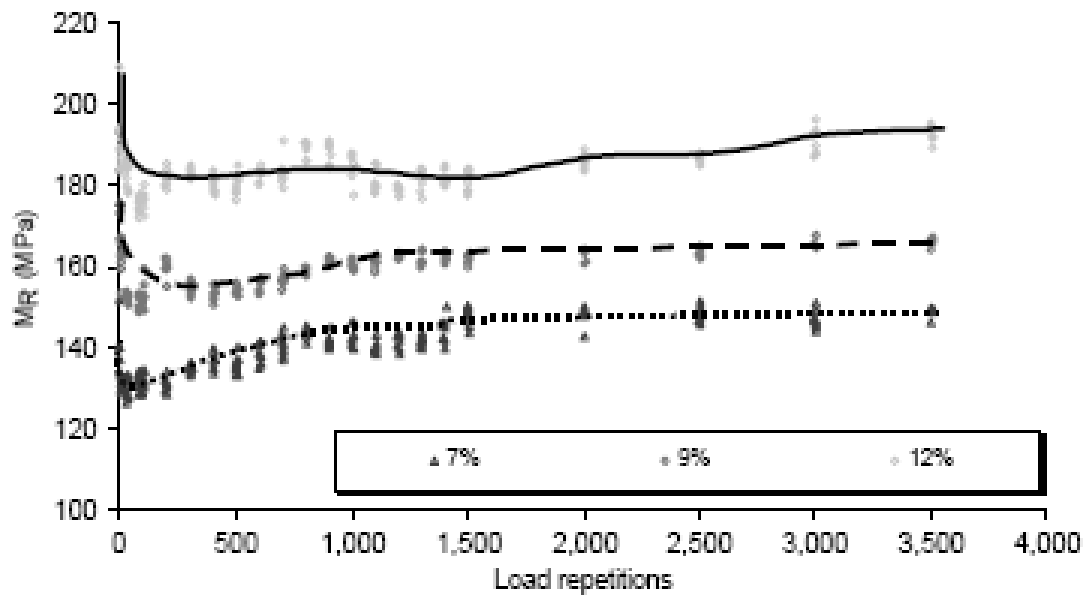
Figure 2.8   **Effect of fines content on resilient modulus (Walaa and El Hussein 2004).**
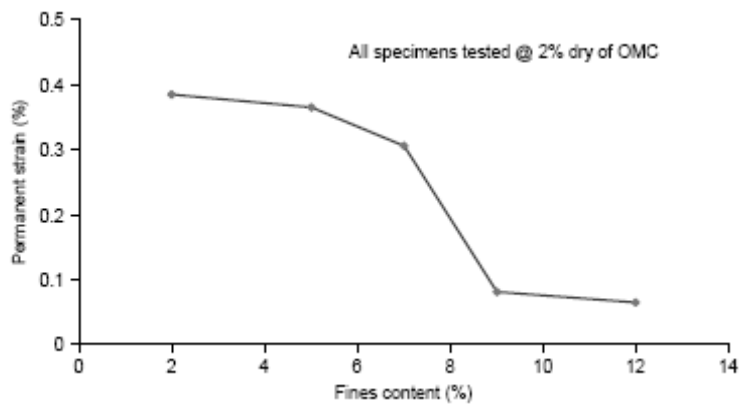

Figure 2.9   **Effect of fines content on permanent strain (Walaa and El Hussein 2004).**
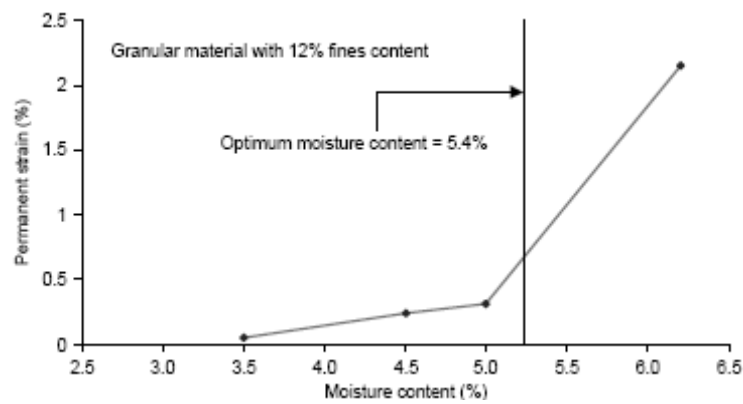

Figure 2.10   Effect of moisture content on permanent strain for an aggregate with 12% clay fines (Walaa and El Hussein 2004).

21

A RLT study looking at freeze-thaw effects on granular materials (Kolisoja et al. 2002) found the modulus values of dry samples increased as the fines content increased. However, in samples which had absorbed water or those which had gone through a freeze-thaw cycle, the modulus values instead tended to decrease as the fines content increased. Permanent strain tests on the same samples prior to a freeze-thaw cycle found the permanent strain clearly increased with increasing fines content.

Elastic and plastic RLT and failure tests were conducted and analysed on unbound mixes using different gradations between the two extreme limits of the envelope currently used in the United Kingdom (Kamal et al. 1993). It was found that the resistance to permanent deformation generally increased for well-graded materials compared with open-graded (coarse-grading) materials. The stiffness also increased from the finer to the coarser end of the envelope.

The results of Monismith et al., reported in FHWA (2006), showed an increase in modulus as the fines content increased from a coarse (2–3% fines) to a medium (5–6% fines) gradation for both crushed and partially crushed materials. However, a decrease in modulus occurred as the fines content was further increased from a medium gradation to a fine gradation (8–10% fines).

Ishibashi et al., reported in FHWA (2006), investigated four soils. Two were gravelly, silty, sand-base materials differing only in fines content. Two were subgrade materials, which were identical but for the substitution of a sandy silty clay material for the minus number 8 fraction (2.36 mm sieve) of the original gravelly sandy silt in one soil. The materials having the highest fines content exhibited the lowest modulus for both the base and subgrade materials.

Early research work at CAPTIF by Seddon, reported in Salt (1977) looked at the effect of grading percentage broken faces and fines content on rutting and densification of basecourse under wheel loading. Aggregate from Miners Road Quarry in Christchurch and Stevensons Quarry in Drury was used in the trial. This report concludes that the most adequate means of predicting susceptibility of aggregate to excessive densification is examination of the shape of the particle size distribution curve (*n*-value). The sand equivalent test which reputedly determines the amount of undesirable fines in aggregates was found to correlate poorly with basecourse performance. Table 2.4 shows some results from Salt (1977) and it can be seen that both aggregates end up at the same amount of total voids regardless of the initial total voids. The denser gradings with *n*-values of 0.5 and 0.4 started with the lowest amount of total voids of 14% and hence had the least amount of further densification showing as surface rutting.

Table 2.4   Aggregate gradation effect on densification after trafficking (Appendix 1, Salt 1977).

Aggregate	Total voids %	
	Initial	After 16,300 EDAs (wheel passes)
[1]Drury – *n*-value = 0.65	16	10.5
Drury – *n*-value = 0.55	17	10
Drury – *n*-value = 0.5	14	10
[2]Christchurch – 95%CF – *n*-value = 0.65	15	7
[2]Christchurch – 94%CF – *n*-value = 0.5	16	8
[2]Christchurch – 94%CF – *n*-value = 0.4	14	8

1 Note all Stevensons Drury material is a quarried product with 100% crushed.

2 CF = Crushed faces

2.6   Physical properties of aggregate particles

Although not the prime purpose of this study the literature review was extended to include the physical properties of aggregate particles such as grain shape and particle surface roughness as these affect permanent deformation behaviour.

2.6.1   Grain shape

A significant factor influencing the grain shape of coarse-grained aggregates is the mineralogical composition of the aggregate particles. The crushing technique used to produce the aggregates affects the grain shape of the crushed materials. The grain shape can be defined into two general groups: the first is composed of natural sands and gravels and the second of crushed materials. In the first group, particle contact is between two smooth surfaces, while in the second group (ie crushed aggregates) the particle edges can be very sharp. This difference between the natural aggregate having rounded grains and crushed aggregate having sharp-edged grains is of most significance in long-term and permanent deformation behaviour. The crushed material is likely to have more grain abrasion than the natural aggregate, especially at high stresses.

The two tests used in basecourse specifications to control the grain shape are the Flakiness Index and the percentage of broken faces. The Flakiness Index is a measure of the length to width ratio of aggregate particles. The Transit New Zealand specification for basecourse aggregate (TNZ M/4) requires the aggregate broken face content (two or more broken faces as per NZS 4407: 1991, Test 3.14 Broken Face Test) in the aggregate fractions between the 37.5 mm and 4.75 mm sieves to be not less than 70% by weight.

2.6.2   Surface roughness of the particles

Many investigations have addressed the effect of macro- and micro-roughness of particles on deformation behaviour (Thom and Brown 1989; Cheung 1994). The surface friction angle can be measured using a sliding type test performed by pulling a representative specimen of aggregate loaded on a rough test surface (Cheung 1994). The surface friction at the contact points of particles can be assumed to affect the resilient deformation behaviour, especially

when the external load reaches the value which makes the particles slide. It should be mentioned that the surface friction angle in the friction test proposed by Cheung (1994) does not necessarily correlate with the visual assessment of surface roughness. However, the macro-level surface roughness (visible roughness of the individual particles based on the number of protrusions in the surface) probably correlates better with the ability of the material to resist permanent deformations (Thom 1989).

2.6.3   Effect of water

High pore water pressures that reduce the amount of suction will occur in granular materials in the laboratory and field due to a combination of a high degree of saturation, low permeability and poor drainage. These conditions result in lowering the effective stress and consequently reducing stiffness and resistance to deformation (Haynes and Yoder 1963; Barksdale 1972; Maree et al. 1982; Thom and Brown 1987; Dawson et al. 1996). Haynes and Yoder (1963) found the total permanent strain rose by more than 100% as the degree of saturation increased from 60 to 80%. A more modest increase of 68% in permanent strain was observed by Barksdale (1972) for tests on soaked specimens compared with those partially saturated. Thom and Brown (1987) reported that a small increase in water content could trigger a dramatic increase in permanent strain. Analysing RLT permanent strain tests from Dodds et al. (1999) found the aggregate contaminated with 10% clay fines did not survive for 100,000 loading cycles when saturated, while the aggregate free from clay fines did survive.

2.6.4   Effect of fines

The effect of increasing fines content generally increases the magnitude of deformation in RLT permanent strain tests (Barksdale 1972, 1991; Thom and Brown 1988). Dodds et al. (1999) confirm this result as the material with 10% fines added showed the highest deformations. Pore water pressure development was observed by Dodds et al. (1999) to be higher for materials with higher fines content. Pore water pressure reduces the effective stress and thus reduces the beneficial level of confinement.

As found by Theyse (2002) and discussed earlier, the type of fines, either plastic (clay) or non-plastic (crusher dust/sand), affects the performance of an aggregate in different ways. Non-plastic fines up to 9% increase shear strength while any quantities of plastic fines decrease strength. Hence TNZ M/4 limits the quantity and type of fines.

2.6.5   Effect of density

Results for RLT permanent strain tests by Dodds et al. (1999) showed in all cases larger permanent strains for the samples compacted to a low density of 90% of MDD (maximum dry density with vibrating hammer, BS 1377–4: 1990) compared with samples compacted at 95% of MDD. Other researchers reported that the resistance to permanent deformation under repetitive loading improved as a result of increased density (Holubec 1969; Barksdale 1972, 1991; Allen 1973; Marek 1977; Thom and Brown 1988). Barksdale (1972) observed an average of 185% more permanent axial strain when the material was compacted at 95% instead of 100% of the maximum compactive density obtained from the drop hammer test.

2.7   Summary of literature review

Generally, the results from the literature show that the mid-range to the coarse side provide the best results in terms of resistance to permanent deformation/rutting. There are exceptions to this where dry materials with high fines contents can reduce the amount of permanent deformation. Resilient modulus results are more scattered and tend to increase with increasing fines contents. This increase in modulus for higher fines contents is probably caused by the increase in density due to the fines filling the voids and thus giving less secondary densification. Also resilient modulus decreases and permanent strain increases with increasing water content. Finer gradings are more affected by increasing water content.

3.   Repeated load triaxial study

3.1   Introduction

Multi-stage RLT permanent strain tests following the procedure developed by Arnold et al. (2007) in stage 1 of this project were conducted on basecourse aggregates mixed to specified particle size distributions (gradings). The RLT apparatus applied repetitive loading on cylindrical materials for a range of specified stress conditions. The output was deformation (shortening of the cylindrical sample) versus the number of load cycles (usually 50,000) for a particular set of stress conditions. Multi-stage permanent strain RLT tests were 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 the computer program CIRCLY and in finite element models.

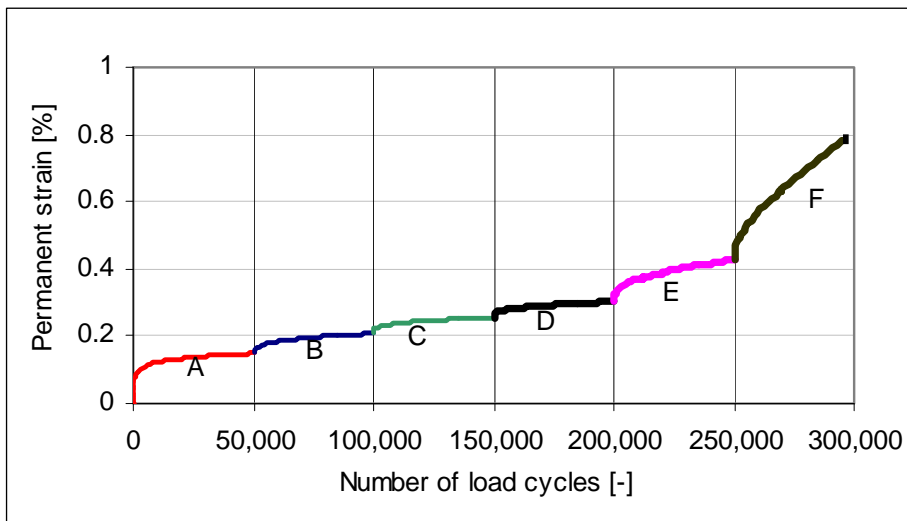**Figure 3.1   RLT apparatus and setup.**

Figure 3.2   Typical output from permanent strain RLTT (A, B, C ... etc denote each loading stage of the RLT test as detailed in Table 4.1).

The method developed by Arnold (2004) has been used in this study to interpret the results from RLT testing and to predict the rut depth in a pavement. This method has been refined and validated for a range of New Zealand basecourses as detailed in stage 1 of this project in the report titled, *Performance tests for road aggregates* (Arnold et al. 2007). The RLT test method is a six-stage test at 50,000 load cycles per stage as detailed in Table 3.1.

Table 3.1   RLT testing stresses for 6-stage test.

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					
Record and report data electronically in Microsoft Excel	Permanent strain versus load cycles and resilient modulus versus load cycles					

p = mean principle stress $(1/3*(\sigma_1 + 2*\sigma_3))$

q = principle stress difference $(\sigma_1 - \sigma_3)$

To predict the pavement rut depth from RLT tests required a series of steps. The first step was to develop a mathematical relationship between stress (both vertical and horizontal) with permanent strain rate (slope of each deformation curves (Figure 3.3), eg % deformation per 1 million load cycles).
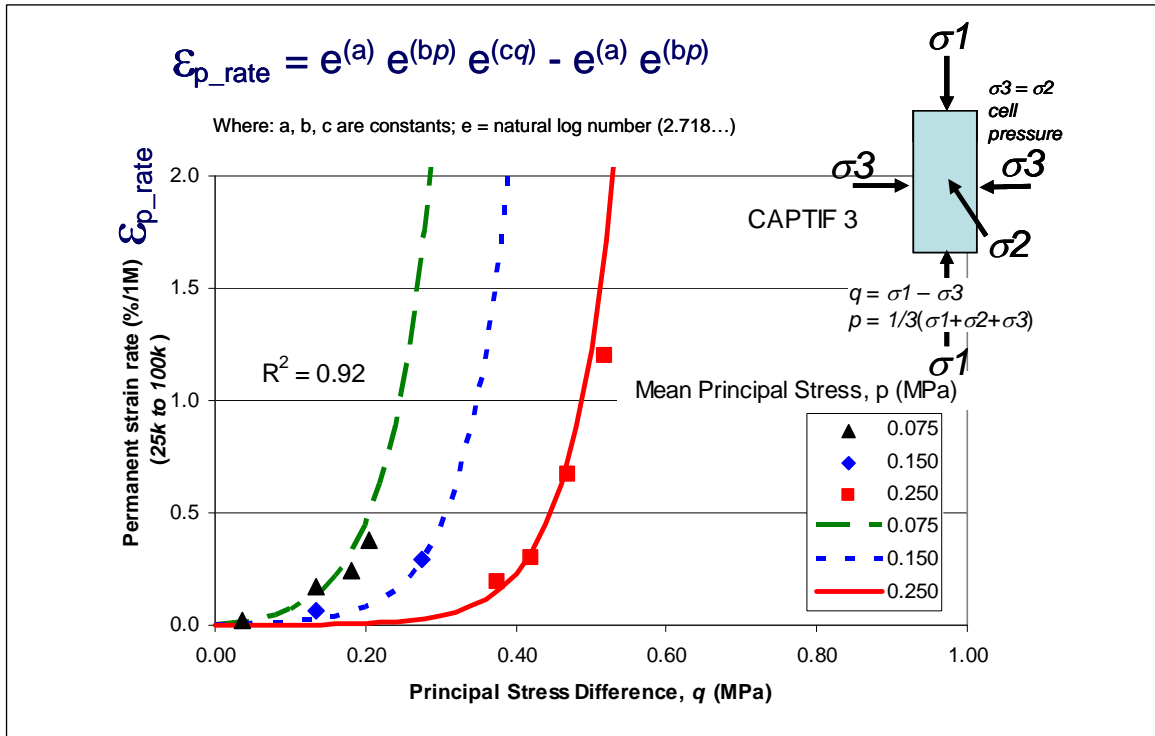
Figure 3.3   Fitting the permanent strain rate model to RLT test results.

The next step was to use a finite element model *(to model the non-linear elastic behaviour of a pavement material and to avoid discontinuities as occurs in CIRCLY which results in high tensile stresses)* to compute the stresses (both vertical and horizontal underneath a standard axle load (8.2 tonne dual tyred axle or higher if designing for ports etc). Stresses were exported into a spreadsheet to calculate the deformation rate at depth increments in the pavement from the permanent strain model (Figure 3.3). Results showed that very good predictions of rutting occurred at CAPTIF (Arnold 2004).

Pavement rut depth was predicted for all RLT tests on basecourse aggregates with a range of aggregate basecourse gradings.

3.2   Aggregates and grading ranges tested

The aggregate used primarily in this study was a TNZ M4 AP40 basecourse from Pounds Road Quarry in Christchurch. Tests at CAPTIF commonly use this basecourse due to the close proximity of the quarries to the site. Using this basecourse allowed the rut depth prediction models to be validated against actual rut depth measured in CAPTIF tests. An added advantage in using the Pounds Road basecourse was there have been many other RLT tests on this material which supplemented the results. The Pounds Road aggregate is referred to as material #1 or CAPTIF 1 as used in stage 1 of this research project.

After initial RLT tests on the Pounds Road basecourse it was decided to undertake further tests on a different aggregate from the upper North Island of New Zealand to see if the same pattern of results could be obtained. This aggregate will remain anonymous and in keeping with names chosen in stage 1 of this project the other aggregate shall be referred to as: material #7.

Talbot's exponent *n*-values (Equation 1) were used as a guide for calculating grading curves to test. Gradings targeted were *n*-values in the middle and just outside the limits of TNZ M/4 basecourse. These *n*-values were 0.55 (middle), 0.3 (fine side) and 0.8 (coarse side).

3.3   Vibrating hammer compaction tests

The standard for compaction of road basecourse is Transit New Zealand's specification TNZ B/2 Specification for the Construction of Unbound Pavement Layers. Compaction targets are a minimum of 95% of maximum dry density (MDD) and 70% of optimum moisture content (OMC). MDD and OMC are determined by a vibrating hammer compaction test in accordance with NZS 4402: Test 4.1.3. targets for repeated load triaxial testing are 95% MDD and 100% OMC (wet) or 70% OMC (dry).

Vibrating hammer compaction tests (NZS 4402: Test 4.1.3) were conducted for the range of aggregate gradings tested in the RLT apparatus. Table 3.2 and Figure 3.4 show the results of the compaction tests which indicate a definite trend in relation to the grading curve. The maximum density does appear to occur when the Talbot's *n* is 0.5 being the same value as the theoretical maximum density grading curve.

Table 3.2   Vibrating hammer compaction test (NZS 4402 : Test 4.1.3) results.

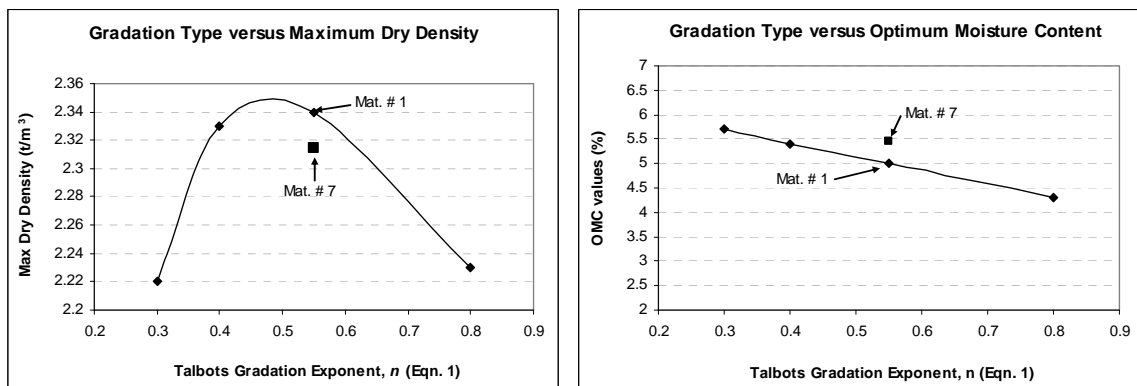| Material #	*n*	Maximum dry density (t/m^3)	Optimum moisture content (%)
1	0.3	2.22	5.7
1	0.4	2.33	5.4
1	0.55	2.34	5
1	0.8	2.23	4.3
7	0.55	2.31	5.45

Figure 3.4   Effect of Talbot's grading exponent, *n* (Equation 1) on maximum dry density (MDD) and optimum moisture content (OMC).

3.4   RLT results

RLT tests conducted including densities and moisture contents are summarised in Table 3.3. A test number reference is included to enable reference to the raw results shown in Figures

3.5 to 3.14. Interpretation of these results in relation to pavement rut depth prediction are summarised in the next section.

Table 3.3   Summary of RLT tests and identification.

University of Canterbury test reference #	Material # n (equation 1) value and wet (100%OMC targeted) or dry (70%OMC targeted)	Actual dry density achieved	Actual moisture content achieved
4	Mat. #1, n=0.8 (coarse), dry.	99% (2.20t/m³)	60% (2.6%)
5	Mat. #1, n=0.8 (coarse), wet.	97% (2.16t/m³)	84% (3.6%)
7	Mat. #1, n=0.3 (fine), wet.	96% (2.13t/m³)	83% (4.7%)
6	Mat. #1, n=0.3 (fine), dry	99% (2.20t/m³)	63% (3.6%)
17	Mat. #1, n=0.55 (middle), wet	98% (2.29t/m³)	71% (3.6%)
3	Mat. #7, n=0.55 (middle) + 15% fines (AP5), dry	96% (2.23t/m³)	62% (3.7%)
13	Mat. #7, n=0.55 (middle), wet	99% (2.28t/m³)	79% (4.3%)
8	Mat. #1, n=0.4 (fine), wet	96% (2.24t/m³)	77% (4.2%)
14	Mat. #7, n=0.3 (fine), dry	102% (2.26t/m³)	74% (4.6%)
15	Mat. #7, n=0.8 (coarse), dry	96% (2.13t/m³)	70% (3.3%)

Univ of Cant Test # 4
Material # 1, n=0.8 (coarse), dry (CAPTIF 1)
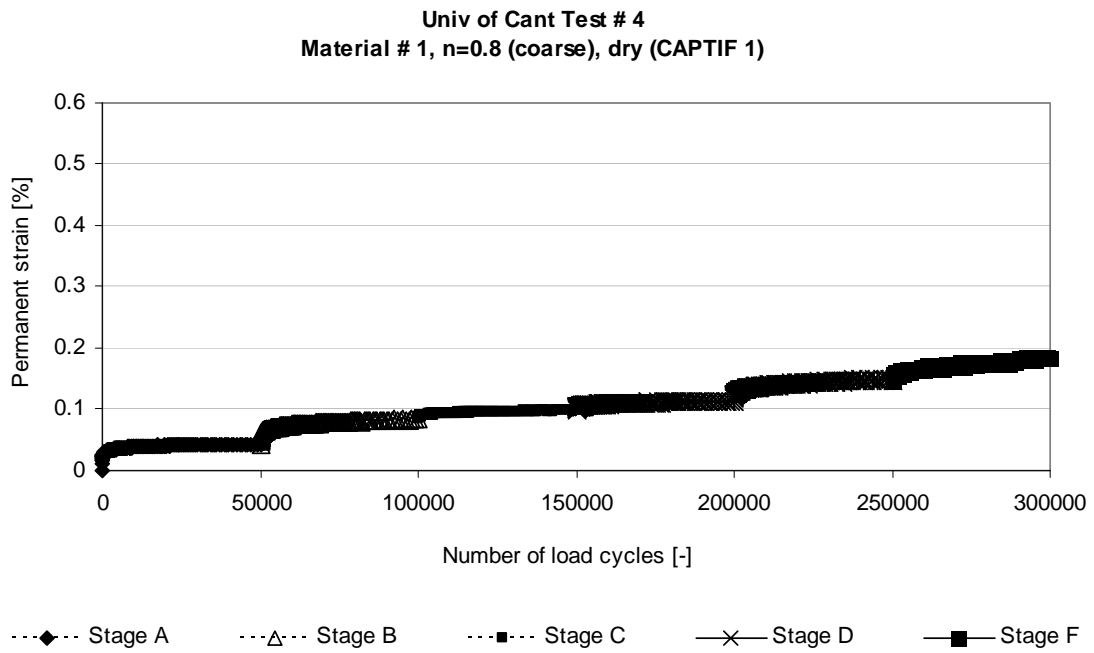
Figure 3.5   RLT result for test #4.

Univ of Cant Test # 5
Material # 1, n=0.8 (coarse), wet (CAPTIF 1)
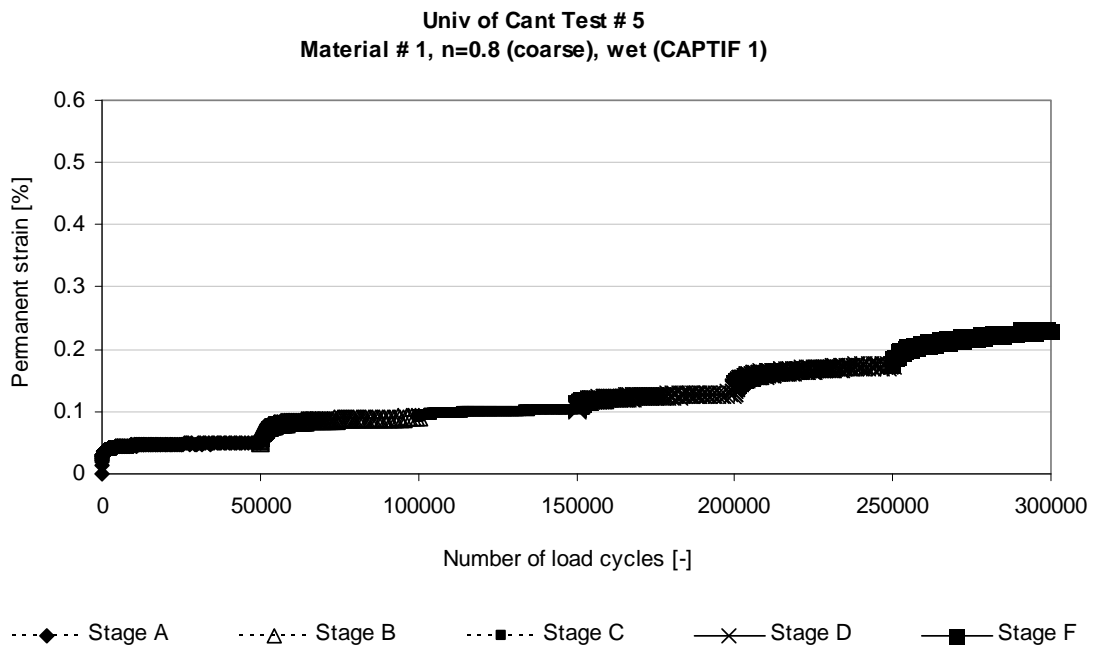
Figure 3.6   RLT result for test #5.

Univ of Cant Test # 7
Material # 1, n=0.3 (fine), wet (CAPTIF 1)
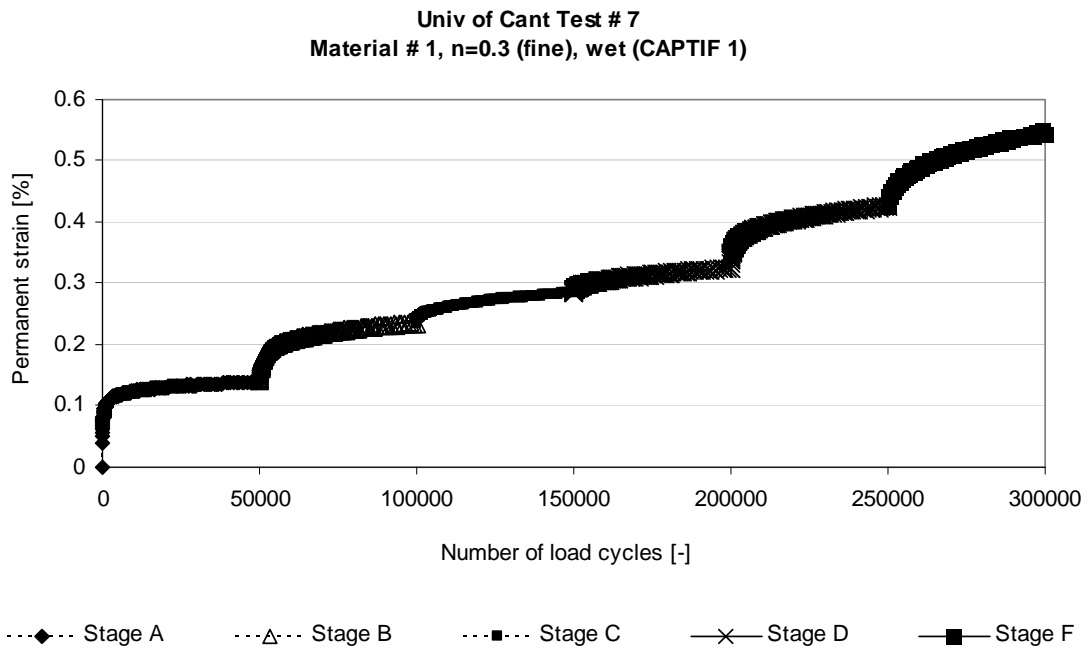
Figure 3.7   RLT result for test #7.

Univ of Cant Test # 6
Material # 1, n=0.3 (fine), dry (CAPTIF 1)
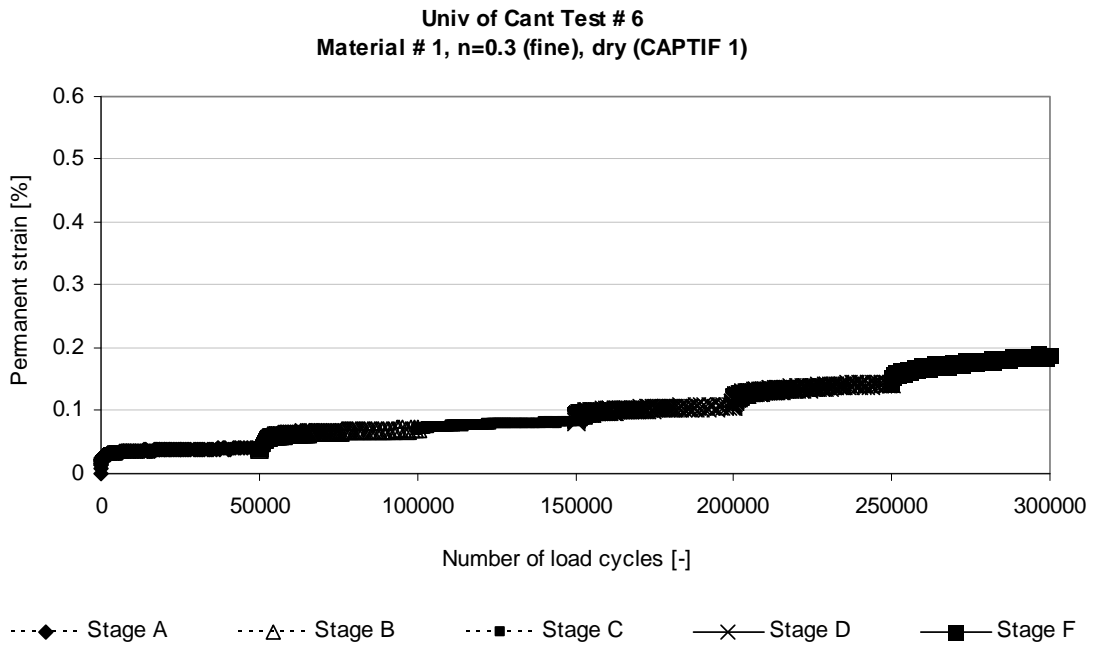
Figure 3.8   RLT result for test #6.

Univ of Cant Test # 17
Material # 1, n=0.55 (middle), wet (CAPTIF 1)
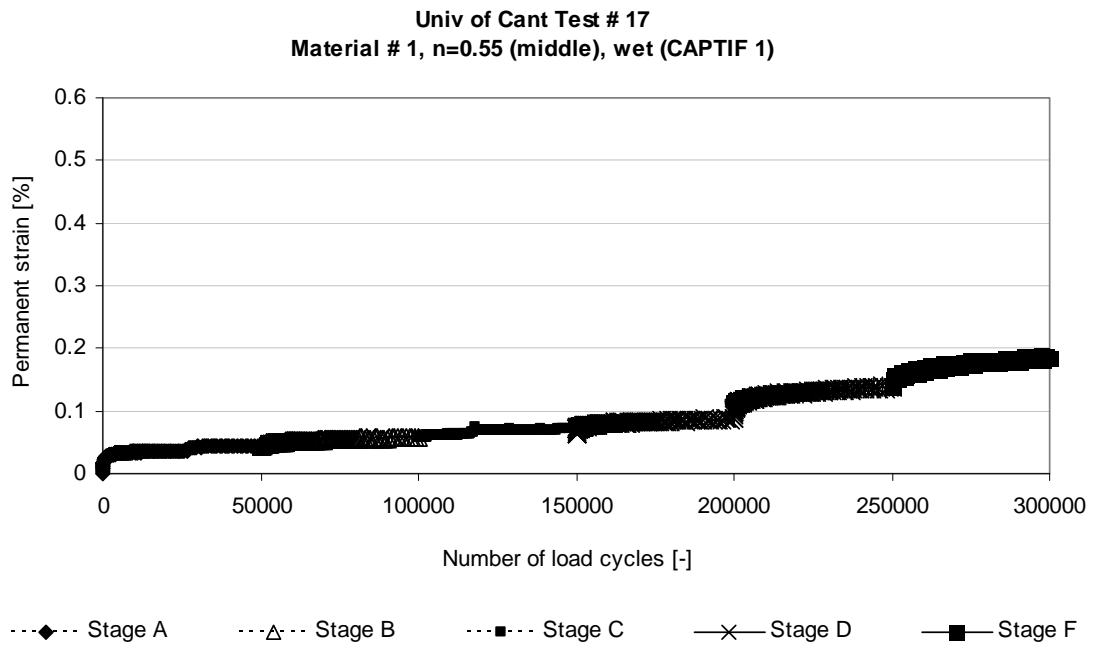
Figure 3.9   RLT result for test #17.

Test # 3
Material # 7, n=0.55 (middle) + 15% fines (AP5), dry
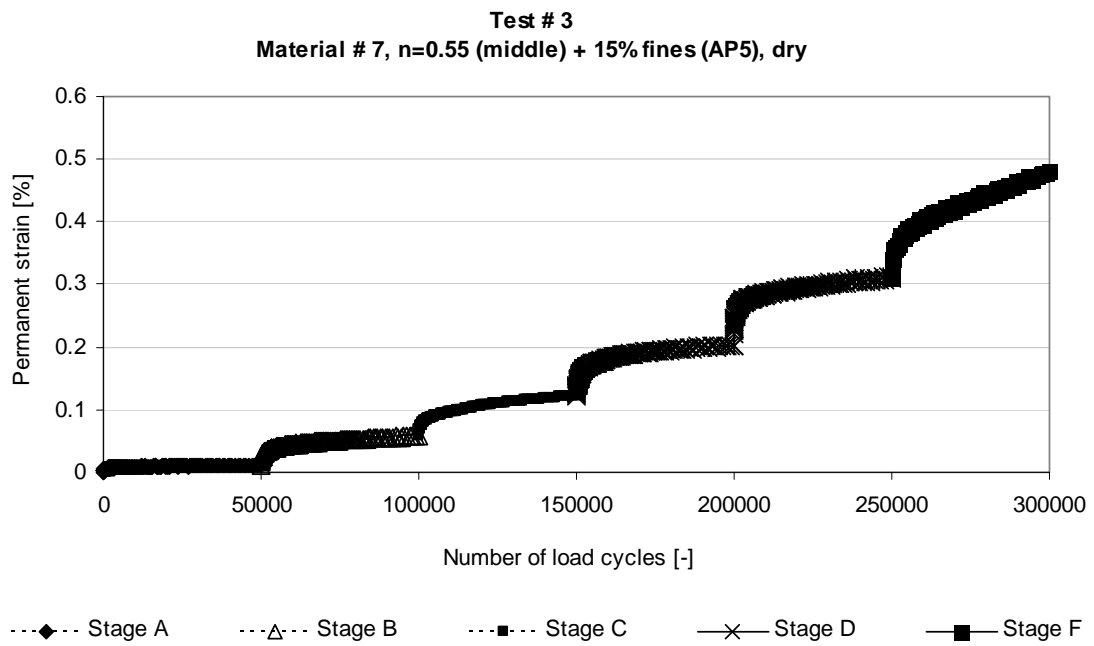
Figure 3.10 RLT result for test #3.

Test # 13
Material # 7, n=0.55 (middle), wet
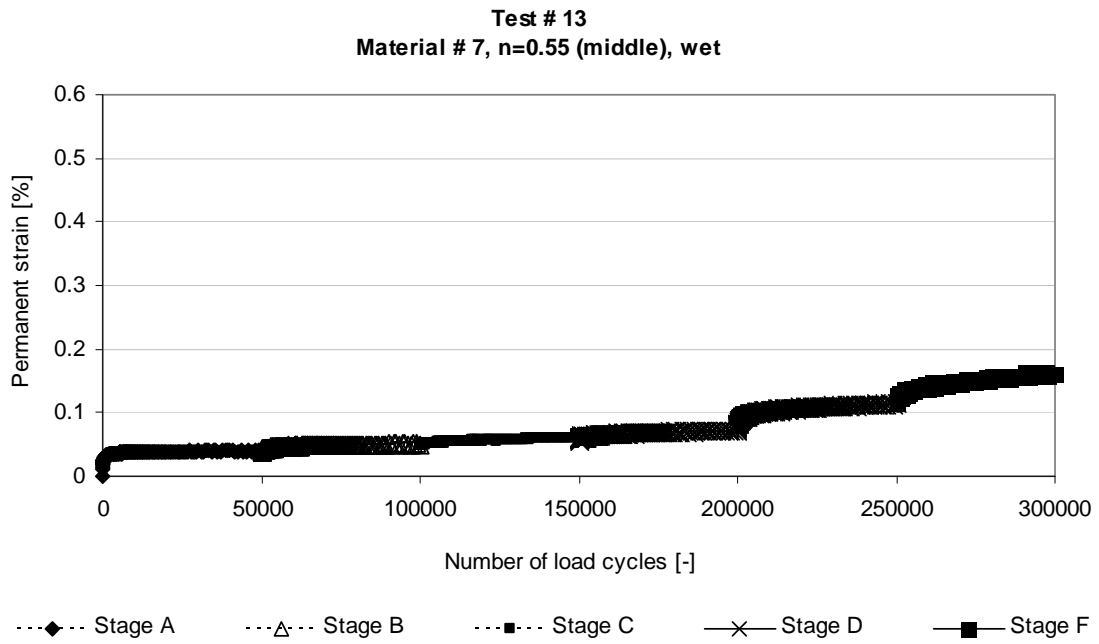
Figure 3.11 RLT result for test #13.

Test # 8
Material # 1, n=0.4 (fine), Wet (CAPTIF 1)
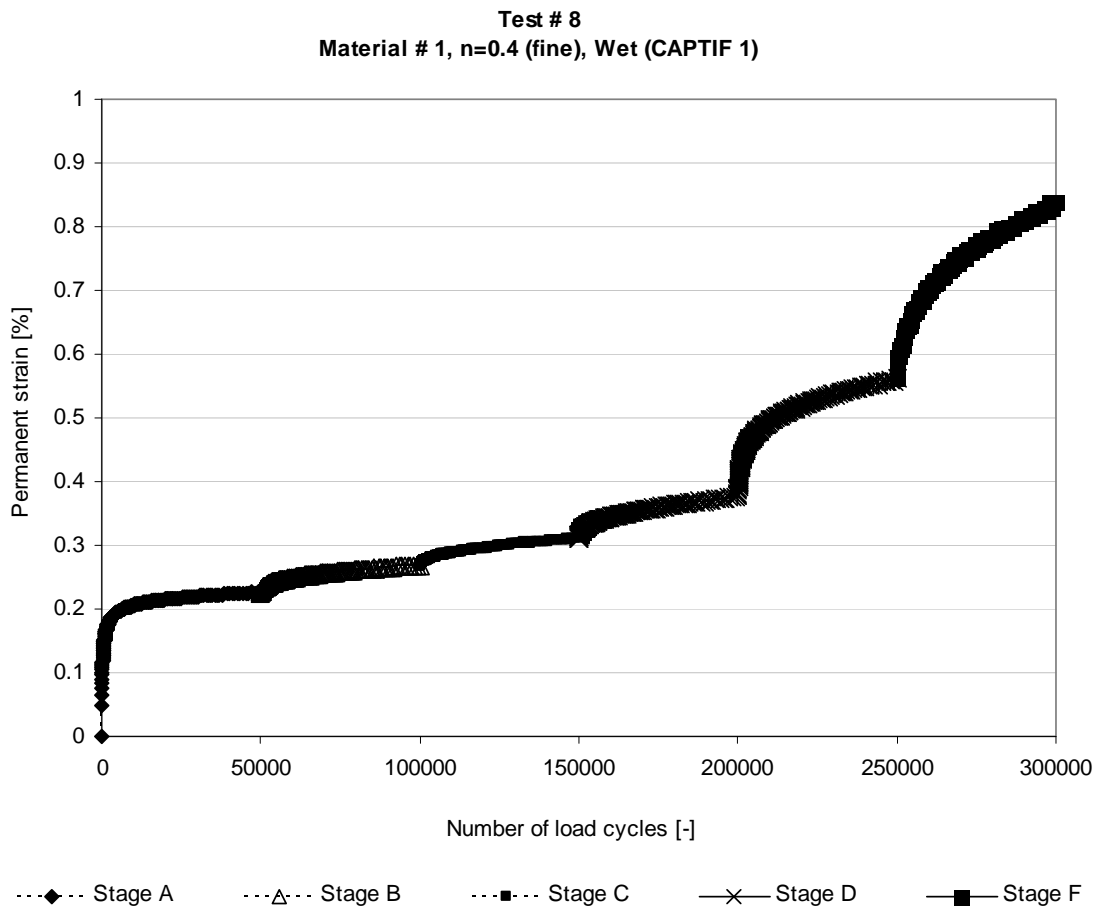
Figure 3.12 RLT result for test #8.

Test # 14
Materials # 7, n=0.3 (fine), dry
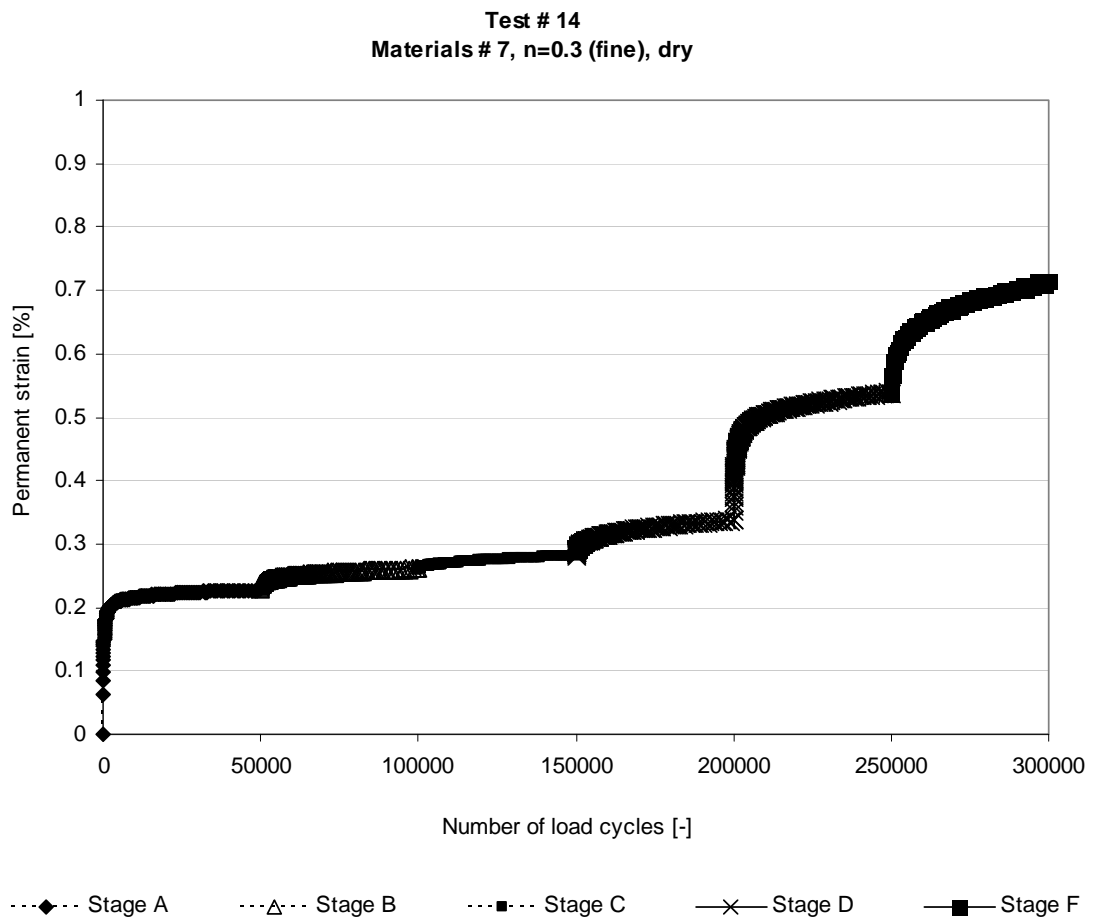
Figure 3.13 RLT result for test #14.

Test # 15
Material # 7, n=0.8 (coarse), dry
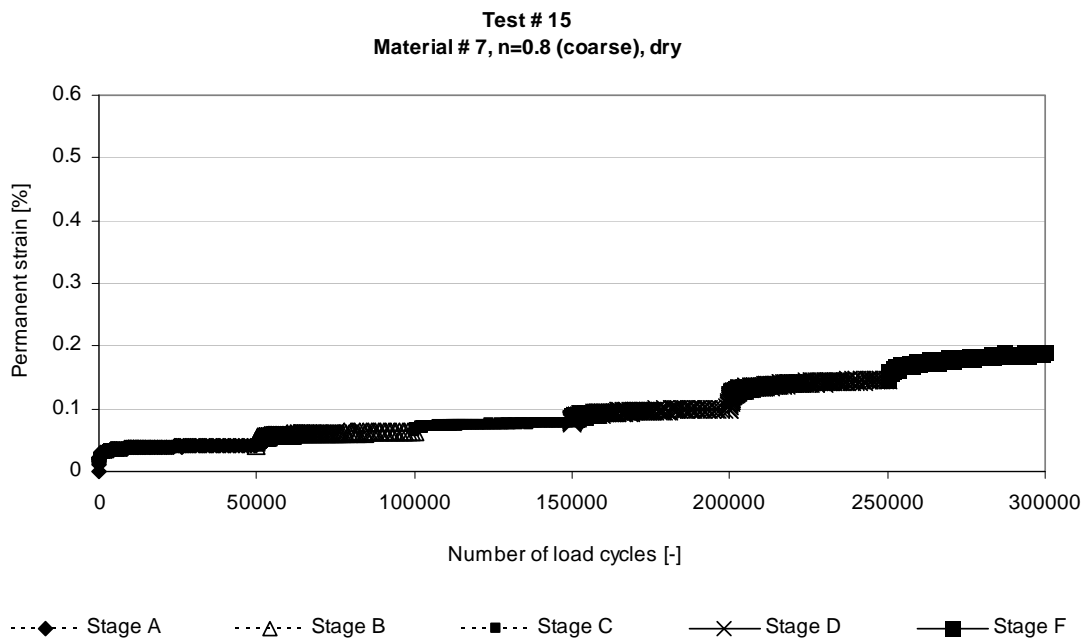
Figure 3.14 RLT result for test #15.

3.5   Rut depth predictions

Rut depth from RLT tests was predicted for a typical pavement constructed at CAPTIF (Figure 3.15). Rut depths within the subgrade and aggregate were predicted with the results shown in Table 3.4. Reviewing the results for material #1 in the wet state shows that the strength/resistance to rutting was significantly improved for coarser gradings with *n*-values >0.5 as shown in Figure 3.16.
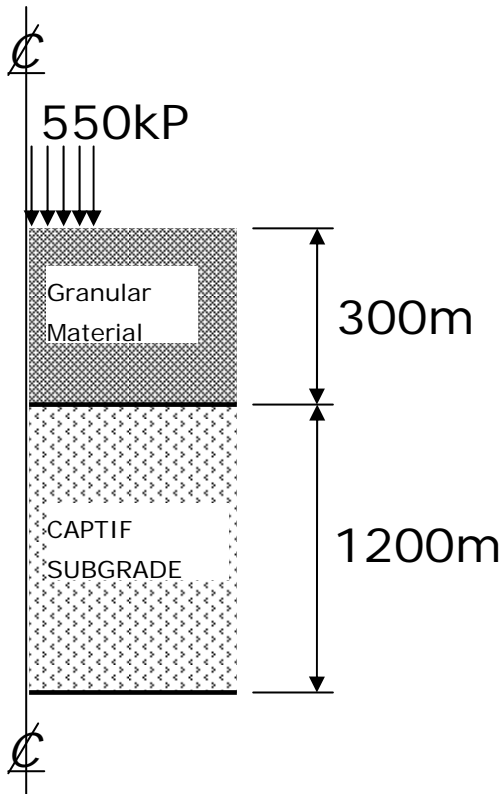
Figure 3.15 CAPTIF pavement cross-section for rut depth prediction.

Table 3.4   Rut depth predictions from RLT results.

#	Material # (Table 3.3)	CAPTIF pavement 300 mm aggregate over 10CBR subgrade					
		Total pavement	Aggregate only	Aggregate only			
		N, ESAs to get 25 mm rut	N, ESAs to get 10 mm rut in aggregate.	Long-term rate of rutting within aggregate	Resilient modulus	Average RLT slope	Maximum slope
		Million ESAs	*Million ESAs*	*mm per 1 million ESAs*	*MPa*	*%/1M*	*%/1M*
4	Mat. #1, *n*=0.8 (coarse), dry.	3.33	24.66	0.4	672	0.182	0.433
5	Mat. #1, *n*=0.8 (coarse), wet.	3.42	28.60	0.3	760	0.193	0.471
7	Mat. #1, *n*=0.3 (fine), wet.	3.18	10.36	0.7	882	0.594	1.157
6	Mat. #1, *n*=0.3 (fine), dry	3.49	37.16	0.3	741	0.206	0.424
17	Mat. #1, *n*=0.55 (middle), wet	3.39	26.14	0.4	698	0.207	0.363
3	Mat. #7, *n*=0.55 (middle) + 15% fines (AP5), dry	2.19	2.46	3.7	583	0.604	1.878
13	Mat. #7, *n*=0.55 (middle), wet	3.07	10.97	0.9	566	0.167	0.448
8	Mat. #1, *n*=0.4 (fine), wet	2.75	6.17	1.2	608	0.978	2.708
14	Mat. #7, *n*=0.3 (fine), dry	3.03	13.57	0.5	756	0.496	1.235
15	Mat. #7, *n*=0.8 (coarse), dry	3.52	48.73	0.2	784	0.157	0.339
	CAPTIF Test 2001, Section 1A, 40kN wheel load	3.8	*Unknown*	*Unknown*			
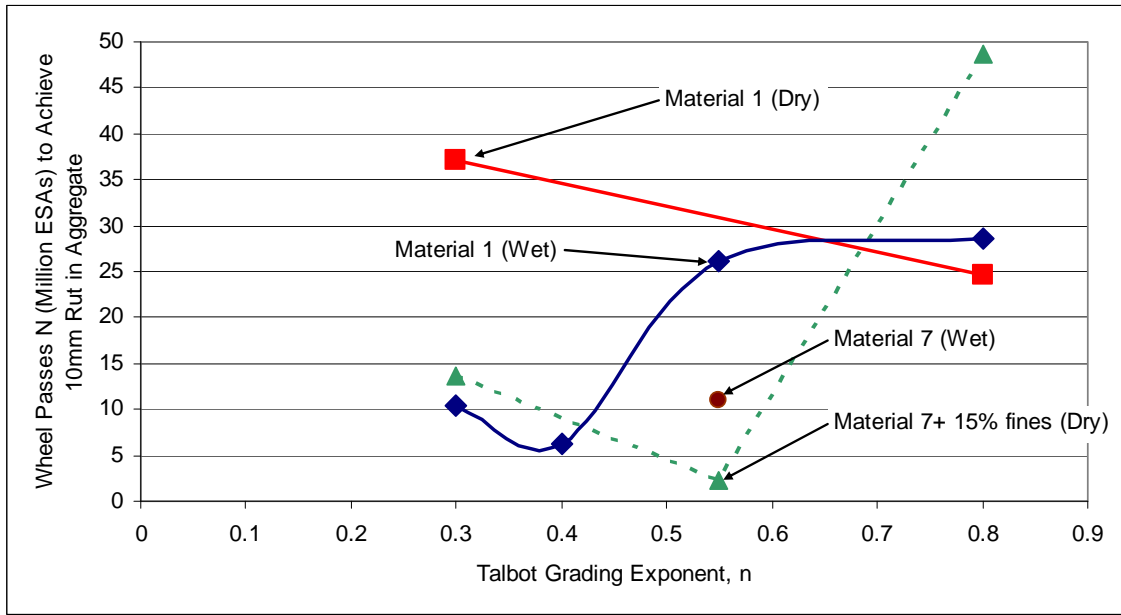
Figure 3.16 Effect of Talbot grading exponent _n_ (equation 1) on rutting performance for materials #1 and #7.

The various lives predicted in the rut depth modelling showing the different contributions of subgrade and aggregate deformations for the various materials can be seen in Figures 3.17 to 3.19.
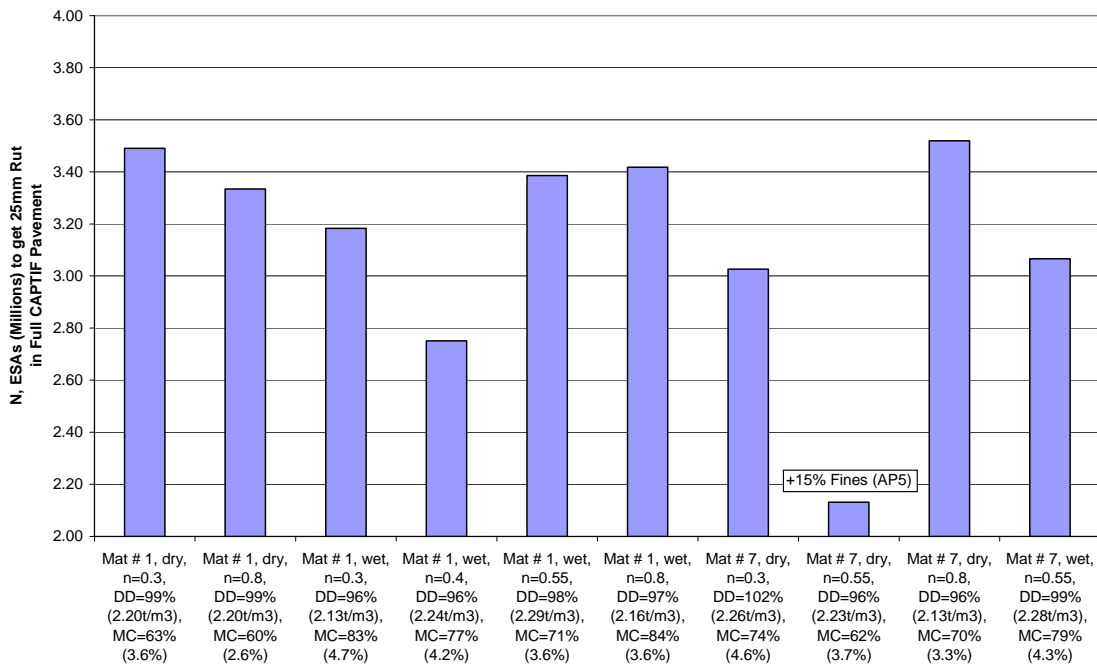
Figure 3.17 Effect of aggregate grading on life of CAPTIF pavement (includes both aggregate and subgrade deformation).
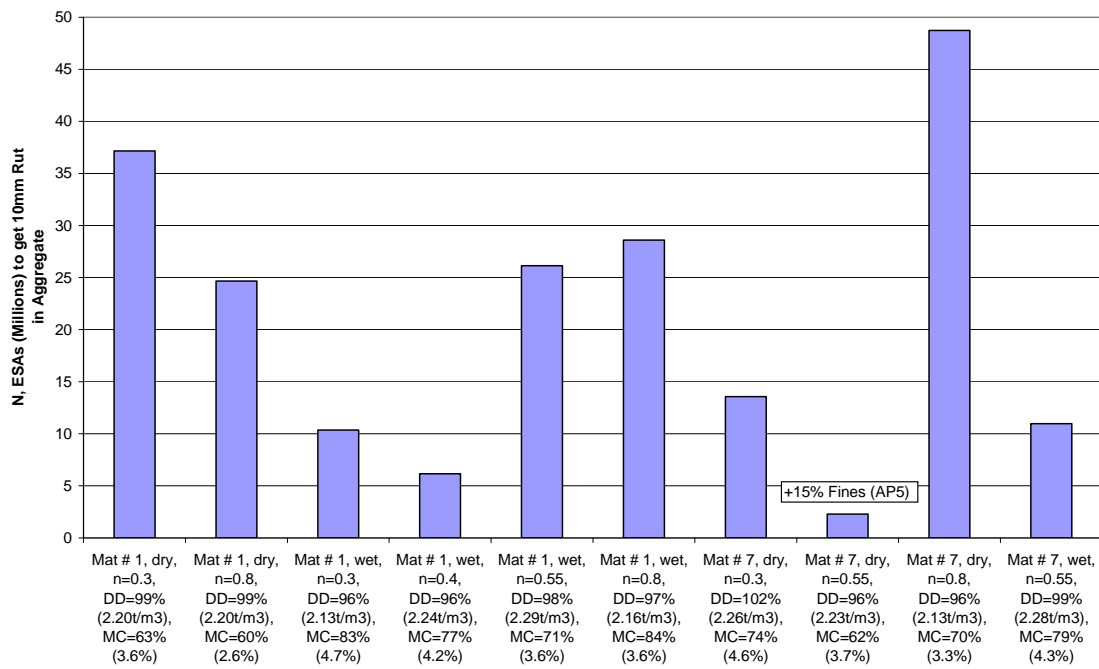
Figure 3.18 Effect of aggregate grading on number of wheel loads to achieved a 10 mm rut within the aggregate layer of a CAPTIF pavement (includes aggregate deformation only).
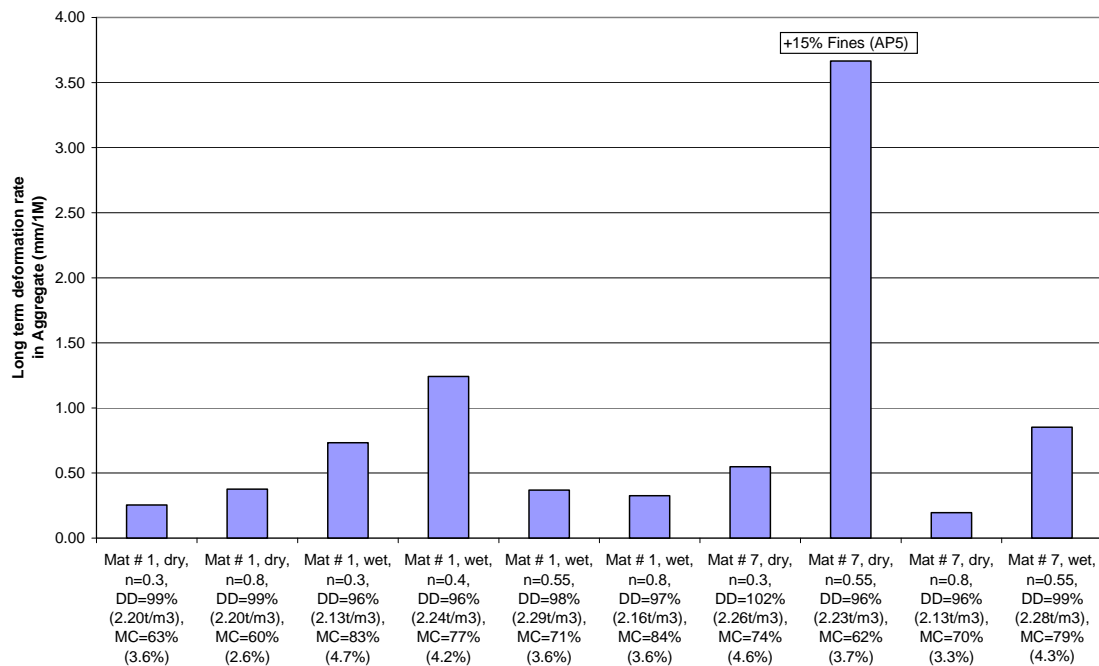
Figure 3.19 Effect of aggregate grading on the long term rate of deformation within the aggregate layer of a CAPTIF pavement (includes aggregate deformation only).

The CAPTIF material #1 showed a typical 'textbook' response. In wet conditions higher deformations or shorter life occurred for the finer grading due to the lubricating effect of the fines. Further, the fine gradings were more sensitive to moisture showing large differences in

performance between the wet and the dry tests. The coarse gradings for material #1 gave the highest life in wet conditions with little difference between the wet and dry tests. For the dry tests, as predicted, a fine grading proved to be best for material #1 due to the cohesive nature of the additional fines.

Material #7 did not follow text book rules for the limited number of tests in dry conditions, probably because the nature of the fines did not have beneficial cohesive strength. This was probably due to the 4% moisture in the material which could cause the fines to have a lubricating effect, hence the results could be considered a wet test. Adding 15% fines had a negative effect on performance as expected. The coarse grading for material #7 showed the best performance in the test.

4.   Specifying grading envelopes for RLT tests

The original purpose of this research was to determine the acceptable limits around a grading that was tested in the RLT apparatus. Ideally one RLT test would be undertaken to determine the acceptability as a basecourse and to determine production limits so that at the extremes of the limits the performance would be nearly the same. A review of the RLT results gave a conservative estimate that Talbot's *n*-value could vary around +/- 0.06 at the low/fine end and +/- 0.1 at the high/coarse end (Figure 4.1). These variations were estimated based on the results of material #1 in wet conditions where it was assumed that the predicted life at *n*=0.3 (higher fines) should be the same as *n*=0.4 (less fines) and the differences in performance were due to sample preparation. Although gradings around an *n*-value of 0.5 can be seen in Figure 4.1, in fact fewer variations of *n* were required to achieve similar results. In this limit test data set the results were grouped as either fine or coarse and anything in between would be placed between the two extreme cases in the grading envelope. This equates to around +/- 13% variation in Talbot's *n*-value where the performance predicted by the RLT test would be similar. In comparison the TNZ M/4 basecourse specification allows for a variation in Talbot's *n*-value of +/- 20% or from 0.4 to 0.6.
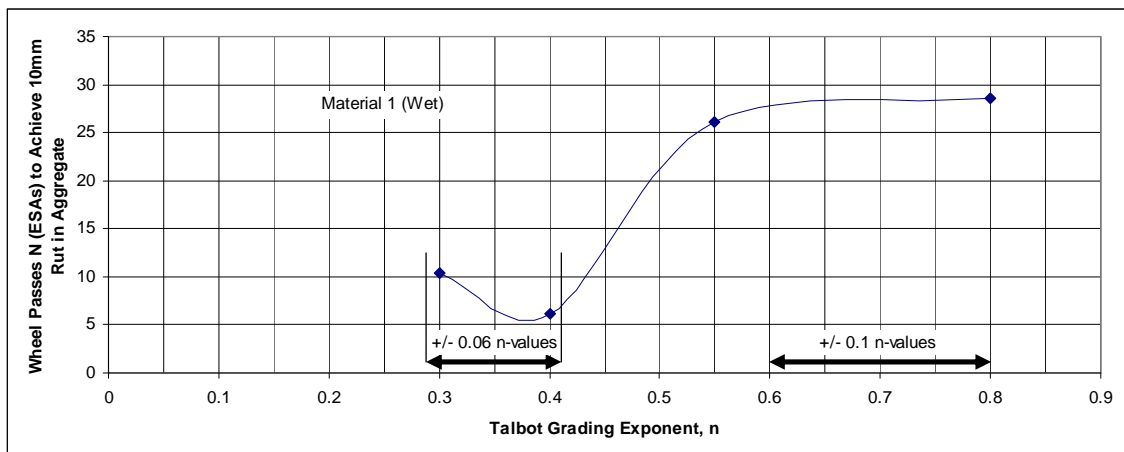

Figure 4.1   Proposed variation in n-values where RLT results are similar.

For simplicity, production limits should be specified in variations around each sieve size as done in TNZ M/4 basecourse specification (Table 4.1). Based on varying Talbot's *n*-value +/- 13% grading envelopes for the fine side and coarse side were determined (Tables 4.2 and 4.3).

Table 4.1   Particle size distribution envelope limits for an individual sample.

Test sieve aperture	Maximum and minimum allowable percentage weight passing	
	AP40 (Max size 40 mm)	AP20 (Max size 20 mm)
37.5 mm	100	-
19 mm	66–81	100
9.5 mm	43–57	55–75
4.75 mm	28–43	33–55
2.36 mm	19–33	22–42
1.18 mm	12–25	14–31
600 μm	7–19	8–23
300 μm	3–14	5–16
150 μm	0–10	0–12
75 μm	0–7	0–8

Table 4.2   Variation in grading envelopes for +/- 13% of Talbot's exponent *n*-values on the coarse side.

n	0.70	0.80	0.90	Variation			
D	37.5	37.5	37.5	Absolute		Percentage	
				plus	minus	plus	minus
Sieve sizes	Fine of middle	Middle	Coarse of middle	Fine side	Coarse side	Fine side	Coarse side
37.5	100	100	100	0.0	0.0	0	0
19	62	58	54	4.3	4.0	7	7
9.5	38	33	29	5.1	4.4	15	13
4.75	24	19	15	4.6	3.7	24	19
2.36	15	11	8	3.6	2.7	33	25
1.18	9	6	4	2.7	1.9	43	30
0.6	6	4	2	2.0	1.3	54	35
0.3	3	2	1	1.4	0.8	65	39
0.15	2	1	1	0.9	0.5	78	44
0.075	1	1	0	0.6	0.3	91	48

Table 4.3   Variation in grading envelopes for +/- 13% of Talbot's exponent *n*-values on the fine side.

n	0.26	0.3	0.34				
D	37.5	37.5	37.5	Absolute		Percentage	
				plus	minus	plus	minus
Sieve sizes	Fine of middle	Middle	Coarse of middle	Fine side	Coarse side	Fine side	Coarse side
37.5	100	100	100	0	0	0	0
19	84	82	79	2	2	3	3
9.5	70	66	63	4	3	6	5
4.75	58	54	50	5	4	8	8
2.36	49	44	39	5	4	11	10
1.18	41	35	31	5	4	14	13
0.6	34	29	25	5	4	18	15
0.3	28	23	19	5	4	21	17
0.15	24	19	15	5	4	24	19
0.075	20	15	12	4	3	27	22

Table 4.4   Variation in grading envelopes for +/- 13% Talbot's exponent *n*-values shown as absolute values.

	Acceptable absolute variation for each sieve size, based on Talbot's *n*-value.							
Sieve sizes	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
	%	%	%	%	%	%	%	%
37.50	0	0	0	0	0	0	0	0
19.00	2	2	3	3	4	4	4	4
9.50	3	4	4	4	5	5	5	5
4.75	4	4	5	5	5	4	4	4
2.36	4	5	5	5	4	4	3	3
1.18	5	5	5	4	3	3	2	2
0.600	5	5	4	3	3	2	2	1
0.300	5	4	4	3	2	2	1	1
0.150	5	4	3	2	2	1	1	0
0.075	5	4	3	2	1	1	0	0

Table 4.5   Variation in grading envelopes for +/- 13% Talbot's exponent *n*-values shown as percentages.

	Acceptable % variation for each sieve size, based on Talbot's *n*-value.							
Sieve sizes	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
37.5	0	0	0	0	0	0	0	0
19	2	3	4	4	5	6	7	8
9.5	4	5	7	9	11	13	14	16
4.75	5	8	11	13	16	19	22	24
2.36	7	11	14	18	22	25	29	33
1.18	9	14	18	23	27	32	37	42
0.6	11	16	22	27	33	39	44	50
0.3	13	19	25	32	39	45	52	60
0.15	14	22	29	37	44	52	61	69
0.075	16	24	33	42	50	60	69	79

Tables 4.4 and 4.5 show the calculated acceptable variations in sieve sizes for the full range of Talbot's exponent *n*-values. It is interesting to note that less variation is allowed for the large sieve sizes for the fine gradings (low *n*-values), while the opposite occurs for the coarse gradings. A difficulty with determining acceptable variations based on Talbot's *n*-values is the difficulty in calculating the *n*-value based on the received grading curve for the RLT specimen. Thus, a more simplified method of calculating the acceptable variation is required.

The Road Traffic Authority in New South Wales, Australia allows alternative basecourse aggregates, either modified or unmodified, based on acceptable shear strength. Limits on grading envelopes for materials based on shear strength are specified in RTA NSW 3051 and are summarised in Table 4.6.

Table 4.6   Grading limits specified by RTA NSW 3051.

Sieve sizes (mm)	Limits around nominated particle size distribution
	%
37.50	-
26.50	± 10
19.00	± 10
13.20	± 8
6.70	± 5
2.36	± 4
0.425	± 3
0.075	± 2
% passing 6.70mm	50–80

A quick scan of grading envelopes from the three methods reported, TNZ M/4 (Table 4.1); Talbot's *n*-value derived from RLT tests (Tables 4.5 and 4.6); and RTA NSW 3051 (Table 4.6) shows the Tabot's *n*-value derived from RLT tests is the most restrictive. Limits derived from RLT testing were used to develop recommended grading envelopes as these were based on actual performance tests and were more conservative. As the amount of fines had the most effect on wet strength then this was limited further. Reviewing grading envelopes in Tables 4.5 and 4.6 and limiting the variation in fines led to the following recommended grading envelopes in Table 4.7.

Table 4.7   Recommended grading envelope variations around a measured particle size distribution (PSD) for the RLT sample.

Sieve sizes	Acceptable variation around measured particle size distribution (PSD).
	%
37.50	-
19.00	± 5
9.50	± 5
4.75	± 4
2.36	± 3
1.18	the > of ± 14% of the measured psd *or* ± 2
0.600	the > of ± 14% of the measured psd *or* ± 1
0.300	the > of ± 14% of the measured psd *or* ± 1
0.150	the > of ± 14% of the measured psd *or* ± 1
0.075	the > of ± 14% of the measured psd *or* ± 1

Table 4.7 shows a reasonably tight grading envelope that may not be achieved in practice but it does represent a grading envelope within which a similar result to the RLT test should be obtained. Therefore, it is considered that Table 4.7 is appropriate for setting grading boundaries on a RLT test result and if an aggregate supplier requires a larger grading envelope for production control then additional RLT tests to expand the grading range will be required.

Variations in grading envelopes were derived to be similar values as the variations in Table 4.5 for the coarse materials >2.36 mm in sieve size. A less conservative approach near the maximum variation allowable for the coarse materials (Table 4.5) was chosen as both TNZ M/4 and RTA NSW 3051 allow larger variations in the coarse materials. For the fines a more conservative approach was taken of the lowest variation values shown in Tables 4.5 and 4.6 with allowance for original fine gradings with a percentage variation (± 14% of PSD value) as ± 1% would be too restrictive for fine contents >15%.

5. Conclusions

This research project included a literature review, RLT tests and rut depth modelling to predict the rutting performance primarily for one basecourse aggregate at different gradings. Limited RLT results for another aggregate were added to supplement the results of the main test. The objective was to determine the extent at which the grading could be changed without affecting the rut depth performance found from RLT tests. Based on this knowledge grading envelopes were established around the gradings of samples tested in the RLT apparatus where the results from the RLT tests would be nearly the same for all gradings within the envelope. The grading envelope found from this project is very tight and it is likely that a wider grading envelope would be required for specifications. Wider grading envelopes can be established by quarry owners by undertaking several RLT tests to cover the full range of gradings expected to occur when developing an alternative in TNZ M/4. From a literature review and RLT test the following conclusions were found:

- Talbot's exponent *n*-value is a convenient way to describe the grading of an aggregate, where *n*-values >0.5 are a coarse grading while *n*-values <0.5 are considered fine.

- Current Transit New Zealand specification for basecourse aggregate has a Talbot's *n*-value varying from 0.4 to 0.6, while the majority of overseas roading authorities allow finer gradings with a Talbot's *n*-value of 0.3 to 0.6.

- Previous research has revealed that fine gradings have a lower strength when wet compared with coarse gradings, while the opposite is the case when dry and/or when a modifier like cement is added (ie fine gradings are best).

- The literature reported that finer gradings could be more easily constructed with less segregation and lower total voids to minimise any further densification after opening a pavement to traffic.

- Vibrating hammer compaction tests for determining target densities for RLT testing indicated that the highest density occurred with an *n*-value of around 0.5 which is the same as the theoretical maximum density curve.

- RLT testing on a TNZ M/4 basecourse from Pounds Road, Christchurch at gradings with Talbot's *n*-values of 0.3, 0.4, 0.55 and 0.8 showed that the best performance (least rutting) in wet conditions was obtained with the coarse grading with an *n*-value of 0.8. The best performance in dry test conditions was obtained at a finer grading with an *n*-value of 0.3.
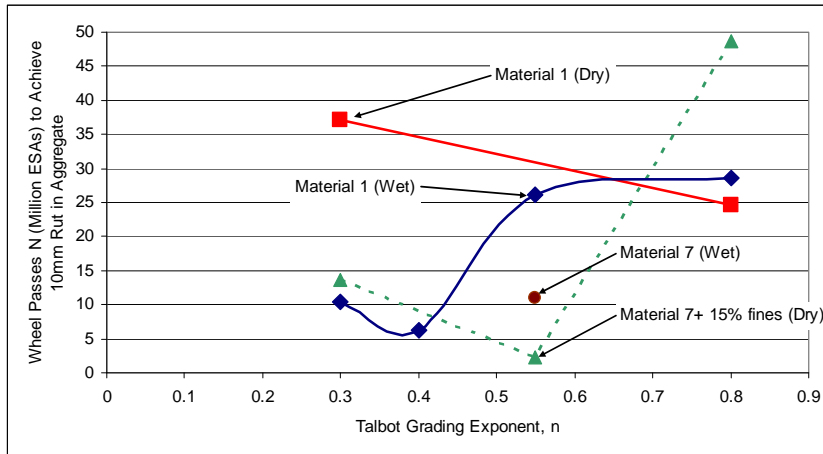
Figure 5.1   Effect of Talbot grading exponent *n* (equation 1) on rutting performance for materials #1 and #7

- The RLT test results showed that Talbot's *n*-values can vary by ±13% without greatly affecting the resulting rutting performance.

- Based on varying the *n*-value by ±13% and limiting the amount of fines, a grading boundary was determined around a RLT result where it was predicted the rut depth performance would be nearly the same. This grading boundary is about half the variation allowed for in the TNZ M/4 and RTA NSW 3051 specifications and additional RLT tests will be required at the extremes of a grading envelope to establish practical production limits.

6.   Recommendations

This research project recommends:

- an amendment to the Transit New Zealand specification being developed to detail the boundaries in grading around a RLT test result where it is expected the results would be similar to those detailed in Table 6.1

- RLT testing at a range of gradings to determine suitable production limits. More discussion is required with industry to determine the most appropriate limits on grading in the specification development process

Table 6.1   Recommended grading envelope variations around a measured particle size distribution (PSD) for the RLT sample.

Sieve sizes	Acceptable variation around measured particle size distribution (PSD).
	%
37.50	-
19.00	± 5
9.50	± 5
4.75	± 4
2.36	± 3
1.18	the > of ± 14% of the measured PSD *or* ± 2
0.600	the > of ± 14% of the measured PSD *or* ± 1
0.300	the > of ± 14% of the measured PSDd *or* ± 1
0.150	the > of ± 14% of the measured PSD *or* ± 1
0.075	the > of ± 14% of the measured PSD *or* ± 1

- further study be undertaken to establish suitable limits for both the quantity and type of fines (plastic or non-plastic) and to establish appropriate limits on the control of size fractions similar to TNZ M/4 requirements.

Results from this research are based on limited testing primarily on one basecourse aggregate and consideration should be given to expanding this study to other aggregates to confirm the results.

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