Extending pavement life: investigation of premature distress in unbound granular pavements

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Abbreviations and acronyms

AASHTO	American Association of State Highway and Transportation Officials
AC	asphaltic concrete
ALF	accelerated loading facility
AP	all passing
APT	accelerated pavement testing
ARRB	Australian Road Research Board
CBR	California bearing ratio
DOS	degree of saturation
ESA	equivalent standard axles
EWC	equilibrium water content
FWD	falling weight deflector
LTPP	long-term pavement performance
M/3 notes	Transit NZ (1986) *Notes on subbase aggregate specification TNZ M/3*
M/4	Transit NZ (2006) *Specification for basecourse aggregate*. TNZ M/4
M-EPDG	NCHRP (2004) *Mechanistic-empirical pavement design guide*
MESA	millions of equivalent standard axles
MRWA	Main Roads Western Australia
NCHRP	National Cooperative Highway Research Program
NDM	nuclear density meter
NZ supplement	Transit NZ (2007) *The New Zealand supplement to the document, Pavement design, a guide to the structural design of road pavements*
NZTA	New Zealand Transport Agency
OGPA	open graded porous asphalt
OWC	optimum water content
PET	potential evapo-transpiration
RLT	repeated load triaxial (testing)
SSD	saturated surface dry
SGE	sand grading exponent

Contents

Executive summary

Premature distress in unbound basecourses has occurred regularly in New Zealand and the motorway north of Auckland (ALPURT) was a recent notable example resulting in costly rehabilitation. Accordingly, in 2008, the New Zealand Transport Agency (NZTA) commissioned the assembly of an inventory of problem basecourses and subbases where suitable documentation could be obtained. Study of the inventory found the long-term degree of saturation of basecourse was highly significant in the case histories of premature distress, ie the pavements failed through shear instability (shoving) in basecourses even though the underlying layer(s) may have provided good drainage. A common feature in basecourses with a high degree of saturation was that they tend to be gap graded in the sand fraction. The existing basecourse specification (which has been changed little in the last 40 years) limits gap grading through grading shape control requirements but the case histories demonstrate that tighter control is required. Appropriate changes to the current NZTA basecourse specification have been recommended.

The particle size distributions and properties of the fines were evaluated in detail in order to see whether performance could be predicted from the basic soil parameters. The basecourse inventory was used to establish regression equations for predicting:

- the likely density of a basecourse after construction compaction
- the long-term refusal density, ie the density of a basecourse after sustained trafficking
- the long-term equilibrium water content of a well drained basecourse
- the in-situ long-term degree of saturation of a basecourse.

This approach appears to be very promising as another check to identify problem basecourses before or during construction. Timely decisions can now be made on acceptance or the need for corrective measures such as cement modification prior to sealing.

In the repeat load triaxial (RLT) test, testing at 95% of maximum dry density may not give conservative results, as the relevant degree of saturation after bedding-in is the over-riding consideration. Recent accelerated pavement testing raises significant issues regarding the limitations of repeat load testing, but these could probably be addressed by appropriate sample selection and preparation. Further work is required in this area. Because a change in degree of saturation from 65% to 70% will approximately double the rate of permanent deformation, it is imperative that degree of saturation is calculated and reported for any RLT test used to predict likely performance of a basecourse in practice. The greatest risk with any non-standard basecourse adopted on the basis of RLT testing is that if it proves to be susceptible to shear instability (shoving) then premature pavement distress may develop within months of opening to traffic.

A practice adopted by other roading authorities is to confirm the quality and quantity of fines in the basecourse post-compaction, thereby reducing the risk of premature failure. With increasing demands on pavements there are increasing requirements for basecourse performance.

The above considerations have been used for preparing revised drafts of the NZTA basecourse specification, subbase specification notes as well as a set of recommendations for the compaction specification and the *New Zealand supplement to the document, Pavement design – a guide to the structural design of road pavements (Austroads 2004)* (Transit NZ 2007a). A durable aggregate in a well constructed unbound granular pavement should be capable of carrying well in excess of 10 million equivalent standard axles. The relatively minor changes to the specifications provide a practical solution to premature distress in unbound basecourses.

Abstract

Premature distress in unbound basecourses has occurred regularly in New Zealand. In 2008, the New Zealand Transport Agency (NZTA) commissioned the assembly of an inventory of problem basecourses and subbases. Study of the inventory found that the long-term degree of saturation of basecourse was highly significant in the case histories of premature distress, ie the pavements failed through shear instability (shoving) in the basecourses. A common feature in basecourses with a high degree of saturation was gap grading in the sand fraction.

Existing basecourse specifications limit gap grading through grading shape control requirements but the case histories demonstrate that tighter control is required.

The basecourse inventory was used to establish regression equations for predicting the in situ long-term degree of saturation of a basecourse. This approach appears to be very promising. Timely decisions can now be made on acceptance or the need for corrective measures prior to sealing.

The above considerations have been used for preparing revised drafts of the NZTA basecourse specification, subbase specification notes as well as a set of recommendations for the compaction specification and the *New Zealand supplement to the document, Pavement design – a guide to the structural design of road pavements (Austroads 2004)* (Transit NZ 2007a) to implement practical solutions to premature distress in unbound basecourses.

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1 Introduction

Basecourse shear instability in unbound granular pavements with sprayed seal surfacing is becoming an increasing issue as traffic volumes and loadings rise on the roading network. Although it may occur on a relatively small proportion of a road, its development is usually sudden and dramatic; therefore, when it occurs on major arterials and highways the consequences to both the travelling public and maintenance programmes are significant.

Shear instability (otherwise known as shallow shear or shoving) soon leads to a depression with cracking which allows ponded surface water to enter. This accelerates the instability and forms a pothole soon after.

Recently, there have been some very significant cases where premature pavement distress has been attributed to shear instability in the granular layers (usually basecourse but subbases can also fail similarly). Several of these failures have been extremely costly and unexpected in that both the initial design and the approved rehabilitation have complied with current practice, but neither has achieved the intended design life. The substantial direct cost has also been accompanied by the usual indirect cost of lane closures and traffic delays for the public.

There is a strong indication of common factors pointing to ‘gaps’ in specifications and practices currently used in New Zealand. It is now becoming clear there are effective solutions; tighter controls will positively address the problem. The current specifications were intended to ensure each pavement achieves its design life, so it is imperative they address all the likely mechanisms of pavement failure. The NZ Transport Agency’s (NZTA) guides, standards and specifications need to include positive safeguards against premature failures (or at least provide forewarning to the client there may be a risk of failure inherent in a specific design).

The current NZTA basecourse specification, TNZ M/4 (Transit NZ 2006, referred to throughout this report as M/4), has changed little over the last 35 years. The current research was undertaken to establish appropriate basecourse criteria that reflect usage (ie where there are high traffic demands on roads, stricter requirements should apply to certain basecourse characteristics, while still allowing basecourses with lesser physical characteristics to be utilised in less demanding situations). This will allow more sustainable use through employing higher quality aggregates only where needed and also utilise more marginal aggregates or recycled materials in situations with less intense traffic.

2 Basecourse inventory

2.1 Origin

Following a discussion of premature failure of a section of the motorway north of Auckland (ALPURT) at a pavement analysis workshop in July 2008, the NZTA commissioned the establishment of an inventory of data from many unbound basecourses and some subbases including those that had failed through shear instability. The data collected at that stage showed promising lines of research to address the premature failure issue.

However, it was considered important to build on the database to see that the full spectrum of unbound granular pavement types and circumstances had been encompassed. For this reason, a broader number of failures was sought to expand the database, as well as carrying out more general consultation throughout the industry.

The inventory is available for download (Tonkin & Taylor 2010a).

2.2 Inventory site coding

The sites are all anonymously coded to promote the sharing of data which may be confidential or from contentious projects. Each material at a given site is numbered with subsidiary alphabetic coding of the form A#Aa (uppercase letter, site number, uppercase letter, lowercase letter) which denotes the following:

1 The first character is an uppercase letter signifying the type of data:

   I – in-service pavement or planned for such use

   A – accelerated pavement test site (heavy vehicle simulator)

   R – repeated load triaxial test data

   L – limits of a particular specification

2 The second character is the unique site/material number.

3 The third character is an uppercase letter starting with A and increasing sequentially indicating sequential but similar samples from the same site.

4 The fourth character is a lowercase letter representing the amount of trafficking at the time of sampling:

   s – stockpile sample

   c – construction compaction, or early life traffic only

   r – ‘refusal condition’, ie after bedding-in is completed, usually regarded as after at least 10,000 equivalent standard axles (ESA) trafficking

2.3 Basic parameters

The current NZTA basecourse parameters in M/4 are the primary fields recorded in the inventory. Not all parameters are available for all sites, but the following basic properties were sought for inclusion in the inventory, as well as secondary parameters calculated from these base parameters.

2.3.1 Source and production material parameters

- particle size distribution
- sand equivalent
- liquid and plastic limit
- clay index
- crushing resistance
- weathering quality index, percentage of weathering resistant material and cleanness value
- California bearing ratio (CBR)
- broken faces
- solid density
- maximum laboratory dry density and optimum water content

2.3.2 Repeated load triaxial (RLT) parameters

- ESA to 10mm rut
- long-term rutting rate (mm/MESA).

2.3.3 In situ parameters

- dry density as compacted
- water content as compacted
- dry density at 'refusal'
- annual rainfall and potential evapo-transpiration
- long-term equilibrium water content.

Most of these parameters are definitive and meaningful measures but some do require further consideration.

2.4 Specific parameters

2.4.1 Plasticity index

This is a useful measure of the sensitivity of the soil in the presence of water and the test is carried out on the passing 425 micron fraction. Although a measure of the nature of this fraction, the parameter takes no account of how much 425 micron material is in the whole sample. If there is only a minute amount of 425 micron material in the whole sample then plasticity will have little relevance to the performance of the all-in grading of the in situ basecourse. This drawback has been addressed in Australian practice by considering the 'weighted plasticity index' defined as the product of the plasticity index and the percentage passing 425 microns. Accordingly this custom was adopted for this study and the weighted plasticity index is also listed in the summary data table of the basecourse inventory. As the 425 micron sieve is not used for standard grading analysis, interpolating from the graphical particle size distribution is appropriate for calculating the weighted plasticity index.

2.4.2 Clay index

Examination of the basecourse inventory suggests the clay index is a particularly important parameter. It is a measure of the nature of the ultra-fine particles, and in particular their ability to attract and hold airborne or capillary water within the matrix. As with the plasticity index, the clay index is measured only on a portion of the material (particles passing 75 microns) and hence again takes no account of the quantity of fines[1] present in the whole sample. Accordingly, for this study a corresponding 'weighted clay index' was calculated as the product of the clay index and the percentage finer than 75 microns. When carrying out the grading, the recording of the passing 75 micron fraction should be to two significant figures to improve the weighting calculation.

2.4.3 Weathering quality index

This is a measure of the resistance of aggregate to the effects of wetting, drying, heating and cooling. High-quality basecourses are required to fall within six of the nine categories, ie classed as AA, BA, CA, BA, BB, or CA. For the comparison of parameters between different basecourses, or for use in regression equations, this non-numeric classification is limiting. However, the weathering quality index is calculated from two distinct numeric variables, ie the cleanness value and also percentage of weathering resistant material (in the 4.75mm fraction). Rows for both of these separate parameters have been allocated in the inventory (labelled CV and WRM).

2.4.4 Sand equivalent

The sand equivalent is carried out on the 4.75mm fraction which includes the entire matrix and hence at this stage, no weighting is considered appropriate.

2.4.5 Moisture ratio

Australian practice is to use a moisture ratio (defined as the ratio of the water content of the basecourse to the optimum water content for compaction) as a criterion for dry back prior to sealing. Midgely (2009a) promotes a moisture ratio of not less than 85% for compaction, then an average value of 60% (with no single result over 70%) prior to the sealing of highways.

Sampson et al (1985) reported on a number of failures, and suggested an indicator where if the moisture ratio long term approached 70%, shear instability was likely to occur. Therefore this parameter was also added to the inventory, but to conform with current terminology was termed the percentage of optimum water content (OWC).

2.4.6 Equilibrium water content

This concept is discussed further in the following sections, and refers to the expectation that in the long term, the basecourses of many well sealed pavements will tend to stabilise at what can be regarded as the 'equilibrium water content' (MRWA 1993). There will be some seasonal fluctuations slightly either side of the characteristic equilibrium value and In some instances the situation will not be applicable, eg where the water table can fluctuate over a wide range and occasionally rise to the point where additional water will migrate (either directly or by capillary action) into the basecourse layer, or where normal resealing has

[1] Note the term 'fines' is used to refer generally (as used in TNZ M/4 specification) to the fraction being discussed (passing 75 micron for CI, passing 425 micron for PI and passing 4.75mm for sand equivalent). Where distinction needs to be made the Austroads term 'fines fraction' refers to the passing 4.75mm material and 'cohesive fines' refers to passing 75 micron.

not been carried out and the waterproofing of the seal is compromised. The concept of an equilibrium water content is likely to be more relevant in temperate climates than in zones with more extreme seasonal fluctuations.

2.4.7 Dry density at 'refusal'

The refusal condition referred to in this study is the condition of a basecourse which has been 'bedded in', ie has been subjected to moderate trafficking (say 10,000 ESA or perhaps 5% to 10% of its design life). In reality the dry density of a sound, well-graded basecourse can be expected to increase progressively throughout its mature life, but the rate of change after the bedding-in phase is usually very small and to the accuracy of normal measurement of in situ density, can be considered relatively constant throughout much of the mature phase of a pavement's life.

2.4.8 Solid density

When calculating the degree of saturation for compaction quality control, an accurate value of solid density is essential. Many of the values supplied for the basecourse inventory are reported as 'assumed' in the documentation supplied. Regular verification testing is required for realistic quality assurance, a point that has been emphasised recently by Bartley (2007).

2.5 Further work

The intention is to maintain the inventory and build on it as more case histories become available on both well and poorly performing basecourses. It will therefore be available for refinement of the recommendations in this report and also serve as a basis for future research.

RLT testing was commissioned at the outset of this project, comparing coarse, medium and fine gradings of basecourse with poor performance record. The intention was to test the samples at their respective water contents that would represent long-term equilibrium values. However, how to assess this realistically presented an issue, which was not initially resolved.

The results of the RLT testing are listed in the inventory, along with a large dataset of other RLT tests carried out on New Zealand basecourses (Greg Arnold, pers comm). Standard New Zealand RLT testing does not include recording of the standard particle size distribution, or the degree of saturation at any stage of the test. This limited the use of historic RLT data for this project in relation to the considerations given in chapter 4.

3 Non-destructive in situ testing

Where practical, the basecourse inventory has been supplemented with in situ parameters using non-destructive methods (mainly deflection testing). This is to allow an understanding of the combined effect of aggregate source properties and the in situ environment. In addition, in situ testing for other case histories has also been assembled, even though in many of these the source aggregate properties are not available at present. Further destructive testing may be warranted.

3.1 Empirical parameters

Background to empirical deflection parameters is given in part 5 of Austroads (2008–2009):

- central deflection
- curvature.

3.2 Parameters from analysis of deflection bowls

Detailed explanation for parameters derived from back analyses of deflection bowls is given in part 5 of Austroads (2008–2009) or Ullidtz (1978):

- layer moduli
- modular ratios
- subgrade modulus (and equivalent CBR)
- predicted terminal distress mode (rutting, roughness, cracking or shear instability)
- predicted life (ESA to a terminal condition)
- rehabilitation options.

The ELMOD software (Dynatest 2007) was used for the back analyses, primarily because of its speed of execution when doing a series of sensitivity analyses and also as it is one of the few packages that realistically models non-linear subgrades (as studies show these predominate in New Zealand).

3.3 Format

The database of the in situ deflection testing results is extensive and therefore only the interpretations and key findings are summarised in this report. All data sets are available electronically in spreadsheet format. Findings from the non-destructive testing are given in chapter 7.

4 Literature review

4.1 Relevant research

The most relevant articles for this project have been assembled in chapter 10 ‘Bibliography’. Relevant issues are summarised below.

4.2 Austroads TT1163

A recently completed project at the accelerated loading facility (ALF) is documented by Jameson et al (2009) report TT1163. In this trial, four basecourses were compared, which were expected to give a good range of characteristics with respect to RLT compliance standards currently in place. The trial investigated the degree to which RLT testing could determine basecourse performance.

The expediency of the test led to reliance on RLT testing for both research and for a more immediate assessment of a new product. Formerly the RLT test had been generally accepted as the best laboratory test to predict the performance (ie shear strength, resilient modulus and permanent deformation) of aggregates. However, Jameson et al (2008; 2009) noted that:

> *A significant hindrance to the widespread adoption of the RLT test has been the lack of data linking the results of the laboratory test to field performance.*

Therefore the trial compared the rutting performance of four different aggregates using both accelerated pavement testing (APT) and RLT testing. APT was carried out in the Australian Road Research Board (ARRB) accelerated loading facility ALF and RLT testing was carried using three different methods (Austroads 2007); the Department for Transport, Energy and Infrastructure South Australia (2008); and Transit NZ (2007b) TNZ T/15. The researchers considered none of the RLT methods consistently predicted the ALF performance.

It has been suggested (D Alabaster, pers comm) that the assessment of the results rather than the methods themselves could be an issue. G Arnold (pers comm) considers Jameson et al (2008; 2009) used different tyre loading compared with the laboratory axial loading. Irwin (2006) considers the reason RLT tests do not simulate in-service behaviour of basecourses is that traffic compaction induces much greater horizontal stress than is used in the laboratory RLT. At the same time, density may be greater after relatively few cycles as the rotating principal stresses induced by each passing wheel load would result in greater compaction (as well as increased degradation) compared with a pure axial load. The latter does not simulate the kneading action resulting from the rotating stress field imposed under normal traffic. Also RLT testing is carried out on a rigid base, but in-service the modulus of an unbound aggregate varies depending on the modulus of the underlying layer. Hence elastic strains will vary and so will the accompanying plastic strains. The average diameter of the RLT sample may change during the test as a result of contraction or dilation and this would need to be measured to understand changes in density and hence degree of saturation.

Another limitation of the RLT test when applied to an ‘average’ sample of basecourse as traditionally sampled, is that it will not be representative of the lower bound quality from the source. Shear instability typically occurs in less than 5% of a pavement by the stage it is regarded as requiring rehabilitation. Most of a newly constructed pavement should have a particle size distribution and fines content that will give good performance. However, as a consequence of the inevitable statistical variation in quality that is inherent in an assemblage of particles, localised clusters will exist in the pavement where the percentage of fine material is higher than average. The variation may be aggravated where construction practices have contributed to segregation. In all cases there will be parts of the pavement which will be more predisposed to shoving, particularly in the outer wheelpath where the long-term water content is likely to be higher and more variable. RLT testing of the average particle size distribution cannot therefore be expected to give meaningful evaluation of the potential for shear instability. Selection and sampling of say the 5th percentile of the finest clusters from the placed basecourse layer may be more appropriate but that nominated percentile and the selection process are both likely to be contentious. It is not surprising, therefore, that ARRB research has led to the finding that the RLT (as currently performed) has proven ineffective for evaluating the potential for shear instability.

Until improvements are made to the RLT test and it is shown to match in-service performance, it should be regarded as essentially an index test for densification but unlikely to be a reliable indicator of shear instability, which is a primary consideration of this project.

This research included the assembly of the inventory of basic parameters (M/4 tests results in particular) in order to study the ways in which the various parameters affect the in-service performance of basecourses. Emphasis was on both particle size distribution and the nature of the fines for the reliable prediction of permanent deformation. However, this emphasis was placed not so much on vertical surface deformation or rutting (caused by densification) but deformation resulting from lack of lateral shear resistance (shear instability or shoving) with all three terms regarded as describing the same process in this report.

It is significant that in the Better Basis project about 80% of the permanent surface deformation occurred due to deformation of the top 150mm of base. The basecourses were placed on a very stiff subbase with deflections of about 0.5mm. Much shorter life would be expected on typical more yielding highway pavements with characteristic deflections more in the range of 0.8mm–1.4mm.

The data from the Better Basis trials was added to the inventory of basecourses. That research also identified the need for a performance index that would adequately characterise basecourse shear stability (an objective also of this study).

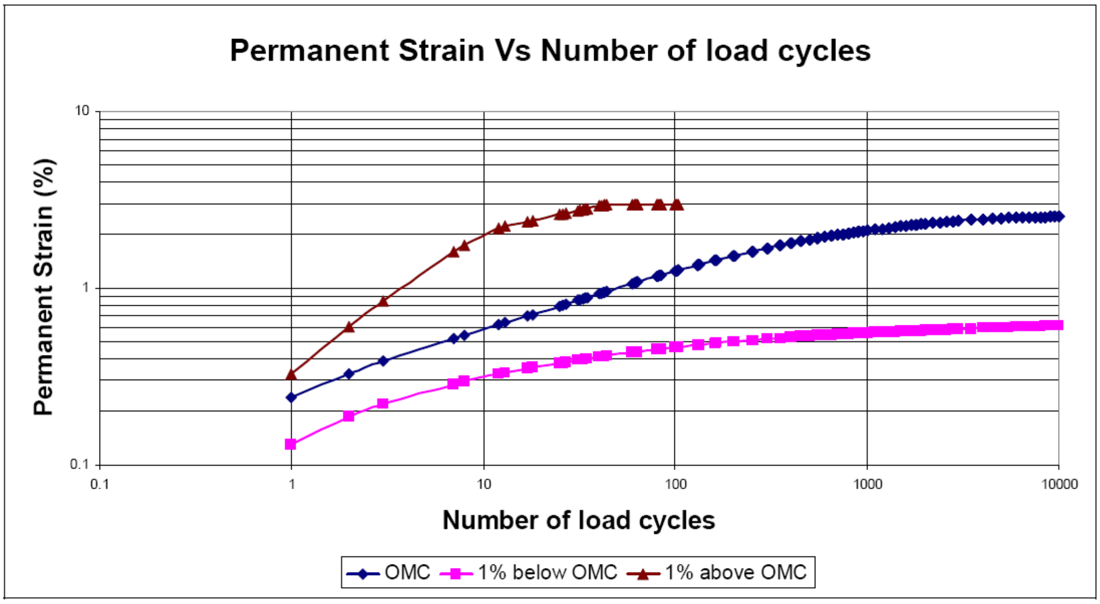
Through the compilation of the inventory of basecourses, common features of aggregates with shear instability are emerging. The continual addition of case histories and their associated basecourse properties will help refine a predictive model.

4.3 Other RLT testing

4.3.1 Effect of basecourse water content

A detailed study of the parameters that affect basecourse performance in laboratory cyclic loading was reported by Kancherla (2004), and the following figure extracted from his thesis clearly demonstrates the extreme sensitivity of basecourse materials to a change in water content of just 1% either side of optimum.

Figure 4.1 Sensitivity of deformation to changes in water content (Kancherla 2004)



Australian practice (Midgley 2009a) is to ensure dry-back to 60%–70% of OWC before sealing which translates to about 1.5% decrease in water content.

A more comprehensive RLT study by Theyse (2002) produced a regression for the number of load cycles to a given permanent deformation as:

$$Log\ N = -13.43+0.29RD-0.07S+0.07PS-0.02SR \qquad \text{(Equation 4.1)}$$

R2 = 97%; SEE =0.313

Where: N is number of load repetitions

RD is relative density (%)

S is degree of saturation (%)

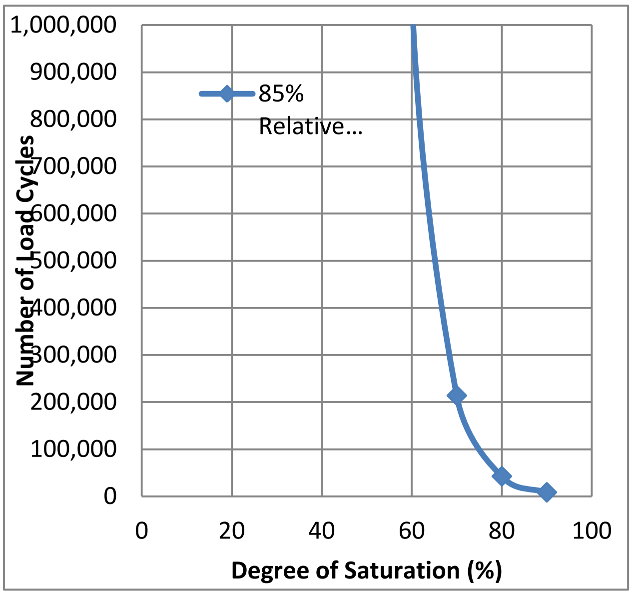
PS is plastic strain

SR is stress ratio (%)

Typical values for this regression in the following figure, illustrate the critical dependence of pavement life on the degree of saturation, especially beyond about 70%:

Figure 4.2 highlights the importance of ensuring the degree of saturation used in any RLT test is the same as that which will apply long term in the pavement. If the relative density and degree of saturation are not accurately known and clearly related to the in-service state, the test may have little value to the practitioner. Note that for this model, going from a degree of saturation of 65% to 70% will approximately halve the pavement life.

Figure 4.2 Influence of degree of saturation in triaxial tests



A logical expectation given in equation 4.1 for RLT testing, is that the prediction of the in-service life of basecourses located in the top layer of a pavement is likely to be given by an equation of a similar form but with N being the lifetime ESA to a given terminal rut depth. The various terms in the above regression require consideration:

- Relative density: Once any basecourse reaches its refusal density it is likely to be at, or marginally over, 100% relative density. This term is likely therefore to be relatively invariant in practice for any basecourse at refusal.
- Degree of saturation: The degree of saturation will be a highly variable parameter between different basecourses depending on the grading, percentage of fines, nature of the fines and environment.
- Plastic strain: The plastic strain is not readily measured in an in-service pavement. It will be relatively variable but approximate correlations may possibly be derived from the basecourse laboratory test parameters and response of the basecourse during passage of a standard axle load (estimated from deflection testing).
- Stress ratio: The average stress ratio at any given depth within a basecourse layer subject to a standard axle would be expected to be relatively invariant.

Accordingly, to predict the ESA to a terminal rut depth in an in-service pavement, a starting point may be to consider a regression of the form:

$$Log(ESA) = a0 - a1\ Sr - a2\ PS \qquad \text{(Equation 4.2)}$$

Where PS is the plastic strain which is a function of the pavement response to heavy traffic loading and a0, a1, a2 are constants, constrained to be positive for a logical regression. (A starting point for a1 would be approximately 0.29 as obtained with the RLT study.)

Note that saturation and plastic strain are not independent variables, ie plastic strain is likely to be largely due to excess pore pressures which in turn would be strongly related to the degree of saturation. Therefore the important inference from the RLT testing is that focus should be primarily on saturation and the fines (their nature and amount) to establish any meaningful method for predicting the life of an in-service basecourse. This concept is developed in detail in section 5.4.

The basecourse inventory has been supplemented with New Zealand and Australian triaxial test data. These include RLT results from the Better Basis for Bases project, specific tests on problem New Zealand basecourses and routine testing of marginal aggregates. The sources acknowledged are the ARRB Group, DTEI, PaveSpec Ltd, Stevenson Resources Ltd and Winstone Aggregates Ltd.

4.3.2 Density prediction

Theyse (2002) also reports on the RLT studies by Semmelink (1991) on the prediction (derived from regression analyses) of maximum density and optimum water content for both modified AASHTO and vibratory table compaction effort:

$$MDD_{vib} = -39.34GF^{0.85} + 20.01C - 1.54LS - 11.05C^3 + 107.82 \quad \text{(Equation 4.3)}$$

$$MDD_{mod} = -33.73GF^{0.85} + 19.28C - 1.21LS - 12.31C^3 + 99.94 \quad \text{(Equation 4.4)}$$

$$OWC_{vib} = 23.14GF^{0.85} - 15.90C + 1.09LS - 11.16C^3 - 7.63 \quad \text{(Equation 4.5)}$$

$$OWC_{mod} = 7.18GF^{0.85} + 0.035C - 0.55LS + 2.86C^3 + 0.80 \quad \text{(Equation 4.6)}$$

Where:

C = (percentage passing 0.425 mm/100)/(LL/100)$^{0.1}$

GF = sum (percentage passing a particular sieve size/nominal sieve size/100 for the 75, 63, 53, 37.5, 26.5, 19, 13.2, 4.75, and 2mm sieves

LS = linear shrinkage

LL = liquid limit

These are of interest conceptually, but the forms adopted are difficult for immediate adaptation to New Zealand conditions, first because linear shrinkage is not currently available for many of the case histories in the basecourse inventory and second the OWC regressions are not logically consistent in the dependence of the terms, ie the C and LS terms use different signs in the two regressions for OWC, suggesting the correlations may work for the set of data from which the equation was derived but may not be meaningful for a wider data set.

4.4 Basecourse specifications

It has long been recognised that many basecourses, particularly sound greywackes, will not meet the M/4 basecourse requirement for a sand equivalent of 40 and this has traditionally been addressed by confirming the fines are non-plastic. However, there is some concern within the industry that basecourse stability is not always ensured by following the M/4 specification that requires compliance with only one of the three options for fines quality (clay index, sand equivalent or plasticity index). One non-standard practice to address this issue is to specify that, for arterials roads and highways, compliance with all three of the fines criteria is to be met. However, this is not practicable for some quarries which produce aggregates of known good performance, yet fail on sand equivalent.

New Zealand and Australian practices are poles apart on the plasticity issue. Australian specifications for Class 1 rock (VicRoads 2008b) reject non-plastic basecourses because VicRoads deems some plasticity to be essential – a stand that is contrary to New Zealand practice. The difference is probably due to environmental effects, as much of Australia (including Victoria) has significantly less rainfall and the long-term equilibrium water contents of their basecourses are likely to be lower than in New Zealand. This is further supported by the observation that during the rare wet periods, Australian roads often show marked distress (D Mangan, Chadwick T&T, pers comm).

4.5 US National Cooperative Highway Research Program (NCHRP) model

The US National Cooperative Highway Research Program (NCHRP) (2004) has adopted a mechanistic-empirical pavement model in its *Guide for mechanistic-empirical design of new and rehabilitated pavement structures* (M-EPDG). All inputs are pavement properties that define the response of pavement to traffic and climate loads, primarily by assessing the moduli of the pavement materials and hence allowing determination of stress and strains throughout the pavement under a standard axle load. Empirical criteria are then derived from observed performance to complete the model.

The M-EPDG assesses structural adequacy based on:

- load-related distress
- trafficking spectra
- material durability and climate
- back-calculated layer elastic moduli
- visual examination of pavement cores
- physical testing of samples to determine moduli and strength.

For the structural modelling of a pavement, critical stresses, strains and displacements (due to traffic loading and climatic factors) are calculated over the total pavement thickness in a layered model using layered elastic theory. The M-EPDG uses either JULEA (linear elastic) or the finite element package DSC2D for non-linear materials. The pavement is modelled to accumulate monthly damage over the design period. This 'incremental damage' is then related to specific distress modes with calibrated empirical models relative to pre-defined treatment criteria. The model for each distress mode incorporates only those physical properties that current research has shown contributes to that mechanism of pavement failure.

Since there is a large degree of uncertainty in the input data, much of the modelling utilises probability distributions for the data, and the designer can then select the level of design reliability they wish to proceed with. The model has been designed with the data available from the US long-term pavement performance (LTPP) sites. In particular the enhanced integrated climatic model uses a large body of climatic data (temperature, precipitation, solar radiation, cloud cover and wind speed) collected alongside structural and traffic data for pavements, to give inputs for seasonal variation in pavement stresses and strains. Pavement layer temperature, frost penetration, moisture predictions are calculated hourly over the design period and used to estimate material properties for the subgrade and pavement layers throughout the design life. Sub- surface drainage design is included in the process by incorporating the FHWA software DRIP (FHWA 2003).

Ultimately the M-EPDG is likely to provide an effective and comprehensive means of pavement design and performance prediction. However, so far it has reportedly been poorly supported, apparently due to its complexity.

For the purposes of the current study, the NCHRP LTPP data sets were examined to see if sufficient detail for basecourse parameters was available to add to the New Zealand basecourse inventory. However it was concluded that the parameters stored were either missing or they were too dissimilar to the New Zealand standard parameters to usefully include any of the database at this stage. However, this may well be a future source for expansion of the inventory.

4.6 Total voids concept

Total voids as a measure of basecourse compaction has been promoted by many in the industry for some years and was the topic of research by Bartley (2007). He recommended several radical departures from the current M/4 basecourse and B/2 compaction specifications, and these are all summarised below. Examination of the basecourse inventory now provides strong support for some of them while others are either modified or varied to some degree in chapter 8.

Bartley's recommendations:

> *a) the limits for particle size distribution set out in TNZ M/4 be changed so that the upper limit is described by an n value of 0.5 and the lower limit by an n value of 0.35;*
>
> *b) quarry operators be permitted to add clean quarry fines or a suitable sand to ensure that their product fits well within the PSD limits and is as well graded as possible;*
>
> *c) use of the laboratory density test for the control of compaction should be discontinued;*
>
> *d) specification for the compaction of both subbase and basecourse should be controlled in terms of total voids;*
>
> *e) total voids should be calculated using the apparent specific gravity determined using ASTM C127:1980 and C128:1980 as a reference;*
>
> *f) apparent specific gravity should be measured by the aggregate producer and reconfirmed on an annual basis. The results should be made available to the construction industry.*

Percentage total voids is applicable to free-draining granular materials yet percentage air voids is a common requirement for compaction of cohesive materials (Pickens 1980). To avoid any confusion, in this report the terminology adopted is percentage of solid density (ie dry density as a percentage of solid density). This is simply related to total voids as 100% minus the percentage of solid density. (Note, the terms solid density and specific gravity are interchangeable, and the reason both are used here is solely to maintain the integrity of the quoted references.) The solid density can be defined in at least three ways (AASHTO 2000), ie:

- apparent specific gravity
- bulk dry specific gravity
- bulk saturated surface dry (SSD) specific gravity.

Opinion on which to use is divided:

Bartley (2007) states: 'Total voids should be calculated using the apparent specific gravity'.

Dongol and Pattrick (1999), present a contrary view: 'In terms of packing efficiency of the aggregate in the field, the voids in the aggregates themselves (intra-granular voids) have virtually no effect. The only effect is the inter-granular voids. This is obtained from the bulk dry basis density'.

Jennings et al (1998) give yet a third opinion: 'The solid density method based on bulk SSD will yield the effective saturation and total voids of the aggregate matrix'.

Because accurate determination of total voids is an essential part of the calculation for degree of saturation, and saturation is a critical parameter affecting basecourse performance, it is disturbing that contradictory views are expressed by such experienced practitioners. Because the writers were also unsure, enquiries to a number of testing laboratory managers were made but these resulted in either conflicting opinions or uncertainty regarding which specific gravity was used to calculate voids or saturation in B/2.

AASHTO (2000) is an informative reference. The apparent specific gravity is defined as the ratio of the weight in air of a unit volume of the impermeable portion of aggregate at a stated temperature to the weight in air of an equal volume of gas-free distilled water at a stated temperature. The volume measurement includes only the volume of the aggregate particles; it does not include the volume of any water permeable voids within particles. However it is generally impractical to force fine material into these voids. Therefore comparison of percentage solid density between aggregates with high versus low absorptions would not be meaningful. Accordingly it is suggested that the logical standard to adopt (when assessing percentage of solid density) is the bulk dry specific gravity. In this, the volume measurement includes the overall volume of each aggregate particle (including the volume of water permeable voids in each particle). The writers therefore concur with the Dongol and Patrick viewpoint, above. The term 'total' voids is misleading in this context and Dongol and Patrick's term of inter-granular voids would be more appropriate. However, to avoid the issue, the term percentage solid density is adopted in this study and the solid density used in the calculation is the bulk dry value. Further development of this concept in relation to the degree of saturation is given in chapter 6.

The three alternative forms of specific gravity are linked by absorption and the inter-relationships are given in appendix 2 of AASHTO (2000). Many New Zealand high-quality basecourses have low absorption values (about 1%) and the difference between the three specific gravities is minor. However, an absorption of 3% can result in significant changes in total voids (depending on the adopted definition of specific gravity) and in some cases the calculated saturation may alter by 20%. If apparent specific gravity is used, saturation will always be on the high side (ie conservative for B/2 pre-sealing assessment). To address the uncertainty in the industry, a standard functional spreadsheet highlighting the correct parameters could be promoted nationally, for use with B/2 acceptance testing.

Bartley (2007) recommends the M/4 grading limits should be finer (n=0.35, Talbot and Richart 1923) but this would increase the allowable percentage of fines (passing 75 microns) to 11%, ie about 50% more fines than permitted at present. The data presented by Barley supported finer gradings only in the coarser fraction of the grading and many of the pavements he studied were relatively new, without sustained trafficking. The case for adjusting the fines limit may therefore be appropriate for particle sizes larger than 4.75mm, although a check would need to be made that the CBR is not too adversely affected. However, no documentation has been presented for raising the percentage passing 75 microns to 11%.

There are arguments for increasing the amount and nature of fines for cases where new basecourses or overlays are planned for stabilisation anyway (because they have an historic record of poor unbound performance). This has been done in some regions with the development of local specifications largely targeted to specific source materials. Ideally these should be included as regional variants in M/4 to allow the practices to be considered for trials in other regions where meeting M/4 requirements is unduly onerous.

Bartley also recommended changing the coarse limit from a grading exponent of 0.6 to 0.5. This may be appropriate for some regions, as successful laying of basecourse with a grading exponent of 0.6 is particularly dependent on the experience of the contractor, methodology and type of equipment used. Basecourses with exponents as high as 0.7 have been built (with cement treatment, van Blerk et al 2010) but at present the practice is not widely adopted. Placing without segregation is essential and with unbound basecourses the surface needs to be dressed with appropriate fine basecourse or crusher dust kneaded into place with a heavy pneumatic tyred roller to lock the surface and form an effective stone mosaic. Provided the compaction is effective, coarsely graded aggregates will perform quite adequately and there are numerous case histories in the basecourse inventory that do not support a change in the coarse limit of the M/4 grading. The choice to use a coarse-graded aggregate can then be left to the contractor but should relate to skills available for the project.

Bartley promoted fine gradings probably because his focus was on short-term rutting and hence there was less concern with the issue of degree of saturation increasing as a basecourse tended towards its refusal condition. This is discussed further in section 5.3.

5 Interpretations from the inventory

5.1 Fines criteria

This section examines possible alternatives to the present M/4 fines criteria.

Option 1

The present specification (here termed option 1) requires any one of sand equivalent, clay index or plasticity index to be compliant. However, there clearly are cases where this approach has led to premature distress.

Option 2

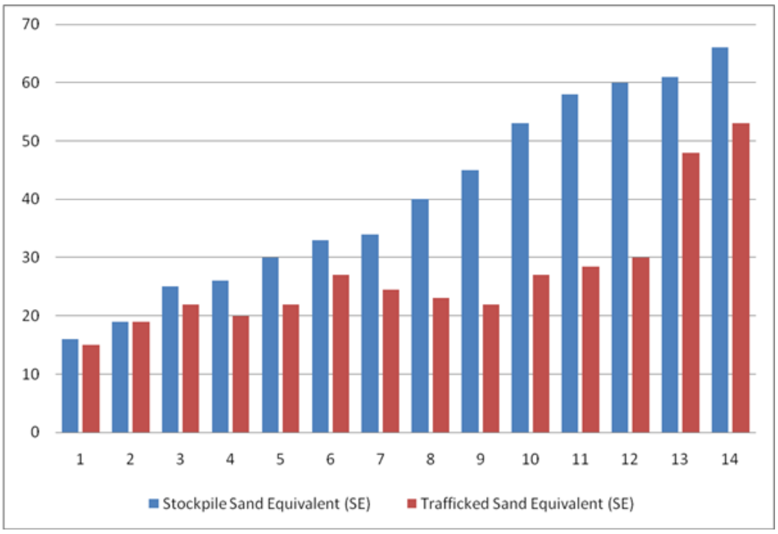
Drawing on the practice of post-compaction testing (for both grading and any one of the three fines criteria) provides a second option.

Option 3

The practice of requiring all three of these fines criteria to be satisfied (option 3) appears conservative but is known to be overly onerous and unnecessary in some regions.

The origin of the sand equivalent test was as a predictor for the plasticity index. Examination of the data (PI-SE correlations) from which the sand equivalent of 40 was derived, indicates that specifying a sand equivalent of 40 produces a very high probability of excluding materials with plastic fines or a high percentage of fines. However even with a value as low as 20, some aggregates (but a minority) can still be non-plastic (O'Harra 1955). The ASTM limits are 35 for basecourse and 30 for subbase.The concern is that many basecourses with proven good performance (viz all those with stockpile sand equivalents of over 20 in the following plot) can readily revert to sand equivalents much lower than 40 after relatively minimal trafficking. Using those cases from the inventory for which both stockpile and trafficked sand equivalents were available, the majority degraded markedly:

Figure 5.1 Reduction in sand equivalent in 14 New Zealand basecourses after trafficking



Others (Hudec et al 2008) also report a rapid drop in sand equivalent from 60 to 30 after minimal trafficking and this has been included in the plot above.

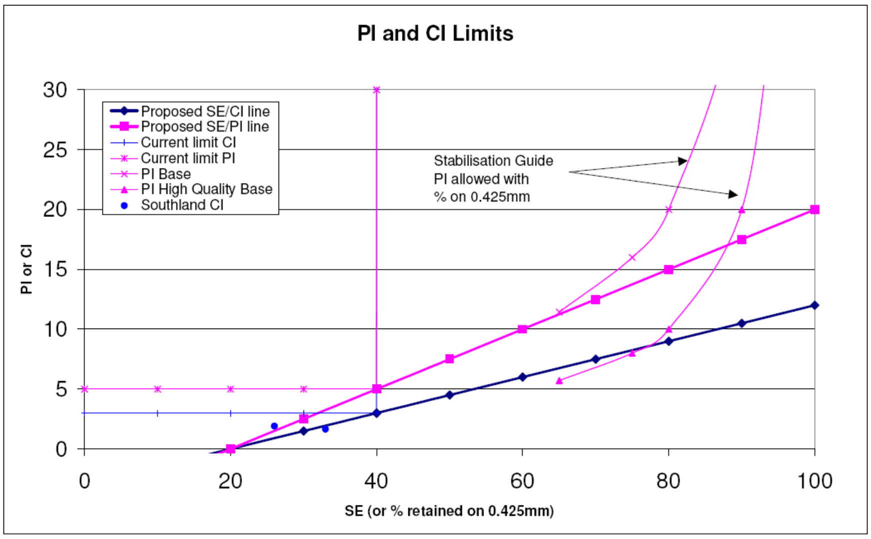
The inventory indicates that many well performing basecourses with stockpile sand equivalents of over 40 will soon degrade to less than 30 after relatively minor trafficking.

Hence, a prescribed value of 40 or more before compaction is difficult to justify as a rational criterion for rejection of any source. The industry needs to consider whether there is in fact any evidence that the present limit should be retained, especially for roads with low to moderate traffic loading. Sand equivalent (if over 20) is basically a measure of the probability that a basecourse will have a low percentage of fines that are non-plastic, rather than giving any absolute measure of suitability for a long-life basecourse.

Option 4

One approach to the issue of appropriate fines criteria (D Alabaster pers comm) is to apply a trade off between sand equivalent and clay index, and also between sand equivalent and plasticity index, as follows:

Figure 5.2 An approach to providing fines criteria based on interaction of indices



The origin of the graph appears to be based on the observed satisfactory performance of basecourses with sand equivalents in the range of 25–35 in Southland and Otago. The point of interest is that the study also regards 20 as being a suitable cut-off for sand equivalent, provided clay index and plasticity index are suitably low.

The criteria in the above figure may be quantified as follows:

1 SE >20

2 PI <0.25* SE - 5

3 CI< 0.15* SE -3

with all three to be satisfied for compliance.

Option 5

There may be considerable extrapolation that needs to be verified with more case histories but in view of earlier discussion on the merits of weighting both the plasticity index and the clay index, (section 2.4.1), an interim translation (that should tend to be conservative) of the three criteria in option 4 leads to:

1 SE> 20

2 WPI< 3* SE – 60

3 WCI< 0.75* SE -15

with all three criteria to be satisfied.

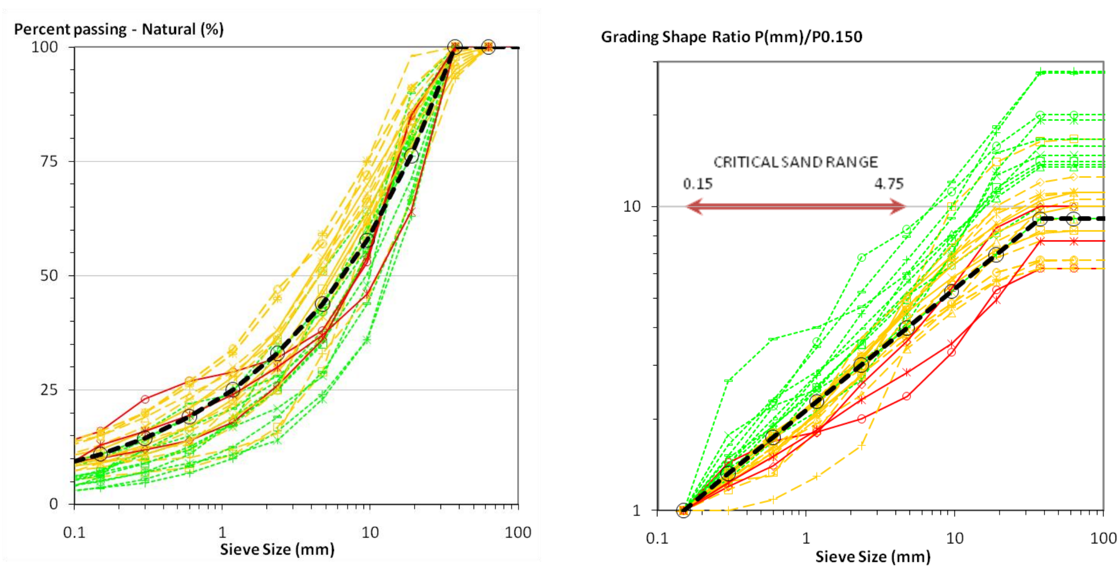
An alternative approach to the issue is discussed in section 5.4.

5.2 A grading indicator of basecourse instability

Many of the poorly performing basecourses in the inventory show either excessive fines or a deficit of sand sizes, as shown by a flattening (in the customary cumulative particle size distribution curve) between the 0.15mm and 4.75mm sieve sizes. Many of these are from hard rock quarries where extraction (either ripping or blasting) often (but not always) will generate a mix of angular gravel sizes and silt. The consequence of a sand deficit is the tendency to either have the gravel particles floating in a silt matrix or, if there is no silt, to have sparse point to point contacts between gravel particles that will inevitably encourage degradation. *Ensuring sufficient sand to increase the frequency of point to point contacts in the structural matrix is therefore fundamental to shear stability*. This is consistent with Bartley's (2007) promotion of the concept of adding sand to basecourses. Figure 5.3 shows plots for some of the basecourses in the inventory using a dotted (green) line for well performing aggregates, a dashed (orange) line for marginal aggregates and a solid (red) line for poorly performing aggregates.

A simple way to illustrate the gap-grading characteristic is to plot the particle size distribution curves using the 0.15mm sieve (the primary sieve size affecting permeability, Cedergren 1989) as a base value and show the 'grading shape ratio', ie the ratio of the percentage passing each subsequent sieve size to the percentage passing 0.15mm. The right graph shows the same distributions as the left graph but re-plotted in the above manner. A continuous straight line on this plot implies a constant value of Talbot's grading exponent 'n' between adjacent pairs of sieve sizes (Talbot and Richart 1923). Grading exponents of about 0.4 and 0.6 correspond to the fine and coarse limits of the M/4 specifications. Talbot's exponent is a measure of the degree to which the coarse particles float in a matrix of fines. Values less than 0.3 always give unacceptable stability for unbound granular layers while greater than 0.4 (throughout the sand range in particular) is normally associated with good stability. Values higher than about 0.6 become difficult to construct without segregation.

Figure 5.3 Particle size distributions for variably performing basecourses



From the above figure, it is evident from the left graph that the coarser gradings often perform better than finer gradings, but this is not a consistent finding. However, the right graph provides a much more reliable separator of good and poor performers when the sand range (0.15mm to 4.75mm) is considered, and provides a simple way of discerning gap grading in the critical range.

A critical lower bound value for Talbot's grading exponent 'n' of 0.4 is identified on both graphs as a dashed black line. A useful feature of this type of plot is the same figure may also be used for assessing finer materials (running courses or all passing (AP)20 basecourses) or coarser aggregates (AP65 subbases).

The gap-grading concept was explored by trialling a number of parameters to see which best ranked the relative performance of aggregates. The 'incremental n' value (n_{1-2}) has been calculated over specific sieve intervals and found to be relevant:

$$\textit{Incremental grading exponent } n_{1-2} = Log_{10}(P_1/P_2) / Log_{10}(d_1/d_2) \qquad \text{(Equation 5.1)}$$

where:

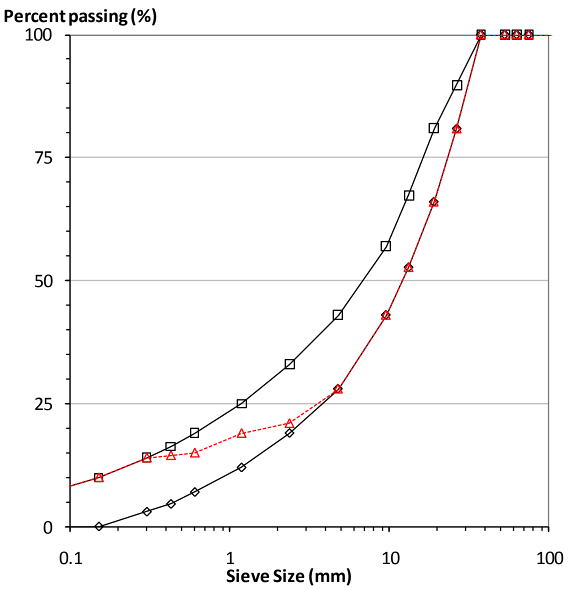
- P is percentage passing the specified sieve sizes
- d is the chosen sieve size.

For example if the fractions passing the 4.75mm and 0.3mm sieve sizes are 44% and 14% respectively then the incremental grading exponent, $n_{4.75-0.30}$ is:

$$n_{4.75-0.30} = Log_{10}(44/14) / Log_{10}(4.75/0.30) = 0.41 \qquad \text{(Equation 5.2)}$$

M/4 grading shape control implicitly allows incremental n values that are much less than 0.4 as shown in figure 5.4.

Figure 5.4 Gap-grading (dashed line) currently permitted under TNZ M/4 (Transit NZ 2006) (upper and lower envelope limits shown as solid lines)



Ironically, for hardrock quarries the above form of grading may often be the cheapest solution for producers to comply with M/4, ie minimum processing which results in maximum silt and minimum sand. Yet this is the most adverse grading for shear stability.

It should be noted that the origin (40 years ago) of the M/4 grading shape control was based on experience and judgement rather than a systematic study: 'we put it in because we thought it ought to be there'. (Norm Major, pers comm). Therefore while the importance of avoiding gap grading at that time was appreciated, case histories now demonstrate that grading shape control is imperative and now needs to be slightly more restrictive for the increasingly heavy traffic loadings. M/4 grading shape control implicitly requires the incremental grading exponents (n) for a four-fold increase in particle size (two standard sieve intervals) to be between about 0.2 and 1.2, ie this allows a substantial departure either way from the desirable exponent of about 0.5. A simple criterion found to be associated with virtually all cases of shear instability in basecourses, which are otherwise compliant with M/4, is where the incremental grading exponent (n) is less than 0.40 over the sand range, ie between 0.150mm and 4.75mm. Four 'governing' sieve ranges for calculation of n, have been determined by regression. If any two of these are less than 0.40, or are likely to become so after compaction, particular caution appears to be indicated by the case histories of in-service pavements.

A simple functional calculator is provided in table 5.1 and also at Tonkin & Taylor Ltd (2010b). This will identify whether any given grading curve will meet the proposed tighter grading shape control limits. Note: Only the second column (yellow cells) should be modified to determine a pass or otherwise.

Table 5.1 Calculator for suitability of particle size distribution for base course

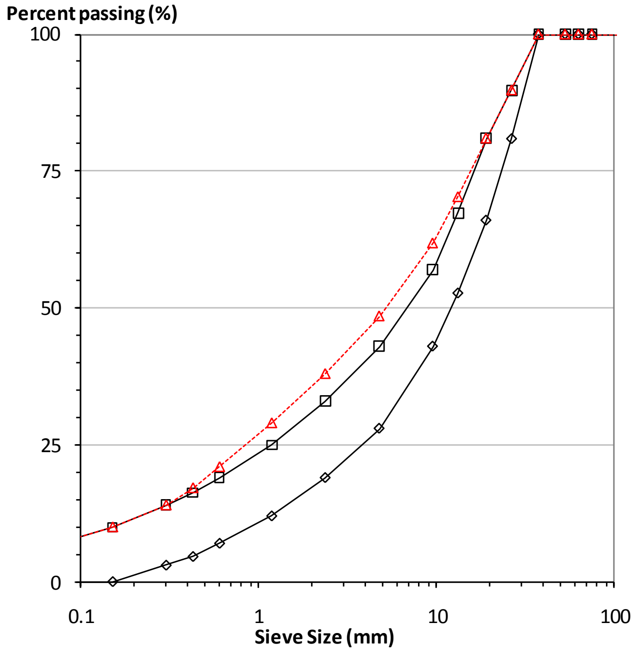
Sieve size (mm)	% Passing	Critical range	Grading exponent
4.75	44	4.75mm-0.30mm	0.41
2.36	33	2.36mm-0.15mm	0.43
1.18	25	1.18mm-0.15mm	0.44
0.6	19	0.60mm-0.15mm	0.46
0.3	14	SGE (average of 2 lowest)	Pass/uncertain
0.15	10	0.42mm	Pass

The average of the lowest two exponents in the above set is here termed the 'sand grading exponent' (SGE).

In this example the two lowest grading exponents in the table are 0.41 and 0.43, and the SGE is given as the average of these, ie 0.42. At this stage it is suggested that if any proposed basecourse aggregate does not pass this criterion, (ie the SGE is less than 0.4) then particular care should be taken to ensure there is good evidence of precedent performance of the same basecourse (material and grading) in similar situations (traffic and environment) to that intended. If not, logical solutions would be cement modification (or foamed bitumen stabilisation) of the basecourse. However, as cement production requires intensive use of energy, and bitumen comes into the fossil fuel category, there will be growing expectations for the industry to focus on alternatives, namely production of unbound aggregates that are well graded in the first place.

The M/4 specification implicitly allows an SGE as low as 0.26 (figure 5.4). As a minimum SGE of 0.4 will provide a tightening of the current specification, it may benefit the industry to introduce at the same time an accompanying relaxation. As discussed in section 4.6, Bartley (2007) recommends shifting the M/4 limit from the present grading exponent of n=0.4 to n=0.35. From the mid-range of the grading to the coarse end, there are numerous well performing aggregates that support the call for increasing the upper limit. The following figure shows a compromise fine grading (dashed) in relation to the current M/4 limits (solid lines).

Figure 5.5 Proposed increase in the upper limit of the M/4 grading to n=0.35 above 4.75mm (dashed line)



5.3 Correction of gap grading

An environmentally favourable method for correcting gap grading in the sand fraction is to add cullet (recycled glass, crushed to pass the 4.75mm sieve). Up to 5% cullet is permitted in M/4, while any greater percentage requires NZTA approval. At present, M/4 requires that cullet must contain at least 35% and less than 88% passing the 2.36mm sieve, and while these values are convenient for production anyway, the reason for such criteria is unclear and possibly can be relaxed with further research or trials over short sections of in-service pavements. The 5% limit is not likely to be a significant constraint in practice, but greater proportions would still be expected to enhance the performance of gap-graded basecourses. The cullet grading is probably immaterial if the resulting blend complies with M/4 in all respects, and because of the lack of clay minerals, increasing the proportion passing the 0.75mm sieve above the normal M/4 limit of 7%, may also be practicable if the other fines criteria remain satisfied. Further research appears warranted. Ideal cullet gradings that would best remedy the gap grading in typical Auckland greywacke hard rock quarries and also in Canterbury alluvial sources are given in figure 5.6a.

Figure 5.6a Ideal gradings to complement typical sand deficits with required blend percentages

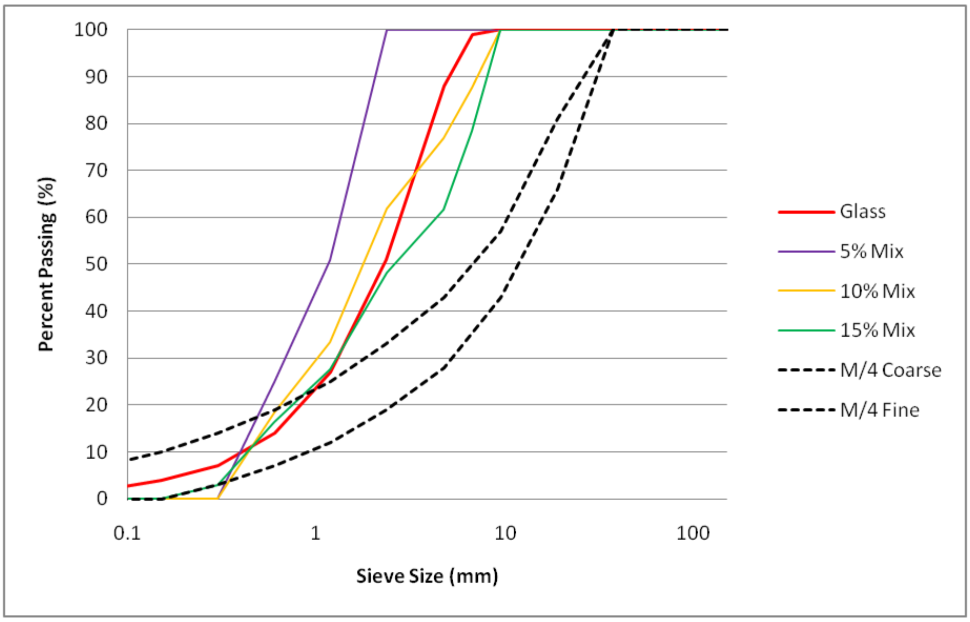
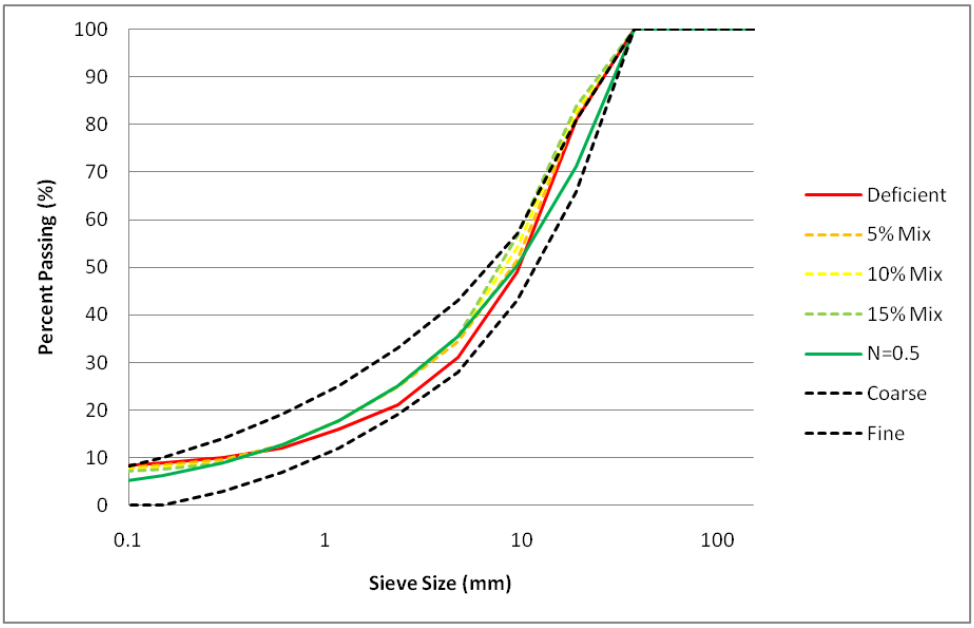


Figure 5.6b Resulting gradings showing the improvement of the original deficient mix with the different blends



5.4 General grading shape control

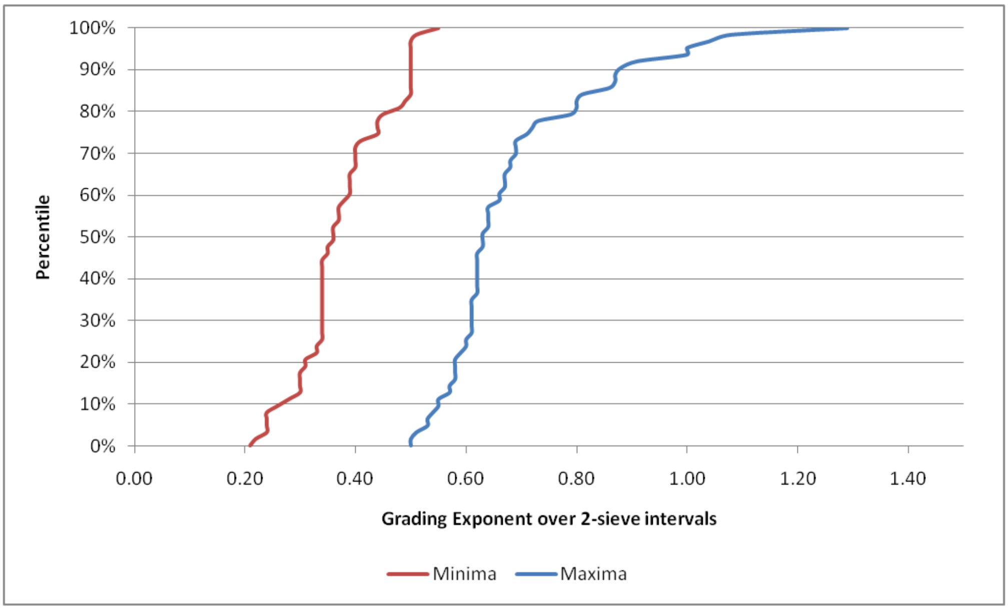
The criteria for ensuring general grading shape control for the full range of sieve sizes is currently provided by table 3 of M/4, shown here as table 5.2.

Table 5.2 Grading shape control in current M/4

Fractions	Maximum and minimum allowable percentage weight of material within the given fraction	
	AP40 (max size 40mm)	AP20 (max size 20mm)
19mm–4.75mm	28–48	–
9.5mm–2.36mm	14–34	20–46
4.75m–18mm	7–27	9–34
2.36mm–600μm	6–22	6–26
1.18mm–300μm	5–19	3–21
600μm–150μm	2–14	2–17

The absolute percentage passing ranges given in M/4 table 3 (or table 5.4 above) result in different acceptable incremental 'n' values, depending on whether the grading is following either the upper or lower grading limit. More consistent grading shape control could be obtained alternatively by specifying that the *incremental grading exponent over the range of sieve sizes should be constrained between specified limits*. The merits of this approach have been put forward by others (Major et al 1990). To provide a similar shape control as intended with M/4, table 3, the minimum and maximum grading exponents over the full range of sieve sizes would need to be set at 0.24 and 1.2 respectively. The criteria should be applied in the traditional manner, ie over two standard sieve increments rather than one, in order to avoid unnecessary sensitivity. The following figure shows the distribution of maximum and minimum grading exponents (over two standard sieve increments) for all M/4 compliant basecourses from the inventory.

Figure 5.7 Typical distribution of upper and lower bound incremental grading exponents over two sieve sizes



These suggest that more restrictive limits of 0.3 and 1.0 would still encompass the vast majority of basecourses. Tighter shape control to ensure well graded material is considered fundamental for long-life basecourses. The change from M/4 would be to simplify table 3 with the following:

Table 5.3 Proposed simplification for M/4 grading shape control

Incremental sieve size range	Incremental grading exponent for each combination of sieve sizes	
	Minimum	Maximum
19mm–4.75mm 9.5mm–2.36mm 4.75mm–1.18mm 2.36mm–600μm 1.18mm–300μm 600μm–150μm	0.3	1.0

Notes:

The criteria apply for every combination of 2 standard sieve size increments where the percentage passing is < 100% and > 0%.

An incremental grading exponent of less than about 0.4 indicates a deficit of particles in that range while greater than about 0.7 indicates a surplus.

Calculations are shown as part of the basecourse inventory spreadsheet at Tonkin & Taylor Ltd (2010a).

The above limits could then be simply tightened for the specification of higher quality aggregates, or relaxed where appropriate for lesser demand including specifications for subbases. The maximum value could, if necessary be increased to 1.2 if the industry finds difficulty in meeting the target. There should not be a difficulty in achieving the minimum of 0.3. Any reduction to 0.2 (the limiting value effectively permitted with M/4:2006) should be subject to further study, should the producers confirm a need to revert to this very low bound.

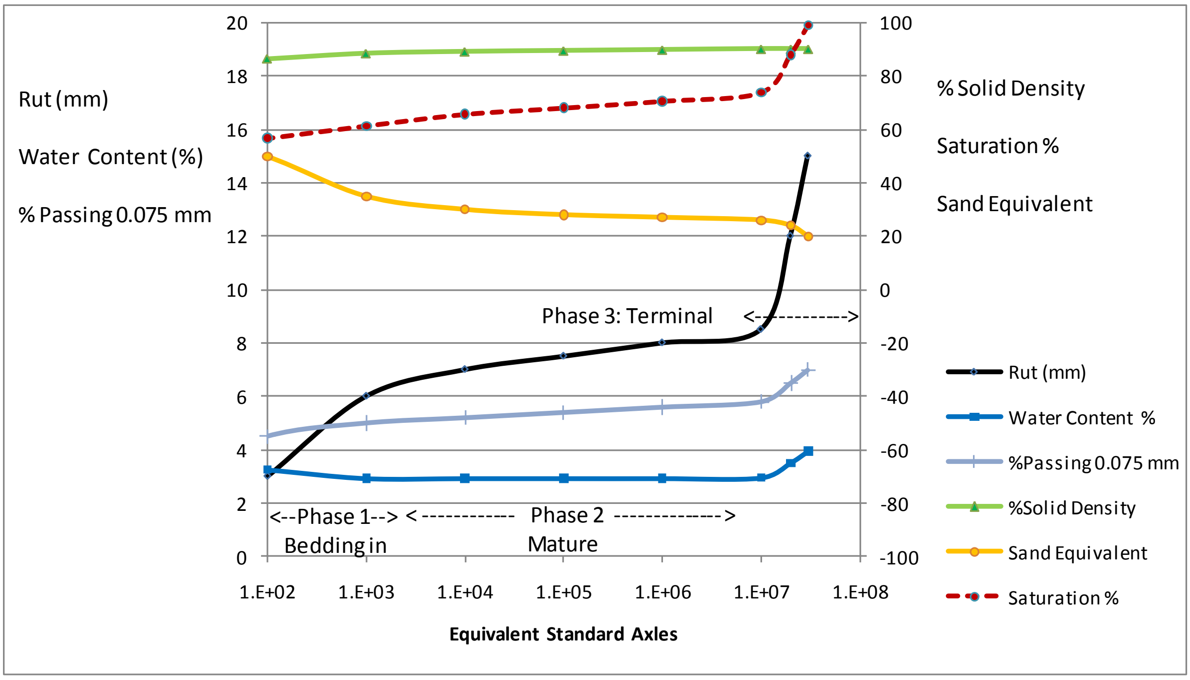
The proposed process is more systematic and allows grading shape control to be very simply quantified, when comparing the performance of two aggregates. Grading shape control addresses multiple issues, namely shear instability which is governed by the lower limit while the upper limit governs constructability (segregation), compactability, and surface interlock. With grading shape control expressed in this form there will be no need to have a separate table for AP20 grading shape control, as the limits in this form are independent of topsize.

5.5 Pavement life phases

Basecourse performance is a compromise. A dense grading is important to avoid rutting in the bedding-in stage (Bartley 2007). An open grading with low fines may rut marginally more but will be better able to accommodate the inevitable increase in fines as degradation is induced by trafficking. The open grading will therefore postpone the terminal long-term condition when the accumulated fines leave insufficient voids to keep the degree of saturation lower than 70%.

Ultimately, when the saturation limit is approached, further densification will not occur but rutting will continue as a result of shear instability (lateral shoving). This mechanism can be illustrated graphically in conjunction with the traditional presentation of the three phases of basecourse life as shown in figure 5.8.

Figure 5.8 Basecourse life phases



To look at the interaction of the variables and their sensitivity, a median example for average dry density, solid density and water content for a basecourse beneath a thin surfacing was adopted from the inventory. Taking these as initial values, assuming a 125mm thickness of basecourse and a typical rut progression curve as shown in figure 5.8, the corresponding change in degree of saturation can be calculated directly. Other typical parameters are also shown. This assumes the rut is only that portion of the total rut that is attributable to the basecourse alone (deducting the component from deeper layers).

The initial percentage of solid density is assumed to be 85% (the average from the inventory) and during the phase 1 bedding-in period, this increases to 89% (11% total voids) and the value apparently increases very slowly thereafter during the mature phase 2. The initial degree of saturation is calculated to be under 60% which increases along with % solid density until a value of 70%–80% is generated towards the end of the phase 2 life. At this point (marking the beginning of the terminal phase 3) the dynamic pore pressures from trafficking will be sufficient to initiate shoving, followed soon by cracking, ingress of ponded water and consequently the saturation rises more rapidly.

Taking typical values for basecourse parameters therefore gives a reasonable mechanism for the widely recognised process of accelerated rutting (other factors may well apply also).

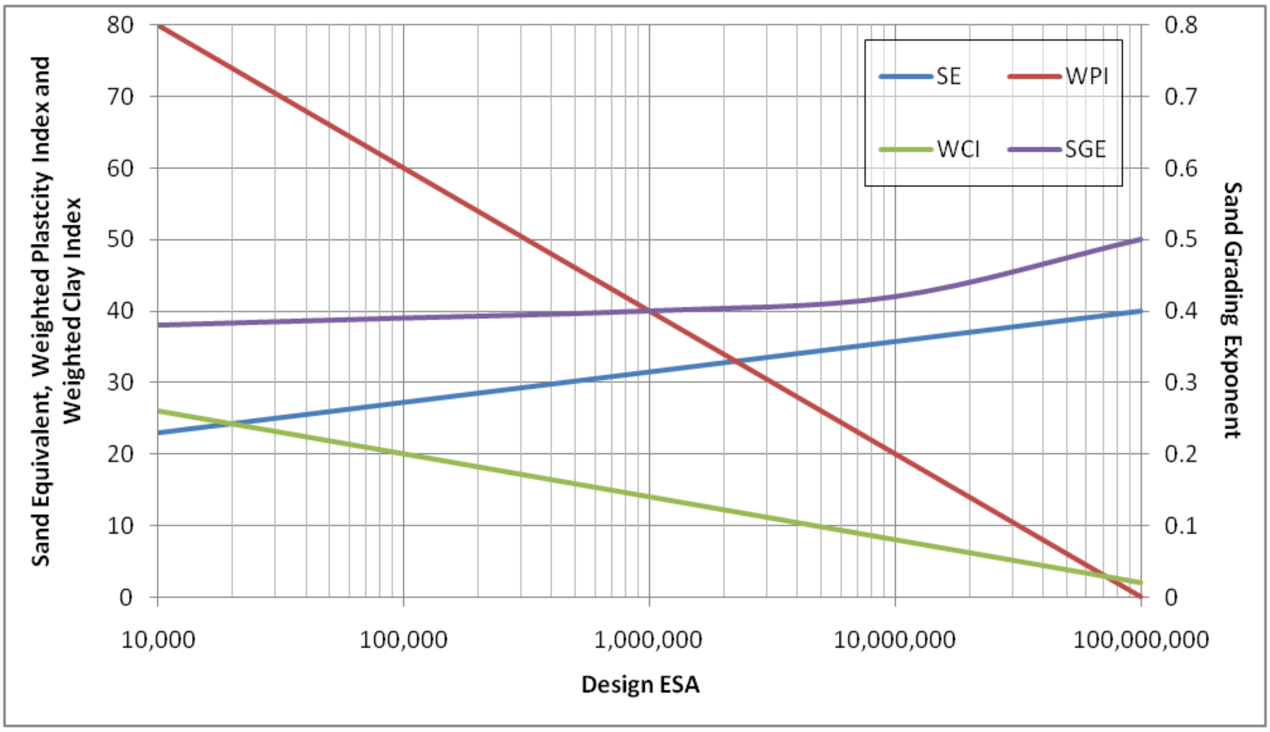
The point from this typical case is that saturation of a densely graded basecourse will change from being no problem (50%–60%) saturated to well over 70% saturated as a result of a 5% change in density induced by trafficking during phases 1 and 2. On the other hand, if there is no densification and if fines (from traffic degradation) are generated that will increase the equilibrium water content by just 2% then the critical saturation limit will again be reached. In practice, what is likely to occur is a combination of both of these, ie say a 2%–3% increase in density and 1% increase in equilibrium water content, to reach critical saturation. There is a relatively fine margin between stable performance and severe shoving, and this is the reason for the difficulty in predicting the performance of marginal basecourses.

5.6 Relating fines criteria to design traffic loadings

If the stages in basecourse life are as represented in figure 5.8, then all basecourses with thin surfacings will eventually experience shear instability as a result of inevitable particle degradation under heavy traffic loading.

This suggests that, rather than assigning a simple set of pass/fail categories to the fines criteria, there is a case for a shift in focus to the ranking of basecourses in relation to the number of ESA that are required in the design life of the pavement. This has support from a number of members of the Aggregate and Quarries Association and has substantial implications for the optimum use of resources, as low-quality aggregates can be used on minor roads while premium aggregates are preserved for the more demanding pavements. The basecourse inventory would require more case histories to provide a definitive set of ESA-related criteria. However a provisional concept for ESA ranking is shown in figure 5.9:

Figure 5.9 Provisional fines criteria related to design traffic loading



Where the weighted plasticity and clay indices, WPI and WCI (discussed in section 2.4) are:

WPI = PI x %Passing 425 micron

WCI = CI x %Passing 75 micron

For a basecourse specification, a technically appealing alternative (option 6, continuing the numbering given in section 5.1) is to require all four fines criteria to be satisfied for the design ESA.

Option 7 would be to require a stepped ESA requirement such as:

Table 5.4 Suggested fines criteria in relation to lifetime design traffic

Design ESA	Fines criteria to be satisfied (SGE, WCI, WPI & SE)
< 50,000	any one criterion
50,000 – 1 million	any two criteria
1million – 5 million	SGE, WCI and one other
> 5 million	SGE, WCI, WPI & SE

Option 8

If the above is considered to be too big a departure from the present standard, an interim improvement would be to require that any two of the four fines criteria are satisfied as follows: SGE >=0.4, WCI <=15, WPI <= 40, SE>=40 or maybe >=30. This allows a simple alternative to revert to a supply specification only, (which does not require design inputs such as ESA loading). This option has the advantage that almost all basecourses complying with M/4 would still comply while those most likely to exhibit premature shoving would not comply. However, some basecourses that would probably experience shear instability after about 5 million ESA would also be classed as compliant.

Where gap grading is marginal (SGE close to 0.4), additional representative sampling (as-delivered and preferably also as-compacted, rather than from stockpiles) to assess the variation and range of aggregate quality used during construction is important, as a relatively small shift in gradation can result in a major change to shear stability performance.

Each of the four fines criteria should be determined after compaction (preferably field compaction, otherwise in the laboratory) if any of the following applies:

- the aggregate source does not have a record of satisfactory performance
- the design traffic is greater than 1 million ESA
- the crushing resistance is less than 180kN
- the aggregate is blended (comes from more than one source).

Post compaction testing from samples recovered from the shoulder during construction has the disadvantage that it may cause contractual issues between the supplier and the construction contractor but post-compaction QA of this type is being practised elsewhere (VicRoads 2008a) and there is a substantial advantage: the risk can be confronted (either at the design stage, or early in the construction stage), rather than at the time of premature failure.

The *New Zealand supplement to the document, Pavement design, a guide to the structural design of road pavements* (*NZ supplement*) (Transit NZ 2007a) refers to Queensland practice where the practice is to use only structural asphaltic pavements if the design traffic exceeds 14 MESA. The *NZ supplement* implies a limit for New Zealand of about 10 MESA because a rutting rate of about 2mm per MESA is indicated as the expectation from the granular layers (based on RLT tests). However the in-service rut depth measurements now available from New Zealand LTPP sites on state highways provide evidence that, while pavements will exhibit 2mm/MESA rutting rates in the early part of their life, in well built pavements this tends to reduce by an order of magnitude to more like 0.2mm/MESA over the more mature stages. This probably explains why Bailey et al (2006) found, when looking at a wide selection of 79 sites due for rehabilitation throughout the New Zealand state highway network, that none of them had reached their terminal condition as a result of rutting.

When producers are targeting TNZ M/4, reduced life will occur in some instances. However, projecting the rutting and roughness trends in the NZTA's LTPP sites indicates there is no reason (subject to close attention to grading shape control and fines criteria as above), that a durable aggregate in a well constructed pavement should not be capable of substantially greater trafficking than 10 MESA. A credible upper bound for unbound granular pavements may be postulated by taking a reasonable early life bedding of 8mm of rutting, then provided the rutting rate is the same as found on the better LTTP sites at 0.2mm/MESA, the expected rutting life to 20mm rut would be at over 60 MESA.

Midgley (2009b) commenting on VicRoads experience, alludes to a 70 million ESA capability for unbound aggregates and attributes the current shortfall in pavement performance to limitations in technology transfer:

> *It was clearly evident that the expertise built up over past generations involved in the construction of heavy duty (up to 8000 heavy vehicles per day equivalent to 70 million ESAs) unbound flexible pavements comprising crushed rock had not been adequately transferred to the current generation involved in constructing such pavements nor had the knowledge been very well recorded in guides and publications.*

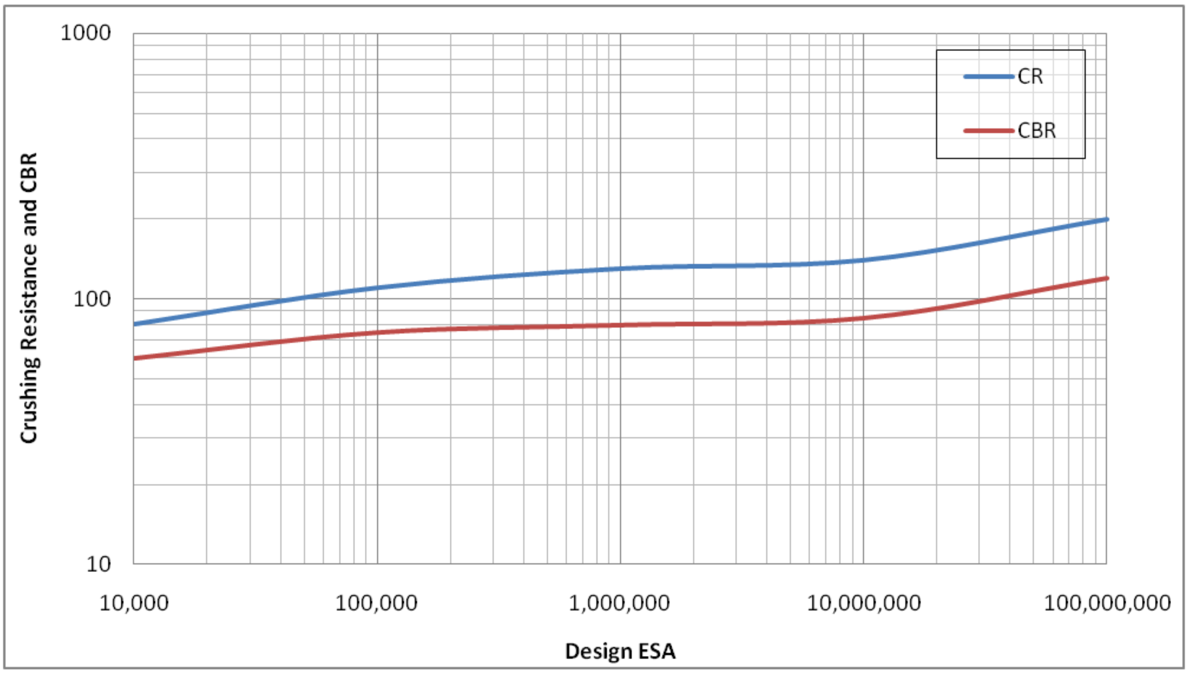
VicRoads also specifies different classes of aggregates depending on traffic loading. The current M/4 is primarily a product supply specification, independent of traffic loading. This provides contractual simplicity, but compromises sustainable use of resources and tailoring of designs. There is some precedent with the existing M/4 regional basecourse variants, namely, for those which already have prescribed maximum ESA ratings.

From inspection of the M/4 regional variants, M/22 notes and case histories of pavement performance (section 7.2), other parameters that would also be reasonable to specify in relation to design traffic are:

- crushing resistance (CR- allowed below 130kN for design traffic less than 1 MESA, in M/22)
- CBR – if the crushing resistance exceeds 130kN the CBR test can be omitted for less than 0.1 MESA
- percentage of broken faces
- grading limits (including percentage passing 75 micron sieve)
- grading shape control (in terms of maximum and minimum grading exponent over two successive standard sieve size intervals).

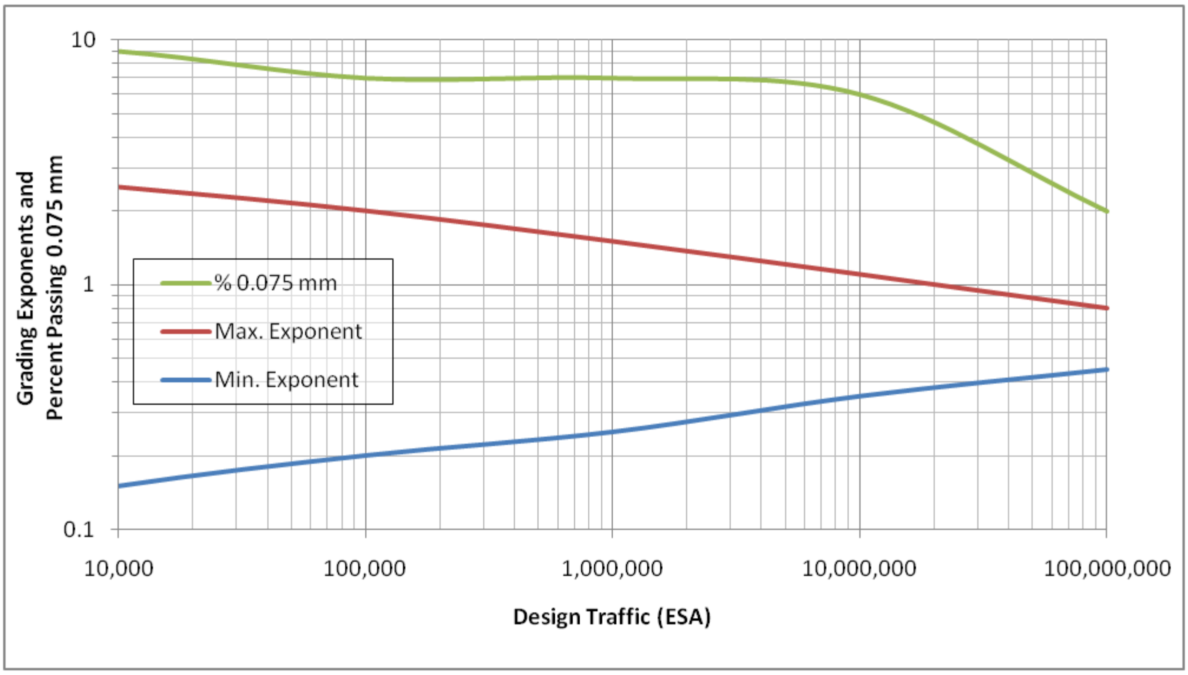
Suggested criteria are shown in the following figures.

Figure 5.10 Crushing resistance and CBR criteria



Similarly for grading exponents and percentage of fines passing 0.075mm fines, see figure 5.11.

Figure 5.11 Percent fines and grading exponent criteria



Eventually there will be substantial cost-benefit resulting if a basecourse specification is developed so the criteria within it are linked to design ESA. However, this may take some time for widespread acceptance throughout the industry and implementation. Therefore, in view of the ongoing cases of severe premature distress of basecourses being reported, an interim simplified upgrade of M/4 may be a more practical course in the short term. A draft revised specification constrained in this manner is given in appendix B.

However, the M/4 notes already acknowledge that pavement design may require departures from the basecourse specification to suit circumstances: ie

> *where above minimum quality (eg stone quality), less severe service conditions (eg loading or drainage) occur, or where M/4 materials have resulted in poor performance, alternative specifications may well be in order. For assurance with such variants two prerequisites are required:*
>
> *(i) compensating properties or loadings,*
>
> *(ii) demonstrated (or inferable) performance,*

Approval from the NZTA's Policy Manager is required

> *by conducting agreed tests to prove the suitability of the material*

And in a later section:

> *Although materials which have a crushing resistance less than 130 kN cannot be classified as M/4 basecourse this does not necessarily preclude their use where stronger aggregates are not available. The use of these materials however will require other specification changes representing a significant departure from the [NZTA] standard pavement design procedure.*

These concepts have been developed further by incorporating the ESA dependent parameters above, into the draft M/4 notes in appendix B.

6 Quantifying basecourse life

Using the standard basecourse (M/4) test results from the inventory (especially the particle size distribution after trafficking), preliminary evaluation of the data from failed sites was carried out to explore the extent to which it would be possible to predict pavement life (in terms of ESA) where shear instability of basecourse is the governing mode of distress at the terminal condition.

6.1 Saturation

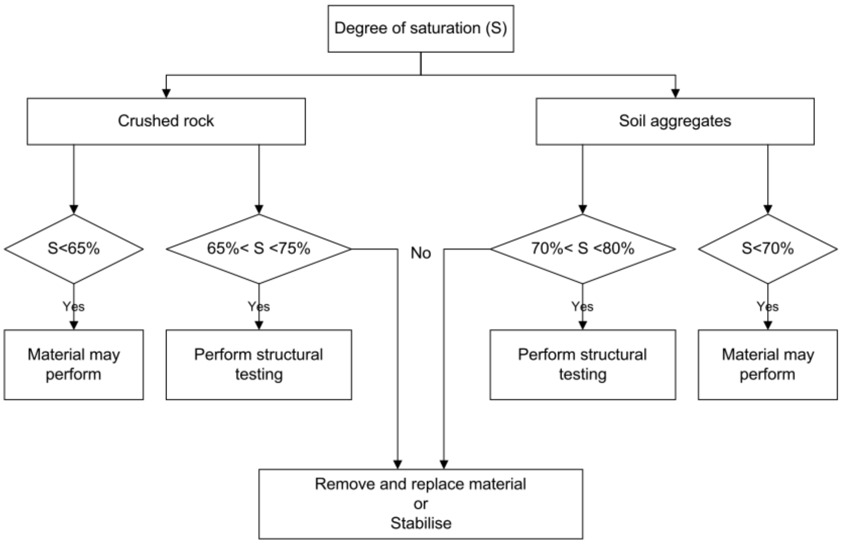
Some of the most premature failures of New Zealand pavements have been attributed to basecourse shear instability. Two schools of thought have prevailed, ie those that relate basecourse instability to excess dynamic pore pressures when degrees of saturation are high (Transit NZ 2005, B/2 and B/2 notes; Transit NZ 2006, M/4 notes; ARRB Group 2003; Readshaw 1967; Buckland 1967; Toan 1975; MRWA 1993; Theyse 2002; Hutchison 2006; Craciun and Lo 2008; Gartin and Raad 1992), or those that consider instability to be governed by the water content of the fine fraction relative to its plastic limit. The inventory suggests saturation is the key issue. The current concern is, while most aggregates complying with the NZTA basecourse specification tend to have acceptable values of saturation at equilibrium water content, some evidently do not. Some tightening of the specification is therefore indicated. The objective is to provide a specification that will ensure all compliant aggregates will have an acceptable level of saturation both at the time of sealing and, more importantly, at equilibrium water content once they attain refusal density (after bedding-in trafficking). The latter is also implicit in the *Performance Based Specification For Structural Design And Construction Of Flexible Unbound Pavements* (TNZ B/3 provisional: 2000) clause 12.1, ie 'the Contractor shall design the drainage requirements and demonstrate how this will ensure full (100%) saturation of the pavement materials will not occur over the life of the pavement'.

MRWA (1993) provides a concise summary on acceptable saturation:

> *There is considerable evidence that significant pore pressures will develop in paving materials under traffic loadings where the degree of saturation exceeds 80%. Even at low saturation levels, traffic loadings may pump water in porous pavements and cause high pore pressures in localised parts of the pavement. Generally levels of saturation above 70% cause problems. However, levels of saturation as low as 60% in crushed rock have also been known problems on occasions. Measurement of field moisture content can be misleading as materials can be at or above 80% of saturation at moisture contents slightly dry of optimum.*

Vorobieff et al (2001) provided the following flow chart:

Figure 6.1 Vorobieff et al 2001: Response to saturation



'Perform structural testing' presumably relates to undertaking RLT testing. However, the concerning result from the recent Austroads 'Better Basis for Bases' research project discussed above (Jameson et al 2008; 2009) was that of three methods investigated for RLT evaluation, none was capable of reliably predicting premature failure of basecourses. The research identified the need for a 'performance index' that would adequately characterise basecourse stability. A prime contender for such a performance index could be the expected degree of saturation in the basecourse at refusal once equilibrium water content is reached, determined as described below.

It should be noted however that field measurement of density and water content both have some degree of error hence the calculated percentage of saturation will typically be accurate only to about 10% (Jennings et al 1998).

ARRB Group (2003) in *Technical note 13* also identifies the 80% limit and comments:

> *The stability of the majority of unbound pavement materials, significantly improves when the pavement is dried back to 70% or 60% in the case of very moisture sensitive pavements….. For major highways and freeways with a traffic loading in excess of 5x10^6 ESAs, a maximum degree of saturation of 60% for the base pavement prior to bituminous surfacing is recommended. For other roads the maximum may be increased to 65%.*

TNZ B/2 notes (Transit NZ 2005) quotes directly from *Technical note 13* (as above) and provides the following additional comment:

> *The original requirement in TNZ B/2 was to prevent the sealing of saturated pavements and did not consider rutting performance. Agreement as to the practicality of this (ie 60-65% limit) could not be reached within the industry and thus this concept will be trialled separately.*

The outcome of any associated trials is not known. However, in accordance with the intention of the above comment, the equilibrium degree of saturation (DOS) has been examined closely for those basecourses in the inventory that have experienced premature distress. The equilibrium DOS is taken as the saturation achieved after substantial trafficking when the basecourse can be expected to be close to its 'refusal' density. To predict the refusal DOS at the time of design/construction when selecting a suitable basecourse, it is of course necessary to provide means of estimating:

- equilibrium water content

- refusal dry density (preferably expressed as a percentage of solid density).

These two parameters, taken in conjunction with solid density lead directly to the equilibrium degree of saturation at refusal (S_{rr}) using the standard relationship which for the purposes of this study can most conveniently be expressed in the form:

$$S_r = (EWC\text{-}A) * G_{bd} * \%SD_r / (100\text{-}\%SD_r) \qquad \text{(Equation 6.1)}$$

where:

- EWC is the equilibrium water content in % (at refusal)
- A is the absorption (%)
- G_{bd} is the solid density (taken as bulk dry specific gravity as defined in section 4.6)
- $\%SD_r$ is the dry density at refusal expressed as a percentage of bulk dry solid density.

The reason the absorption is deducted is because the water absorbed in any voids within (rather than between) individual particles will not adversely affect the performance of an aggregate under cyclic loading.) In view of the difference in opinion regarding the calculation of 'total' voids and degree of saturation, equation 6.1 was derived using first principles as set out in appendix D.

It may be noted, where absorption is minimal, saturation is directly proportional to water content. For example, if the EWC is 3 (a typical value from the basecourse inventory) and ingress of water increases this by just 1%, the degree of saturation will increase by 33%.

This research therefore explores whether it is possible to predict with reasonable accuracy, both EWC and $\%SD_r$ using the parameters that are readily available from standard testing for compliance with basecourse requirements (M/4 specification).

6.2 Key parameters for saturation

6.2.1 Equilibrium water content

MRWA (1993) provided a regression equation based on a detailed study of in situ equilibrium water contents (EWC) of unbound granular aggregates in a well drained pavement:

$$EWC = 0.7\ OWC + 0.29\ (AR\text{-}PE) + 0.58\ P_{0.425}/P_{0.075} - 0.02\ P_{2.3} \qquad \text{(Equation 6.2)}$$

where:

- OWC is the optimum water content % (mod. AASHTO)
- $P_{2.36}$ is the percent of fines passing the 2.36mm sieve size
- $P_{0.425}$ is the percent of fines passing the 0.425mm sieve size
- $P_{0.075}$ is the percent of fines passing the 0.075mm sieve size
- AR is the annual rainfall (m) – appendix A
- PE is the potential pan evaporation (m) – appendix A.

MRWA (1993) also explored the performance of other than well drained pavements. Water will enter through the surfacing layer to some degree as well as migrating from the subgrade and shoulders (Hutchison 2006). Current basecourse inventory information is insufficient as yet to provide useful quantifiable data for other than well drained pavements, but revisiting this issue will be important as the inventory is expanded.

Charts showing rainfall and potential evaporation in Australia are given in MRWA (1993), reproduced in this report as appendix A. The annual average pan evaporation was used in that study rather than annual average actual evapo-transpiration (generally a much lower figure). The OWC is a relatively sensitive parameter and, in view of differences between rammer and vibratory compaction, should be able to be replaced by alternative terms relating to the percentage of fines. Various likely parameters were trialled for the present study involving mostly New Zealand aggregates with some from Australia.

Using linear regression analysis, the equation for predicting the EWC of New Zealand basecourses in well drained situations with intact seal is:

$$EWC = A - 0.2 + 0.134\ P_{0.075} + 0.045\ P_{0.150} + 0.02\ CI\ .\ P_{0.075} + 2.9\ AR - 3.19\ PET \qquad \text{(Equation 6.3)}$$

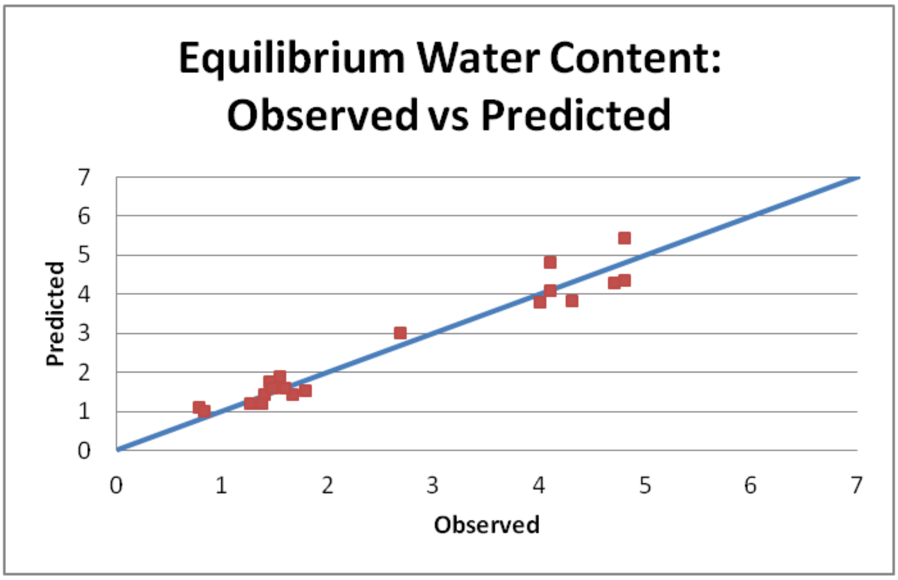
where:

- A is the absorption (%) of the aggregate
- $P_{0.075}$ is the percent of fines passing the 0.075mm sieve size
- $P_{0.150}$ is the percent of fines passing the 0.150mm sieve size
- AR is the annual rainfall (m)
- PET is the potential evapo-transpiration (m) (appendix B).

Potential evapo-transpiration (PET) used as pan evaporation (as in the Australian study) is not available in the form of a national chart of New Zealand. Numerically, PET is about half PE, so different coefficients would be necessary. An alternative, and perhaps more applicable, climatic indicator would be the Thornthwaite Index which is available for both countries and will be included in ongoing studies.

The correlation, using data currently available from the basecourse inventory, is shown in figure 6.2.

Figure 6.2 Prediction of equilibrium water content (%) for basecourses under intact chip seals



6.2.2 Density as compacted at construction

The laboratory density calculated from the compaction curve should provide the most convenient measure of as-compacted density in the field. Alternatively, estimation may be made from basecourse parameters. Suitable data from the basecourses in the inventory were analysed to see the degree to which dry density (expressed as a percentage of solid density: %SD) could be predicted – first at the end of construction compaction, and then after reaching refusal density after trafficking.

Interim regression on the relatively few data points gives:

$$\%SD_c = 105.5 - 14\,n - 5\,SGE - 0.02\,P_{19} - 0.025\,BF - 0.5\,P_{4.75}/\,P_{0.150} - 0.1\,P_{0.075} - 0.4\,P_{0.150} \quad \text{(Equation 6.4)}$$

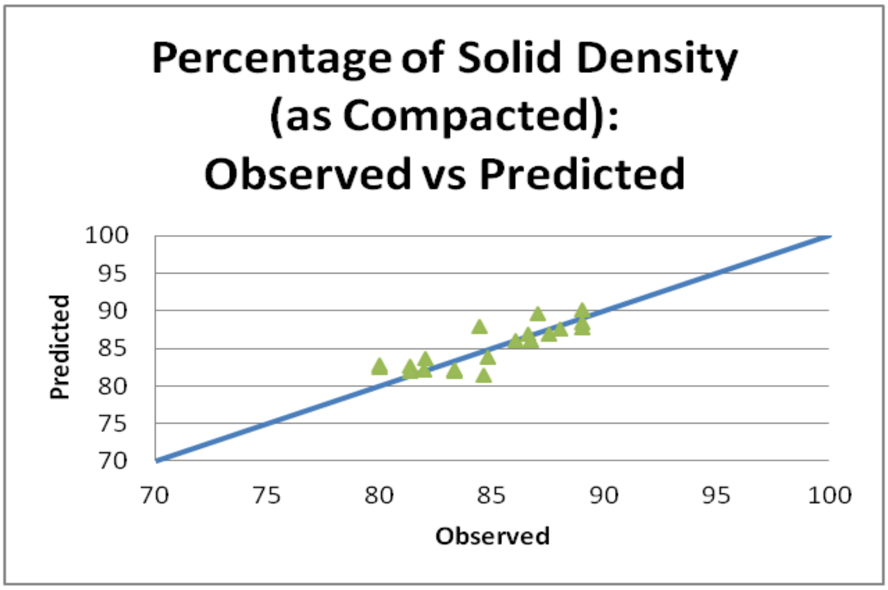
where:

- $\%SD_c$ is the percentage of solid density at the end of construction compaction
- n is Talbot's grading exponent
- SGE is the sand grading exponent
- BF is the percentage of broken faces (taken as 200% for quarried rock)
- P_{19} is the percent of fines passing the 19mm sieve size
- $P_{4.75}$ is the percent of fines passing the 4.75mm sieve size
- $P_{0.075}$ is the percent of fines passing the 0.075mm sieve size
- $P_{0.150}$ is the percent of fines passing the 0.150mm sieve size.

The SGE is similar to the Talbot exponent (slope of the particle size distribution curve on a log-log plot) but considers only the sand fraction (section 4.2).

The correlation for percentage of solid density using data available so far from the basecourse inventory is shown in figure 6.3.

Figure 6.3 Prediction of percentage of solid density as compacted (at construction)



As the inventory is expanded, additional terms may be warranted to incorporate aspects which are additional to the standard M/4 test parameters. A likely candidate would be a better descriptor of particle shape and eventually it may be practical to calibrate to general rock types or individual quarry sources.

The above regression should also be of use for checking the basecourse density achieved as part of the quality assurance process during compaction. Some authorities already specify a compaction criterion of around 20% total voids (80% solid density) and the inventory supports this as a good initial target for a coarse basecourse, but it will be unconservative in many cases. A more realistic criterion can be obtained using equation 6.4. A functional spreadsheet is available which calculates this target density from the results of standard basecourse (M/4) parameters, Tonkin & Taylor Ltd (2010a).

6.2.3 Density at refusal

At the end of construction but before trafficking, density may often by lower than laboratory density which in turn is generally significantly lower than refusal density after traffic compaction. Further study may enable a simple direct measure of refusal density using, for example, a modified gyratory compactor with suitably intense loading and number of cycles. Meanwhile, regression equations have been investigated.

If the laboratory compaction curve is used to determine the likely percentage of solid density after construction compaction (SD_c) then an approximate estimate of the ultimate percentage solid density at refusal after trafficking can be simply approximated by:

$$\%SD_r = 0.72\ \%SD_c + 28 \qquad \text{(Equation 6.5)}$$

Alternatively, using regressions from M/4 parameters in the inventory:

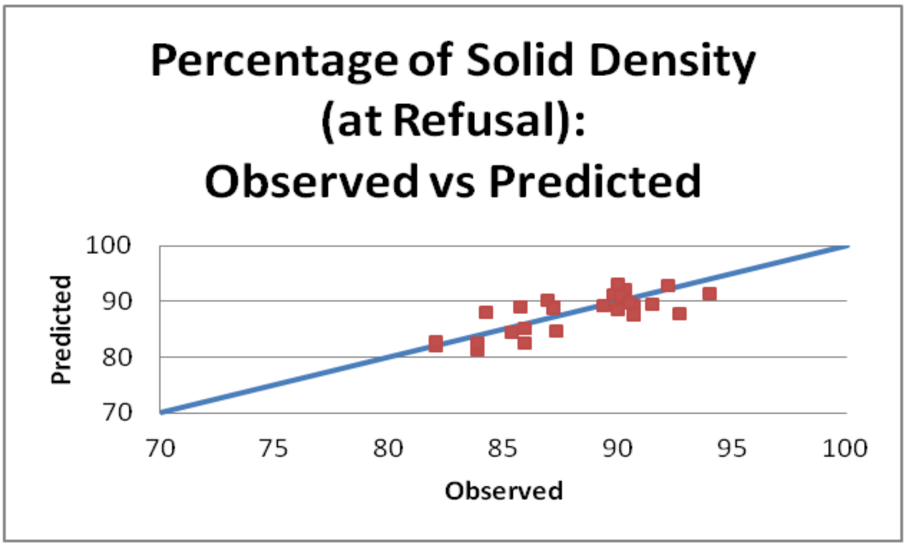
$$\%SD_r = 137.8 - 49.5\ n - 23.3\ SGE - 0.013\ BF - 1.7\ P_{0.150} \qquad \text{(Equation 6.6)}$$

where:

- $\%SD_r$ is the percentage of solid density at refusal
- n is Talbot's grading exponent
- SGE is the sand grading exponent
- BF is the percentage of broken faces (taken as 200% for quarried rock)
- $P_{0.150}$ is the percent of fines passing the 0.150mm sieve size.

The correlation using data available so far from the basecourse inventory is shown in the figure 6.4.

Figure 6.4 Prediction of percentage of solid density at refusal (after traffic compaction)

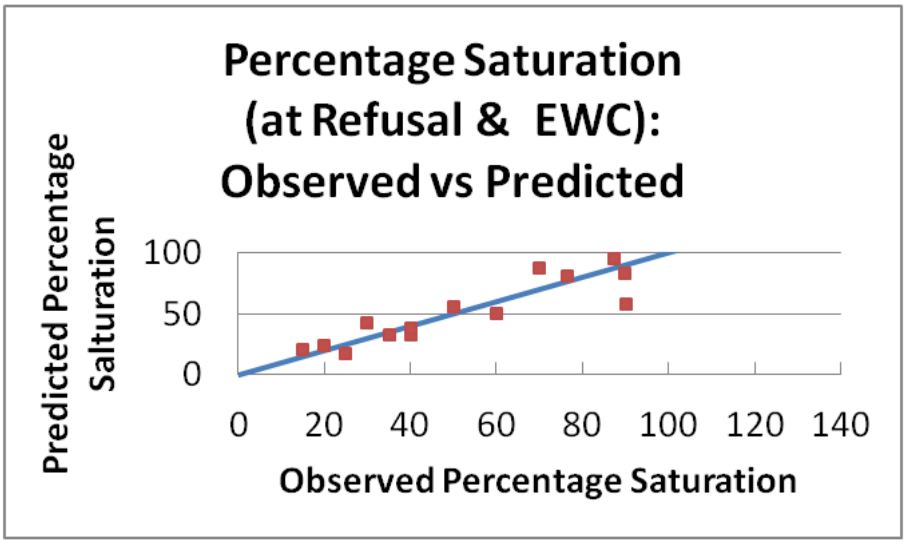


If the parameters are available for the prediction using both equations, then the subsequent calculation of degree of saturation should be conservative if the adopted value is weighted towards the greater value for percentage of solid density at refusal.

6.2.4 Degree of saturation at refusal

Using equation 6.1 and the regression results from figures 5.1 and 5.3, the final prediction for percentage saturation at refusal (S_r) is shown in figure 6.5.

Figure 6.5 Prediction of equilibrium degree of saturation (at refusal after trafficking)



A limitation at this stage is the number of case histories on which the regressions are based, and the pronounced change in some of the key parameters as basecourse is trafficked. All of the measures of the properties of the fines are susceptible to change in-service. In practice the absorption in the field may be lower than that determined in the laboratory test, and this issue may warrant further study as the basecourse inventory is expanded.

From the inventory data so far, it appears that for refusal saturation less than about 60% (the ideal range for reliable pavements), prediction is reasonably good, although still subject to a typical accuracy of 10%. Above 60%, prediction is poorer. This prediction of saturation should be regarded as interim and further case histories are needed. The concept appears sufficiently reliable to be used as a guide to practitioners who wish to explore the likelihood of saturation developing in a given basecourse that has no precedent from RLT testing or in-service performance.

To summarise the procedure the steps are:

1 Determine crushing resistance, solid density and compaction curve for the aggregate, retaining all material extracted from the compaction mould.

2 If the crushing resistance is less than 180kN, or the aggregate is a blend from two different sources, or the aggregate source is suspect or the design traffic is high, undertake M/4 testing on the samples recovered from compaction tests (or from a field trial). Otherwise carry out M/4 testing on the stockpile sample.

3 Calculate the estimated equilibium water content from equation 6.3

$$EWC = A - 0.2 + 0.134\ P0.075 + 0.045\ P0.150 + 0.02\ CI\ .\ P0.075 + 2.9\ AR - 3.19\ PET \qquad \text{(Equation 6.3)}$$

4 Calculate the refusal percentage solid density from equation 6.6.

$$\%SD_r = 137.8 - 49.5\ n - 23.3\ SGE - 0.013\ BF - 1.7\ P_{0.150} \qquad \text{(Equation 6.6)}$$

5 Calculate the estimated long-term equilibrium saturation using equation 6.1.

$$S_r = (EWC-A) * G_{bd} * \%SD_r / (100-\%SD_r) \quad \text{(Equation 6.1)}$$

6 Follow the decision tree in figure 6.1.

If there is any doubt, in situ modification (say 1.5% cement) would be a likely solution.

A functional spreadsheet which executes this procedure is available at Tonkin & Taylor Ltd (2010a).

The concept of degree of saturation at refusal could also be considered as an effective key performance indicator for performance-based design and construction of basecourse layers.

6.3 Other issues

The focus of this study is on the basecourse parameters, particularly for thin chip-seal pavements. There are of course many other issues that relate to basecourse life, including:

- waterproofing of surfacings
- lateral drainage
- vertical drainage
- stability of multiple seal layers
- flushing.

Patrick (2009) and Midgely (2009) address most of the relevant issues, including the lack of waterproofing in a first coat seal, documenting measures that may be adopted for limiting the ponding that can occur, and the desirability of multiple seal coats to ensure the integrity of the basecourse. (As part of further studies it would be useful to include the number of seal coats, and percentage crossfall in the regression.) The use of subsoil drains or deep water tables are well recognised measures to improve lateral drainage. Vertical drainage is addressed through standard subbase requirements for a material with permeability of at least 10^4 m/s (TNZ M/3 notes, Transit NZ 1986, referred to as 'M/3 notes'). Extensive detail is contained in a guide developed by the US Federal Highway Administration (FHWA 2003) and many of these concepts have been incorporated in a recent draft revision of the M/3 notes (appendix C).

Stability of multiple chip-seal layers (Gray and Hart 2003) has been investigated preliminarily using the falling weight deflectometer (FWD) to identify characteristics of the full stress/deflection time history. Very few examples have been identified so far and some of these are dominated by subgrade deformation. Further study is in progress by the authors of this report.

Flushing has recently been shown to be significantly affected by loss of voids as a result of degradation of chip or contamination from basecourses which have degraded or been inadequately broomed (Ball and Patrick 2005).

The change in clay index with trafficking has also been investigated but with little success. No literature on this subject was obtained in the search, or from contacts in the industry.

6.4 Quantitative prediction of basecourse life

It may be inferred from the laboratory RLT testing by Theyse (2002), discussed in section 3.3, that basecourse life should be able to be predicted quantitatively using an equation of the form:

$$Log(ESA) = a_0 - a_1 S_r - f(PS) \quad \text{(Equation 6.7)}$$

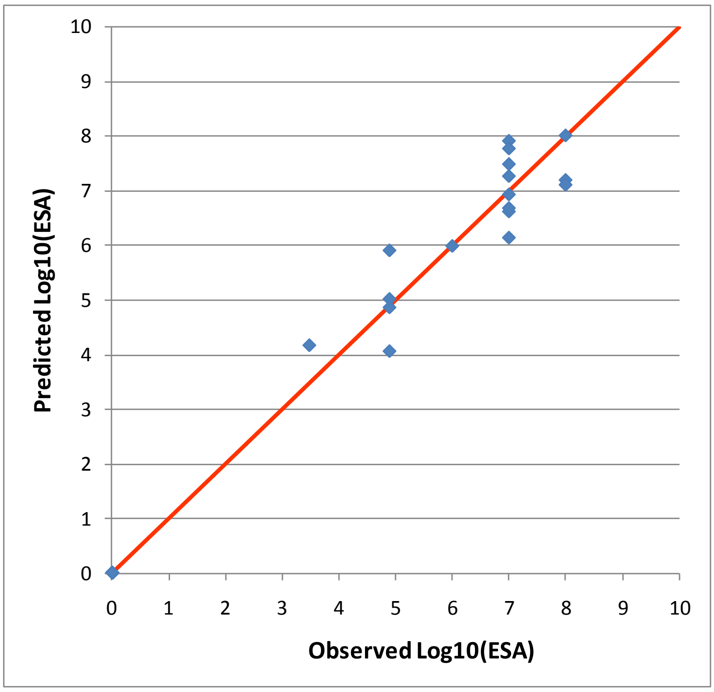
where:

- ESA is the number of equivalent standard axles to a terminal condition for shear instability
- S_r is the equilibrium degree of saturation
- f(PS) is a function of the plastic strain induced by traffic loading
- a_0, and a_1 are constants, constrained to be positive for a logical regression. (A starting point for a_1 would be approximately 0.07 as obtained with the RLT study.)

The plastic strain term should be able to be replaced by undertaking a regression analysis of the relevant parameters generated by standard M/4 and B/2 quality assurance testing as listed in the basecourse inventory. These should be used with other terms which relate plastic strain to in situ characteristics, in particular the FWD bowl shape and climatic parameters (mean annual rainfall and potential evapo-transpiration). The reliability of this process has been evaluated using the spreadsheet containing the inventory of premature basecourse failures to deduce the regression equation for the number of ESA to a terminal condition. The approach used has been to constrain the sign (whether logically positive or negative) for each term of the expression so that the regression process will still be robust, given the limited database.

The results are promising and extension of the inventory is warranted in order to reach the stage where a more robust prediction of lifetime ESA to a terminal condition can be demonstrated.

Figure 6.6 Regression relationship for predicting basecourse life (Tonkin & Taylor 2010a)



Meanwhile, the important issue to address is that the M/4 basecourse specification should include a requirement for some form of well documented, logical check that the aggregate is not susceptible to shear instability. Chapters 5 and 6 of this report set out a number of checks but, where practical, the best check is still likely to be precedent performance of the same aggregate in similar environmental conditions.

7 Case histories of premature distress

7.1 General

This section summarises the findings from analyses of unbound granular pavements that have experienced premature distress and considers the type of quality assurance that would limit any similar recurrence in future. In particular, the measures considered are those that might be included in the standard specifications, ie M/4, M/3 notes and B/2. Non-destructive testing has been carried out where possible to enable consideration of any additional testing that could take place during construction, to provide additional assurance or at least allow timely corrective action to extend pavement life. The sites are numbered rather than specifically referenced, to avoid any potential contractual issues. The detailed analyses are extensive and hence are available electronically as spreadsheets.

The earliest research on the concept of predicting pavement shear instability from non-destructive testing is in an FHWA LTPP TechBrief by Richter (1997), who postulated that the full-time history from an FWD deflection test could be used by plotting the plate stress versus deflection of the central deflector. The area within the stress-deflection hysteresis curve is a direct measure of the 'dissipated work' or energy lost during the simulated 1 ESA loading. At that stage the energy loss parameter was examined by the writers and has since been available in FWD interpretive output, as shown below.

Figure 7.1 An example set of full-time history curves from an FWD deflection test

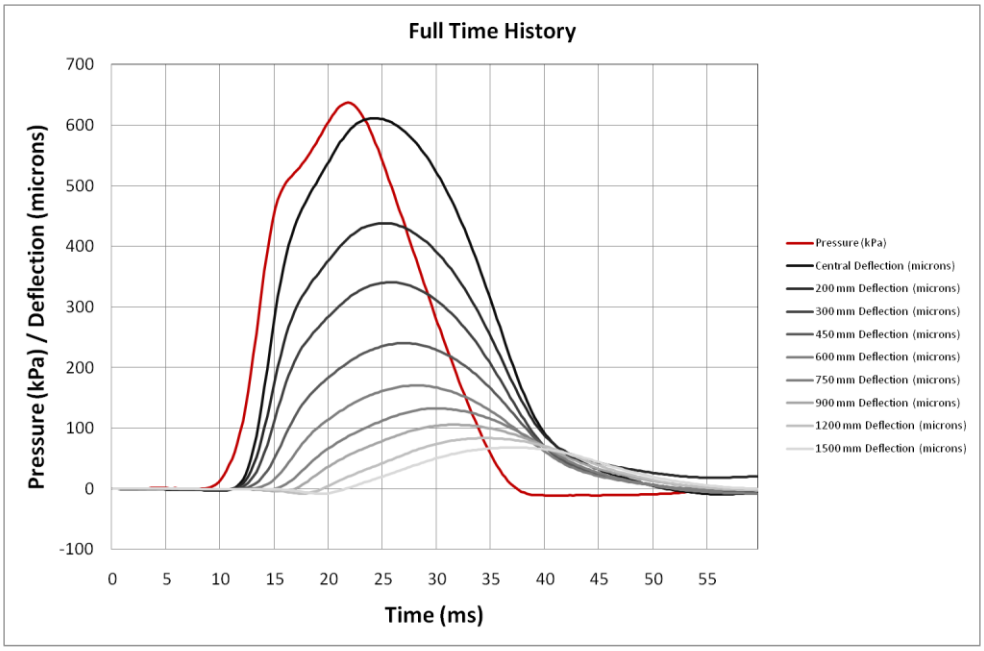
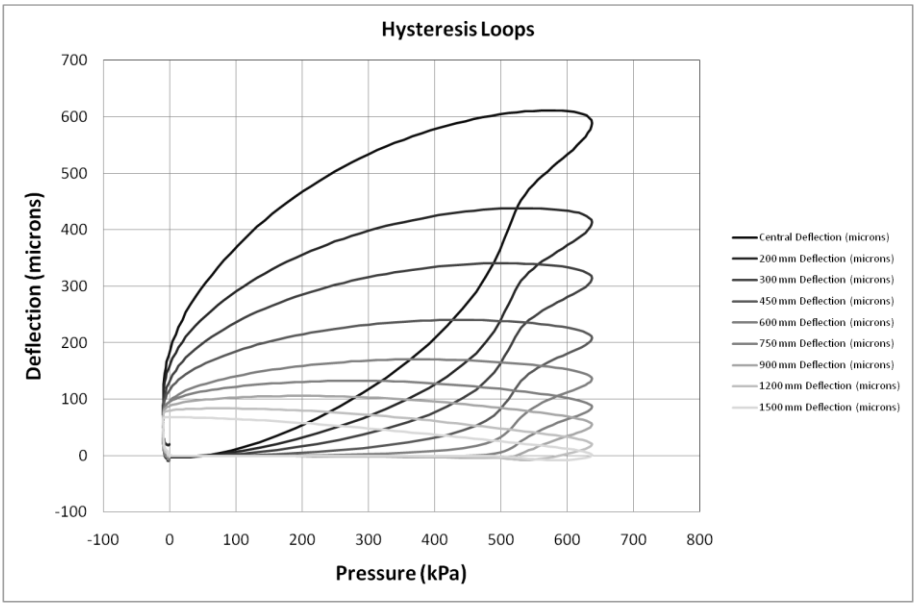


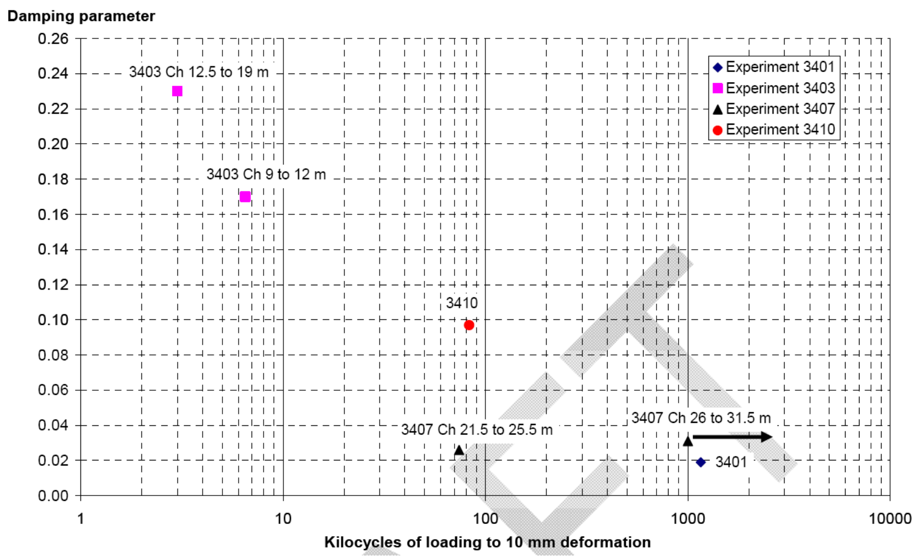
Figure 7.2 An example set of hysteresis loops calculated from figure 7.1



However, examples studied at that time showed no useful correlation with shear instability in basecourses that were more significant than the basecourse modulus determined using standard methods (Ullitdz 1978). Unusually low moduli tend to be predictors of excessive silty fines, but do not tend to show saturated granular layers because water has low compressibility in the short period dynamic test. Therefore the full-time history was used to derive a series of empirical parameters that related to the lag in response of the deflectors to the stress pulse (Tonkin & Taylor in preparation). This resulted in a provisional shear instability index that tended to indicate the probability of shear instability rather than a uniquely definitive measure. The index is therefore promising and further development has continued as relevant case histories have become available.

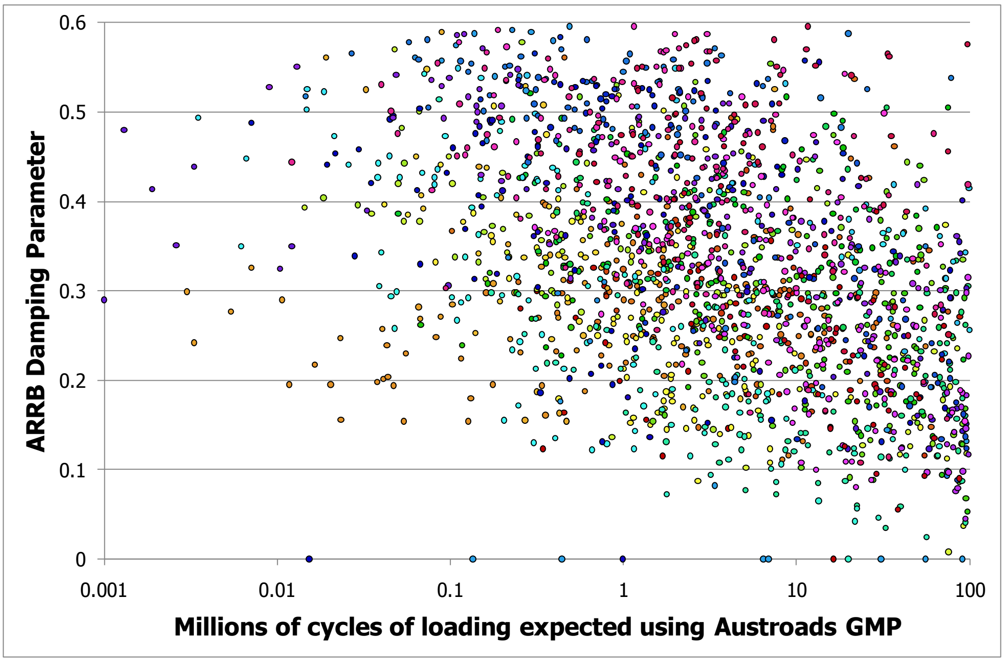
Jameson et al (2009) proposed a damping parameter that is a partial hysteresis loop, and found that pavement life under accelerated testing in a controlled environment showed a trend for decreasing life as the damping parameter increased.

Figure7.3 - ARRB ALF Trial comparison of damping and life (Jameson et al 2009)



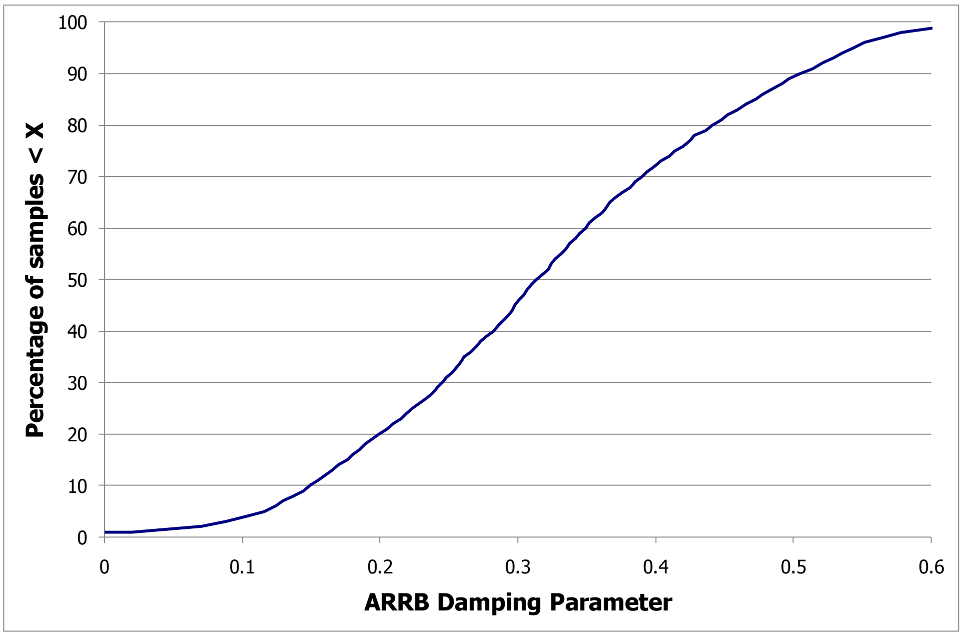
The ARRB damping parameter was investigated for all NZTA's LTPP sites, initially by comparing it with pavement life calculated using Austroads procedures, but no significant trend was identified.

Figure 7.4 ARRB damping parameter versus Austroads GMP life for all NZTA LTPP sites



The cumulative distribution (figure 7.5) also shows the median damping on the LTPP sites is about 0.3 and it is expected that most of these New Zealand highway pavements will perform for well over 100,000 ESA, or 100 kilocycles. That does not relate at all to the ARRB parameter where values of less than 0.05 are indicated as necessary to achieve 100 kilocycles.

Figure 7.5 ARRB damping parameter cumulative distribution over all NZTA's LTPP sites



The basecourses for the ALF trial were all formed on a uniform, very stiff foundation and that may be the reason the damping parameter could apparently discern differences for those basecourses, ie once subgrade stiffness is introduced as another variable, a correlation no longer appears viable. The damping parameter would need to be assessed from a family of curves for different subgrade stiffnesses, and also for varying thicknesses of subbase.

No trends could be identified in any of the New Zealand case histories examined when plotting the ARRB damping parameter versus severity of shear instability. Further investigation of this form of the damping parameter does not appear warranted for New Zealand conditions. However the principle of interpreting the full deflection-time history of the FWD test to predict basecourse instability is sound and is being investigated as more case histories become available.

7.2 Case histories of premature distress with unbound basecourses

Most of the following are recent examples of premature distress, with further details of most obtained from internal files (Tonkin & Taylor, Beca and Opus, with acknowledgements to others including John Hallett and William Gray). The focus is where subgrade deformation was not a significant factor, ie deformation within the pavement layers was involved, especially shear instability in the basecourse.

Where deflection bowls have been measured, the expected total pavement life is reported, assuming the tests reflect the condition of the pavement immediately after bedding-in is complete (ie after initial in-service trafficking). The model used is based on Austroads principles supplemented by performance data from CAPTIF and the state highway LTPP sites. The pavement total life is predicted in terms of ESA to a terminal condition by addressing four discrete structural distress modes (rutting, roughness, flexure/cracking and shoving), see Salt et al (2010).

For pavements that are not new it is important to distinguish between total life and remaining life. The remaining life takes into account the condition of the pavement at the time of FWD testing and is the additional number of ESA expected until the treatment length reaches a terminal condition for each structural distress mode if there is no addition or replacement of bitumen bound layers. (Additional non-structural sprayed seal layers may be applied if markedly reducing seal lives are not being experienced.) On the other hand the total life or ‘potential life’ ignores the current condition of the pavement and is the number of ESA expected from the time of FWD testing until the treatment length reaches a terminal condition for each distress mode. This assumes the pavement is either new or if not, any departure from a new condition is first rectified by smoothing of the surfacing, but without any form of structural strengthening. Total life is the principal parameter of interest when deciding whether or not structural strengthening is required for a pavement surface that is in a terminal condition. For this reason total life expectations for each distress mode are reported below, plotted versus chainage along the road and also as a cumulative distribution so the characteristic design value (usually the 10th percentile) can be readily identified.

For each case history possible corrective actions for tightened quality assurance are summarised.

Site 1

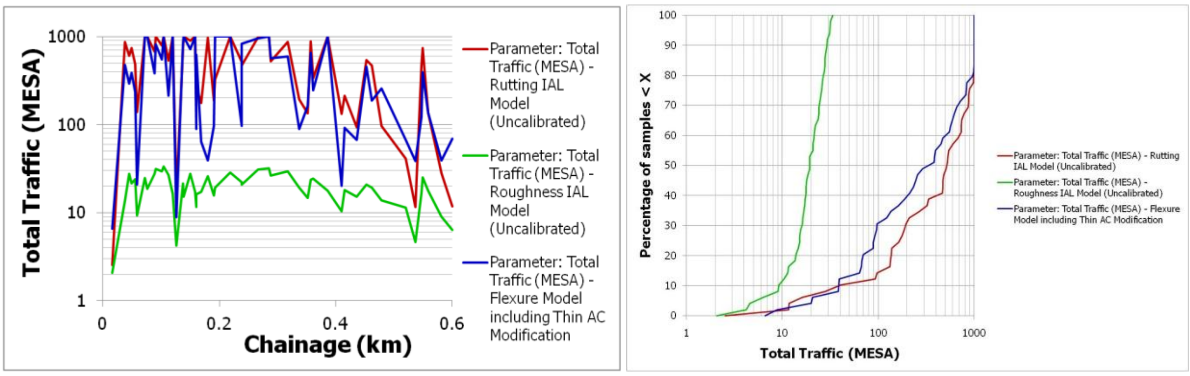
This pavement was constructed two years ago using a cement-bound subbase and unbound granular basecourse that complied fully with M/4 with a surfacing of 35mm of asphaltic concrete (AC). B/2 compliance documentation has not been inspected. Severe wheelpath cracking developed soon after construction but negligible rutting. Some destructive testing was carried out and the conclusion was reached that premature distress was because the basecourse was essentially an AP20 with very little coarser stone, but the in situ density and degree of saturation were not measured.

A report was prepared by the consultant for the contractor, concluding that although the basecourse was originally compliant with M/4, breakdown of the basecourse with release of plastic fines during trafficking was the primary cause of failure. The possibility also remains that the material delivered to site may have differed from the stockpile samples tested. The sand equivalent had reduced from a mean of 45 to 22. No plasticity index tests were reported in production tests but a value of 10 was reported after trafficking. The clay index of 9 after trafficking was three times higher than the M/4 limit and the weighted clay index was excessive (well over 50) drawing focus on the need not to rely on only one measure of fines quality for densely trafficked roads. However the construction quality assurance records clearly demonstrate the degree of saturation at the time of sealing was non-compliant with the B/2 specification: many values measured over 80%, yet sealing was carried out on the same day.

The in situ degree of saturation was not measured so supplementary post-trafficking testing was carried out. These in situ tests, two years after construction, demonstrated the degree of saturation averaged 93%, ie well in excess of acceptable values. Particle size distributions carried out on the samples showed a gap grading with SGEs of 0.37 during production and after trafficking values were as low as 0.26. Deflection testing was carried out showing a very stiff pavement. It was noted that the permeability of the subbase had not been addressed, and this may have contributed to the premature distress, but the over-riding consideration appeared to be basecourse saturation.

Although the contractor should have raised the saturation issue at the time of construction, it is probable the road would have failed even if it had been possible to dry back prior to sealing. The reason is that, after examination of the gradings and fines parameters, the SGE would have led to saturation and the high post-construction weighted clay index raises doubts as to the likely weighted clay index during production. A check for appropriate weighted clay index may have provided forewarning or at least the need for close quality assurance of the product as delivered for this heavily trafficked road. Analysis of deflection bowls indicated good predicted life for all other distress modes (roughness life was the lowest but that was generally more than 10 million ESA).

Figure 7.6 Predicted lifetime traffic from deflection test interpretation



Corrective action: Tighten the M/4 specification to avoid gradings susceptible to saturation, or include a specific check in M/4 or the *NZ supplement* for the expected degree of saturation equilibrium in situ. Enforce the B/2 saturation limit of 80%, and if dry-back appears unusually difficult (say if saturation cannot be readily brought back to 70% or less), undertake post-construction verification of the basecourse quality, prior to sealing. Testing for all four fines criteria at this stage from samples recovered from the shoulder would determine whether the product delivered was similar to the stockpile sample and whether degradation had developed during compaction. The importance of checking for subbase permeability needs to be emphasised, or perhaps included specifically as part of B/2 compliance documentation. Consider assessment of marginal aggregates in terms of weighted clay index.

Site 2

Site 2 is a recently constructed unbound granular pavement on a passing lane on a state highway subject to moderately heavy traffic.

Deflection testing was carried out as soon as the surface was sealed, and again at three- and six-monthly intervals to deliver a sequence of tests that would provide documentation of the characteristics of the bedding-in phase of a new full-depth unbound granular pavement.

The road experienced premature distress within a year in the form of severe rutting to depths exceeding 25mm. A slurry infill was used to restore shape and the pavement has since performed well, but eventually its life will be shortened once the pavement matures and delamination begins.

The improvements in layer moduli were distinct and provide guidelines to the improvements in stiffness that can be expected in unbound granular pavements. A very useful measure of the overall compaction (including both the subbase and basecourse layers) of an unbound granular pavement is the ratio of the layer moduli. Austroads (2008–2009, part 2, section 8.2.3) gives expected values, eg the modulus of each successive layer should double for every 125mm of pavement from the subgrade to the basecourse. That ratio has been demonstrated as very applicable to New Zealand's LTPP sites (Salt and Stevens 2007). At this site, the modular ratios at the time of sealing were much less than expected from the Austroads relationship. When retested at six months the modular ratios were almost exactly as expected by Austroads and the pavement was showing severe premature rutting. The lack of construction compaction was ultimately achieved by normal highway traffic, but with significant cost for rut filling subsequently. Examination of B/2 compaction documentation indicated no results for the lower subbase. For the upper subbase no correction for oversize material was used, ie unconservative estimates of percentage of maximum density would result.

Corrective action: Emphasise in B/2 that compaction documentation is necessary for each layer of the subbase as well as the top of the subbase and the basecourse. Include in B/2 notes, the applicability of the Austroads modular ratio guidelines as a reliable measure of the overall stiffness achieved in the combined subbase and basecourse and that a check for these should be made prior to sealing for any unbound granular pavement where documentation of B/2 compaction (especially for the deeper layers) is not provided or is marginally compliant. Ensure that subbase oversize particles (not represented in the laboratory target compaction) are rationally accounted for in quality assurance procedures for the in situ percentage of maximum density.

Site 3

This is a new unbound granular passing lane which exhibited significant roughness within a year of construction. FWD results showed the design CBR was 3.5, yet only relatively thin pavement had been constructed, giving subgrade strain ratios up to twice that allowable using the Austroads subgrade strain criterion. The analysis indicated that more than a third of the lane would fail prematurely through both rutting and roughness modes predominating. Acceptably high calculated modular ratios for the granular layers, (relative to the Austroads expectation) confirmed this was a design issue rather than a construction fault.

Corrective action: CBR design for passing lanes should be reliable if FWD testing is undertaken on the adjacent lane, and the design pavement thickness should be determined from the 10th percentile in situ CBR or the existing pavement thickness whichever is the greater. Deflection testing prior to sealing would also have identified this issue.

Site 4

This section of motorway was new unbound granular pavement with 30mm of open graded porous asphalt (OGPA) surfacing on an M/4 compliant basecourse that experienced shear instability after six years. After trafficking the basecourse was found to be gap graded with a SGE commonly about 0.33, significantly less than the proposed 0.4 limit. The weighted clay index was excessive (commonly over 20) as was the weighted plasticity index (over 50).

Corrective action: Include in M/4 a specific check for shear instability, such as a SGE greater than 0.4, and consider post-compaction testing of all fines criteria for heavily trafficked roads. In particular, use weighted clay index and weighted plasticity index as an aid to judgment.

Site 5

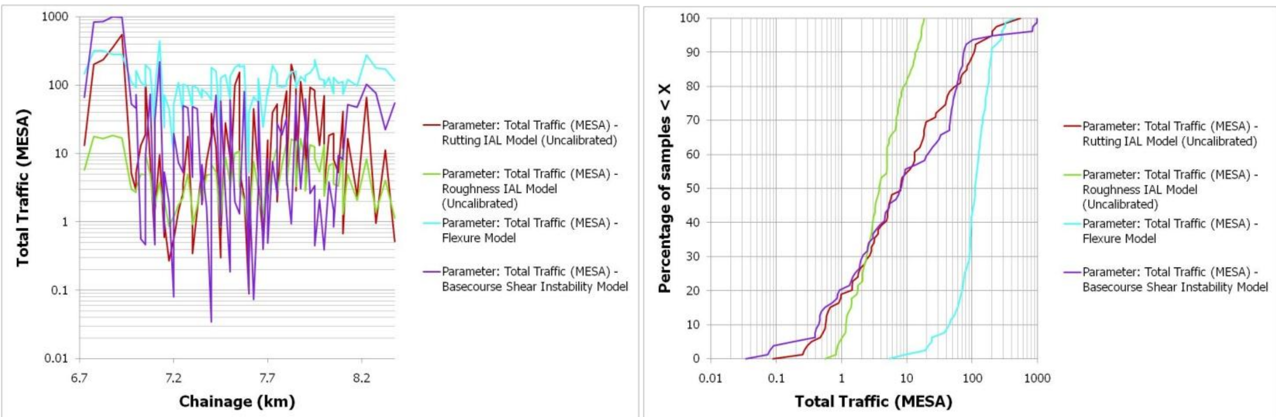
This pavement was rehabilitated six years ago with an M/4 compliant 125mm granular overlay. Within a few years there was marked rutting then localised shoving. In situ density was measured using both replacement methods and nuclear density meter (NDM) on direct transmission mode showing a degree of saturation between 75% and 100% in the basecourse. The original M/4 compliance tests showed the stockpile fines were non-plastic but the clay index was 4.9. The clay index after trafficking was reported as 3.8, ie no apparent degradation, but significant sample variation was indicated. The weighted clay index appeared significant as relatively high values (27–42) were present in all samples.

Corrective action: For moderate to heavily trafficked roads consider adding a check for the weighted clay index as well as plasticity index where sand equivalent is less than 40.

Site 6

This site is a new unbound granular pavement with chip seal, constructed two years ago as a passing lane on SH1. The pavement failed (rut depths over 80mm) with pronounced rutting and shoving. Deflection testing indicated many points with a high index for shear instability. Logging of test trenches across distressed parts of the pavement showed intrusion of subgrade silts up through more than 400mm of aggregates. Testing showed well in excess of 10% silt and saturation conditions in the basecourse. B/2 documentation showed compliant subbase and basecourse. However the design had not followed the requirements of the M/3 notes for compatibility at the subbase/subgrade contact. The subgrade was a low angle fan of micaceous clay silt of low plasticity. This terrain may be subject to springs, but in any case compaction water would not have been easily drained away.

Figure 7.7 Predicted lifetime traffic from deflection test interpretation



Corrective action: M/3 notes state 'Instead of ensuring compatibility an alternative acceptable strategy is to assume that some subgrade intrusion is going to take place and to check that the assessed reduction in CBR for the "intrusion zone" at the bottom of that subbase layer does not invalidate the pavement design'.

One school of thought interprets M/3 notes as implying any intrusion zone will be only 75mm thick. However M/3 notes only mention 75mm as the minimum thickness of a compatibility layer, not the maximum depth of intrusion. Subgrade intrusion (in non-plastic silt at least) has the potential to push up through many hundreds of millimetres of open-graded subbase and then penetrate the basecourse. Compatibility needs to be reinforced in M/3 notes to ensure compliance with the compatibility criteria in all designs where a fine-grained subgrade has the potential to reach a high water content either during addition of compaction water, or long term.

A simple spreadsheet that will perform the compatibility check between the lower subbase layer and the subgrade would be a useful supplement for M/3 notes.

Site 7

This is a motorway with modified basecourse and about 50mm–60mm of AC surfacing. It exhibited severe cracking within five years of construction. The AC was rehabilitated but failed again within two years. The SGE was less than 0.4 for six out of seven samples of basecourse (and subbase) and the weighted clay index was in the range of 20–40, ie excessive for motorway use, as was the weighted plasticity index (well over 50). The pavement was inferred to have failed from excessive tensile strains in the AC layer as a result of the unstable basecourse. Samples were recovered at the outset of the extending pavement life project for further study of alternative gradings in the RLT tests. However the issue of the applicability of the results then arose, given the arbitrary water content and degree of saturation used for each different gradation. This led to an appreciation of the need for a design procedure to assess applicable preparation of RLT samples as discussed in chapter 4.

Corrective action: Ensure a check for shear stability is included in M/4, both for chip-sealed and AC surfacings less than say 80mm, at least. Adopt weightings for the fines criteria for compacted samples and use them to decide if stabilisation is warranted prior to sealing. When contemplating RLT testing, if the relative density and degree of saturation are not accurately known and clearly related to the in-service state, the test may have little value to the practitioner.

Site 8

This was a rehabilitation site comprising an unbound granular basecourse (400mm thick) and 40mm of asphaltic surfacing. The pavement showed premature cracking and rutting within a year (traffic of 0.7 MESA). FWD deflections indicated subgrade strain was not an issue. No B/2 compaction results were provided for the subbase which had a sand equivalent of 21. There was only one set of basecourse compaction tests which indicated the compacted density was satisfactory but the majority of points were between 70% and 95% saturated at the time of sealing. No particle size information has been located at this stage. Subsequent test pits in the distressed pavement confirmed saturated conditions in the basecourse.

Corrective action: Ensure B/2 compaction documentation is supplied for all granular layers. Ensure subbase permeability criteria are met. Investigate fines criteria in the compacted pavement for any basecourse which cannot be readily dried back to less than 70% saturation.

Site 9

One year after application of an M/4 compliant unbound granular overlay (150mm after wheelpath ripping), extensive rutting, cracking and shoving developed with associated potholing. Elapsed traffic was only 0.5 MESA. FWD results indicated strains were not excessive in the volcanic subgrade. The source quarry was known to produce aggregate with marginal performance. Observation of trenches indicated deformation was confined to the new basecourse layer which appeared wet. Ripping of the old seal appeared ineffective. The investigations did not report the degree of saturation in the trafficked state. The stockpile SGE was 0.40 and this decreased with trafficking to 0.37 as the fines content increased from 6% to 12% passing 0.075mm.

Corrective action: Ensure old seal is appropriately ripped and if a former basecourse is required to act as subbase, then it should meet permeability criteria of M/3 notes, or compensatory measures adopted to increase water resistance of the surfacing. Include a check for basecourse shear stability (SGE>0.40) taking this from a sample that has been compacted where the source rock does not have a good historic performance record. Measure degree of saturation at compaction and estimate the expected saturation at refusal. Apply additional seal coats in the same construction season for a basecourse where a high degree of saturation is predicted, or if subbase permeability is marginal.

Site 10

This site involved a 150mm granular overlay after ripping the seal in the wheeltracks. The source quarry had a record of providing aggregates of marginal performance. Continual maintenance was then required for wheel track cracking, rutting and potholing after trafficking of much less than 0.5 MESA. FWD results indicated strains were marginal but not excessive in the volcanic subgrade, given the low traffic level applied before distress began. Compaction was to 84% of solid density. Saturation prior to sealing was in the satisfactory range of 40 to 60%. Trenching showed the rutting was confined to the basecourse and the old seal ripping was ineffective. Recovered samples showed substantial degradation. The stockpile SGE was 0.35 and this reduced further as the trafficked samples gave values of 0.14 to 0.4 with an average of 0.31.

Corrective action: Uniformity of the basecourse grading should be tracked closely with each delivery to site. This can be done initially using sand equivalent, followed when changes are indicated with full particle size distribution check. Shear stability should be checked for SGE > 0.40, using post compaction gradings for any quarry with a marginal performance record. Check subbase permeability criteria (M/3 notes) are met especially where old seal layers are present, otherwise specify conservative fines criteria for the overlay aggregate and apply multiple seal coats in the same sealing season. If the aggregate is marginal, consider cement modification.

Site 11

Rehabilitation of this site involved 150mm of granular overlay and two coats of chip seal. Ongoing maintenance was required for rutting, cracking and potholing after minimal trafficking. FWD results indicated strains were not excessive in the volcanic subgrade. The source quarry did not have a good aggregate performance record. Compaction was compliant to an average of 86% of solid density. Trenches after trafficking revealed moist to wet basecourse (pumping locally) and showed that ripping of the seal prior to overlay was evidently ineffectual. Deformation was predominantly confined to the basecourse. The pavement had become over-wet during construction and was re-worked to meet specification. The SGE from the stockpile was poor at 0.35, and degraded to a range of 0.35 to 0.39 after trafficking with fines content doubling from 6% to 12%–13%.

Corrective action: Ensure a check for shear instability in M/4, using post-compaction gradings for aggregates from sources with marginal performance. If subbase permeability is marginal specify conservative fines criteria for the overlay aggregate and use multiple seal coats in the same construction season for marginal conditions.

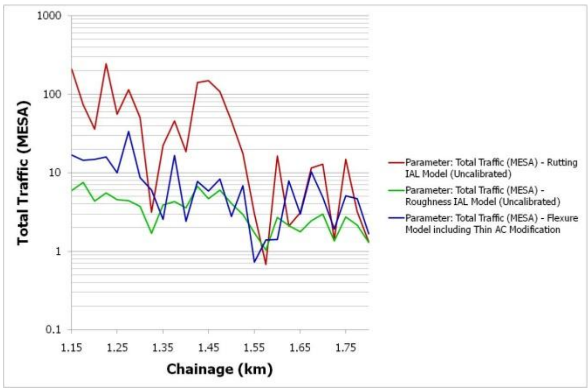
Site 12

Prior to rehabilitation this pavement comprised chip seal over 25mm asphaltic concrete over 90mm–200mm basecourse, 240mm–300mm subbase, on stiff clay silt subgrade with characteristic CBR of 8. The 90 percentile deflection was 1mm. The intention was to remove existing seal then overlay with 50mm mix 14 and install subsoil drains as the subbase and basecourse were noted as very moist. The rehabilitated pavement then failed within six months by widespread crocodile cracking, pumping and slight to moderate deformation. Investigations focused on the waterproofing of the AC and seal. The AC was not milled to full depth as intended and a tack coat was used rather than membrane seal. The degree of saturation in situ was not measured. The basecourse had a low SGE (0.36 to 0.39). This together with the reported pumping implies saturation of greater than 80%, ie a combination of potentially unstable basecourse and inadequate waterproofing. Calculated strains in the AC were excessive, but no shift factor was applied.

FWD testing was carried out prior to rehabilitation and figure 7.8 shows the expected life if the pavement was to be resurfaced without strengthening.

This indicates the design was slightly unconservative but most of the road should have survived for at least half of the design life of 2 MESA, hence reinforcing the view that rather than excessive tensile strain in the surfacing, waterproofing (lack of membrane seal) and shear instability were more likely factors in the rapid failure.

Figure 7.8 Predicted lifetime traffic from deflection test interpretation



Corrective action: Ensure a check for shear instability in M/4 and ensure effective water proofing especially for any marginal aggregates. Check any AC design for fatigue cracking, using the correct layer thickness.

Site 13

This highway was constructed with SGEs of 0.29 in the basecourse and 0.34 in the subbase. Widespread distress was observed shortly after opening.

Corrective action: Ensure fines criteria are satisfied in both basecourse and subbase.

Site 14

An unbound basecourse with stockpile SGE of 0.26 failed immediately after construction. Note that 0.26 is the lower limit currently allowed by M/4 grading shape control limits.

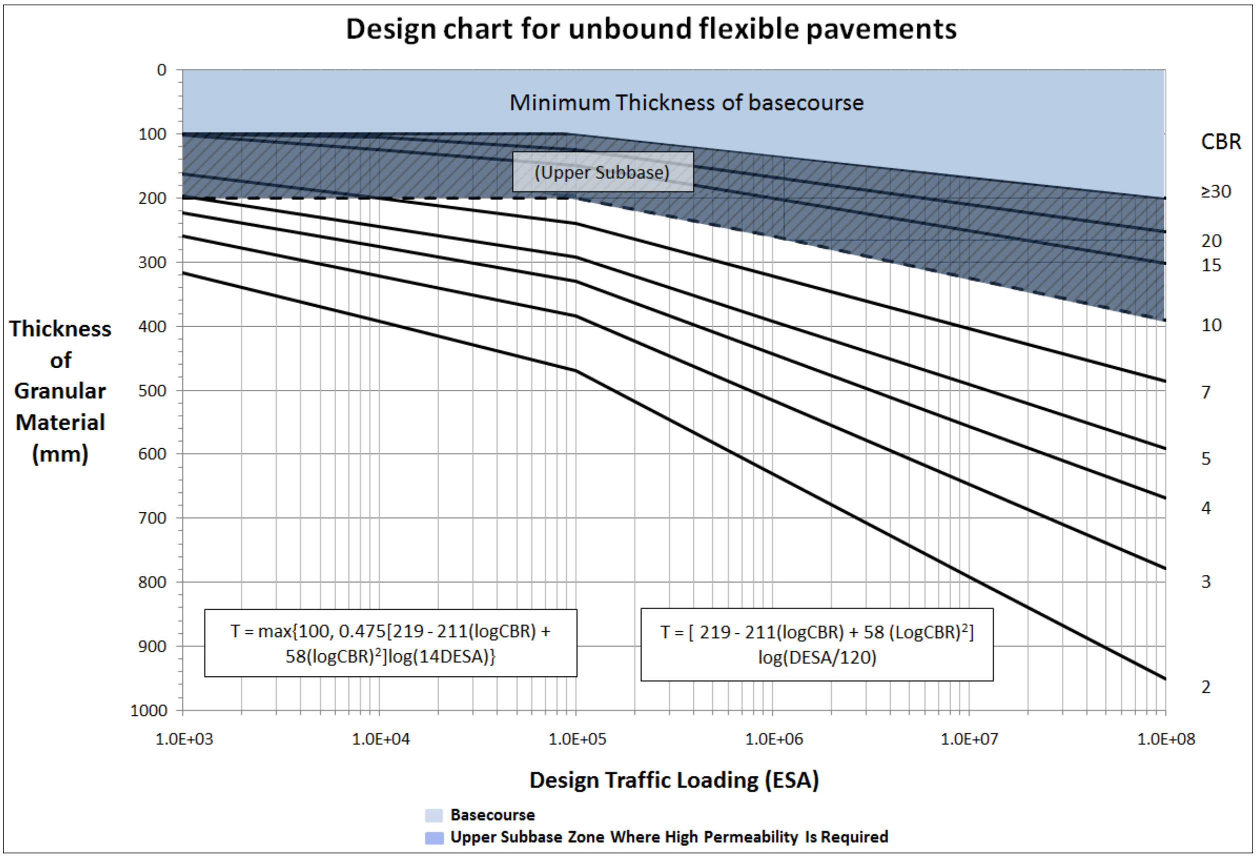
Corrective action: Ensure minimum SGE of 0.4 or other check on potential for basecourse saturation.

Site 15

A basecourse variant approved by a local authority but was trialled as a subbase, beneath high-permeability good quality M/4 basecourse. The stockpile SGE of the subbase was 0.22 and the pavement failed with saturation conditions in the subbase within 10,000 ESA, with the source of the shoving established by trenching.

Corrective action: There is less emphasis in Austroads regarding the importance of a free-draining subbase, but it was well highlighted originally by NRB S/4 and is also clear in the VicRoads design chart. A compromise for the design depth of the permeable upper subbase (between these two charts) is shown in figure 7.9 and it would reinforce the importance of drainage if this is reproduced in the *NZ supplement*.

Figure 7.9 Combined design chart showing upper subbase zone where high permeability is required



Ensure a check for saturation and shoving in upper subbase as well as basecourse. Shoving can clearly develop at greater depths than the normal thickness of basecourse (100mm–150mm). Because B/2 compaction quality control requirements include density and water content of the subbase there would be no additional cost to include as a standard requirement, the saturation calculation for the subbase as well as the basecourse. Because subbase is intended to be free draining, any test showing saturation over 70% should immediately flag the permeability of the supplied/compacted product is likely to be unsatisfactory.

Site 16

An unbound granular overlay with one seal coat exhibited localised but severe distress (wheelpath cracking and potholing) within the first season after 10,000 ESA. The SGE from test pits ranged widely from 0.32 to 0.42 with mean of 0.37. Basecourse saturation was inferred with contribution from porous seal to account for the rapid failure.

Corrective action: Carry out post-compaction checks on fines criteria and multiple seal coats for marginal aggregates.

Site 17

A site with 250mm of unbound aggregate on a resilient volcanic subgrade was rehabilitated by adding 100mm of make-up metal then adding 2% cement to form a 250mm thick modified layer. It failed by rutting in the first year after about 150,000 ESA. The basecourse was M/4 compliant. Compaction records were lacking. Investigations into the distress found the supposed 350mm thick pavement was only 250mm (the same as the original pavement). Because quarry records showed the correct volume of aggregate had been delivered to site it was inferred that the bulk of the makeup metal had been diverted into building up the shoulder. The point of interest was that the same subgrade with 250mm of old aggregate exhibited performance, which was superior to subgrade with the same thickness of newly modified aggregate. This case history also raises the question of what should be the modular ratios between successive granular layers on volcanic subgrades where strains much higher than normal can usually be accommodated. This issue is not covered by the Austroads (2008–2009) or the *NZ supplement*. In this case, analysis of deflection testing showed the horizontal tensile strains in the base of the modified layer averaged 1200 microstrain, ie at least twice that normally encountered in modified basecourses. The associated pulling apart evidently destroyed any cohesion in the mix which was found to be almost free running in investigation trenches.

Corrective action: For highly resilient volcanic soils, check that the tensile strains in the base of any cement modified layer are in the normal range (ie less than 500 microstrain, unless higher values are justified by precedents in similar conditions). Ensure compliance with B/2 density requirements. Where subgrade strains higher than allowed by Austroads are contemplated, compaction to B/2 may not be practical in a cement modified single layer of 250mm compacted thickness. Note the single lift practice (usually 200mm–250mm compacted depth) with cement modified layers is contrary to B/2 which requires a maximum of 200mm loose depth (equivalent to about 150mm compacted depth) for any basecourse layer, ie the consistency of specifications may need to be addressed.

Site 18

A new pavement fully compliant with M/4 and B/2 with only 40% saturation showed deformation as soon as it was opened to traffic. Inspection revealed excessive fines in the thin running course (non compliant with B/2) with chip pushing down into the fines.

Corrective action: Ensure testing of running course for compliance with both B/2 grading and M/4 fines criteria, in accordance with the standard requirements of B/2, ie 'The quality of fines shall be not less than that required for the basecourse below the running course when tested in accordance with the requirements for the base course'.

Sites 19 and 20

A new unbound granular pavement with thin AC surfacing was constructed in two adjacent sections by two separate contractors. The heavily trafficked arterial exhibited alligator cracking and pumping with minimal rutting within two years on one section and within four years on the other, after trafficking of 1–3 MESA. Many volumes of quality assurance records were supplied in hard copy requiring considerable time to peruse the data, but it became evident that neither contractor had provided the B/2 test results for the degree of saturation of their basecourses prior to sealing. The stockpile grading and fines criteria indicated susceptibility to shear instability.

Figure 7.10a Particle size distribution ex stockpile

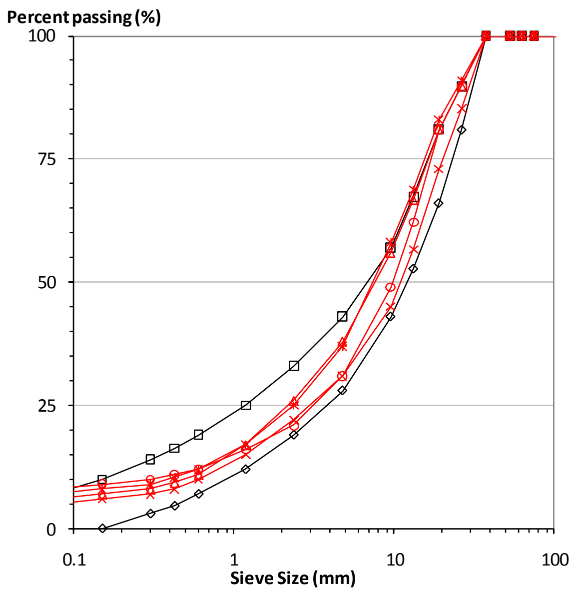
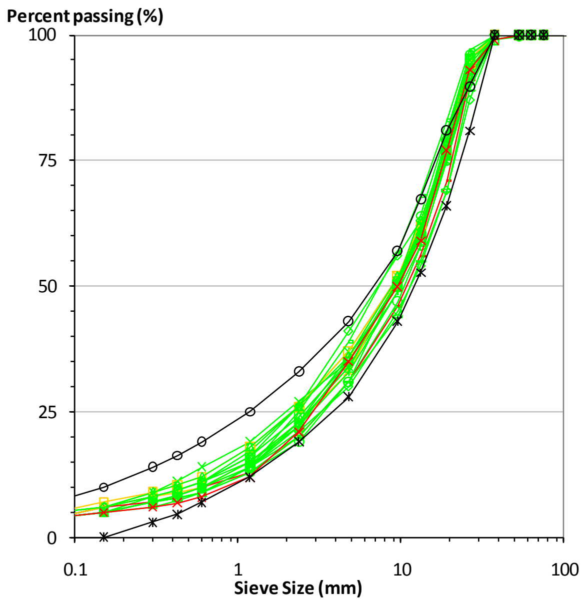


Figure 7.10b Particle size distribution after trafficking



The trafficked samples tended to be gap graded, with low SGEs and two being very low (0.33 and 0.24). These two had sand equivalents of 25 and 27, clay indices of 4.7 and 6.3, and plasticity indices of 13 and 10. Other major factors were found to be a lack of bond between the surfacing and the basecourse and excessive curvature in the thin AC in relation to the very high loading (about 0.5 million ESA per year).

Corrective action: Use of thin AC on unbound granular pavements for this level of loading is now discouraged in the *NZ supplement*. However neither Austroads (2008–2009) nor the supplement provide definitive guidelines for the design of thin AC surfacing (less than 40mm). This is a likely reason for the too frequent incidence of premature distress in new pavements of this type. The risk could be reduced if appropriate checks are prescribed in the *NZ supplement*, for example:

- Specify a premium basecourse then sample from the shoulder of the compacted basecourse (preferably in a pilot trial but at least prior to surfacing) to confirm the basecourse is capable of carrying the design traffic (chapter 6).
- Carry out deflection testing to record curvature (not just central deflection) at the top of the compacted basecourse prior to surfacing.
- Correct the measured curvature for post-construction densification under traffic using Austroads modular ratios for unbound granular layers.
- Determine the suitability of the structure for accommodating the design traffic with the design thickness of AC either mechanistically, or empirically, using as a minimum the 40mm thickness in figure E-A2.2 of part 5 of Austroads (2008–2009), (see figure 7.11) but ensure likely construction tolerances are included. Typically, as-built thin AC thickness will vary by +/- 5mm from that specified.
- The same checks should be used for the full length of the constructed road, immediately prior to application of the thin asphaltic concrete. Intervention (eg with a modified basecourse) could then be invoked if the checks fail.
- Ensure basecourse saturation complies with B/2 and an effective bond is achieved between the surfacing and the swept basecourse, as well as provision of an effective waterproofing membrane (verified with an acceptance test).

Figure 7.11 Asphalt overlay fatigue lives (Austroads 2008–2009)

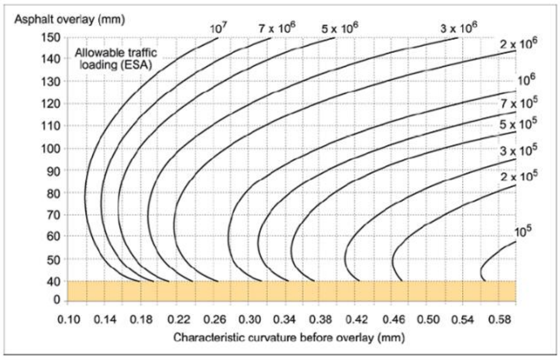


Figure E-A2.2: Asphalt overlay fatigue lives WMAPTs 20–25°C

For thin AC it should be noted that Austroads (2008–2009) requires that basecourse must be 'primed prior to placing the asphalt' and that 'it is critically important to the achievement of an adequate bond between the substrate and the surfacing'.

Other quality assurance related to this site would be to ensure the provision of B/2 test results is enforced. To address the current omissions as well as uncertainty in the industry, a standard functional spreadsheet could be promoted nationally, for use with B/2 acceptance testing. The correct input fields would be highlighted, the equations would be transparent to both users and reviewers and the entire compaction history for any project could be filed electronically in a compact form to allow subsequent understanding of any distress. The necessary consistency in this important stage of pavement construction would be promoted. Pass/Fail fields would display automatically to allow rapid recognition as well as assisting any less experienced supervising staff. An associated comment field would be used to record responses to any non-complying result and facilitate subsequent tracking. If there is a difficulty meeting saturation requirements prior to sealing, contractors should be made aware of the importance of making non-compliant results available as a priority because it is likely to be an indicator that immediate corrective measures (ie basecourse modification stabilisation prior to sealing) are necessary. A similar industry-standard electronic spreadsheet for all M/4 results for each project would also be of value. Ensure checks take place of the compacted basecourse for shear instability.

Site 21

This heavily trafficked site is an unbound granular pavement surfaced with 30mm asphaltic surfacing. FWD deflection testing showed very low deflection of 0.3mm but moderate curvature. The new route was opened in late 2008. Actual ESA is unknown but in the order of 0.5 MESA/year. Failure in the form of alligator cracking with neglible rutting affected 5% of the pavement within 18 months. In situ saturation measurements ranged from 82 to 100%. Sand grading exponent after trafficking ranged from 0.33 to 0.41. Sand equivalent 30, plasticity index 12 and clay index 6.The subbase had been stabilised with 2% cement, and permeability may not have been high.

Corrective action: If subbase permeability is low, use compensatory measures including SGE over 0.4, and effective sealing of the surfacing.

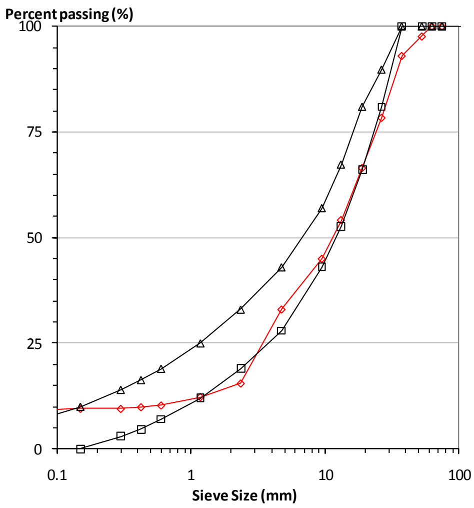
Site 22

This site is the earliest record (1962) located for a New Zealand basecourse saturation failure so although all other case histories are from the last decade, this case highlights the unresolved solution to the most common cause of premature failures over a span of 50 years. The Redoubt Rd section of the Auckland Motorway was meant to be constructed with SGE of 0.39 (stockpile) but when sampled after about 3 million ESA was found to be dramatically less at 0.09. (The relative contributions of the as-delivered product versus degradation are unknown, but a contribution from both factors is likely.)

Progressive densification occurred along with generation of additional fines until cracking was initiated by basecourse instability at an average degree of saturation of 80% (Buckland 1967). Thin asphaltic concrete surfacing was applied after incipient cracking was first evident in the seal, but widespread cracking soon followed.

Corrective action: The particle size distribution shown for this notable case history is an extreme example of a low SGE. Ensure post-compaction evaluation of M/4 fines criteria. Applying relatively modest (eg 40 mm) thicknesses of impermeable asphaltic concrete overlay early in the life of the basecourse will increase the basecourse life markedly, but if applied once incipient shear instability becomes evident, the extension in life will be minimal.

Figure 7.12 Gap grading in Auckland Motorway basecourse (1962)



Site 23

An unbound granular overlay failed by severe flushing and shoving within 18 months of construction. The old seal layer was not scarified and deflections were very low. Trenches showed no deformation of the old seal layer, but a 50mm crust of fines capped the basecourse. Embedment of chips in the soft base had resulted in excessive binder and flushing. The fines recovered had average values of PI 12, SE 27 and CI 3.8.

Corrective action:

If overworking is suspected, check for degradation (whether the reworked material still complies with M/4) and if not, cement modify prior to sealing.

7.2.1 Summary of findings from the case histories.

The most common form of premature distress in the case histories studied was shear instability of basecourses due to densification under trafficking resulting in high degrees of saturation. Once this state is reached, cracking of the surfacing will inevitably allow the ingress of surface water accelerating the failure. If the surfacing is a thin bound asphaltic layer that has cracked, it can be difficult to deduce whether the primary failure mechanism is excessive tensile strain in the surfacing, or if the cracking is a response to a primary failure mechanism of excess pore pressure in the basecourse. Other issues in approximate order of decreasing frequency were:

- failure to dry back basecourses prior to sealing
- porous surfacings (including insufficient seal coats in the first season)
- basecourse shoving due to impermeable subbases (including ineffective ripping of old seal coats)
- cracking of thin asphaltic concrete surfacings due to excessive curvature and/or lack of bond to the basecourse
- sub-standard compaction of subbase layers
- failure to correct the target field density for AP65 subbase when oversize was removed for determination of laboratory maximum density
- subgrade intrusion into incompatible subbase
- excessive subgrade strain resulting in rutting (pavement design too thin OR pavement construction thickness less than designed)
- subbase shoving due to shear instability in the subbase layer
- excessive fines capping the basecourse, as a result of over-working or non-compliant running course.

Where shear instability was absent rutting densification of basecourse compacted in accordance with B/2 was not a factor in any of these case histories.

8 Conclusions

- Premature distress in the form of shear instability (wheelpath cracking or shoving) has occurred in some unbound granular pavements with chip-seal or thin asphaltic surfacing even though documentation for those aggregates complied fully with the current specification (M/4) at the time of construction. Therefore an inventory of the basecourse parameters for a number of case histories has been collated to obtain a better understanding of the factors related to shear instability.
- A collection of field information including both destructive and non-destructive testing has been assembled for a number of case studies of pavements focusing on those with premature distress.

8.1 Basecourse saturation

- Saturation of basecourse is a common factor in many cases. The problem occurs not only at the time of sealing but also in well drained situations at later stages of pavement life when the water content has reached a steady state long-term equilibrium value that relates to the pavement environment.
- After dry-back prior to sealing, basecourse water content will normally increase with depth and NDM back-scatter will only record the lesser water content of the top half of the basecourse layer. There could be benefit in reducing the B/2 requirement for basecourse degree of saturation prior to sealing from 80% to 70%, at least for construction during the warmer months. However if this leads to a tendency to compact dry of optimum, with associated more intense compaction needed (with greater degradation of the aggregate), then a change in the saturation limit would be counterproductive.
- If there is a difficulty achieving less than 70% saturation when weather conditions are favourable at the time of sealing, then that should be regarded as a trigger to check the fines criteria for the basecourse post-compaction and also verify that the subbase provides effective drainage.

8.2 Basecourse gap grading and deficit in the sand fraction

- A common factor in poorly performing aggregates (both subbase and basecourse) is that they tend to be silty gravels, ie with gap grading in the sand fraction. It is commonly suggested a constant grading exponent provides the densest packing of particles. However this is contrary to Fuller and Thompson (1907) who found some degree of gap grading in the sand sizes produced maximum density. This provides an explanation for the high levels of saturation found in many gap-graded aggregates.
- There is evidently a compromise needed between a dense aggregate to minimise rutting through densification, and more importantly, a sufficiently open-graded aggregate that will not experience the more dramatic distress mode (shear instability) which will occur as soon as the critical degree of saturation (75%–80%) is reached.
- Gap-graded basecourses that are saturated are not always visually recognisable as free water is not necessarily seen in excavations. A basecourse with a typical value of 4.5% water content requires only a 0.5% increase to exceed the saturation threshold that instigates shoving.
- Gap grading can be quantified by calculating the incremental grading exponent (n) between successive sieve sizes in the range of 0.15mm–4.75mm. (M/4 overall grading limits lie generally between n values of about 0.4 and 0.6.) If the exponent is less than 0.4 in the sand range, performance is frequently poor. The term ‘sand grading exponent’ (SGE) is used to describe the characteristic exponent in the sand range. Grading shape control requirements in M/4 allow a SGE as low as 0.26. A tighter form of grading shape control is

therefore necessary to ensure aggregates do not deviate markedly from the targeted exponent of about 0.5. A basecourse with SGE of more than 0.4 will also be much more resistant to deformation in the event that water resistance of the surfacing is marginal, or if plastic fines develop as a result of either degradation or contamination.

- Many of the gap-graded aggregates are from hard rock quarries where extraction (either ripping or blasting) tends to generate a mix of angular gravel sizes and silt. The consequence of a sand deficit is the tendency to either have the gravel particles floating in a matrix of moisture-holding silt, or if there is no silt to have only sparse point–to-point contacts between gravel particles that will inevitably encourage degradation. Ensuring sufficient sand to increase the frequency of point-to-point load-bearing contacts in the structural skeleton is therefore fundamental to shear stability. A gap-graded aggregate is inherently a double detraction from basecourse performance: the sand that is favourable is not present and the silt that is unfavourable is present.
- Over time, most unbound basecourses under thin chip seals will experience a reduction in grading exponent (n value) with trafficking, but if this is only gradual, the additional waterproofing obtained once multiple seal layers are in place will tend to maintain the long-term degree of saturation at acceptable levels. Performance of marginal basecourses can be improved by providing multiple seal layers in the first season.
- If aggregate degradation does not occur and if the basecourse is kept below about 70% saturation, gap-graded aggregates can, and do, perform adequately. However, in New Zealand, it appears one or both of these conditions are often not realised in practice. A relatively small shift in grading can dramatically affect pavement life (in terms of ESA to a terminal condition) by an order of magnitude or more. Applying relatively modest (eg 40mm) thicknesses of impermeable asphaltic concrete overlay, early in the life of the basecourse will increase its life markedly. However, if applied once incipient shear instability becomes evident, the extension in life will be minimal.
- Aggregate suppliers who have been contacted indicate that the tighter grading shape control as proposed should not be onerous. The grading shape control criteria for New Zealand basecourses have been unchanged for over 40 years. Traffic is now generally greater in both volume and loading with premature distress due to basecourse shear instability becoming all too frequent. It is reasonable to expect costs will increase for at least some producers, but the overall cost benefit is large. A trade off may be to increase the upper limit of the grading envelope for the fraction coarser than 4.75mm, subject to appropriate trials, and CBR in particular would need to be verified.
- A recent proposal to raise the upper limit of the current M/4 grading generally (to a grading exponent of 0.35 over all sieve sizes) is contrary to the findings of this study. Further case histories would be needed to understand the conditions under which there would be assurance that such fine gradings would perform well in New Zealand conditions.
- Adding clean quarry fines or sand is recommended for gap-graded aggregates with deficits in these fractions. However, if there is more than one source, the crushing resistance (determined only on a discrete size range) will no longer be meaningful, hence the particle size distribution and M/4 fines criteria should be determined from samples which have previously been subject to compaction.
- Recycled glass, crushed to fractions within the 4.75mm–0.15mm range made may well provide an economic and environmentally sound solution to gap-graded aggregates.

8.3 Quantifying pavement life

- The degree of saturation of basecourse is widely acknowledged as a critical factor in basecourse stability, and this is consistent with the cases in the inventory. The long-term degree of saturation must remain less than about 70% for good performance. In view of the inherent limit of accuracy for determination of the degree of saturation, a reasonable target is a degree of saturation less than 65% at refusal (after traffic compaction).
- Producers should be encouraged to develop substantiated models that will enable practical estimates of design ESA loadings for any basecourse they produce, using the onset of lateral shear instability rather than vertical rutting as the key performance measure. If this can be done with reasonable accuracy then the net-present-value approach may be used to quantify the benefits or otherwise of basecourses that are either superior or inferior to standard M/4. This will allow efficient use of resources directing higher quality aggregates to only those sites with high traffic demand as well as the converse. A logical consequence of being able to utilise less than premium aggregates is that costs and haul distances will necessarily reduce (along with reductions in both pavement wear and carbon emissions). At present in some regions, basecourses with marginal shortfalls in fines criteria can be produced for less than half the cost of complying with M/4 specifications, once both processing and haul distances are taken into account. Clearly in these circumstances there will inevitably be a substantial difference in any benefit–cost evaluation and hence to viability of a project. This report sets out the basis of two methods for evaluating alternative basecourses, using either appropriate fines criteria (figure 5.9) or the prediction of refusal saturation (chapter 6).
- A literature review found existing methods for predicting i) the compacted dry density of an aggregate from grading and other index test, and also ii) the equilibrium water content (for well drained aggregates beneath an intact seal). These two parameters (together with solid density) define the degree of saturation and its potential for further change. Therefore the inventory was used to explore the feasibility of predicting the expected long-term degree of saturation of New Zealand basecourses in well drained pavements.
- Using the inventory parameters of particle size distribution, M/4 index tests and environmental parameters, interim regression equations have been formulated to predict the following characteristics for the unbound basecourse (or subbase) in a completed pavement (equations 6.1 to 6.6):
  - dry density (as a percentage of solid density) immediately after construction
  - dry density (as a percentage of solid density) long term after bedding in (refusal trafficking)
  - equilibrium water content in a well drained pavement (long term)
  - equilibrium degree of saturation at refusal trafficking (long term)
- The predicted equilibrium degree of saturation provides a fundamental design criterion that will safeguard against shear instability. Values under 60% should give good service while those over 70% indicate the need for convincing evidence that they can perform. Values between 60% and 70% need to be investigated with observations of precedent performance or field trials. RLT testing may also become an option once procedures are refined.

8.4 Repeat load triaxial testing

- In the RLT test, the over-riding consideration is that testing at 95% of maximum dry density may not give conservative results as the relevant degree of saturation after bedding-in. The ARRB Better Basis for Bases project (Jameson et al 2009) raises significant issues regarding the limitations of repeat load testing:

  The Austroads RLT permanent strain test is not suitable for characterising the resistance of granular bases to lateral shoving under thin bituminous surfacings. This is an important deficiency that significantly influences the usefulness of the test as a means of characterising rut resistance.

- A limitation of the RLT test when applied to an 'average' sample of basecourse as traditionally sampled, is that it will not be representative of the lower bound quality from the source. Shear instability typically occurs in less than 5% of a pavement by the stage it is regarded as requiring rehabilitation. The greater part of a newly constructed pavement should have a particle size distribution and fines content that will give good performance. However, as a consequence of the inevitable statistical variation in quality that is inherent in an assemblage of particles, localised clusters will exist in the pavement where the percentage of fine material is higher than average. The variation may be aggravated where construction practices have contributed to segregation. Generally therefore, there will be parts of any unbound granular pavement which will be more predisposed to shoving, particularly in the outer wheelpath where the long-term water content is likely to be higher and more variable. RLT testing of the carefully sampled 'average' particle size distribution would not give meaningful evaluation of the potential for shear instability. Selection and sampling of say the 5th percentile from the finest clusters from the placed basecourse layer may be more appropriate but this nominated percentile and the selection process are both likely to be contentious. The above concept may well have some bearing on the ARRB research findings.
- Once in-service basecourse density and, in particular, degree of saturation at equilibrium water content are appropriately replicated in the laboratory in the advanced stages of the test, there may be a better prospect of ultimately characterising performance. Without knowing these two parameters, the RLT may be of little value to the practitioner. If RLT testing is carried out on samples with less than 65% saturation, it should still be possible to rank basecourses meaningfully in terms of their rutting due to vertical deformation. At higher degrees of saturation, rutting rates due to lateral deformation (shear instability) will be acutely sensitive to the test density and degree of saturation.
- Because a change in degree of saturation from 65% to 70% will approximately double the rate of permanent deformation under cyclic loading (equation 6.7), it is important that the degree of saturation is calculated consistently (ideally using the absorption and bulk dry specific gravity) and reported for any RLT test that is used to predict likely performance of a basecourse in service. Determining degree of saturation at the beginning and in particular for the advanced stages of the RLT test is not current practice in New Zealand but should be feasible (G Arnold, pers comm).
- The Better Basis for Bases project identified the need for a performance index that would adequately characterise basecourse shear stability. The equilibrium degree of saturation, determined as described above, should serve as such a performance index.

8.5 Case histories of premature distress

The most common form of premature distress in the case histories studied was shear instability of basecourses due to densification under trafficking, which resulted in high degrees of saturation in well drained pavements. Other issues in approximate order of decreasing frequency were:

- failure to dry back basecourses prior to sealing

- porous surfacings (including insufficient seal coats in the first season)
- basecourse shoving failure due to impermeable subbases (including ineffective ripping of old seal coats and/or overwatering during construction)
- cracking of thin asphaltic concrete surfacings due to excessive curvature and/or lack of bond to the basecourse
- sub-standard compaction of subbase layers
- failure to correct the target field density for AP 65 subbase when oversize was removed for determination of laboratory maximum density
- subgrade intrusion into incompatible subbase
- excessive subgrade strain resulting in rutting
- subbase shoving due to shear instability in the subbase layer
- excessive fines capping the basecourse, as a result of over-working or non-compliant running course and overwatering during construction.

In the absence of shear instability, rutting densification of basecourse compacted in accordance with B/2 was not a critical factor for the premature distress in any of these case histories.

Following on from the above, if basecourse rutting is not a primary distress mechanism in any locality, then M/4-20 or even M/4–30 basecourse overlays could be considered as an alternative to AP40 (regardless of specified layer thickness). Most Australian basecourses are 20mm maximum-sized aggregates in view of the perceived reduction in segregation and improvements to the finished characteristics such as roughness. In some South Island alluvial pits, producers find that to obtain the requisite broken faces, wastage is commonly 60% to produce M/4–40 but this reduces to 35% or less for M/4–30, thus greatly reducing production costs as well as more sustainable utilisation of the limited resource. (In sources with traditionally marginal CBR values, reduction of maximum size would not of course be acceptable if they then fail M/4 CBR requirements.)

8.6 Fines criteria for basecourses

- Many sound basecourses with sand equivalents well above 40 will degrade to provide values of about 30 after they have been trafficked for only a small proportion of their design lives. Therefore while the stockpile sand equivalent can reasonably identify a good performer (SE>40) the converse does not apply; a value of <40 does not necessarily confirm a poorly performing aggregate. To reasonably reject any basecourse, its sand equivalent would need to be below 20–25. Between 20 and 40, alternative measures to characterise the fines quality and quantity should be used. However, the sand equivalent is a useful index for progressive quality assurance of the consistency of the fines quality/quantity from an individual quarry.
- Plasticity index is determined on the passing 0.425mm fraction. The proportion of this fraction in basecourses varies considerably, therefore an improved characteristic (used by other roading authorities) for assessing basecourses is the ‘weighted plasticity index’ calculated as the product of the plasticity index and the percentage passing the 0.425mm sieve.
- Clay index is determined only on the fraction passing 0.075mm sieve. Focus for basecourse performance should therefore be on the weighted clay index calculated as the product of the clay index and the percentage passing 0.075mm.
- Because M/4 is limited to stockpile properties, subsequent changes in parameters are seldom monitored. A practice adopted by VicRoads is to confirm the quality and quantity of fines of the basecourse post-

compaction, thereby reducing the risk of premature failure. Post compaction testing also provides assurance that the material which reaches the site is consistent with that from a previously approved stockpile and directly addresses issues such as source variability, segregation, extent of working/reworking and over-compaction with steel rollers. Post compaction testing presents contractual issues, but there is no practical alternative that will address the current issues with premature distress of M/4 compliant basecourses.

- Another limitation of M/4 is that the quality of aggregate required is independent of ESA loading, and is also independent of the environment or any compromising factors such as poor drainage. Former practice has been to modify the 'fines criteria' in some regions to reduce risk. A modified set of four fines criteria has been established, namely weighted clay index, weighted plasticity index, sand equivalent and SGE. An interim guideline for appropriate fines criteria as a function of design ESA is given in figure 5.9. This interim figure will change with further research, but in the near future, should be able to be used systematically by designers to either tighten or relax the standard M/4 specification, depending on design ESA and drainage conditions. An associated spreadsheet for sensitivity analyses is available, Tonkin & Taylor (2010a), for net present value calculations when comparing alternative quality basecourses, and this can be used to demonstrate there are cases where the increased cost of well graded basecourses has a good return, but there are also cases where accepting only say 10–15 years life with a poorly graded basecourse can also have a net benefit when the initial cost-differential is sufficiently large.
- To ensure long-life basecourses, a simple and effective approach that could be put in place by aggregate producers would be to focus in particular on processing that maximises the SGE and also minimises the weighted clay index.

8.7 Unbound granular overlays

- Unbound granular overlays that are failing through saturation are likely to be affected at least partly by the fact that the underlying layer is unlikely to meet the permeability requirements of M/3 notes (regardless of the effectiveness of seal ripping). Compensation is essential and this could be achieved by specifying a higher quality basecourse than required for full depth construction (with permeable subbase). An interim procedure would be to increase the design ESA by a factor of at least 2, then specify the corresponding set of fines criteria (given for that factored ESA in figure 5.9) to override those in the draft M/4 specification (see appendix B).

8.8 Basecourse and subbase specifications and construction quality control

- The findings of this report have been used in a draft revision of the M/4 basecourse specification, (appendix B), and M/3 notes (appendix C). Ongoing compilation of the basecourse inventory is in progress to confirm or extend the interpretations to date, hence providing future refinement of M/4 and the procedures for quantifying basecourse life prediction.
- The proposed addition to the grading shape control requirements has been discussed with many of the larger aggregate producers and testing laboratories. They advise that the changes can be accommodated with little or no impact on prices. The aggregate producers acknowledge the need to provide high levels of confidence in their products and acceptance will not be an issue for them, but it will be important to also canvas other producers. Another issue raised during the consultation process was the need for a minimum crushing resistance for an upper subbase.

- Where new basecourses are being produced for projects where cement modification or stabilisation is intended, a finer grading envelope than that for unbound basecourses is warranted and relaxed fines criteria are acceptable. Cost reductions should also result.
- The reasons for ignoring the procedures for assessing compatibility in M/3 notes, as found in the case histories studied, may be that it is not an issue addressed by Austroads, or there may be uncertainty about the calculation. Therefore the procedure has been assembled into a functional spreadsheet to facilitate the compatibility calculations. There are cases where compatibility is not essential (eg some lime or cement stabilised subgrades) but any departure from M/3 notes should be appropriately documented in design reports.
- A simple, pragmatic check on upper subbase quality as supplied/compacted in the pavement is to include reporting on the degree of saturation at the same time as in situ density (in the same manner as required for basecourse). No additional effort or cost is involved and any saturation over 70% will provide warning that permeability is likely to be unsatisfactory.
- Guidelines for the design and construction supervision of thin asphaltic concrete or OGPA surfacings (less than 40mm thick) are not well defined in the Austroads Guide (2008-2009) or the *NZ supplement*, as these documents favour thick structural asphaltic concrete for heavily trafficked roads. Nevertheless thin surfacings are in widespread use on New Zealand arterials and motorways, hence definitive guidelines are required to reduce the increasing incidences of premature cracking.

9 Recommendations

These recommendations include reinforcement and some modifications of those by Bartley (2007) and Gray (2007).

9.1 M/4 basecourse specification

- Adopt weighted plasticity index and weighted clay index rather than unweighted parameters as used in M/4:2006 'fines criteria', using typical values that will provide a 'pass' for most basecourses complying with the former specification. However, the change will allow the use of some source rock that would formerly not have been considered; hence close monitoring of the performance of basecourses from any new sources should be carried out. Any basecourse with mid-range grading (grading exponent 0.5) that complied with M/4:2006 will necessarily be accepted under the proposed M/4.
- In addition to weighted plasticity index, weighted clay index and sand equivalent, add a fourth 'fines criteria' being the SGE. Rather than requiring only one of the fines criteria be met, increase this to two. This may be further increased for heavily loaded pavements.
- Maintain M/4 as an aggregate supply specification in the meantime, but suggest in M/4 notes or the *NZ supplement* that designers may modify (within NZTA prescribed limits) the set of four 'fines criteria' in the revised M/4 (up or down depending on design ESA loading and environment) to allow due consideration of cost efficiency and energy conservation through the appropriate use of locally available resources. Where higher reliability is required, designers should be encouraged to vary the number of fines criteria to be met from two to three. For most pavements it should not be specified that all four fines criteria are to be met unless the minimum sand equivalent is set at 20–25. Note that targeting a design ESA for a particular basecourse is a move away from an 'all purpose' premium basecourse supply specification but is consistent with the application of RLT testing results which also assign an effective ESA rating.
- Specify that the M/4 fines criteria are to be evaluated on post-compaction samples (as practised by VicRoads) if any of the following apply:
  - the aggregate source does not have a record of satisfactory performance
  - the design traffic is greater than 1 million ESA
  - the crushing resistance is less than 180kN
  - the aggregate is blended (comes from more than one source)

  Samples should preferably be recovered from a trial section or from the shoulder. Because M/4 is a material supply specification, in practice it may be more convenient to salvage material from laboratory compaction or RLT tests. Recovering samples from the pavement during construction would also allow issues of uniformity of supply, segregation, contamination and construction degradation to be addressed effectively. That would increase the risk of disputes between the supplier and the contractor placing the aggregate, but would substantially minimise the greater risk of premature distress in marginal aggregates.
- Consider the implication of increasing the upper limit of the M/4 particle size distribution to follow the 0.35 grading exponent between the 4.75 and 37.5mm sieves only, as shown on figure 5.5. Obtain further well documented case histories to investigate changing other fractions.

- Raise the bar for the fines criteria for basecourses to be used as overlays where the underlying layer is not compliant with M/3 notes. Permeability of the underlying layer is likely to be inadequate for most overlays. As an interim measure, a minimum SGE of 0.4 should be specified. Ultimately, with an extended basecourse inventory, the design ESA should be increased by a factor of at least 2 and fines criteria from figure 5.9 used to override the fines criteria in the draft M/4. In addition, multiple seal coats should be applied in the first season. Similarly these enhanced fines criteria should be used for basecourses in areas that are frost prone, or subject to flooding.
- Add an additional particle size distribution for AP30 as a local variant for substitution of AP40 in those regions where there are substantial cost efficiencies to be made.

9.2 M/3 specification notes

- Introduce a minimum crushing resistance for upper subbase materials.
- Emphasise in M/3 notes the importance and means of ensuring adequate permeability of the upper subbase for all new pavement construction (incorporating figure 7.6).
- Emphasise in M/3 notes that compatibility between subgrade and subbase is an essential consideration and can be achieved with a transition layer (lower subbase) at least 75mm thick. Without it, potential intrusion can be many hundreds of millimetres (not just 75mm). Also simplify the procedure by making available an appropriate functional spreadsheet for compatibility design. Ensure any departure from the compatibility requirement is fully documented in the design report.
- A spreadsheet that facilitates compatibility checks for multiple pavement layers as well as a check on permeability of the upper subbase has been established for trial in Tonkin & Taylor (2010a).

9.3 B/2 compaction specification

- Consider the industry implications of reducing the B/2 acceptance criterion for degree of saturation from 80% to 70% prior to sealing, in the warmer months at least.
- If prior to sealing, the saturation of a basecourse cannot be readily reduced to 70%, carry out re-measurement of all of the M/4 fines parameters using samples from the compacted basecourse (shoulder or other non-critical location). If the fines parameters indicate likely long-term saturation, consider carrying out in situ cement modification of the basecourse prior to sealing. Reinforce in B/2 notes the need for documenting compaction testing of each individual lift to avoid the practice of testing only the final lift of each aggregate layer, and amend B/2 clause 7.6 to 'Compaction testing of each lift of the pavement layers shall be carried out....'.
- The current approach in B/2 to under-compaction is:

  Where the Acceptance Criteria is based on laboratory results and cannot be met, the Engineer shall nominate an independent laboratory to repeat the laboratory tests and supervise a repeat of the Plateau Density test. Should the Criteria still appear unachievable the Engineer may accept the Plateau Density tests as the Maximum Dry Density.

  For cases where the B/2 compaction target cannot be achieved in the lower subbase layers because the subgrade is highly yielding, it is suggested that the independent laboratory test is not warranted. Instead, reduce layer thicknesses and roller weight then confirm the necessary number of passes for each layer from plateau testing until a suitable base is eventually established to meet 95% average compaction. As bedding-in will be accompanied by deep seated settlement, sealing should be deferred until substantial trafficking (ideally 10,000 ESA) has been experienced with traffic flow progressively channellised to give full width coverage of the pavement. If earlier sealing is required, a measure of the effectiveness of the state of compaction of the deeper layers (and subsequent rutting likely) can be quantified at any stage, from the ratios of the layer moduli (Tonkin & Taylor 2002).

- Where compaction of the subbase is in doubt (as above) or B/2 results are not provided, carry out deflection testing to determine the modular ratios of successive pavement layers (relative to those given in Austroads (2008–2009) in order to assess (prior to sealing) whether adequate compaction has been obtained for the full depth of the pavement. If not continue with compaction by rubber tyred roller or traffic until the modular ratios meet that standard. (Austroads 2008–2009, part 2, section 8.2.3)
- Carry out quality assurance for the compaction of both subbase and basecourse in terms of percentage of solid density (or 'total' voids). Also, if target values determined from laboratory compaction are unusually low (less than 80% of solid density), use equation 6.4 to decide whether plateau testing or some other form of verification testing needs to be carried out.
- Calculate percentage of solid density (or percentage total voids) consistently with an agreed national standard. Logically, the bulk dry specific gravity (measured not assumed) should be adopted.
- Calculate the degree of saturation consistently with an agreed national standard. For B/2 purposes (immediately after construction) this should assume the intra-particle voids will be saturated and the bulk dry solid density should be used, with due allowance for absorption (equation 6.1). Where absorption is low the choice of which solid density should be used, is not important for B/2 purposes. For long-term studies (when the equilibrium water content has been reached throughout the aggregate) the intra-particle voids may be in a state of similar partial saturation to the inter-particle voids and the apparent solid density should be used for assessment of whether saturation is a factor in any premature distress.
- Include in B/2 the reporting of degree of saturation for the top subbase layer. Any value over 70% should immediately flag that the permeability of the supplied/compacted product is likely to be unsatisfactory, even if watering occurred only shortly before the test.
- Promote nationally, the use of a standard functional spreadsheet (containing the necessary input fields and equations) for use with B/2 acceptance testing to address the current omissions as well as uncertainty in the industry. This would also facilitate investigations of any distress and allow subsequent data mining for improved understanding of basecourse performance.
- If in situ density testing of a subbase encounters significant oversize particles (larger than those used in the laboratory compaction test) ensure the field target percentage density is corrected in accordance with AASHTO T 224.
- For running course, B/2 currently requires that 'The quality of fines (those materials passing 75µm) shall be not less than that required for the basecourse below the running course when tested in accordance with the requirements for the base course'. The only requirement for materials passing 75µm is the clay index. Consideration could be given to changing this requirement to 'The running course (or any fines used for blinding) shall meet the same fines criteria as specified for the basecourse below the running course'.

9.4 Production of new basecourses intended for cement treatment

- Consider development of new specifications for quarry production of basecourse intended for i) cement modification or ii) foamed bitumen stabilisation using finer grading envelopes than those for unbound basecourse. Further research is required to determine appropriate envelopes and fines criteria. An interim measure (subject to local verification prior to adoption) might be to adopt the fines criteria for low traffic values from figure 5.9, ie a SGE of not less than 22, weighted plasticity index of less than 80, weighted clay index of less than 26, or SGE of greater than 0.38 (with only one complying value necessary).

9.5 RLT testing

- Ensure RLT testing includes the 'mature' phase of pavement life, ie after bedding-in, using the relevant long-term predicted refusal dry density (as percentage of solid density) and long-term equilibrium water content.
- Include as part of the test reporting, the dry density (as percentage of solid density using the bulk dry specific gravity) and effective degree of saturation (by allowing for absorption). These should be determined both at the start and finish of the test. A full particle size distribution (ex stockpile) using wet sieving in accordance with NZS 4407:1991, test 3.8.1 is also needed as standard practice to facilitate subsequent quality assurance.

9.6 NZ supplement

- Include in the design process in the *NZ supplement* a requirement for a specific check for the potential for basecourse shear instability in the longer term. The check may be any of the following:
  - a precedence of satisfactory performance of the same aggregate under equivalent trafficking and environmental conditions (eg reference to the expanded basecourse inventory)
  - confirmation that the predicted degree of saturation of the basecourse long term after trafficking to refusal density can be expected to be less than 65% (chapter 6).
  - RLT testing noting that it is imperative to test at the long-term (refusal) density and equilibrium water content expected in-service, and addressing the reservations raised by Jameson et al (2009).
  - a SGE of not less than 0.4
  - some other well substantiated procedure to ensure long-term shear stability.
- Add figure 7.1 to reinstate the previous New Zealand pavement design concept of the upper subbase zone that must have high permeability in any new pavement construction (and also in a rehabilitation design unless the risk of saturation can be adequately mitigated by compensating qualities).
- Where conditions are marginal for either subbase permeability or basecourse shear stability, apply compensatory measures such as multiple seal coats in the same construction season and ensure surface cross-falls are effective. If both subbase permeability and basecourse shear stability are both marginal, cement modify the basecourse.
- When designing any unbound granular overlay, the designer should acknowledge it is likely the supporting layer will not meet the permeability requirements of M/3 notes. If there is no reasonable corrective solution then all practical compensating measures shall be implemented in the pavement design. Such measures would be i) ripping of any existing seal layers, ii) unconstrained lateral drainage, iii) a SGE of greater than 0.4 in the compacted basecourse or other design procedure that gives resistance to long term saturation of the new basecourse, iv) a suitably effective water resistant surfacing (multiple coat seals) and v) surface crossfall that promotes run-off.
- If construction is likely to be carried out late in the season or in wet climates consider specifying basecourse with a SGE of 0.40 or higher to reduce the risk that saturation at the time of sealing will be excessive.

Conversely if all local aggregate sources have marginal SGEs, programme the works early so that construction can be completed in summer (sealing before mid-March).

- In regions where alluvial sources have a shortage of the coarser gravel sizes, encourage suppliers to provide comparative rates for M/4-30 and M/4-20 basecourses as well as for M/4-40. Provided basecourse rutting is not a common distress characteristic in that region, use of reduced topsize can result in substantial cost savings with little or no impact on pavement performance.
- Review the design process for rehabilitation design of thin pavements on highly resilient volcanic subgrades. Include a check to confirm the horizontal tensile strains in any cement modified layer will be in the normal range, ie less than 500 microstrain unless higher values are justified by precedents in similar conditions.
- Provide practical guidelines for the design and construction of thin asphaltic surfacing (less than 40mm thick) subject to high traffic loadings (and always if loading exceeds 5 million ESA) to reduce the current incidence of premature distress in this type of surfacing, for example:
  - Specify a premium basecourse then sample (eg dry core) the shoulder of the compacted basecourse (preferably sampling in a pilot trial but at least prior to surfacing) to confirm the basecourse is capable of carrying the design traffic (chapter 6).
  - Carry out deflection testing to record characteristic curvature at the top of the compacted basecourse prior to surfacing.
  - Correct the measured curvature for post-construction densification under traffic using Austroads modular ratios for unbound granular layers.
  - Determine the suitability of the structure for accommodating the design traffic with the design thickness of AC either mechanistically, or, empirically using as a minimum the 40mm thickness in figure E-A2.2 in part 5 of Austroads (2008–2009), but ensure that likely construction tolerances are included. Typically, specified thin AC thickness will vary by +/- 5mm.
  - Use the same checks for the full length of the constructed road, immediately prior to application of the thin AC. Intervention (eg with a modified basecourse) could then be invoked if the checks fail.
  - Ensure an effective bond is achieved between the surfacing and the swept basecourse, as well as provision of an effective waterproofing membrane (verified with an acceptance test).

9.7 General

- Encourage quarry operators who are currently producing basecourses or subbases that are gap graded in the sand fraction to add clean quarry fines, or a suitable sand to ensure their product fits well within the grading envelope limits and is as well graded as practicable. Promote, where economic, the use of crushed recycled glass for this purpose, particularly in basecourse but also in subbase. Obtain case histories or establish trial pavements to verify the performance of basecourses with the upper M/4 upper grading limit set to follow the 0.35 grading exponent between the 4.75mm and 37.5mm sieves.
- Obtain case histories to determine whether lime or cement stabilisation of a subgrade will necessarily allow the requirement for filter compatibility at the lower subbase interface to be waived or not.
- Measure solid density and absorption at least annually but preferably more regularly, and always for projects where the design traffic is more than 1 million ESA, or where aggregates sources have any history of premature distress, in order to ensure close quality assurance of both compaction (based on bulk dry specific gravity) and degree of saturation (corrected for absorption where applicable).

- Include the date and relevant report reference of the last bulk dry solid density and absorption measurements on all laboratory reports for compaction and degree of saturation of aggregates if an assumed solid density or absorption is utilised in the current report. Include in M/4 and M/3 notes a requirement for the sampling date for stockpile laboratory acceptance testing to be not more than three months prior to the date of delivery to site, unless traceability is documented with relevant details of stockpile management.
- Measure the in situ degree of saturation of the basecourse in the wheelpath in any pavement investigation where shear instability is suspected. Ensure the base reading for the density volume is taken on the irregular surface of the exposed aggregate. If all M/4 results for each project could be routinely stored electronically (perhaps in association with RAMM), subsequent lessons from any premature distress and calibration of methods for the prediction of remaining life using deflection measurements would be greatly facilitated.
- Maintain and expand both the inventory of basecourses which have known performance and the file of premature distress case histories. More wide ranging databases will allow refinement of the regression equations developed so far, and ultimately improve the ability of designers to establish reliable ESA ratings for basecourses. More documentation should be sought on the change of weighted clay index and weighted plasticity index with aggregates of varying crushing resistance and with trafficking on pavements built on both rigid and yielding subgrades.

  Basecourse inventory link: www.pavementanalysis.com/applications/Basecourse/Aggregate Parameters Anon Sites.xls

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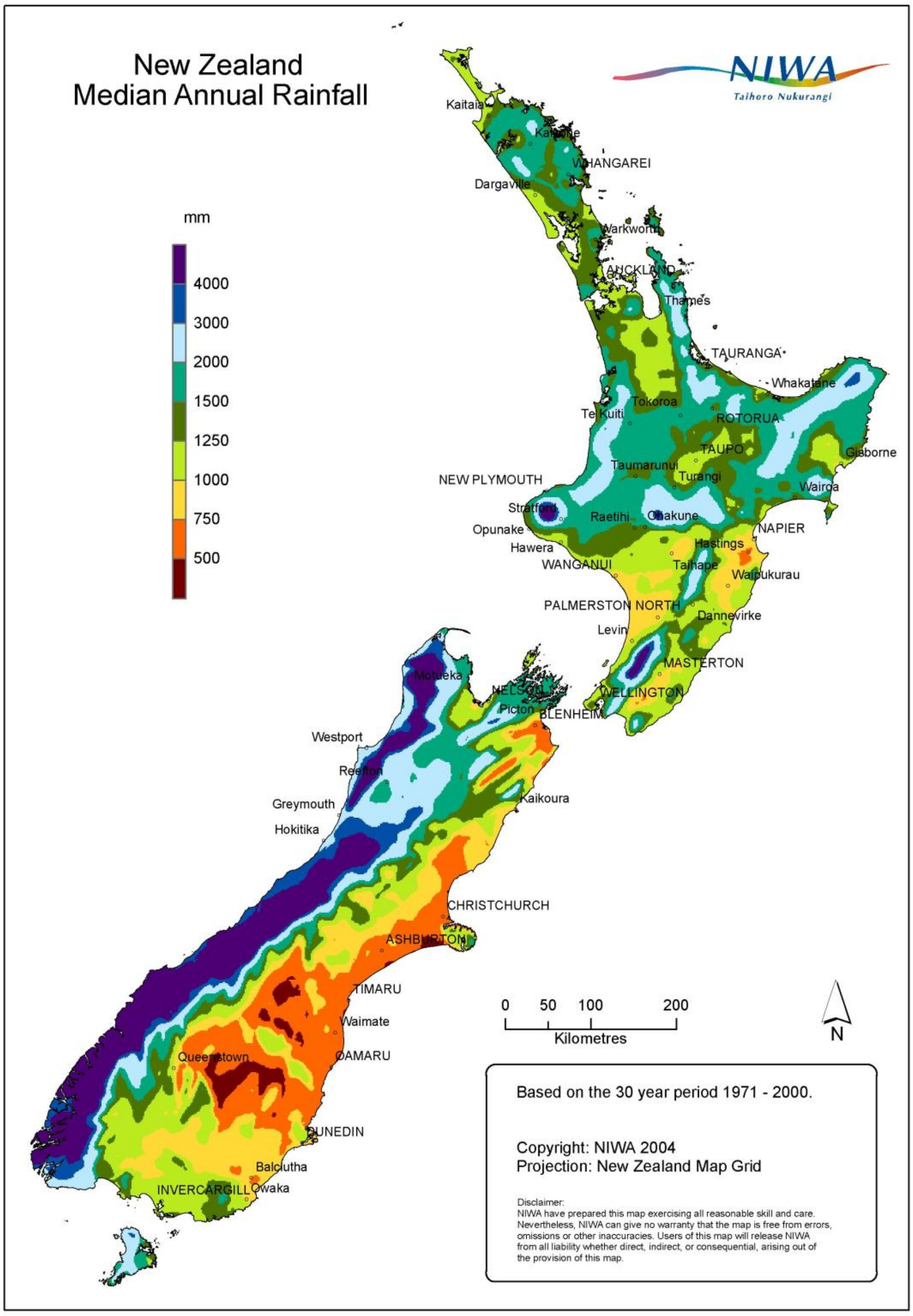
Appendix A: Median annual rainfall and potential evapo-transpiration maps



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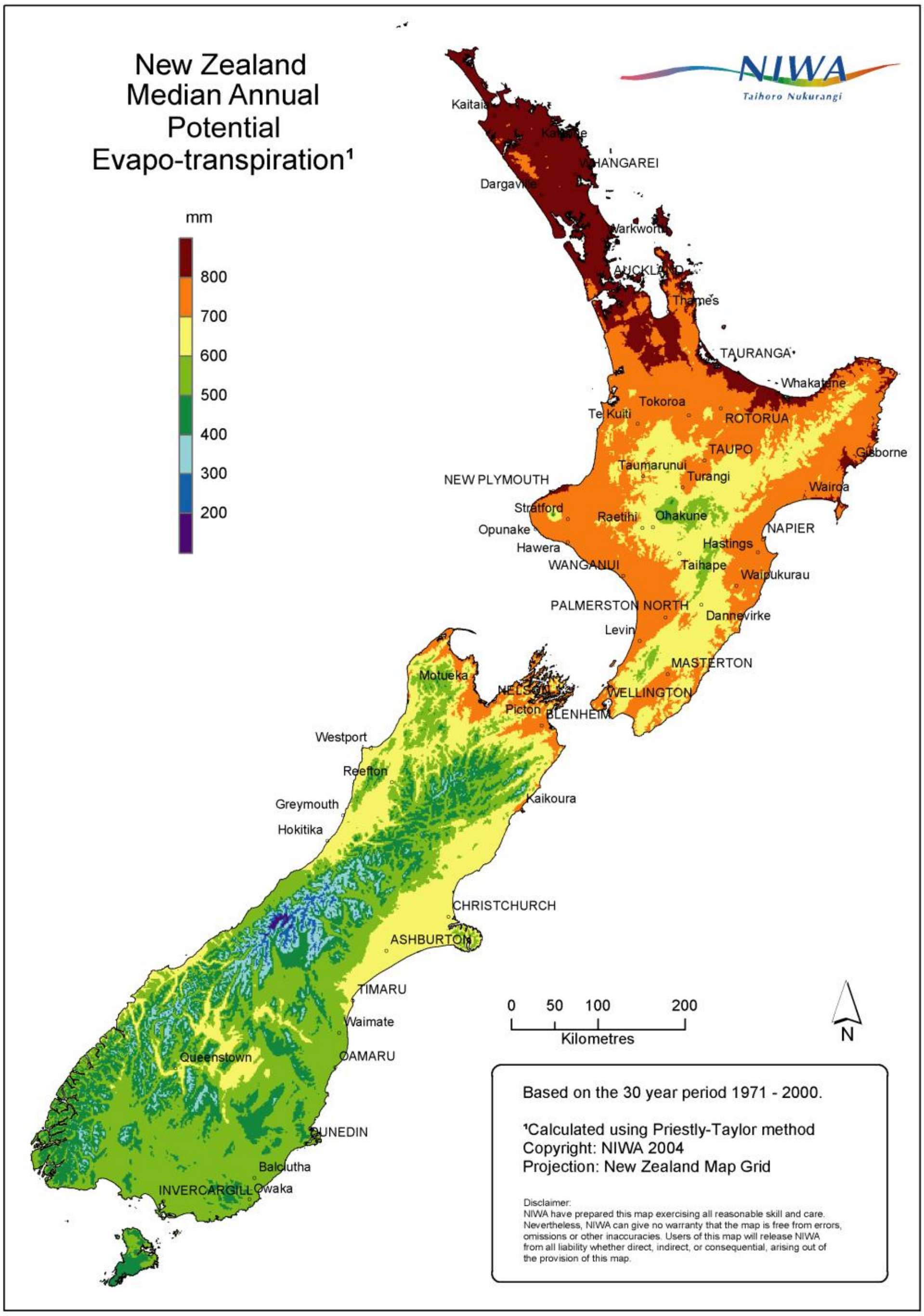

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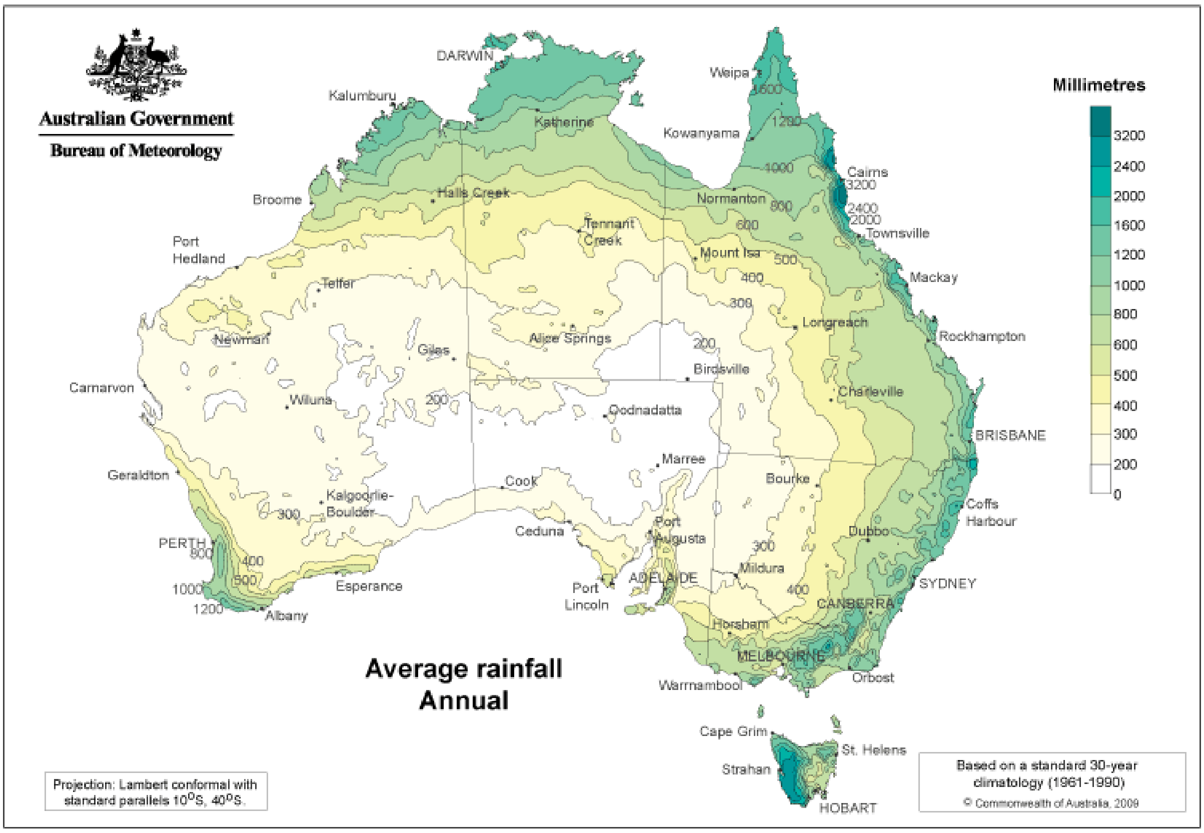
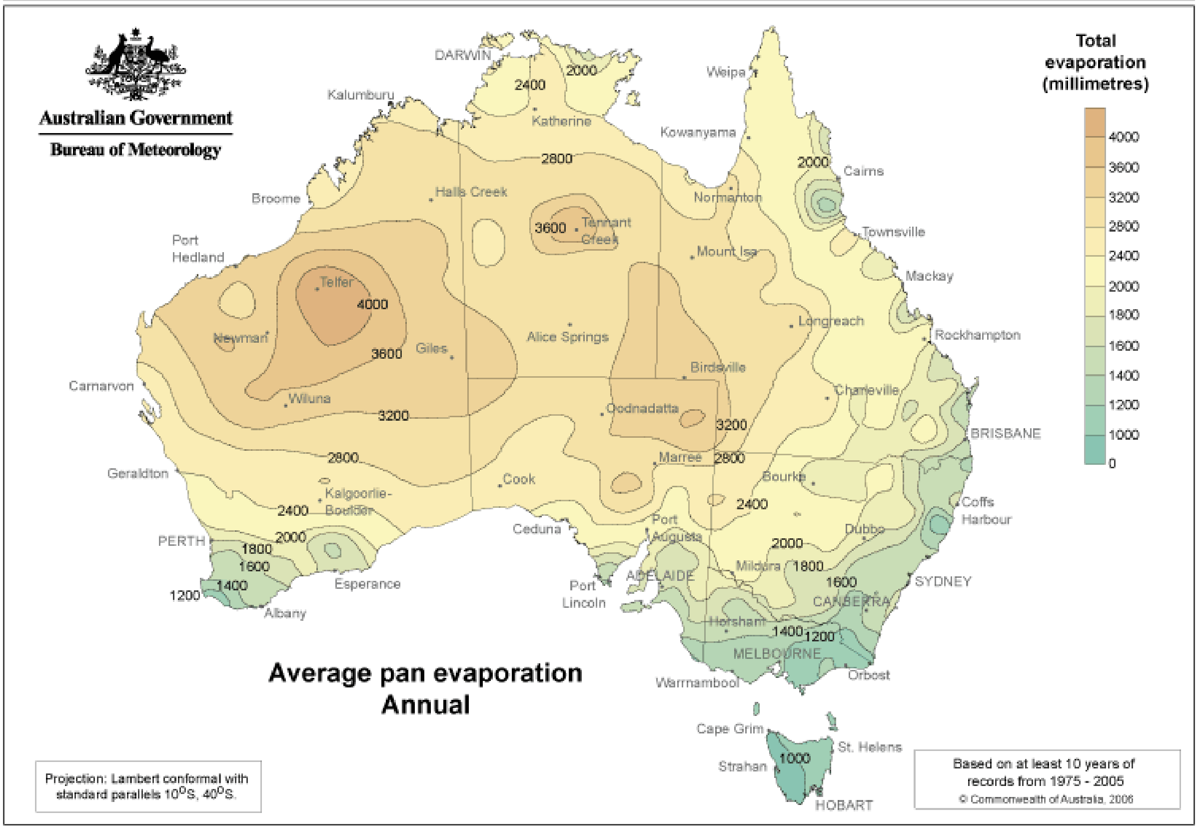
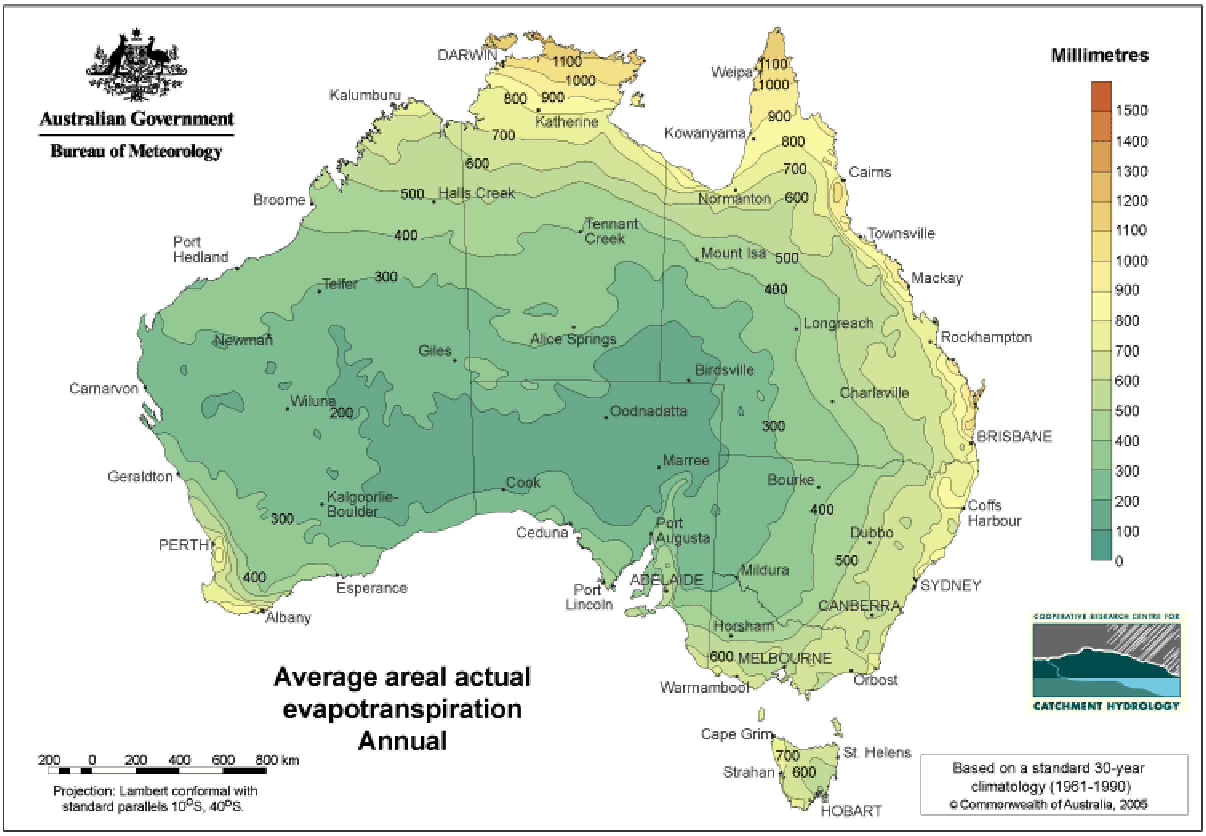




Source of Australian maps: MRWA (1993)



Source of Australian maps: MRWA (1993)

Appendix B: Draft amended M/4 basecourse specification and notes

[NZTA logo]

Proposed NZTA M/4: 2011 draft following industry comment – not current NZ Transport Agency (NZTA) policy

(Note changes in section 4 resulting from recent failure case histories, refer to *NZTA research report 459* 'Extending pavement life'.)

Specification for basecourse aggregate

1 Scope

This specification sets out requirements for basecourse aggregate for use on state highways and other heavily trafficked roadways.

2 General

All sampling and testing shall be performed by an IANZ accredited laboratory for the performance of the relevant test as shown in figure 1.

All basecourse aggregate which does not comply with the requirements of this specification shall be either: tested as agreed by the ~~Transit~~ NZTA's Engineering Policy Manager for consideration as a regional basecourse aggregate for inclusion in table 4 or rejected.

The basecourse aggregate shall be classified as either M/4 or one of the regional basecourse aggregates detailed in table 4. Additional guidance on the use of regional basecourse aggregates is provided in the appendices to the notes for this specification.

3 Source properties

The basecourse aggregate shall be broken or crushed from either: waterworn gravel; quarried rock or from other sources accepted as a regional basecourse aggregate detailed in table 4. Source material shall consist of hard, sound material of uniform quality, free from soft or disintegrated stone or other deleterious material.

3.1 Testing source properties general

Source properties of the aggregate shall be assessed by the testing specified in clause 3.3 on samples of aggregate from current production, which are representative of the processing method.

If the aggregate source or processing method is changed then the source properties shall be tested immediately and the engineer informed. Acceptance of basecourse aggregate from the varied process shall be at the discretion of the engineer until the source properties are shown by test to comply with this specification.

The source property tests shall be performed at periods not exceeding two years unless a comparative petrographic examination of the current aggregate and a sample from the material successfully tested two years earlier shows there has been no significant change in the material.

If a petrographic examination is used as described above the source properties shall be tested at least once every four years.

The petrographic examination must be performed by persons who are qualified by education and experience to employ techniques for the recognition of the characteristic properties of aggregates and minerals. The examination shall follow the guidelines given in ASTM C 295 *Standard practice for petrographic examination of aggregate for concrete*.

When testing source properties a sample of the aggregate suitable for petrographic examination shall be stored for a minimum of two years by the IANZ accredited laboratory performing the test.

The engineer may require some or all of the source property tests to be performed in addition to the testing frequencies stated above. Should the test results show that the material complies with this specification, testing will be at the principal's cost, otherwise testing will be at the cost of the contractor.

3.2 Source property tests and sampling

Source properties shall be sampled and tested at a rate of at least one sample for every 10,000m^3 of source material.

3.3 Source property tests

3.3.1 Crushing resistance

When tested in accordance with NZS 4407: 1991, test 3.10 *The crushing resistance test*, under a load of 130kN less than 10% fines passing 2.36mm sieve size shall be produced.

If the aggregate includes blended fines from another source, this shall be stated because their durability would not be reflected by the size range used for the crushing resistance test. Therefore relevant documentation of precedent performance or alternative assessment of durability shall be provided.

3.3.2 Weathering quality index

The aggregate shall have a quality index of AA, AB, AC, BA, BB or CA when tested according to NZS 4407: 1991, test 3.11 *Weathering quality index test.*

3.3.3 California bearing ratio

The sample shall be:

- compacted in accordance with NZS 4402: 1986, test 4.1.3 *New Zealand vibrating hammer compaction test at optimum water content*, and
- tested in accordance with NZS 4407: 1991, test 3.15 *The California bearing ratio test* (without a surcharge for at least four days). The soaked California bearing ratio (CBR) of the basecourse aggregate shall not be less than 80%.

4 Production properties

Production properties of the aggregate shall be assessed by the testing specified in clause 4.2 on representative samples of the crushed aggregate.

Representative samples of aggregate may be taken from conveyor belt, bin, stockpile or truck. Representative samples of the aggregate shall be obtained in accordance with NZS 4407: 1991.

4.1 Production property test sampling

Stored aggregate shall be subdivided into lots so that aggregates of visible difference are sampled and tested separately. The rate of obtaining samples from lots shall be as in table 1.

Table 1 Minimum sampling rate for production property tests

Lot size		Number of samples
From	**To**	
1 m^3	400m^3	2
400m^3	1500m^3	3
1500m^3	4000m^3	4

Where the lot size exceeds 4000m^3 additional testing shall be at the rate of one sample for every 1000m^3.

Sampling for production acceptance testing shall be carried out not more than three months prior to delivery of aggregates to site. This requirement may be waived if traceability is documented with relevant details of stockpile management, to the satisfaction of the engineer.

The engineer may require some or all of the production property tests to be performed in addition to the testing frequencies stated above. Should the test results show that the aggregate complies with this specification, testing will be at the principal's cost, otherwise testing will be at the cost of the contractor.

4.2 Production property tests

4.2.1 Quality of fines

The basecourse aggregate shall comply with any two of the four 'fines criteria'. The fines criteria are ~~either~~ sand equivalent, weighted clay index, ~~,~~weighted plasticity index and sand grading exponent (SGE) stated below. For materials which are either i) of moderate to low crushing resistance (less than 180kN) or ii) are blends from more than one source, the quality of fines shall be assessed from samples recovered from a field compaction trial or alternatively from material recovered after vibrating hammer compaction.

4.2.1.1 Sand equivalent

The sand equivalent shall not be less than 40 when the aggregate is tested according to NZS 4407: 1991, test 3.6 *Sand equivalent test*.

4.2.1.2 Weighted clay index

The clay index of the fraction of basecourse passing the 75μm sieve multiplied by the percentage passing that sieve shall not be greater than 15~~3~~ when the aggregate is tested according to NZS 4407: 1991, test 3.5 *Clay index test* and test 3.8.1 *Wet sieving test,* with the percentage passing expressed to two significant figures.

4.2.1.3 Weighted plasticity index

The plasticity index of the fraction of basecourse passing the 425µm sieve multiplied by the percentage passing that sieve shall not be greater than 40~~5~~ when the aggregate is tested according to NZS 4407: 1991, test 3.4 *Plasticity index test* and test 3.8.1 *Wet sieving test* with the percentage passing expressed to two significant figures.

4.2.1.4 Sand grading exponent

The SGE is the effective slope of the particle distribution over the sand range and is defined in M/4 notes: 2010. It is a measure of the amount of gap grading in the sand sizes. The SGE shall not be less than 0.40 when the aggregate is tested according to NZS 4407: 1991, test 3.8.1 *Wet sieving test*

The quality of fines check shall be applied both to the base M/4 and separately to any crusher dust or running course fines prior to the use of such materials for final surface preparation.

Note that the SGE criterion applies in addition to overall grading shape control in table 3.

4.2.2 Broken face content

The aggregate broken face content in each of the ~~three~~ aggregate fractions coarser than the ~~between the 37.5mm and the~~ 4.75mm sieve~~s~~ shall not be less than 70% by weight and shall have two or more broken faces, when tested according to NZS 4407: 1991, test 3.14 *Broken face test*.

4.2.3 Particle size distribution

The particle-size distribution of the aggregate shall conform to the envelope limits defined in tables 2 and 3, when the aggregate is tested according to NZS 4407: 1991, test 3.8.1 *Wet sieving test*. Percentages passing each sieve shall be reported to two significant figures.

If testing has been performed to show the dry sieving method is not significantly different to the wet sieving method at 95% confidence limit for the same aggregate then the dry sieving method may be used.

Table 2 Particle size distribution envelope limits for an individual sample

Test sieve aperture	Maximum and minimum allowable percentage weight passing	
	AP40 (max size 40mm)	AP20 (max size 20mm)
37.5mm	100	-
9mm	66-81	100
9.5mm	43-62	55-80
4.75mm	28-49	3-61
2.36mm	19-38	22-47
1.18mm	12-29	14-35
600µm	7.0-21	8.0-25
300µm	3.0-14	5.0-16
150µm	0.0-10	0.0-12
75µm	0.0-7.0	0.0-8.0

Table 3 Particle size distribution shape control

~~Fractions~~	~~Maximum and Minimum Allowable Percentage Weight Of Material Within the Given Fraction~~	
	~~AP40 (Max size 40mm)~~	~~AP20 (Max size 20mm)~~
~~19mm - 4.75mm~~	~~28 - 48~~	~~-~~
~~9.5mm - 2.36mm~~	~~14 - 34~~	~~20 - 46~~
~~4.75mm - 1.18mm~~	~~7 - 27~~	~~9 - 34~~
~~2.36mm - 600μm~~	~~6 - 22~~	~~6 - 26~~
~~1.18mm - 300μm~~	~~5 - 19~~	~~3 - 21~~
~~600μm - 150μm~~	~~2 - 14~~	~~2 - 17~~

Incremental size range	Incremental grading exponent for each combination of sieve sizes	
	Maximum	Minimum
19 mm–4.75 mm 9.5 mm–2.36 mm 4.75 mm–1.18 μm 2.36 mm–600 μm 1.18 mm–300 μm 600 μm–150 μm	1.0	0.3
~~Note: The criteria apply for every combination of 2 standard sieve size increments where the percentage passing is < 100% and > 0%.~~ Note: The criteria apply for every combination of 2 standard sieve size increments where the percentage passing is < 100% and > 0%.		

5 Regional basecourse aggregates

For the regional basecourse aggregates the M/4 criteria shall apply except for deviations as stated in table 4.

The regional basecourse aggregates may only be used in the region detailed if specified in table 4 or as approved by the engineer. The use and source of regional materials must be clearly identified in the contractor's tender. A methodology for dealing with any special considerations must also be included in the tender.

6 Compliance

The contractor shall supply proof of compliance before basecourse aggregate is supplied.

7 Basis of measurement and payment

The basis of payment shall be on the final compacted volume of the basecourse aggregate in place with the method of measurement as defined in the contract documents.

Figure 1 Flow chart for basecourse aggregate tests

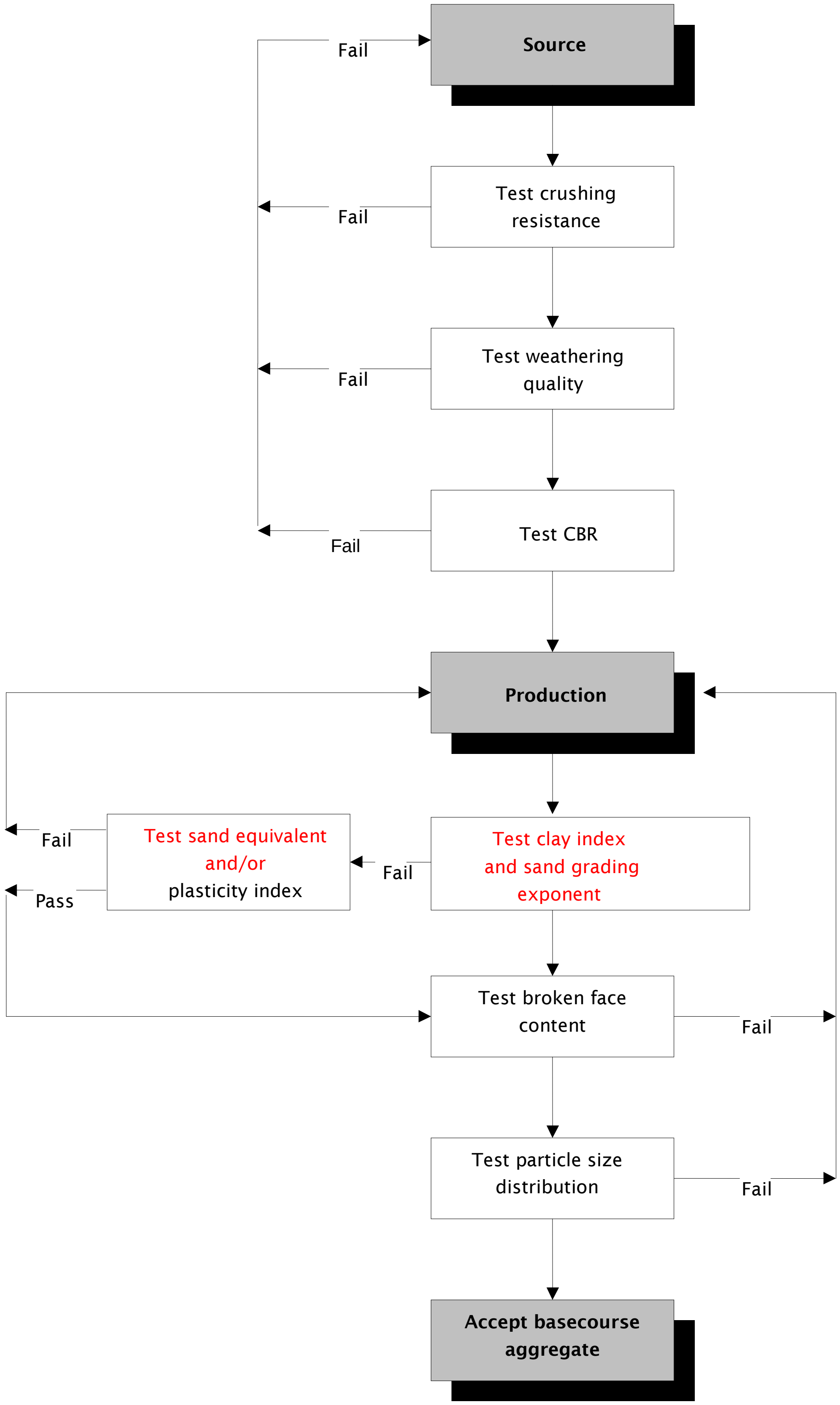


Table 4 Regional basecourses

NZS 4407:1991 test name	Test no.	TNZ M/4		NAPIER river gravel	
Weathering quality index	3.11	AA,AB,BA,BB,CA			
Crushing resistance	3.1	Not less than 130kN			
California bearing ratio	3.15	Not less than 80%			
Broken face content greater than two	3.14				
Sieve size					
19mm – 37.5mm		Not less than 70%		Not less than 50%	
9.5mm – 19.0mm		Not less than 70%		Not less than 50%	
4.75mm – 9.5mm		Not less than 70%		Not less than 50%	
Quality of fines					
Sand equivalent, or	3.6	Not less than 40		Not less than 35	
Clay index, or	3.5	Not greater than 3		If sand equivalent is less than 35	
Plasticity index	3.4	Not greater than 5		If sand equivalent is less than 35	
Wet sieving test	3.8.1				
Test sieve aperture		AP40	AP20	AP40	AP20
37.5mm		100	–		
26.5mm		–	–	78 – 100	
19mm		66 – 81	100		
9.5mm		43 – 57	55 – 75		
4.75mm		28 – 43	33 – 55		
2.36mm		19 – 33	22 – 42		
1.18mm		12 – 25	14 – 31	13 – 25	
600µm		7 – 19	8 – 23	10 – 19	
300µm		3 – 14	5 – 16	7 – 14	
150µm		0 – 10	0 – 12	5 – 11	
75µm		0 – 7	0 – 8	3 – 8	
Particle size distribution shape					
Fractions		AP40	AP20	AP40	AP20
19.0mm – 4.75mm		28 – 48	–		
9.5mm – 2.36mm		14 – 34	20 – 46		
4.75mm – 1.18mm		7 – 27	9 – 34		
2.36mm – 600µm		6 – 22	6 – 26	6 – 20	
1.18mm – 300µm		3 – 19	3 – 21	5 – 15	
600µm – 150µm		2 – 14	2 – 17	2 – 12	
Traffic loading limit					

NZS 4407:1991 Test name	Test no.	Rotorua 1 rhyolite	Rotorua 2 part crushed river gravel
Weathering quality index	3.11		
Crushing resistance	3.1	Not less than 60kN	
California bearing ratio	3.15		
Broken face content greater than two	3.14		
Sieve size			
19mm – 37.5mm 9.5mm – 19.0mm 4.75mm – 9.5mm		N/A N/A N/A	Not less than 40% Not less than 40% Not less than 40%
Quality of fines			
Sand equivalent or	3.6		Not less than 45
Clay index or	3.5		If sand equivalent is less than 45
Plasticity index	3.4		If sand equivalent is less than 45
Wet sieving test	3.8.1		
Test sieve aperture		AP40 AP20	AP40 AP20
37.5mm 26.5mm 19mm 9.5mm 4.75mm 2.36mm 1.18mm 600µm 300µm 150µm 75µm			
Particle size distribution shape control			
Fractions		AP40 AP20	AP40 AP20
19.0mm – 4.75mm 9.5mm – 2.36mm 4.75mm – 1.18mm 2.36mm – 600µm 1.18mm – 300µm 600µm – 150µm			
Traffic loading limit		Less than 1 x 106 ESA	

NZS 4407:1991 Test name	Test no.	Wanganui shell rock		Taranaki andesite – 65kN	
Weathering quality index	3.11				
Crushing resistance	3.1	Not less than 50 kN		Not less than 65 kN	
California bearing ratio	3.15	Not less than 120%			
Broken face content greater than two	3.14				
Sieve size					
19mm – 37.5mm 9.5mm – 19.0mm 4.75mm – 9.5mm		N/A N/A N/A			
Quality of fines					
Sand equivalent, or	3.6				
Clay index, or	3.5				
Plasticity index	3.4				
Wet sieving test	3.8.1				
Test sieve aperture		AP40	AP20	AP40	AP20
37.5mm 26.5mm 19mm 9.5mm 4.75mm 2.36mm 1.18mm 600μm 300μm 150μm 75μm		N/A N/A N/A N/A 70 MAX N/A 50 MAX N/A N/A N/A 10 MAX			
Particle size distribution shape					
Fractions		AP40	AP20	AP40	AP20
19.0mm – 4.75mm 9.5mm – 2.36mm 4.75mm – 1.18mm 2.36mm – 600μm 1.18mm – 300μm 600μm – 150μm		N/A N/A N/A N/A N/A N/A			
Traffic loading limit				Less than 2 x 10^5 ESA	

NZS 4407:1991 Test name	Test no.	Taranaki andesite –85kN	Taranaki andesite–100kN
Weathering quality index	3.11		
Crushing resistance	3.1	Not less than 85 kN	Not less than 100 kN
California bearing ratio	3.15		
Broken face content greater than two	3.14		
Sieve size			
19mm – 37.5mm 9.5mm – 19.0mm 4.75mm – 9.5mm			
Quality of fines			
Sand equivalent, or	3.6		
Clay index, or	3.5		
Plasticity index	3.4		
Wet sieving test	3.8.1		
Test sieve aperture		AP40 AP20	AP40 AP20
37.5mm 26.5mm 19mm 9.5mm 4.75mm 2.36mm 1.18mm 600μm 300μm 150μm 75μm			
Particle size distribution shape			
Fractions		AP40 AP20	AP40 AP20
19.0mm – 4.75mm 9.5mm – 2.36mm 4.75mm – 1.18mm 2.36mm – 600μm 1.18mm – 300μm 600μm – 150μm			
Traffic loading limit		Less than 1 x 10^6 ESA	

NZS 4407:1991 Test name	Test no.	Wellington 1 greywacke	NZ M/4-AP30, AP50 (any region)
Weathering quality index	3.11		
Crushing resistance	3.1		
California bearing ratio	3.15		
Broken face content greater than two	3.14		
Sieve size			
19mm – 37.5mm 9.5mm – 19.0mm 4.75mm – 9.5mm		Not less than 60% Not less than 60% Not less than 60%	
Quality of fines			
Sand equivalent, or	3.6	Not less than 30	
Clay index, or	3.5	If sand equivalent is less than 30	
Plasticity index	3.4	If sand equivalent is less than 30	
Wet sieving test	3.8.1		
Test sieve aperture		AP40 AP20	AP30
37.5mm 26.5mm 19mm 9.5mm 4.75mm 2.36mm 1.18mm 600µm 300µm 150µm 75µm		100 – 95 58 – 85 30 – 65 15 – 45 10 – 35 8 – 25 5 – 20 3 – 15 0 – 10 0 – 8	100 83 – 91 49 – 71 31 – 55 21 – 43 13 – 32 7 – 23 4 -15 0 – 11 0 – 7
Particle size distribution shape control			
Fractions		AP40 AP20	AP30
19.0mm – 4.75mm 9.5mm – 2.36mm 4.75mm – 1.18mm 2.36mm – 600µm 1.18mm – 300µm 600µm – 150µm			Standard (table 3)
Traffic loading limit			

Recycled crushed concrete (RCC)	
[2,3]**Definition**	**RCC** is **recycled crushed concrete** composed of rock fragments coated with cement with or without sands and/or filler, produced in a controlled manner to close tolerances of grading and minimum foreign material content. RCC fragments shall consist of clean, hard, durable, angular fragments of concrete. A basecourse is the upper 150mm layer in the pavement, while the sub-base is below the basecourse layer. Subbases shall conform to the requirements of NZTA M/3 notes, the foreign material contents listed below and the project specific specification. Variations to the following limits are possible should the material meet the requirements of TNZ M22, accepted by the NZ Transport Agency. **It must be approved for use by the appropriate regional council.**
[2]**Foreign material**	The percentages of foreign materials shall be determined by RTA test method T276. The percentages of foreign materials shall not exceed the following percentages by mass: Type 1 materials: glass, brick, stone, ceramics and asphalt < 3%; Type II materials: plaster, clay lumps and other friable material: < 1%; Type III materials: rubber, plastic, bitumen, paper, wood and other vegetable or decomposable matter: < 0.5% No Type II or III materials may be retained on the 37.5mm or above sieves for RCC basecourse materials. In no circumstances shall the RCC product contain any asbestos or asbestos fibre. Testing for foreign materials shall be at the minimum sampling rate for production property tests.

NZS 4407: 1991 Test name	Test no.	RCC 130 kN basecourse	RCC 110 kN basecourse
Weathering quality index	3.11	(N/A)	
Crushing resistance	3.1	Not less than 130 ~~Kn~~ kN	Not less than 110kN
California bearing ratio	3.15	Not less than 80%	Not less than 80%
Broken face content greater than 2	3.14		
Sieve size			
19mm - 37.5mm 9.5mm - 19.0mm 4.75mm - 9.5mm		Not less than 70% Not less than 70% Not less than 70%	Not less than 70% Not less than 70% Not less than 70%
Quality of fines			
Sand equivalent, or	3.6	(N/A)	
Clay index[2], or	3.5	(N/A)	
Plasticity index[2]	3.4	Not greater than 5	Not greater than 5
Wet sieving test	3.8.1		
Test sieve aperture		AP40	AP40
75mm 63mm		100 100	100 100

NZS 4407: 1991 Test name	Test no.	RCC 130 kN basecourse	RCC 110 kN basecourse
37.5mm		98 - 100	98 - 100
19mm		76 - 94	76 - 94
9.5mm		57 - 75	57 - 75
4.75mm		38 - 58	38 - 58
2.36mm		27 - 47	27 - 47
1.18mm		19 - 39	19 - 39
600µm		12 - 32	12 - 32
300µm		6 - 26	6 - 26
150µm		0 - 22	0 - 22
75µm		0 - 14	0 - 14
Particle size distribution shape			
Fractions		AP40	AP40
37.5mm - 9.5mm			
19.0mm - 4.75mm		27 - 47	27 - 47
9.5mm - 2.36mm		17 - 41	17 - 41
4.75mm - 1.18mm		8 - 30	8 - 30
2.36mm - 600µm		6 - 24	6 - 24
1.18mm - 300µm		5 - 21	5 - 21
600µm - 150µm		3 - 19	3 - 19
Traffic loading limit			100,000 ESA

Note: N/A = not applicable and test is not required

1 RCC is generally non plastic as cement dust reacts with any plastic fines present.

2 These requirements for RCC were based on Transport South Australia's Pavement material specification part 215.

3 RCC shows comparable performance to high quality M4 aggregate as proven at ~~Transit~~ NZTA's accelerated pavement testing facility CAPTIF.

Special considerations

Stockpiles of RCC should be separated (a minimum distance) from water courses because of the alkaline nature of RCC leachate.

Where RCC aggregates are used in granular basecourse applications in conjunction with subdrains, the following procedures are recommended to reduce the likelihood of leachate precipitates clogging the drainage system:

- Wash the processed RCC aggregates to remove dust from the coarse particles.
- Ensure that any geotextile fabric surrounding the drainage trenches (containing the subdrains) does not intersect the drainage path from the base course, ie do not fully wrap drains (to avoid potential plugging with fines).

The pH value of the RCC aggregate can exceed a pH value of 11. This can be corrosive to galvanised or aluminum pipes placed in direct contact with the RCC. Galvanised or aluminum pipes shall not be used in RCC pavements.

<table>
<tr><th>NZS 4407: 1991 Test name</th><th>Test no.</th><th>Glenbrook melter slag</th></tr>
<tr><td>Definition</td><td colspan="2">Glenbrook melter slag is a co-product of the iron making operation at NZ Steel, Glenbrook. The material is processed by 'SteelServ' to produce an AP40 aggregate complying with the standard TNZ M4 requirements.
It must be approved for use by the appropriate regional council.</td></tr>
<tr><td>Chemical analysis</td><td colspan="2">To ensure a consistent product, the acceptable ranges of the individual relative proportions of slag are:
| Min % | Max %
Cao 10 20
Fe 0 10
SiO2 9 15
Al2O3 15 21
MnO 0.5 1.7
MgO 11 15
TiO2 27 42
Cr2O3 0.2 0.6
V2O5 0.1 0.5
Note: The Fe content is removed from the slag during the crushing process
The non-ferrous component of every production batch of sub-base and base course slag shall be analysed in a IANZ accredited laboratory for its chemical properties and at an interval of six months or 10,000m3 of production (whichever occurs first), for the source properties, so as to assure the ~~Transit~~ NZTA that the slag remains within the parameters specified.</td></tr>
<tr><td>Potential expansion of aggregates from hydration reactions (preformed as a source test)</td><td>EN 1744-1:1998</td><td>Not greater than 0.5% at seven days</td></tr>
<tr><td>Weathering quality index</td><td>3.11</td><td>>BB</td></tr>
<tr><td>Crushing resistance</td><td>3.1</td><td>Not less than 130kN</td></tr>
<tr><td>Other</td><td colspan="2">As per TNZ M4</td></tr>
<tr><td>Traffic loading limit</td><td></td><td></td></tr>
</table>

Special considerations

Stockpiles should be separated (a minimum distance) from water courses because of the alkaline nature of leachate.

Steel slag aggreate are known to potentially clog geotextile fabric wrapped drains, the reduced amount of free lime in melter slag should reduce this risk. Where melter slag aggregates are used in granular basecourse applications in conjunction with subdrains, the following procedure is required:

- Ensure that any geotextile fabric surrounding the drainage trenches (containing the subdrains) does not intersect the drainage path from the base course, ie do not fully wrap drains (to avoid potential plugging with fines).

The pH value of the melter slag aggregate generally ranges from approximately 8 to 10 in laboratory testing and 7.5-8 in the field; however, leachate from blast furnance and steel slags are often in these ranges and can exceed a pH value of 11. This can be corrosive to galvanised or aluminum pipes placed in direct contact with the slag. With this in mind galvanized or aluminum pipes shall not be used in melter slag aggregate pavements.

While melter slags have reportedly good test results in terms of potential to swell. The use of slag aggregate next to structures (such as bridge abutments) is not permitted.

<table>
<tr><th colspan="3">Aggregate/reclaimed glass blended basecourse</th></tr>
<tr><td colspan="3">Definition</td></tr>
<tr><td colspan="3">Overseas experience suggests that appropriately processed reclaimed glass is well suited for use as a basecourse aggregate. Adding glass to aggregate, in suitable proportions, provides a number of environmental benefits without compromising the mechanical properties of the aggregate.
This extension of the M/4 specification allows up to 5% reclaimed glass (by mass) to be blended with natural or recycled aggregate for road base construction. The aggregate/ reclaimed glass (cullet) blend must comply with the requirements of the M/4 specification except for the variations and additions provided in this table.
Up to 5% reclaimed glass can also be added to subbase aggregate in accordance with the relevant requirements of the M/4 specification.
Proportions of cullet in excess of 5% may be used at the discretion of the ~~Transit~~ NZTA Engineering Policy Manager, provided that the requirements of the M/22 specification have been satisfied. Such applications are likely to be restricted to relatively low traffic volume projects and the material may be subject to higher standards with respect to contamination limits.</td></tr>
<tr><td colspan="3">Cullet properties</td></tr>
<tr><td>Reclaimed glass source</td><td colspan="2">The cullet can originate from a number of glass products, viz: waste food and beverage containers, drinking glasses, window glass, or plain ceramic or china dinnerware. Reclaimed glass from hazardous waste containers, light bulbs, vehicle windscreens, fluorescent tubes or cathode ray tubes shall not be used.</td></tr>
<tr><td rowspan="9">Grading
*Provided the combined grading meets M/4, alternative cullet gradings may be submitted to the NZTA Policy Manager for consideration</td><td colspan="2">The cullet shall be crushed to achieve the following gradation: (NZS 4407:1991 test 3.8.1)</td></tr>
<tr><td>Sieve</td><td>Percent passing*</td></tr>
<tr><td>9.5 mm</td><td>100</td></tr>
<tr><td>4.75 mm</td><td>70 - 100</td></tr>
<tr><td>2.36 mm</td><td>35 - 88</td></tr>
<tr><td>1.18 mm</td><td>15 - 45</td></tr>
<tr><td>0.30 mm</td><td>4 - 12</td></tr>
<tr><td>0.075 mm</td><td>0 - 5</td></tr>
<tr><td colspan="2">The plus 4.75mm component of the cullet must not contain more than 1% of flat or elongated particles, ie particles with a maximum to minimum dimension ratio greater than 5:1. The ASTM D 4791 test is appropriate (except that the test sample shall be taken as the material retained on the 4.75mm sieve).</td></tr>
<tr><td>Contamination limit</td><td colspan="2">Debris, such as paper, foil, plastic, metal, cork, food residue, organic matter, etc can have a significant influence on the performance of the aggregate/glass material.
The cullet shall not contain more than 5% debris, as determined using the procedure described in RTA test method T267 (where ‘reclaimed glass’ is substituted for ‘recycled concrete’).</td></tr>
<tr><td>Cleanliness</td><td colspan="2">The cullet shall be washed to ensure that undesirable odours are eliminated.</td></tr>
<tr><td colspan="3">Production</td></tr>
<tr><td colspan="3">Concentrations of reclaimed glass within the aggregate could have a detrimental influence on the performance of the material in a basecourse layer. Therefore, the aggregate and reclaimed glass shall be mixed thoroughly to ensure that there is an even distribution of glass throughout the basecourse stockpile.</td></tr>
</table>

<table>
<tr><td colspan="3">Cullet quality assurance test frequency</td></tr>
<tr><td colspan="3">Tests for compliance with grading, particle shape and contamination shall be carried out at a frequency of two tests (each) per cullet stockpile.</td></tr>
<tr><td>Additional production testing</td><td colspan="2">As per TNZ M/4</td></tr>
<tr><td>Traffic loading limit</td><td>N/A</td><td></td></tr>
</table>

[NZTA logo]

Proposed NZTA M/4 notes: 2011 Draft following industry comment (not current NZ Transport Agency (NZTA) policy)

Notes to the specification for basecourse aggregate

These notes are for the guidance of supervising offices and must not be included in the contract documents.

1 Scope

~~T~~NZTA M/4 is the reference or standard specification for primary basecourse for heavy duty use in flexible pavements with thin surfacings.

Use of the term 'basecourse' should be restricted to only select quality material suitable for the uppermost granular layer adjacent to the surfacing.

Lower layers, where performance requirements allow a lesser quality material, should be referred to and specified as 'subbase'. Given the variation in materials available from place to place for subbase use, there is no standard specification, but M/3 notes set out the recommended procedure for local specification development with examples to illustrate preferred format.

The requirements of M/4, if all just satisfied, produce an acceptable material for nearly all heavy duty flexible pavements. However, where above minimum quality (eg stone quality), less severe service conditions (eg loading or drainage) occur, or where M/4 materials have resulted in poor performance, alternative specifications may well be in order. For assurance with such variants two prerequisites are required:

1 compensating properties or loadings

2 demonstrated (or inferable) performance.

~~(iii) obtain approval~~Obtain approval to use an alternative material from the ~~Transit~~ NZTA's Engineering Policy Manager by conducting agreed tests to prove the suitability of the material.

For the ~~Transit New Zealand~~ NZTA on state highways, the series of approved variants are given in table 4 of ~~TNZ~~ the M/4 specification. They include uncrushed and part-crushed river source basecourse, and some variants based on rock type.

Roading contract documents should include a requirement that the contractor must state the source(s) of his aggregate. The material should have a history of good performance for the proposed design traffic and environmental conditions.

The ~~Transit~~NZTA's Engineering Policy Manager is David Alabaster at the NZTA's Christchurch Office, email: david.alabaster@nzta.govt.nz

2 Testing

Section 2 requires that each individual sample must meet the specification. Before the contractor can be brought to task for failing to meet the specification it is important to ensure that testing is carried out strictly according to the standard methods specified. Care should be taken when sampling to ensure that the sample is representative of a significant amount of material.

All the tests specified should be carried out for acceptance testing.

Acceptance testing comprises both source material testing and production (quality control testing).

- Source material testing – tests such as crushing resistance, weathering resistance and to a certain extent sand equivalent indicate basic inherent properties of the rock.
- Production (quality control) testing – tests such as particle-size distribution, proportion of broken rock and sand equivalent indicate how the production process, and variations of it, affects the product.

When large amounts of material are being supplied the following procedure should be adopted:

- Source material testing establishes the basic rock properties.
- Production testing is then regularly carried out to monitor the product.
- Source material testing is occasionally carried out to check the material properties, in particular when a change in material or properties is suspected (eg when a seam containing clay is encountered).

The size of the representative sample required depends on what tests are actually going to be carried out. Therefore constant liaison between the testing laboratory and the officer doing the sampling is necessary. (Preferably someone from the testing laboratory should do the sampling but this is often impractical). ‘When’ samples are being taken, it should be understood ‘why’ they are being taken.

In general the material used for the particle-size distribution test can subsequently be used in other tests.

Several tests require test samples which comprise of material in specific particle-size ranges. Hence the mass of a representative sample from which a given mass of test sample can be obtained is dependent on the grading of the representative sample.

The following indicates sample sizes required for particular tests:

Table 1 Sample sizes and distribution

Test	M/4 grading	Minimum test sample mass (kg)	Representative sample (quartered etc to obtain test sample) (kg)
Particle-size distribution NZS 4407: 1991 test 3.8.1	AP40	10	40
Particle-size distribution NZS 4407: 1991 test 3.8.1	AP20	5	20

Table 2 Sample sizes

Test	Particle-size range for test sample (mm)	Minimum test sample mass (kg)	Representative sample (sieved to obtain test sample) (kg)
Sand equivalent NZS 4407: 1991 test 3.6	passing 4.75	0.5	* approx 2
Crushing resistance NZS 4407: 1991 test 3.10	13.2 - 9.5	8	* approx 50
Weathering resistance NZS 4407: 1991 test 3.11	19.0 - 9.5 9.5 - 4.75	2 3	* approx 30

* These approximate figures assume the representative sample has an M/4 grading.

3 Proportion of broken rock

The specified proportion, for each size fraction, is designed to define ~~materials~~ aggregates that will have properties that effectively match a quarried product.

4 Crushing resistance

This test indicates the ease of processing (strength) of the aggregate and the likelihood of attrition. Although materials which have a crushing resistance less than 130kN cannot be classified as M/4 basecourse this does not necessarily preclude their use where stronger aggregates are not available, depending on the intended equivalent standard axle (ESA) loading conditions and environment. The use of these materials however will require other specification changes, representing a significant departure from the ~~Transit~~ NZTA standard pavement design procedure. Conversely experience with local aggregates may show that a minimum crushing resistance greater than 130kN is necessary to ensure a material with adequate strength.

As part of ongoing research, tentative guidelines for estimating the design traffic (ESA) capability of aggregates, which may have inferior (or superior) parameters relative to the standard M/4, are given in *NZTA research report 459*. The relevant criteria are figures 5.9, 5.10 and 5.11 in that report.

If the aggregate includes blended fines from a separate source, this shall be stated with M/4 results. Because there will be no applicable crushing value for this fraction, good precedent field performance in comparable conditions must be established or some other appropriate verification testing undertaken.

5 Weathering resistance

This test is an accelerated laboratory test to assess the resistance of aggregate to the combined agencies of wetting and drying, and heating and cooling. Thus it is some measure of soundness and durability.

It is recognised that the test is far from ideal for ensuring the durability of an aggregate in service, and the meaning of the results is far from certain. It is however the best of many inadequate tests and will have to serve till something better is developed.

6 Sand equivalent

This test measures the relative amounts of silt or clay-size particles in granular soils. Thus it indicates cleanness.

Although the sand equivalent test was originally established as an alternative to ~~the~~ plasticity index test the two are not strictly comparable. The deleterious effect of the presence of clay fractions is to seriously reduce the aggregate's permeability and increase its susceptibility to stability destroying pore pressures under dynamic loading. The fines result in an increased long-term equilibrium water content (and hence increased degree of saturation). The object of the specification is to control the proportion of such ultrafine material in the fine aggregate and the sand equivalent test does this satisfactorily.

7 Grading

An alternative grading has been added to the specification. This grading AP20 (all passing 20mm) has a topsize of 19mm and is intended for use in thin granular overlays.

Non structural, shape restoring overlays require the minimum depth of metal over high spots with sufficient depth over low spots to provide an acceptable riding surface. The minimum depth of metal over high spots will depend primarily on the maximum particle size of the material and considerable savings are possible if the maximum size is 19mm instead of the standard 37.5mm.

Limits on use of AP20 basecourse:

- Not less than 40mm over high spots – to ensure satisfactory workability and compactibility.
- Not more than 125mm depth in depressions – to ensure adequate stability.
- In some areas AP20 basecourse may cost significantly more per cubic metre than AP40 basecourse. In such cases a decision on which material to use should be based on the overall economics, bearing in mind that a smaller quantity will be required of the AP20 material.

The AP40 and AP20 nomenclature is consistent with the recommendations for aggregate naming of the Aggregates and Quarry Association of New Zealand. AP40 and AP20 refer only to the size of the material and not the quality. To specify these materials TNZ M/4 AP40 and TNZ M/4 AP20 must be quoted.

With the gradings shown it will normally be unnecessary to choke the basecourse surface with additional fines. However, where an AP40 aggregate has a much higher than minimum crushing resistance, the standard grading, which includes provision for some gradation change during compaction, may accept materials that appear deficient in fines during construction. In such a case it is proper to vary the specified grading by adding an amending clause to the job specification. A suitable such clause is:

'For basecourse produced from 'named material type' the gradation requirements shall be varied for the specific sieve apertures tabulated below.'

Test sieve Aperture	percent passing (AP40)
300µm	5 – 14
150µm	3 – 10
75µm	2 – 7

This change effectively bars materials with nothing finer than 300µm.

Such a provision is not necessary with AP20 aggregate as it is used in thin layers (less breakdown under compaction) and its grading already includes adequate fine material.

An alternative AP30 is also shown as local variant for AP40 where obtaining sufficient broken faces is costly in alluvial sources.

Grading shape can be expressed by Talbot's 'n' value. The 'incremental n' value (n_{1-2}) may be calculated over the range of any two sieves sizes (1, 2) and is defined as:

$$\textit{Incremental grading exponent } n_{1\text{-}2} = Log_{10}(P_1/P_2) / Log_{10}(d_1/d_2)$$

where:

- P is percentage passing the specified sieve sizes.
- d is the chosen sieve size in any consistent units.

For example if the fractions passing the 4.75mm and 0.03mm sieve sizes are 44% and 14% respectively then the incremental grading exponent, $n_{4.75\text{-}0.30}$ is:

$$n_{4.75\text{-}0.30} = Log_{10}(44/14) / Log_{10}(4.75/0.30) \quad = 0.41$$

~~The g~~Grading shape control sets limits to local deviations from the general shape of the grading envelope, and ensures a well graded product. Poorly graded aggregates, especially those that are gap graded in the sand fraction feature frequently in cases of premature distress. A simple criterion that has been found to be associated with virtually all cases of shear instability in basecourses which are otherwise compliant with M/4 is where the incremental grading exponent (n) is less than 0.40 over the sand range, ie between 0.150 and 4.75mm sizes. Four 'governing' sieve ranges for calculation of n, have been determined by regression. If any two of these are less than 0.40, or are likely to become so after compaction, the risk of premature basecourse distress tends to increase. For this reason an additional fines criterion has been developed, termed the 'sand grading exponent' (SGE). The SGE is defined as the average of the two lowest incremental grading exponents from a set of four sieve size ranges, namely 4.75–0.30mm, 2.36–0.15mm, 1.18–0.15mm and 0.60–0.15mm.

A simple functional calculator is available at the following link: www.pavementanalysis.com/files/SGE_Calculator.xls and is illustrated in table 3 below. This will identify whether any given grading curve will meet the proposed tighter grading shape control limits. Note: only the second column (yellow cells) should be modified to determine a pass or otherwise.

Table 3 Calculator for SGE giving suitability of particle size distribution for base course

Sieve size (mm)	% Passing	Critical Range	Grading Exponent
4.75	44	4.75 mm - 0.30mm	0.41
2.36	33	2.36 mm - 0.15 mm	0.43
1.18	25	1.18 mm - 0.15 mm	0.44
0.6	19	0.60 mm - 0.15 mm	0.46
0.3	14	**SGE (Ave. of 2 Lowest)**	**Pass/Uncertain**
0.15	10	0.42	**Pass**

8 Basis of measurement and payment

Although the specification stipulates that the issue of a cartage docket is not an acceptance that the material supplied complies with specification requirements, the tallyman should be supplied with typical quality control samples so as to avoid as much as possible the need to condemn substandard aggregate after it has been delivered. However, it should also be emphasised that it is not proper to expect a tallyman to carry substantial authority in this matter, and constant liaison will often be necessary.

M/4 is written on the basis of payment by volume at the point of delivery.

It may be more convenient to supply the aggregate by weight by using TNZ G/2 'Conditions for supply of aggregate by weight'.

G/2 applies when the supply of aggregate is a separate contract item and the quantity for payment purposes is specified as the loose volume in delivery trucks. Payment will be made on the volume delivered to the delivery point but the purchaser may agree to the use of a mass per unit volume conversion factor to give delivered volume from certified weights.

Note: Aggregate and Quarry Association recommends that compaction testing should be carried out in conjunction with M/4 testing. This should also include a solid density result or current reference to a recent solid density test (within three months).

Appendix C: Draft amended M/3 subbase notes

[NZTA logo]

These notes must not be included in the contract documents.

1 Introduction

There is no standard ~~TNZ~~NZTA M/3 specification for subbase aggregate. The variety of materials that will serve as satisfactory subbases is too wide to set a rational standard specification and custom written specifications must be used that match the design of each individual project. These notes set out the recommended format of the subbase specification, lay down certain minimum requirements and identify other requirements that may be necessary.

Notwithstanding these guidelines, optimal material utilisation, particularly involving stabilised materials, may require development of specifications beyond the scope of these notes. NZ Transport Agency (NZTA~~Transit New Zealand~~) approval of such specifications shall be sought prior to their use.

The term 'subbase' means that material used in the pavement between the subgrade and the M/4 'basecourse'. As ~~they are~~the subbase is located well below the zone of intense wheel load induced stresses and strains and because ~~they are~~ it is confined by a superimposed layer of basecourse, subbase materials do not have to meet the stringent requirements of the M/4 basecourse specification.

These notes can be used in the development of a specification for use in conjunction with either the supply of subbase material or the supply and construction of a subbase layer. In thick pavements, more cost-effective design may result if an 'upper' subbase layer is used along with one or more lower layers. The upper subbase will require, in particular, a high permeability.

2 General

Materials must be free from all non-mineral matter. Crushed recycled glass up to 5% may be added subject to the same criteria for the cullet given in M/4. Proportions greater than 5% require NZTA approval.

3 Testing

~~A clause similar to that in TNZ M/4 should be included stating s~~Sampling method, sources, sizes etc (refer to section 8).

The place of sampling should be clearly identified in the specification, eg, quarry belt, stockpile or truck; from uncompacted pavement ~~or~~ or from constructed pavement. Sampling from the constructed pavement is ~~not~~ recommended ~~unlessif~~ if there is no information on precedent performance of the source, if particularly nondurable materials are likely to be supplied, or if the site is in a frost prone region. Sampling procedures are given in NZS 4407:1991, section 2.

If the subbase is potentially very variable it is worthwhile to require that a control sample be supplied to the site to which incoming subbase can be visually compared. The volume of the control sample should be at least 4m^3 and must be kept covered and protected from contamination throughout the period of the contract. As visual assessment can be contentious a higher frequency of quality control testing may be a preferable approach if a subbase is particularly variable.

4 Broken rock

It should not be necessary to require that the material be crushed. However, if the particle size distribution is coarse then contractors will need to consider whether a moderate percentage of crushed stone is necessary for some sources in order for them to be successfully compacted to comply with TNZ B/2 requirements or to provide stability in moderately steep shoulders. Unless performance of a particular subbase has been proven, a minimum 30% broken faces on the plus 19mm fractions may need to be specified, for the uppermost layer of the subbase, ie that zone that is required to meet permeability requirements as stated below. An exception may be where a source is already deficient in the coarser gravel sizes and increasing the percentage of broken faces is likely to result in excessive reduction of the effective maximum stone size.

5 Grading

5.1 Minimum requirements

Maximum size must not be greater than 0.75 x (compacted layer thickness).

Maximum size must not be greater than 100mm (or 106mm standard sieve).

5.2 ~~Envelopes~~ Grading envelopes

~~There is no standard grading envelope for subbases and it is intended that grading be left as flexible as possible. Consideration should however be given to specifying some grading requirements to ensure that compactability is not too difficult and that reasonable densities can be obtained. Savings made in buying a material without any grading controls can easily be lost in the additional construction costs necessary to achieve adequate densities and surface shape. Laying and compaction considerations may result in a maximum particle size smaller than 0.75 x (compacted layer depth).~~

For reasonable workability and density the general form of the M/4 grading envelope should be followed. This can be achieved by expressing the particle size to be controlled as a proportion of the topsize and applying the M/4 controls for the particle size of equal proportion. Control is best applied to the fraction passing ¼ ~~I/A~~ topsize. For example, assuming a 75mm top size, control would be effected at 19mm since 19/75 = 0.25.

The equivalent M/4 fraction is 9.5mm (37.5 x 0.25) with envelope limits of 43 and 57%. Control of subbase gradings can be somewhat looser than for the M/4 material ~~so that~~ and the upper subbase specification ~~would~~ should require between say 40% and 65% passing the 19mm sieve.

These requirements can be more readily appreciated if the particle size ~~distribution requirements are is~~ grading envelope is presented on ~~a~~ a log/log ~~paper~~scale graph where well graded materials plot as straight lines. The slope of the straight line particle size distribution describes the coarseness or denseness of the grading. The M/4 grading envelope is bounded by lines of slope 0.41 and 0.63. Limits described by lines of slopes 0.40 and 0.~~70~~ 65 are appropriate for upper subbases, and can be transferred to other topsizes by shifting the upper and lower bounds as pairs of parallel lines on the log/log plot. Limits of 0.35 and 0.70 are appropriate for lower subbases.

~~5.2~~5.3 Grading shape control

It is important to keep the slope of the particle size distribution as straight as practicable, rather than swapping across between the extremes of the envelope limits, as this i) ensures adequate permeability, ii) limits the potential for segregation and iii) promotes ease of laying and compacting.

Grading shape control can be applied in a similar manner to basecourses by requiring that the incremental grading exponent between every second sieve size should also lie within the limits of 0.40 and 0.65 for upper subbases. The incremental grading exponent concept is a quantitative way of ensuring that a material will be neither gap-graded nor dominated by a single size. Further detail is given in section 7 and in an accompanying spreadsheet. For lower subbases the corresponding limits should be 0.30 and 0.75. Grading shape control need not be applied for sieve sizes coarser than 4.75mm.

Laying and compaction considerations may result in a maximum particle size smaller than 0.75 x (compacted layer depth).

It is important that water is not trapped in depressions on top of any subbase layer. Subbase material should therefore be capable of being shaped to provide an even crossfall without birdbaths. This requirement can be relaxed if the material is very open and is unlikely to hold water, or if the layer immediately above is less permeable.

~~5.3 Free Draining Requirements~~

~~5.3 The subbase material immediately beneath the basecourse must meet certain permeability requirements. See section 7 (permeability) for details.~~

5.4 Subgrade compatibility

At the subbase/subgrade interface the two materials should be compatible to prevent the intrusion of fine subgrade particles into the subbase which could in turn reduce the subbase CBR below the value assessed in the pavement design, as well as reducing its permeability.

To ensure compatibility a layer at least 75mm thick should meet recognised~~the following~~ requirements for 'filter compatibility' between the subbase and subgrade. By limiting the d15 size of the lower subbase aggregate to less than five times the d85 size of the subgrade, the larger soil particles of the subgrade will be retained, allowing the soil bridging action to start. By limiting the d50 size of the aggregate separator layer to less than 25 times the d50 size of the subgrade, the gradation curves will be kept in balance:

d_{15} (subbase) < 5 d_{85} (subgrade)

d_{50} (subbase) < 25 d_{50} (subgrade)

or ~~(~~for subgrades of medium and high plasticity clays~~)~~

d_{15} (subbase) < 5

d_{85} (subgrade)

d_{60} (subbase) < 20

d_{10} (subgrade)

where d_{15} is the sieve size 15% of the material passes.

The same compatibility criteria should be applied between all unbound layers in the pavement (eg between the lower and upper subbase).

Designers will need to document the basis for any non-compliance with compatibility criteria (and confirm response with site trials or inspection of test pits within the shoulder). It should be noted that where soaked CBR values are less than 4, particularly in low plasticity silty subgrades, intrusion of many hundred millimetres can develop during construction as compaction water for the subbase is applied. In some cases subgrade stabilisation (lime or cement) can greatly minimise the risk of intrusion. The over-riding considerations are likely to be the amount of intrusion that takes place as water is applied during compaction of the aggregate layers and the level of assurance that the subgrade will remain in an unsaturated condition post-construction.

If compatibility cannot be economically achieved, a geotextile should be considered (NZTA F/7 Geotextiles specification and notes).

6 Bearing strength

The Austroads pavement design method uses the subbase CBR to determine the cover required over any layer. It assumes that at least 90% of the material will have CBRs in excess of the design value. Note however that some NZTA contracts seek 95% reliability for their pavement designs.

6.1 CBR test

CBR tests shall be made on randomly selected samples tested and soaked according to ~~NZS 4407:1991,~~ NZS 4407:1991, test 3.1.~~ NZS 4402 Part 2P: 1981 Test 18(A)~~5, except that CBR values shall be reported to the nearest 1 in all cases thus modifying the requirement of clause 3.15.7.1(h) of the specification. The test samples should be surcharged to simulate the cover over the subbase in the pavement.

~~For the acceptance or rejection of an isolated unit of material one CBR test shall be performed, and the item accepted provided the CBR value is not less than the design value.~~

~~For the checking of larger quantities or areas the size of the lot (pavement area or unit of production) to be accepted or rejected shall be defined by the Engineer, and three to five tests carried out.~~

~~The size of the lot will be based on the Engineer's assessment of a lot which has homogeneous characteristics.~~CBR testing frequency is given in section 8.

The design CBR shall be assessed by the following traditional method where CBR is assumed to have Gaussian distribution, otherwise the percentile method should be used:

Traditional method

Let c = a CBR value

C = mean value of CBRs

C_d = design CBR

n = sample size

s = standard deviation

ie s = square root ((sum of $(c-C)^2$)/(n-l))

k = acceptability constant.

Then the lot will be accepted provided C is not less than C_d + ks, in which k is given in the following table:

Table 1 Acceptability constants

n	k
3	1.3
4	1.2
5	1.1

This procedure gives for all sample sizes a 10% consumer (engineer's) risk of accepting a situation in which 37% of CBR's values are below the design value. The producer (contractor's) risk that an acceptable situation (just 10% of CBRs below design limit) be faulted varies from 42% for n = 3 to 33% for n = 5. This last fact shows the benefit of a larger sample size.

It should be noted that the acceptance criterion described above can be used to select a revised design CBR appropriate to the sample data. The revised design CBR = C – ks.

6.1.1 Percentile method

The percentile method comes from the distribution of CBR typically found in laboratory CBR testing of a single material source. If there are insufficient tests on the material under consideration to determine a cumulative distribution then (from inspection of data collected on NZTA's LTPP sites) for granular materials:

$$\textit{Design CBR} = 0.85 * \textit{Mean CBR}$$

(Note cohesive soils are often much more variable).

The CBR test samples should be compacted to densities and at ~~moisture~~ water contents approximating those applying during construction. This is especially important when the sand equivalent of the material is less than 40 (when tested according to NZS ~~4402 Part 1:1980 Test 7)~~4407:1991, test 3.6 and there is more than 10% passing the 425μm sieve.

6.2 Waiving of CBR test

The CBR test is performed only on that fraction passing the 19mm sieve. If there is not sufficient of this fraction to fill the holes between the larger sized particles the CBR of the passing 19mm material will have little effect on the stability of the subbase. This condition will apply for material with a topsize of 37.5mm or less when the weight passing the 19mm sieve is *55%* or less, and for material with a topsize between 37.5mm and 106mm when the weight passing the 10mm sieve is 25% or less.

In such cases and where the crushing resistance is greater than 110kN (when tested in accordance with NZS 4407:1991, test 3.10, then the CBR test can be waived and the material can be assumed to have a CBR in excess of 40.

6.3 Crushing resistance

A granular subbase material with a crushing resistance of less than 110 may be incorporated in a pavement only if it is located below the depth (as given in the Austroads design chart for depths of unbound granular pavements) for the following corresponding CBR values:

Crushing resistance (kN)	Corresponding CBR
110	30
80	15
70	10
60	8

6.4 Target density and optimum water content

If the topsize of the whole soil exceeds the size fraction used for determination of the laboratory compacted maximum density (19mm for standard compaction or 37.5mm for the vibrating hammer test), then it is important to appreciate that the target field density will need to be adjusted to take into account the proportion of material in the whole soil compared with that used in the laboratory mould.

The target field density and optimum water content shall be calculated as follows:

$$\rho d = \frac{100}{\frac{S37.5}{\rho S} + \frac{(100 - S37.5)}{G}}$$

where:

ρ_d is the field target dry density for the whole soil

$S_{37.5}$ is the percent passing the 37.5mm sieve

ρ_S is the maximum laboratory dry density of the fraction

G is the bulk specific gravity of the oversize fraction (in the same units as the dry density)

If there is more than 20% of oversize material, a check of this correction may be obtained by replacing the oversize gravel with an equal weight of particles selected from the largest undersize sieve interval and carrying out a compaction test on this manufactured material.

Where the 19mm size fraction is used in the laboratory compaction, the same form of expression is used after substituting for the 37.5mm terms.

If there is difficulty meeting the calculated target density in the field, then the target dry density may need to be re-determined using an oversize mould (diameter not less than four times the maximum particle size) with the all-in material compacted using the same energy per unit volume, as used in the standard test. Alternatively, field plateau density testing may be carried out. Plateau tests apply particularly on a yielding subgrade as the ‘anvil’ effect of the base of the laboratory compaction mould will no longer be present. However testing must be carried out at optimum water content for valid results.

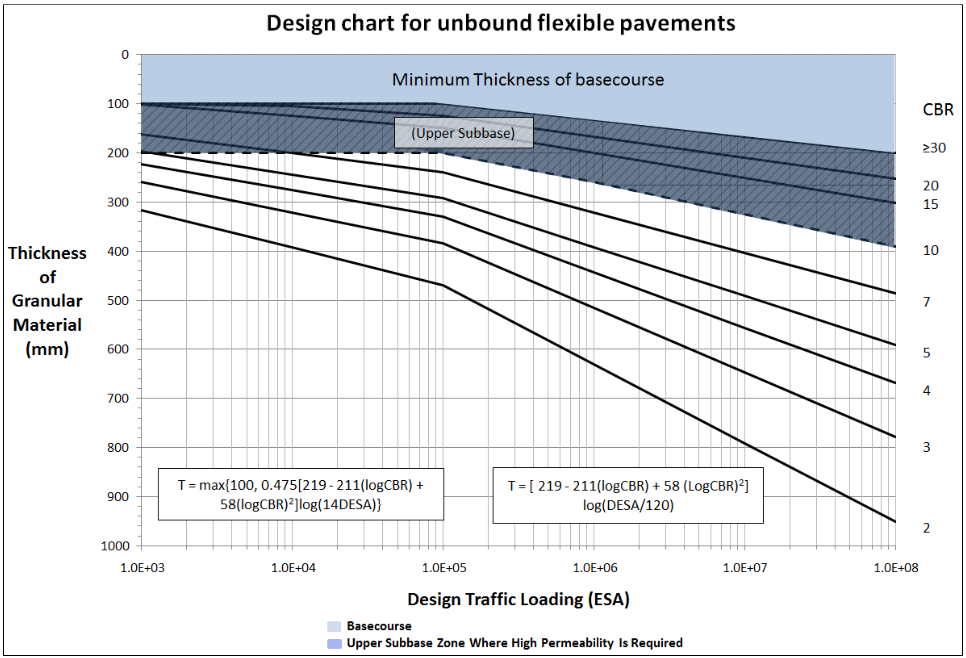
7 Permeability

Subbase material immediately beneath the M/4 basecourse layer must meet three permeability requirements:

1 The drainage channels in the voids of the material must be sufficiently large to prevent the adverse build up of pore pressure when the material is subject to traffic loads. This requirement is best satisfied by specifying a minimum sand equivalent value:

A subbase material ~~hall have a~~ within the ‘upper subbase zone’ shown shaded on figure 1 (between the 10 and 30 CBR range) shall have a sand equivalent greater than 40 ~~or if~~unless there is less than 10% by weight passing the 425µm sieve.

Figure 1 Unbound pavement design



2 To ensure that the M/4 basecourse layer is kept in an unsaturated condition, the percent passing 37.5mm fraction of the upper subbase layer immediately beneath the basecourse must have a permeability of at least 10^{-4}m/s, when compacted to at least 95% of maximum density in a constant head permeability apparatus with cell diameter of a least six times the maximum particle size of the tested material. (For unbound pavements the upper subbase will be at least 100 to 170mm thick according to design loading). A check that the permeability is adequate can be obtained while the maximum density is being determined, by soaking the compacted sample then measuring the degree of saturation after drainage for one hour. This value provides a benchmark for subsequent maximum degree of saturation when testing for construction compliance (B/2.)

3 The ~~isorder of~~required permeability will not be hard to achieve if the weight passing the 150µm sieve in relation to coarser sizes is controlled. This can be achieved by ensuring that the Talbot’s grading exponent (the slope of the particle size distribution on a log-log plot) is greater than 0.4 over the sand sized fraction. The incremental grading exponent n_{1-2} is calculated from:

$$n_{1-2} = \frac{\log_{10}\left(\dfrac{P_1}{P_2}\right)}{\log_{10}\left(\dfrac{d_1}{d_2}\right)}$$

where P_i is the percentage passing sieve size d_i

The incremental grading exponents calculated over the following four sieve sizes ranges have been found to be significant in controlling permeability:

4.75mm – 0.3mm

2.36mm – 0.15mm

1.18mm – 0.15mm

0.6mm – 0.15mm

If the average of the two lowest of these four incremental grading exponents is not less than 0.40 then good permeability should be achieved.

The same criterion is proposed for basecourses (refer M/4 revision in progress).

The~~e~~ ~~is~~ permeability requirement can be waived if:

- a suitable material is unreasonably expensive
- the design loading is less than 1 x 10^5 E~~D~~SA
- there is good precedence for performance of the subject material in a similar environment, and
- the pavement is to be formed in a region which is not prone to flooding or frost.

In frost prone areas, the subbase must satisfy the permeability and the shear stability criteria to at least the depth of expected frost penetration.

If the permeability requirement is waived for the subbase then particular attention should be paid to the basecourse characteristics to ensure a suitably low fines content that will maintain a low degree of saturation both at the time of sealing and long term.

8 Minimum testing requirements

Testing shall be at such a frequency to ensure that the material consistently complies with the specification. The test frequency shall initially not be less than that shown in table 2 except that the test frequency may be halved where the most recent 10 successive test results meet the specification. If any subsequent test result fails, another test shall be immediately undertaken. If the second test fails the test frequency shall revert to the minimum test frequency specified in table 2 and testing shall not return to half the test frequency until a further ten successive test results comply with the specification.

Table 2 Sampling rates for subbase

Subbase	**Sampling rates**
Grading and sand equivalent	Stockpiles < 600 m^3 two samples Stockpiles 600 2000 m^3 three samples Stockpiles >2000 m^3 four samples
Permeability, crushing resistance, laboratory compaction and target compaction with oversize	One per material type
Solid density	One per each source in the last 12 months

CBR (laboratory)	Minimum number of three tests per stockpile
Field compaction testing	One sample for each 500m^3. Minimum three tests Delivered materials.
~~Field~~	~~One sample for each 500m^3. Minimum three tests~~ ~~Delivered materials.~~

~~Guidance : The is to consider the appropriateness of 8.1 and decide on the sampling rates on a case-by-case basis~~

~~Sampling for production acceptance testing shall be carried out not more than months prior to delivery of aggregates to site. This requirement may be waived if traceability is documented with relevant details of stockpile management, to the satisfaction of the .~~

~~Application and examples~~

~~A **flow chart** of the subbase selection process has been developed along with an associated **spreadsheet** that applies the above compatibility criteria to automatically generate acceptable limits for particle size distribution. This uses the subgrade grading, CBR and design ESA as inputs) to generate a full specification for both a lower and upper subbase.~~

~~The spreadsheet is in the testing stage and a web version should soon be available, along with examples of alternatives for upper subbase, lower subbase and transition layer.~~

Guidance note: The consultant is to consider the appropriateness of table 2 and decide on the sampling rates on a case-by-case basis.

Sampling for production acceptance testing shall be carried out not more than three months prior to delivery of aggregates to site. This requirement may be waived if traceability is documented with relevant details of stockpile management, to the satisfaction of the engineer.

9 Application and examples

A **flow chart** of the subbase selection process has been developed along with an associated **spreadsheet** that applies the above compatibility criteria to automatically generate acceptable limits for particle size distribution. This uses the subgrade grading, CBR and design ESA as inputs) to generate a full specification for both a lower and upper subbase.

The spreadsheet is in the testing stage and a web version should soon be available, along with examples of alternatives for upper subbase, lower subbase and transition layer.

Appendix D: AASHTO (2000) and saturation equation

Derivation of effective degree of saturation from first principles.

Symbols

Vw= volume of water between particles

Va= volume of air between particles

Vi= volume of water permeable voids within particles

Vs= volume of solids (excluding water permeable voids within particles)

Ms= mass of solids

Mw= mass of water between particles

Sr= degree of saturation of the inter-particle structure

(The inter-particle structure is nominated because that is the part of the aggregate structure where the ratio of water to air will dictate the magnitude of pore pressure development under dynamic traffic loading, It is assumed that the intra-particle voids remain filled with water.)

All other symbols as in Section 6

By definition:

$Sr=100\ Vw/(Vw+Va)$

$EWC=100\ (Mw/Ms)+A\%$ ie $(EWC-A\%) \times Ms/100 = Mw$

$G_{bd}=Ms/(Vs+Vi)$

$\%SDr=100\ Ms/[Vs+Vw+Va+Vi)]/G_{bd}$ ie $Vw+Va= \{100\ Ms/(G_{bd}x\%SDr)-(Vs+Vi)\}$

$Mw/(Vw)=1$

Substituting:

$Sr= 100x\ Mw/(Vw+Va)$

$SR = (EWC-A\%)x(Ms)/ \{100\ Ms/(G_{bd}x\%SDr)-(Vs+Vi)\}$

$Vw+Va=100\ Ms/(G_{bd}x\%SDr)-(Vs+Vi)$

$Sr= (EWC-A\%)xMs/ \{100Ms /(G_{bd}x\%SDr)-(Ms/G_{bd}) \}$

$Sr= (EWC-A\%)/ \{100 /(G_{bd}x\%SDr\%)-(1/G_{bd}) \}$

$Sr= (EWC-A\%)xG_{bd}/ \{100 /(\%SDr)-(1) \}$

$Sr= (EWC-A\%)xG_{bd}x\ \%SDr/ \{100 - SDr\%) \}$ ......Equation 6.1