Adaptation of the AUSTROADS Pavement Design Guide for New Zealand Conditions

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Contents

Executive summary

New Zealand granular pavement design is currently based on the assumption that all deformation of the pavement shape under traffic loading occurs in the subgrade. To reflect this theoretical behaviour the AUSTROADS document *Pavement Design – A Guide to Structural Design of Road Pavements* (AUSTROADS 1992) is based on limiting the vertical strain on the subgrade. AUSTROADS (1992) was adopted as a design methodology by New Zealand in 1996. In 2004 a newer version of the AUSTROADS guide was issued (AUSTROADS, 2004a) and this document is now the current pavement design guide in New Zealand. Just as with the 1992 document AUSTROADS (2004a) continues with the design methodology of minimising the subgrade strain according to the design traffic.

Observations on the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) and in the field show that repeated vehicle loading produces significant plastic deformation that occurs in the granular base layers as well as in the subgrade. Consequently, design based solely on subgrade strain criteria does not reflect observed pavement performance.

To better understand the application of AUSTROADS (2004a) to New Zealand conditions a review of the available New Zealand and Australia literature was conducted and a summary is presented in this report, which was initiated in 2004. In addition, on the 1st of December 2004 an industry forum was conducted, at which the participants were all acknowledged experts in pavement design. Each participant made a presentation and a summary of these presentations was distributed within the group. Participants were invited to rank the current issues being faced by the pavement community. The discussions at the forum and the issues identified as significant were used to direct the research progress.

The CIRCLY software is used for the mechanistic analysis and design of roads pavements based on the AUSTROADS' design methodology. The damage factor produced by CIRCLY was used to determine the expected pavement life for various pavement designs. These lives were then compared to the predicted lives obtained using a South African pavement software package called mePADs. The mePADS software indicated that pavement layers other than the subgrade were critical for determining the pavement life.

Roughness and rutting are frequently drivers for pavement rehabilitation. The progression of pavement roughness and pavement rutting is examined. In 1998 New Zealand adopted the software platform dTIMS from Deighton Associates for the predictive modelling of pavement deterioration. The basic models used in dTIMS are derived from the Highway Design and Maintenance Standard Series (HDM) models. The roughness model from HDM III indicates that for design traffic levels of 10^5 Equivalent Standard Axles (ESA) AUSTROADS' subgrade strain criteria is highly conservative; for design traffic levels of 10^7 ESA, AUSTROADS is not conservative enough. From another perspective the HDM model indicates that AUSTROADS' pavement thickness is excessive for low traffic of 10^5 ESA while the 10^7 ESA traffic levels require greater aggregate cover. The results for design traffic between 10^5 and 10^6 ESA might help explain the observation that lives greater than 50 years are being achieved in New Zealand (Bailey et al. 2004) since, assuming the modelling is correct, in effect, these roads have been over designed.

Given that the AUSTROADS (2004a) design method only allows for pavement failure in the subgrade it is suggested that the incorporation of the South African design methodology, which utilises empirically determined layer performance models for the unbound granular layers as well as the subgrade, into New Zealand design is worth investigation, particularly given the similarity in the road construction practices of the two countries. Any investigation would require the incorporation of material characterisation from the laboratory, CAPTIF trials, field trials, and would also require the accurate modelling of pavement behaviour.

Most roading data, in New Zealand, are for traffic levels less than 10^6 ESA and there are little data for 10^7 ESA or greater. Therefore, it is recommended that the subgrade strain criteria derived from the HDM roughness model be considered for adoption for traffic levels less than and equal to 10^6 ESA. Prior to any implementation, the validity of the proposed subgrade strain criteria should be determined by examining the performance of actual roads. For traffic levels greater than 10^6 ESA verification is recommended.

From the literature reviewed it appears that the predominant failure indicator is roughness. What are not clear, however, are the physical mechanisms that drive the increase in roughness. Two potential mechanisms are shallow shear and differential settling because of inconsistent compaction. It would be beneficial to determine which of these two, if either, is the dominant mechanism in roughness evolution.

The literature indicates that reduced construction variability is potentially an area where improved pavement performance could be obtained. In the New Zealand context what is unclear is whether pavements are being constructed to a reasonable quality level or if the construction specifications are inadequate. Auditing of quality control during pavement construction should be conducted. If this indicates potential for improving process quality control then there is potential to positively influence pavement construction quality and thereby the achieved pavement life. In doing so the top ranked issue of minimising post-construction rutting, as per the Industry Forum, would likely be addressed. Furthermore, the potential exists for the use of initial high rates of accumulation of permanent deformation to be used as an indicator of inadequate construction quality.

Currently, pavement design in New Zealand is critically dependent on subgrade strain. Both of the subgrade strain criteria proposed by Pidwerbesky et al. (1997) and derived from Salt & Stevens' (2001) statements allow greater, when compared with AUSTROADS, subgrade strains for New Zealand conditions, the latter albeit for volcanic soils only. While the use of these relaxed subgrade strain criteria might be useful in the short term ultimately the linking of plastic strain with resilient modulus should be discontinued. A better characterisation of the plastic behaviour of pavement materials would assist in pavement modelling and ultimately in characterising pavement performance.

Abstract

The AUSTROADS document Pavement Design – A Guide to the Structural Design of Road Pavements does not specifically design for plastic deformation in the basecourse; however, both experiments and field observations demonstrate that, with sufficient traffic loading, plastic deformation accumulates in the basecourse, sub-base, and the subgrade. Furthermore, pavement design in New Zealand is critically dependent on subgrade strength, apparently neglecting the accumulation of plastic strain in the granular layers of the pavement. This study, initiated in 2004, examines the design methodologies presented in AUSTROADS and evaluates them against available New Zealand research. Various subgrade strain criteria are examined for New Zealand conditions. The roughness model from HDM III has been used to generate a pavement design figure similar to 'Figure 8.4' of AUSTROADS. The figure indicates that for lower design traffic levels AUSTROADS is highly conservative; for high design traffic levels AUSTROADS is not conservative enough. The results for design traffic between 10^5 and 10^6 ESA might help explain the observation that lives greater than 50 years are being achieved in New Zealand since, assuming the modelling is correct, in effect, these roads have been over designed.

1 Introduction

1.1 Background

New Zealand granular pavement design is currently based on the assumption that all deformation of the pavement shape under traffic loading occurs in the subgrade. To reflect this theoretical behaviour the AUSTROADS document, *Pavement Design – A Guide to Structural Design of Road Pavements* (AUSTROADS 1992), is based on limiting the vertical strain on the subgrade. AUSTROADS (1992) was adopted as a design methodology by New Zealand in 1996. In 2004 a new version of the AUSTROADS guide was issued (AUSTROADS, 2004a) and this document is now the current pavement design guide in New Zealand. Just as with the 1992 document AUSTROADS (2004a) continues with the design methodology of minimising the subgrade strain according to the design traffic.

Observations on the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) and in the field show that repeated vehicle loading produces significant plastic deformation that occurs in the granular base layers as well as in the subgrade. Consequently, design based solely on subgrade strain criteria does not reflect observed pavement performance.

AUSTROADS (1992, and 2004a) presents an empirically determined relationship between the measured subgrade California Bearing Ratio (CBR), the design traffic, and the pavement thickness. This relationship is displayed graphically and is colloquially known after the figure label, namely as 'Figure 8.4'. Theoretically, pavements designed according to 'Figure 8.4' achieve the reduction in subgrade strain necessary to achieve the design life.

In a 1996 report Jameson (1996) observed that 'Figure 8.4' from AUSTROADS (2004a) evolved from the 1940s California State Highway design thickness curves with the curves being refined and extended by the UK Road Research Laboratory. Jameson (1996) stated that Australia has used the resultant curves for 30 years with no research undertaken to verify them.

The New Zealand Supplement to the AUSTROADS' design guide (Transit New Zealand 2000) states that in both Australia and New Zealand, pavements designed according to 'Figure 8.4' generally achieve their design life. Furthermore, Salt & Gray (1998) observed that rehabilitation according to AUSTROADS design criteria generally achieves the intended reductions in subgrade strain. However, Bartley & Peploe (1998) made the statement 'Designers must be aware that in spite of the pseudo scientific approach of all modern pavement design methods, there is at the heart a very empirical relationship of doubtful legitimacy.'

1.2 Project aims

With these observations and statements in mind, this project, which was initiated in late 2004, addresses the questions:

- does the reduction of *in situ* subgrade vertical strain result in longer lasting pavements;
- are the conditions that were used to produce 'Figure 8.4′ in AUSTROADS applicable in New Zealand;
- is AUSTROADS' subgrade strain criterion appropriate in New Zealand; and
- where, if at all, might it be necessary to adapt AUSTROADS to better reflect New Zealand conditions?

Five tasks were identified for this project:

1. Reviewing Australian and New Zealand research associated with granular pavement design.
2. Convening an Industry Forum to which interested pavement designers and practitioners were invited. The results from the literature review were presented and participants were encouraged to present their experience. Experts in pavement deterioration modelling participated in order to determine their experience in relating pavement structure to performance. The experience of practitioners was obtained and added to the 'current knowledge pool'.
3. Modelling using CIRCLY: The information obtained from the pavement forum formed the basis of modelling using the elastic layer analysis program CIRCLY. Changes to the AUSTROADS' relationships were investigated to determine the changes required to match New Zealand mechanistic designs to performance.
4. Developing failure criteria: the models used in Deighton Total Infrastructure Management System (dTIMS) which relate pavement roughness progression and rut formation were used to predict the 'failure' condition of pavements designed to New Zealand design criteria.
5. Analysing sensitivity and risk: the models developed were used in a sensitivity analysis to determine the critical parameters for a range of pavements. The results were also used to generate a risk analysis scenario that will give pavement designers an appreciation of the factors that are critical in the design process. The sensitivity of the pavement design life to typical measurement errors in subgrade CBR was examined using both CIRCLY and mePADs.

It should be noted that while the AUSTROADS (2004a) Pavement Design Guide includes design methodology for cemented and asphalt pavements the scope of this project has been limited to unbound granular pavements as this is the most common pavement construction technique in New Zealand.

2 Background to the AUSTROADS Pavement Design Guide

2.1 History of AUSTROADS in New Zealand

The document entitled *Pavement Design – A Guide to the Structural Design of Road Pavements* which was published by AUSTROADS (2004a) is used within both New Zealand and Australia to design pavements for motorised traffic. An earlier edition of the document was adopted by New Zealand in 1996 (AUSTROADS, 1992) and since its release AUSTROADS (2004a) has been the current document. Prior to 1996 pavements in New Zealand were designed according to the State Highway Pavement Design and Rehabilitation Manual (National Roads Board New Zealand 1987).

The AUSTROADS document is a distillation of many years of research into pavement construction, field observations of pavement performance, characterisation of construction materials, and pavement modelling. As such, it could be reasonably be assumed that pavements designed and built according to AUSTROADS specifications will provide a satisfactory level of service for the anticipated life of the pavement. However, the assumptions underpinning AUSTROADS need to be understood for New Zealand conditions and their validity assessed.

2.2 Pavement design methodology

2.2.1 Design steps

The majority of pavements in New Zealand are unbound granular pavements and as such the design methods are prescribed by AUSTROADS. There are a number of steps required for the empirical design of granular pavements with thin bituminous surfacing. A methodology is indicated within AUSTROADS that follows the steps

1. Prediction of design traffic,
2. Assessment of design subgrade California Bearing Ratio,
3. Determination of basic pavement thickness,
4. Upon consideration of available pavement materials determine a pavement structure,
5. Assessment of CBR of each pavement material,
6. Determine adequacy of cover,
7. Ensure minimum base thickness satisfied, and
8. Adoption of pavement design.

Pivotal to this design process is Step Three, namely the determination of the pavement thickness. This step, for unbound pavements is typically achieved through the use of 'Figure 8.4' in the AUSTROADS document. Validation and understanding of this figure for New Zealand conditions is thus critical.

2.2.2 Design traffic

The design traffic requirement is formally stated in AUSTROADS (2004a) as 'The design traffic for flexible pavement design is, for each relevant damage type, the total number of Standard Axle Repetitions during the design period which causes the same damage as the cumulative traffic.'

AUSTROADS (2004a) comments that it is well established that light vehicles contribute very little to structural deterioration; consequently design traffic only accounts for heavy vehicles. So AUSTROADS (2004a) defines a standard axle as a single axle with dual tyres applying a load of 80 kN to the pavement. When axles are less than 2.1 m apart they are considered to form an axle group. Axle groups that cause equal damage are taken to be those loads that produce equal maximum deflection of the pavement surface. The equation used to calculate the design traffic is given below:

$$N_{DT} = 365 \cdot AADT \cdot DF \cdot \frac{\%HV}{100} \cdot N_{HVAG} \cdot LDF \cdot CGF \qquad (2.1)$$

Where:
N_{DT} = cumulative number of heavy vehicle axle groups,
$AADT$ = annual average daily traffic,
DF = direction factor,
$\%HV$ = percentage of traffic that are heavy vehicles,
N_{HVAG} = average number of axle groups per heavy vehicle,
LDF = lane distribution factor, and
CGF = cumulative growth factor.

AUSTROADS uses a factor called the Standard Axle Repetitions (SARs) to provide a measure of the damage caused to the road in terms of a standard axle. The SAR is evaluated using Equation 2.2.

$$SAR_{ijm} = \left(\frac{L_{ij}}{SL_i}\right)^m \qquad (2.2)$$

Where:
SAR_{ijm} = number of standard axle repetitions which causes the same amount of type m damage as a single passage of axle group type i with load L_{ij},
SL_i = standard load for axle group type I,
L_{ij} = load on the axle group, and
m = damage exponential which is specific to the mechanism of failure.

There is a damage exponent (m = 4) associated with the empirical evaluation of damage within granular pavements with thin bituminous surfacing. This exponential is derived from field studies of pavement performance. The Standard Axle Repetitions calculated with an exponent m of 4 are commonly referred to as Equivalent Standard Axles (ESAs).

The cumulative number of heavy vehicle axle groups, N_{DT}, is the product of a number of terms many of which have measurement errors; the cumulative error from the product of all these terms will likely produce a N_{DT} that has a significant error. The N_{DT} is multiplied by the *ESA/HVAG* ratio to give the *DESA* or the Design Equivalent Standard Axles. Given this ratio is a measured quantity it is likely that there also is an error associated with it which will produce a further error in the final estimate of design traffic. These errors being noted the design thickness of the granular material determined using 'Figure 8.4' from AUSTROADS is relatively insensitive to design traffic. For example, a doubling of the design traffic only produces an approximate increase of ten percent in the design thickness of the pavement.

2.2.3 Subgrade CBR

The errors associated with the field measurement of subgrade CBR and their influence on the design life are discussed in Chapter 4.5.

2.2.4 Determining the basic pavement thickness.

The required thickness for a granular thin surfaced pavement, for particular conditions, is determined using 'Figure 8.4' of AUSTROADS (2004a and 1992). AUSTROADS (2004a) has an equation presented with 'Figure 8.4' that describes the granular thicknesses required for various traffic and subgrades; this equation is reproduced here as Equation 2.3

$$t = \left[219 - 211(\log CBR) + 58(\log CBR)^2\right]\log(DESA/120) \quad (2.3)$$

Where: t = pavement thickness (mm),
CBR = California Bearing Ratio (%) of the subgrade, and
$DESA$ = design traffic (ESA).

This design methodology does not address the failure of pavement surfacings.

The pavement thickness indicated is the total thickness of the constructed pavement and is a function of both the subgrade CBR and the design traffic. The curves of 'Figure 8.4' can also be used to establish the thickness of new pavement layers. When designing a particular layer all lower layers are considered to act as a subgrade with a CBR value equal to the test CBR of the layer immediately under the one being designed. This, presumably, assumes that the topmost layer obeys the ratio constraints discussed in Chapter 2.2.5.

2.2.5 Material moduli

The AUSTROADS pavement design system is unique amongst pavement design methodologies in that it assumes anisotropic moduli for the unbound granular materials; most other design methods assume isotropic moduli. The assumption of anisotropic moduli is supported by work by Seyhan et al. (2005) who observed vertical resilient moduli that were commonly found to be larger than horizontal moduli. This might be expected because unidirectional compaction produces a preferred void orientation and therefore an anisotropic structure.

The resilient modulus of pavement construction materials may be determined in the laboratory using various tests. In determining the modulus of a pavement material AUSTROADS (1992) used the equation

$$E_r = K_1 \theta^{K_2} \tag{2.4}$$

Where: E_r = resilient modulus,
θ = bulk stress, and
K_1, K_2 = experimentally determined constants.

AUSTROADS (2004a), however, introduced a replacement equation

$$E_r = K_1 \left(\frac{\sigma_m}{\sigma_{ref}} \right)^{K_2} \left(\frac{\tau}{\sigma_{ref}} + 1 \right)^{K_3}, \tag{2.5}$$

where E_r = resilient modulus (MPa),
σ_m = mean normal stress (kPa),
σ_{ref} = reference stress (kPa),
τ = shear stress (kPa), and
K_1, K_2, and K_3 = experimentally determined constants.

Examining Equation 2.4 it can be seen that the resilient modulus of granular materials is a function of the bulk stress. Equation 2.5 indicates that the modulus is a function of both the normal stress and the shear stress. In either case, it can be concluded that the effective modulus of a granular material within a pavement will be increased by the compaction of the *in situ* materials to produce high residual stresses within the material.

Realizing the full potential modulus of a basecourse requires an underlying layer strong enough to compact against. For example, if a high quality basecourse with a high modulus is laid directly on a subgrade with a poor CBR the effective modulus of the basecourse will be reduced. In calculating the effective moduli of the basecourse AUSTROADS (2004a) divides the granular layer into 5 layers of equal thickness. The vertical modulus for the top sublayer is the minimum of either a value from a tabulated value in AUSTROADS (2004a) or the modulus determined according to

$$E_{v\ \text{Top of Base}} = E_{v\ \text{Subgrade}} 2^{(T/125)} \tag{2.6}$$

Where: $E_{v\ \text{Subgrade}}$ = vertical modulus of the subgrade,
$E_{v\ \text{Top of Base}}$ = modulus of the top layer of the basecourse, and
T = total thickness of the granular media (mm).

The denominator in the exponential term was originally proposed to be equal to 100, the value 125, used in AUSTROADS, was a suggested value to ‘tighten the screws more’ (Potter reproduced in Moffatt & Jameson 1998) if a more conservative approach was desired.

A maximum limiting ratio of moduli for adjacent sublayers is determined using Equation 2.7

$$R = \left[\frac{E_{\text{v top of base}}}{E_{\text{v subgrade}}} \right]^{1/5} \tag{2.7}$$

However, if 2.6 and 2.7 are assumed true then

$$R = 2^{(T/625)} \tag{2.8}$$

that is; that the ratio of adjacent moduli is determined only by the total thickness of the granular overlay and not by any physical properties of the aggregate. It also follows that for a thicker sub-base then the ratio of allowed moduli is larger. Given the modulus of the n^{th} layer is given by $E_n = R^n E_{subgrade}$ any error in the total thickness will be five times greater when determining E_5 assuming these relationships are correct. It should be noted that this is a theoretical error; actual construction variations will not have such significant influence on the mechanical behaviour.

2.3 Subgrade strain

AUSTROADS (2004a) has a criteria for the allowable number of ESA loadings, N, namely that

$$N = \left[\frac{9300}{\mu\varepsilon} \right]^7 \tag{2.9}$$

Where: $\mu\varepsilon$ = maximum compressive strain at the top of the subgrade in units of microstrain.

Equation 2.9 behaviour contrasts with the behaviour of the equation given by Shell (1978) for which

$$N = \left[\frac{28000}{\mu\varepsilon} \right]^4 \tag{2.10}$$

For low strains, less than 0.001, the Shell allowable traffic is significantly less than that allowed by the AUSTROADS criteria. These two equations only reasonably agree in the microstrain range of 2000–3000 and this range is outside the typical strains expected for pavement deformation under trafficking. A likely complication is that the calculated strains for AUSTROADS' strain criteria assume anisotropic material characteristics. This is one example of why Wardle et al. (2003) argue that empirically determined strain criteria cannot be directly compared. Each failure criterion is derived under specific environmental and construction conditions. It follows that application of the strain conditions outside the context for which they were developed is not necessarily valid.

AUSTROADS (1992) contained a slightly different form of Equation 2.9 which was derived using data from 'Figure 8.4' of AUSTROADS. Equation 2.9 is a simplified version of the earlier equation; the errors associated with the simplification are typically between 1 and 3%. For strains over 1000 $\mu\varepsilon$ Equation 2.9 predicts 40% greater traffic loadings than were allowed by the earlier equation. These two equations, from the 1992 and 2004 editions of

AUSTROADS, were derived assuming different tyre pressures; however, it is not expected that the tyre pressure variation will produce a strong change in allowable loading.

2.4 CIRCLY

The software package CIRCLY can be used as an aid in the design of pavements according to the guidance of AUSTROADS (2004a). The software provides mechanistic analysis tools for the design of road pavements and uses integral transform techniques to perform linear elastic analysis. The package calculates the cumulative damage induced by a traffic spectrum based on AUSTROADS' subgrade strain criterion as mentioned in Chapter 2.3. The software allows the use of anisotropic material properties and automatic sub-layering of unbound granular materials.

3 Pavement design literature review

3.1 Introduction

This chapter is a summary of the literature published in New Zealand related to pavement design. This literature was reviewed for information to reinforce or compare with information contained in the AUSTROADS Pavement Design Guide (AUSTROADS 2004a), a summary of which has already been presented in Chapter 2. Particular emphasis was given to literature describing unbound granular pavements, as this is the prevailing pavement type in New Zealand. Flexible unbound granular pavements are also common in Australia and South Africa.

The Highway Design and Maintenance (HDM) methodologies are used to model pavement performance in New Zealand and are discussed in Chapter 3.12. The South African methods are briefly discussed in Chapter 3.13 as these design methods might potentially be applied in New Zealand.

3.2 General observations

The majority of roads in New Zealand are constructed from a number of layers. The top surface layer is a thin coat of a wearing course, which may be asphalt or, as is common in New Zealand, may be a type of chipseal. Directly below the seal is a high modulus unbound granular layer called the basecourse. Below the basecourse is a lower quality unbound aggregate called the sub-base, and below this is the subgrade. The subgrade can either be the *in situ* soil or it might be modified to attain better performance characteristics. The structural courses must be able to withstand the applied traffic loads and transform it into acceptable vertical stress on the soil.

Traditional pavement models assume elastic material behaviour. AUSTROADS (2004a) uses analytical design methods in the form of elastic layer analysis. Many countries use empirical design methods including the United States of America, Germany, France, Spain and Italy. South Africa, Australia and New Zealand all use mechanistic-empirical design methods and construct similar pavements with thin surfacing and granular layers. Australia and New Zealand both use the AUSTROADS design methodology. The AUSTROADS analytical method has been calibrated primarily against empirical data obtained from 1940s California State Highway research; South African design methodology utilises linear elastic multilayer analysis but is calibrated against empirically derived transfer functions. So both AUSTROADS and the South African design methods have a mechanistic basis for their design that is calibrated against empirical data. Currently, there are no fully mechanistic pavement models being used to design pavements.

Per Ullidtz (2002), in a keynote address, commented that most current analytical methods are derived from continuum mechanics and assume static equilibrium, continuity, and that Hooke's Law is applicable. The validity of these assumptions is tenuous since traffic loading is dynamic, roads are predominately granular in nature, and in addition to elastic deformation Unbound Granular Materials (UGM) exhibit plastic behaviour.

It has been frequently noted (for an example, see Bartley Consultants 1997) that UGM cannot support tensile strain, but that linear elastic analysis predicts tensile strains in these materials. Using laboratory experiments and numerical modelling Wallace (1998) demonstrated that an unbound granular pavement can sustain some tensile strain, with the tensile strength decreasing with increasing pavement thickness.

Dawson (1999) commented that, under loading, a granular pavement layer cannot be characterised as having uniform layers since mechanical characteristics will vary with both depth and radial distance from the loaded area. Ruts generally result from deformations in both the granular layer and the subgrade as they interact. Deformation of the granular layer is related to the ability of the subgrade to resist permanent deformation and vice versa. Both the sub-base and subgrade moduli vary with stress. In fact, a cohesionless granular material does not have a modulus as a material characteristic but only as a function of the stress condition (Ullidtz 2002).

Horizontal loading of the road is typically not addressed in pavement design but can be quite significant. Different surfaces are used at roundabouts and intersections where these forces can be significant. The software CIRCLY can address these types of loading. Beyond this brief mention these forces are not addressed in this report.

3.3 Pavement performance

Typically, granular pavements are designed to last between 20 and 25 years while rigid pavements are designed to last for 30 to 40 years. In New Zealand it is not uncommon to observe pavements that were constructed 50 years ago still performing satisfactorily (Bailey et al. 2004).

While discussing pavement performance, Yeo et al. (1995) observed that increased construction process control produced a significant increase in the expected life of the pavement. For an increase in control costs of approximately 5.4%, the life of the pavement was increased by an order of magnitude. Auff & Laksmanto (1994) had earlier concluded that improving process quality control would either result in a significant increase in either the estimated life or in construction savings. Given that some pavements within New Zealand have failed within six months of construction such estimates do not seem too unrealistic. These pavements that have rutted prematurely might have failed because the specifications were inadequate rather than poor construction quality.

After examining test results obtained at CAPTIF de Pont et al. (1999) concluded that the AUSTROADS Pavement Design Guide (AUSTROADS 2004a) provided the most accurate predictions of pavement life when compared to most available design models. The construction of the pavement indoors means that weathering is not a factor in the pavement deterioration at CAPTIF. CAPTIF experiments indicate that variation in pavement wear was related to both dynamic load and variation in the structural capacity of the pavement (de Pont et al. 1999).

Tonkin & Taylor Ltd. (1998) examined how the back analysis of the deflection bowl generated by a Falling Weight Deflectometer (FWD) may be used to estimate the elastic properties of the *in situ* pavement materials in order to determine rehabilitation requirements.

Pavement performance can be assessed in a number of ways. Examples include: skid resistance, noise, minimisation of disruption, roughness, rutting, cracking and potholes. In economic terms the pavement roughness is the most important characteristic (Ullidtz 1998), however, safety might also be regarded as a significant factor. The public assumes that pavements are constructed to provide maximum safety and skid resistance.

3.4 Roughness

The general public arguably considers pavement roughness as the most important indicator of pavement condition (Huang 2004) although there is evidence that this perception might be influenced by the number of complaints received rather than actual public opinion (Cleland et al. 2005). The evolution of pavement roughness is governed by a number of factors: traffic volume and load magnitude, pavement construction, pavement materials and the environmental conditions. There are a number of methods used to measure roughness. The International Roughness Index (IRI) is progressively being adopted in New Zealand which has traditionally used the NAASRA[1] roughness measurement.

When considering all failure modes, Hajek et al. (2004) observed that a 25% improvement in smoothness increased pavement life by less than one year for asphalt pavements when all failure modes are considered. The introduction of a smoothness specification results in an improvement in the post-construction smoothness.

Roughness is caused by variations in the characteristics of pavement materials, which result in uneven pavement response to traffic.

[1] See Glossary in Appendix B.

3.5 Ruts

Rutting is created through material compaction, plastic shear and dilation in both the granular layers and the subgrade materials (Dawson 1999). While AUSTROADS (2004a) does not define a failure limit for rut depth the Technical Basis for AUSTROADS (AUSTROADS 2004b) states that a terminal rut has a depth of 20 mm. For the CAPTIF experiments de Pont et al. (1999) defined a failure rut as when the permanent vertical deformation from the original profile exceeded 25 mm. The South African software package, mePADS, allows a terminal rut to be either 10 or 20 mm. Other researchers have adopted a failure rut depth of 15 mm. The Indian specification for a failed pavement sets a maximum limit for a rut of 20 mm (Maji & Das 2005). Given the asymptotic development of ruts the life of a pavement will be highly sensitive to a small change in the rut depth failure measurement.

While performing experiments at CAPTIF de Pont et al. (1999) observed that deformation of the basecourse was a significant failure mechanism. This finding is supported by overseas researchers. Fwa et al. (2004) make the statement that rutting is caused by the accumulation of irreversible deformation in all pavement layers, not just the subgrade. Using load applications by the Texas Mobile Load Simulator, Chen (1998) observed that the majority of the rutting, developed over a six month period, accumulated in the base. Those sections of pavement that exhibited the greatest FWD deflections prior to trafficking produced the deepest ruts.

Watanatada et al. (1987) make the statement that modern roads should not fail through rutting. Fwa et al. (2004) have successfully developed a model to predict rut progression in the asphalt layer of asphalt pavements within the laboratory; they are currently verifying the model in the field. They reiterate the five factors affecting pavement rut development, namely; magnitude of traffic loads, loading speed, number of load repetitions, pavement temperatures, and rutting resistance of asphalt mixture. Not all of these factors are necessarily applicable to thin surfaced granular pavements.

Haddock et al. (2005) discuss the determination of the contribution from different pavement layers to the surface rut profile. The initial evidence is that the relative contributions of the layers to rutting could be determined from an analysis of its transverse surface profile.

The paucity of models for plastic deformation of granular layers under traffic loading means that AUSTROADS (2004a) does not account for permanent deformation in the granular layers. The document does, however, acknowledge that permanent deformation is a primary distress mode for granular layers. The contribution of the basecourse to unrecoverable strain is potentially implicitly accounted for in AUSTROADS (2004a) design since it is generally thought that the use of AUSTROADS' 'Figure 8.4' produces pavement designs that achieve their design life (Transit New Zealand, 2000).

3.6 Shakedown effect

Under constant amplitude cyclic loading, an aggregate will tend towards a constant density state. If the loading amplitude is increased then the aggregate will tend towards a new state: it is said to be shaking down. As early as 1993 Bartley & Cornwell (1993) proposed shakedown theory as a possible mechanism to describe pavement deformation. Bartley Consultants (1997) strongly encouraged further research into shakedown theory. Collins and Boulbibane (2000) concluded that shakedown theory could provide a rational approach to the analysis of unbound pavements. During a discussion on shakedown analysis Werkmeister et al. (2003) comment that there is field evidence that pavements can achieve steady-state conditions where no further deterioration occurs. Werkmeister et al. (2003) modelled vertical plastic strain in the unbound granular layer. Strain was greatest at least depth and decayed with depth; strain accumulated at approximately the same rate for all depths.

There are four material response categories for repeated loading: pure elastic, elastic shakedown, plastic shakedown, and incremental collapse. The existing shakedown theory can account for some but not all of the observed responses (Werkmeister et al. 2004).

3.7 Distribution of strain

The theoretical purpose of unbound granular layers in a pavement is the transfer and distribution of the stresses imposed by wheel loads at the pavement surface. Each pavement layer must be able to accept the applied stresses and distribute the stress to any lower layers at magnitudes that can be sustained for the design life. The stress applied to the foundation (subgrade) must be limited to sustainable levels. Load transfer is through interparticle contact, either point contact or intergranular friction (Bartley 1980), and is accompanied by both recoverable and non-recoverable strain.

Salt (1979) made the general observation that basecourses harden under traffic loading. For a pavement tested at CAPTIF by de Pont et al. (1999) with an asphaltic surface most permanent vertical deformation was observed in the unbound granular layer. A more conventional pavement (with respect to conventional New Zealand pavement construction) had between 30 and 52% of the permanent deformation occurring in the subgrade which infers that between 70 and 48% of the deformation is in the basecourse. After conducting accelerated pavement tests on pavements constructed with 76 mm of asphalt concrete and 229 mm of crushed gravel base Janoo & Cortez (2003) concluded that approximately 54–58 % of the pavement's total vertical deformation was in the subgrade. The obvious implication is that there are significant levels of vertical deformation distributed between the asphalt layer and the gravel base. Huang (2004) also noted that with increasing traffic load and tyre pressures, permanent deformation predominantly occurs in the upper pavement layers rather than in the subgrade.

Numerous experiments have observed permanent strain in the unbound layers; see, for example, Pidwerbesky et al. (1997). Furthermore, the resilient modulus is not closely related to permanent deformation (Alabaster 2000). Both Dodds et al. (1999) and Khogali & Mohamed (2004), while performing Repeated Load Triaxial (RLT) tests observed that the influence of moisture on the plastic deformation accumulated was dramatic while its effect on the resilient modulus was slight. Furthermore, Dodds et al. found the degree of saturation influenced the plastic behaviour.

From the examples given in the preceding paragraphs it follows that the assumption that all permanent deformation occurs in the subgrade is not valid, not just for New Zealand pavements but for pavements in general.

While measuring the strain response of subgrades Pidwerbesky (1995) observed that the magnitude of the resilient strains measured in the CAPTIF test track were greater than those predicted by the then current design models, assuming the same pavement life in terms of traffic loading. In a subsequent paper Pidwerbesky et al. (1997) proposed a less conservative subgrade strain criteria; the consequence of this is discussed in Chapter 4. Using data from the Western Bay of Plenty, Hallett & McIlroy (2003) produced a very similar subgrade strain criteria to that of Pidwerbesky et al. (1997).

3.8 Basecourse and sub-base

3.8.1 General notes

Patrick et al. (1998) determined that under traffic, basecourse density tends towards 95% of the maximum dry density. Dry density normally decreases towards the bottom of the layer, where the compaction stresses are the smallest. The density of the lower levels may be critical to the satisfactory performance of the pavement but confirmation of the density of the sub-base under traffic has not been obtained. According to Khogali & Mohamed (2004) there is potential for the use of initial high rates of accumulation of permanent deformation to be used as an indicator of inadequate construction quality.

While examining the performance of unbound basecourse Salt (1979) commented that lateral movement contributed no measurable contribution to rut depth unless either high saturation or excessive pavement deflection had occurred.

3.8.2 Performance criteria for unbound granular materials

Bartley (1979) made the statement that pavements become unstable for two reasons, excessive loading or excess water levels. There are currently no performance criteria for UGM beyond trying to minimise permanent deformation. The M/4 (Transit 2006) specification prescribes a pavement material that, experience has demonstrated, has the characteristics necessary to function effectively as a pavement material, characteristics, for example, such as permeability and durability. Bartley (1979) suggested that the policy of using M/4 in all situations is not necessary. In some conditions, where the pavement performance is not critical, marginal materials could potentially be used without significantly increased risk.

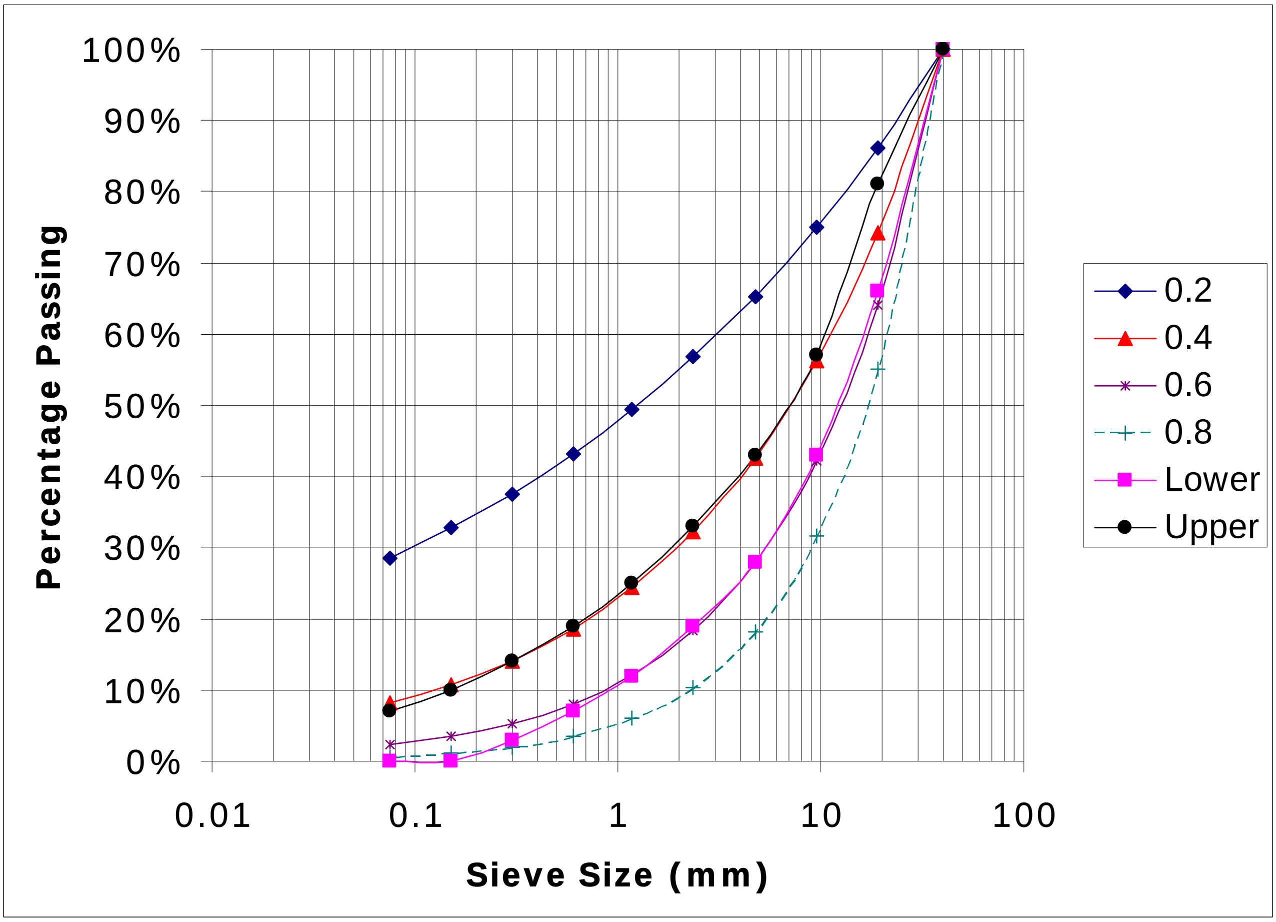

Figure 3.1 Variation of grading curve with Talbot number.

Numbers in legend refer to Talbot numbers.

'Lower' and 'Upper' refer to the lower and upper limits allowed by TNZ M/4.

The Talbot formula, which is used to characterise particle size distribution, is given by

$$P = \left(\frac{d}{D}\right)^n \tag{3.1}$$

Where: P = percentage passing a sieve with opening *d,*

D = maximum particle size in the material, and

n = constant.

The Talbot number, *n*, is frequently used as one of a number of specifications for granular media when used in road construction. Figure 3.1 displays the variation of grading curve with various Talbot numbers; also included in the plot are the upper and lower limits allowed by the TNZ M/4:2006 specification published by Transit (2006). The M/4 specification dictates basecourse aggregate characteristics for materials used on State Highways and other heavily trafficked roadways. After normal pressure the shear strength is most strongly influenced by particle shape, followed by the grading exponent (Seddon 1979). Research by the New Zealand Road Research Unit (1979) determined that gradings with n=0.6 produced higher rutting than the other gradings tested; this n=0.6 grading follows close to the lower limit of the M/4 specification.

Using RLT testing Dodds et al. (1999) established a New Zealand protocol to distinguish between the performance of various materials and this protocol is being used in the M/22 specification (Transit 2000).

AUSTROADS (2004a) provides presumptive moduli for unbound granular materials and modelling by CIRCLY is strongly dependent on the resilient moduli of pavement materials. In fact, the use of resilient modulus in roading is questionable since plastic deformation is responsible for most of the wheelpath rutting that is observed (Khogali & Mohamed 2004). Bartley & Peploe (1998) have observed that 'The non-linearity of most subgrade soils and unbound aggregate makes the determination of the elastic modulus somewhat of a moving target'.

3.9 Subgrade

The purpose of a pavement is to reduce the vertical stress on the subgrade so that deformation of the subgrade will not detrimentally affect the easy transport of traffic. Limiting the vertical compressive strain has been used as a design criterion for many design methods. Paterson (1987), however, observed that limiting the vertical compressive strain in the subgrade was not useful for performance modelling. This was because rather than predicting a limit it indicated the trend of accumulated deformation during the life of the pavement.

Volcanic soils have various relationships between the CBR and the modulus (Bailey & Patrick 2001). Thus, the use of the relationship from AUSTROADS where *Modulus* = 10 *CBR* is not appropriate for all situations. Instead of using a factor of 10 Sutherland et al. (1997) recommend the use of the equation *Modulus* = 3 *CBR* to estimate the subgrade modulus when calculating pavement deflections and strains for materials sourced from Wanganui. For volcanic soils, Bailey & Patrick (2001) suggested that the following relationships between isotropic modulus and CBR were appropriate: M_r = CBR for a typical pumice/sandy soil, M_r = 3 CBR for a mixture of silty soils and brown ash, and M_r = 10 CBR for clayey ash soils. They present a number of different relationships depending on the intended purpose of the results.

Alabaster et al. (2000) commented that there is little relationship between subgrade modulus and the accumulation of plastic strain in volcanic soils. Furthermore, they observed that when the failure criteria of AUSTROADS are applied, the high strains observed on roads constructed on North Island volcanic subgrades should produce fatigue failure. They state their desire to quantify the relationship between the elastic and plastic strains generated by the load.

Subgrade strain criteria for volcanic soils may, according to Salt & Stevens (2001), be conservative by at least a factor of 1.5 and, in some soils, up to a factor of 1.75. This would have the consequence of increasing the design traffic levels, according to Equation 4.6, by a factor between 17 and 50. To account for this effect during the calculation of the structural number they have proposed the scaling of subgrade moduli for different subgrades.

A study by Bartley Consultants Ltd. (1998) has demonstrated that where the subgrade has been stabilised the improved performance will continue in the long term.

Bartley & Peploe (1998) demonstrated, using CIRCLY, that it is the modulus of the subgrade that dominates the life of the pavement. They however state, 'Designers must be aware that in spite of the pseudo scientific approach of all modern pavement design methods, there is at the heart a very empirical relationship of doubtful legitimacy.' Within the subgrade the average depth of significant stress is typically about 300 mm below the top of the subgrade (Bartley & Peploe 1998).

3.10 Seasonal and environmental effects

The influences of seasonal effects are still predominately unknown according to Patrick & McLarin (1998). Freeze-thaw effects are not addressed in AUSTROADS (2004a); the New Zealand Supplement, however, states that where the potential exists for freezing, the aggregates used in construction must not be susceptible to freeze-thaw effects.

As mentioned earlier Khogali & Mohamed (2004) observed that moisture influence on pavement performance is more significant than the initial selection of resilient modulus. Pidwerbesky (personal communication[2]) has commented that he personally has never observed a subgrade fail without moisture being present in the subgrade. Bartley (1979) has stated that the stability of materials is controlled by their water content. Dennison (2005) found no correlation between moisture variation and FWD deflection, but acknowledged that the moisture variations may have been insufficient to produce an effect. Furthermore, Peploe (2002) found few clear relationships between measured parameters when examining subgrade moisture conditions.

De Pont et al. (1999) observed that some pavement wear can be attributed to environment effects. The HDM roughness model (Paterson 1987) has a term that specifically accounts for environmental deterioration.

3.11 Tyre and road interaction

3.11.1 Stress application

The stress application of traffic is applied to the pavement through the tyres of the vehicles. For low pressure tyres the walls are in compression; while for high pressure tyres, the walls are in tension. Tyre stress patterns can have twice the stress at the edge of the tyre than at the centre (AUSTROADS 2004a). This stress can have a marked effect on thin surfacings, especially in settings with high lateral loadings.

The design tyre pressure for the pavement analysis adopted by AUSTROADS (2004a) is 750 kPa, although it is acknowledged that the actual pressures can range from 500 to 1200 kPa. Using a finite element model Saleh[3] has found that the influence of tyre

[2] Statement made at 2004 Pavement Forum

[3] Unpublished report entitled *Three Dimensional Finite Elements Model and Fractorial Analysis to Study the Effect of the Lateral Support on the Pavement Structural Response.*

pressure on pavement response is significant only near the pavement surface, not at depth.

AUSTROADS (2004a) defines a standard axle as a Single Axle with Dual Tyres (SADT) applying a load of 80 kN to the pavement. This load is applied over four tyres so each tyre is providing a downward force of 20 kN. Huang (2004) provides a calculation to determine the contact area of each tyre. Following AUSTROADS in assuming a tyre pressure of 750 kPa the equation determines a contact area of 0.0267 m^2, or an equivalent rectangle of 136 mm by 197 mm, as the footprint of each tyre for a SADT in New Zealand.

3.11.2 Traffic loading

Timm et al*.* (2005) acknowledged the complex distribution of axle weights and observed that such distributions were not well represented by individual theoretical statistical distributions. Instead they proposed a mixed distribution model of two or more theoretical distributions. Such an approach represents a shift away from Equivalent Single Axle Load (ESAL). Huang (2004) regards traffic as being the most important factor in pavement design, which is an interesting statement to make considering the relative insensitivity of AUSTROADS to traffic variation. De Pont et al. (2003) found that for both compaction and wear mechanisms within pavements the design traffic is proportional to the load to the power of approximately 2 rather than 4. Johnsson (2004), after examining another researcher's data, states that for flexible pavements the fourth power law seems to be justified.

Given the insensitivity of the AUSTROADS design methodology to traffic and the apparent insensitivity of the design to the power law used, it is the authors' opinion that AUSTROADS' traffic evaluation needs no adaptation at this stage.

3.12 Highway Design and Maintenance Standard Series

3.12.1 Introduction

The Highway Design and Maintenance Standard Series are prepared on behalf of the World Bank. The models developed for these series are known as HDM models and each new generation is incrementally numbered. Thus the models presented by Paterson in 1987 are known as the HDM III models. The roughness evolution model remained relatively unchanged in the HDM IV document (Lea International Ltd. 1995). The models discussed in this document are generally HDM III models. The HDM models are used in the New Zealand dTIMs modelling package to predict pavement condition with the model constants being calibrated by New Zealand experience.

While discussing mechanistic models, Paterson (1987) notes that repeated loading tests using constant stress cycles commonly have the form:

$$\varepsilon_p = aN^b \tag{3.2}$$

Where: ε_p = plastic strain,
a = estimated coefficient,
b = estimated coefficient, and
N = number of stress applications.

The variable N does not allow any different weighting for different stress loading conditions and therefore Equation 3.2 implies that each stress application has the same effect. No account is explicitly made for vehicle speed, mass or load distribution. Instead, these factors are implicitly accounted for with the estimated constants a and b, and are consequently fixed.

The form of this equation is, upon rearrangement, similar to the Equations 2.9 and 2.10 presented to describe the allowable subgrade vertical elastic strain. However, in the form presented in Equation 3.2 the implication is that the total plastic strain is a function of the total number of stress applications rather than the total number of stress applications being a function of strain magnitude.

3.12.2 Structural number

The Structural Number is an indicator of pavement strength and is used frequently in HDM models. The Structural Number, SN, defined by Huang (2004) is given by:

$$SN = a_1 D_1 + a_2 D_2 m_2 + \ldots. + a_n D_n m_n \tag{3.3}$$

Where: a_i = is the i^{th} layer coefficient,
D_i = D_i is the i^{th} layer thickness (in inches), and
m_i = the i^{th} layer drainage coefficient.

While the above equation does account for drainage it does not account for subgrade contribution to pavement strength. The AASHTO structural number presented in both HDM III (Paterson 1987) and Bailey & Patrick (2001) is modified to account for subgrade strength. The Structural Number is defined as

$$SNC = \frac{1}{25.4} \sum_{i=1}^{n} a_i h_i + SN_{sg} \tag{3.4}$$

In which

$$a_i = a_g \left(\frac{E_i}{E_g} \right)^{1/3} \tag{3.5}$$

And

$$SN_{sg} = -0.85 (\log CBR)^2 + 3.51 (\log CBR) - 1.43 \tag{3.6}$$

Where: SNC = modified Structural Number,
SN_{sg} = Structural Number contribution from subgrade,
a_i = layer coefficient,
h_i = thickness of layer i (mm), and where $\Sigma h_i \leq 700$ mm,
a_g = layer coefficient of standard materials (AASHTO Road test),
E_i = layer modulus,
E_g = modulus of standard materials (AASHTO Road test), and
CBR = California Bearing Ratio of the subgrade (as a percentage).

The factor of 25.4 is used to convert the equation, derived for imperial units, into SI[4] units.
Note that Equation 3.4 does not, however, specifically account for drainage. This was amended in HDM IV where factors were included to account for the influence of moisture in the various pavement layers.

Cenek & Patrick (1991) and then Bailey et al. (2004) proposed a slight modification to the standard strength coefficients for the sub-base from 0.11 to 0.12 to reflect New Zealand conditions. The strength coefficients are displayed in Table 3.1 with the modification made.

Table 3.1 Strength coefficients and resilient moduli for New Zealand conditions from Bailey et al. (2004).

Layer type	Strength coefficient a_i	Resilient modulus E_a (MPa)
Asphalt concrete surface course	0.44	3100
Unbound basecourse	0.14	207
Granular sub-base course	0.12	104

Patrick & Dongal (2001) discussed the determination of the structural number for New Zealand pavements. They observed that volcanic subgrades, common in the central North Island of New Zealand, appear to have higher shear strengths than those normally associated with lower modulus materials. This additional strength would mean that pavement thicknesses determined from AUSTROADS for these subgrades are greater than are actually required.

3.12.3 Rutting

Paterson (1987) provides an equation to describe the rutting of a pavement, namely:

$$RDM = 1.0AGER^{0.166}SNC^{-0.502}COMP^{-2.3}NE_4^{ERM} \qquad (3.7)$$

Where:

$$ERM = 0.0902 + 0.0384DEF - 0.009RH + 0.00158MMP^{CRX} \qquad (3.8)$$

Where:
- RDM = mean rut depth of both wheelpaths (mm),
- $AGER$ = age of the pavement since or time since last overlay,
- SNC = modified Structural Number,
- $COMP$ = compaction index,
- NE_4 = cumulative number of equivalent 80 kN standard axles (ESA),
- DEF = mean peak Benkelman beam deflection under 80 kN standard axle load of both wheelpaths (mm),
- RH = rehabilitation state (=1 for overlaid pavements and =0 for original pavement),
- MMP = mean monthly precipitation (m/month), and
- CRX = area of indexed cracking (%).

[4] SI = Système International (d'Unités). Standard international units, e.g. metre, kilogram and second.

Assuming that cracking is insignificant and that the pavements are original it follows that the last two terms in Equation 3.7 can be neglected. In this document, where rut values have been calculated, using Equations 3.7 and 3.8, zero cracking and an original pavement have both been assumed.

3.12.4 Roughness

Equations to determine the evolution of pavement roughness are presented in Paterson (1987), namely:

$$RI(t) = [RI_0 + 725(1 + SNC)^{-4.99} NE_4(t)]e^{0.0153t} \tag{3.9}$$

Where:
- $RI(t)$ = roughness at time t,
- RI_0 = roughness at t=0,
- SNC = modified Structural Number,
- $NE_4(t)$ = cumulative equivalent standard axle loadings until time t (10^6 ESA lane^{-1}), and
- t = age of pavement since overlay or construction.

Equation 3.9 produces results in terms of the International Roughness Index (IRI) rather than the NASSRA units (see Glossary in Appendix B), which are commonly used in New Zealand. An equation derived from the HDM roughness models but producing an output in NAASRA is presented in Chapter 4.2.3.

3.13 South African design methods

The pavement design methodologies used in South Africa are of interest since the pavements constructed there are similar to those built in New Zealand and Australia, namely thin-surfaced granular pavements.

South African pavement design is complemented by a software package called Pavement Analysis and Design Software, also known as mePADS. While the original intention was to develop a deterministic model, the model is a probabilistic one. Final selection of pavement design in the approach is based on the life cycle costs. The South Africans essentially use a critical layer approach where the life of the pavement is determined by the lives of the individual layers. The South African approach utilises linear elastic multilayer analysis but incorporates empirically determined layer performance models for the unbound granular layers as well as the subgrade.

The software allows the user to set the level of a terminal rut, either 10 or 20 mm with empirical curves being available for either rut depth. Similarly reliability of the road design is preselected using the Road Category, and the Climatic Region options are Dry, Moderate or Wet. The design traffic is a variable with upper and lower bounds.

The document associated with mePADs (Theyse & Muthen 1996) identifies two deformation mechanisms for granular materials, namely densification and granular shear. Shear is accounted for using a safety factor based on Mohr Coulomb theory. Climatic conditions are defined as dry, moderate and wet and the pavement life is highly sensitive to the choice of climate.

The bearing capacity of the subgrade is empirically determined based on the vertical strain experienced at the sub-base/subgrade interface. The influence of moisture levels on the bearing capacity does not appear to be modelled.

Given that the AUSTROADS (2004a) design method only allows for pavement failure in the subgrade, it is suggested that the incorporation of the South African design method into New Zealand design is worth investigation, particularly given the similarity in the road construction practices of the two countries.

4 Discussion

4.1 Industry forum

4.1.1 Attendees and issues

As mentioned earlier an industry forum was held to discuss pavement design in New Zealand and current practices. Participants attending the forum are listed in Table 4.1. All participants made presentations and the issues facing pavement design and construction were discussed. Seventeen issues were listed as being of particular importance to the industry and these are included in Appendix A. Attendees were asked to rank these issues in order of importance and the rankings of those who responded are presented in Table A.1 in Appendix A. The top five ranked issues were identified as being:

1. the need to stop post-construction rutting,
2. reducing shear in the granular layer,
3. determining the maximum traffic loading possible on granular pavement coupled with intensity,
4. that models used in design be based on observed performance, and
5. the quantification of stabilisation performance.

Table 4.1 List of attendees at the 2004 Pavement Forum.

Attendee	Employer
Allen Browne	Opus International Consultants
Bruce Steven	University of Canterbury
Bryan Pidwerbesky	Fulton Hogan
Clarence Morkel	New Zealand Institute of Highway Technology (NZIHT)
David Alabaster	Transit New Zealand
David Hutchison	Works Infrastructure
Frank Bartley	Bartley Consultants
Graham Salt	Tonkin and Taylor
Greg Arnold	Transit New Zealand
John Hallett	Beca
John Patrick	Opus International Consultants
Ken Hudson	Duffill Watts and Tse Ltd.
Martin Gribble	Opus International Consultants
Mofreh Saleh	University of Canterbury
Norm Major	Retired (previously Works Consultancy)[5]
Ross Peploe	Bartley Consultants

[5] Norm Major died 2006 while this report was being prepared for press.

4.1.2 Stopping post-construction rutting

Rapid premature rutting of pavements occurs occasionally. This post-construction rutting is strongly linked with low compaction energies, high moisture content, low material quality and the construction of Greenfield pavements. At the forum, David Alabaster commented that Transit does not see a lot of rutting in pavements in New Zealand. Calculations using HDM III (results presented in Chapter 6.1) also indicate that the rutting is a insignificant contributor to the failure of pavements.

These pavements that have rutted prematurely might have failed because the specifications were inadequate rather than through poor construction quality. Provided adequate construction specifications can be determined then there is potential, through the use of construction process control, to increase the lives of pavements (Yeo et al. 1995). Furthermore, initial high accumulation of permanent deformation might be used as an indicator of inadequate construction quality (Khogali & Mohamed 2004), again provided the construction specifications have been demonstrated to be satisfactory.

Participants at the forum reiterated the observation that not all the permanent strain occurs in the subgrade. Instead, rutting within the aggregate layer is an important element of the overall distress.

4.1.3 Reducing shear in the granular layer

The reduction of shear in the granular layer potentially may be solved using stabilisation (which was ranked fifth in importance). Shear strength is strongly influenced by normal stress, particle shape, plasticity of fines, and grading of the aggregate. Careful control of these four parameters would likely minimise shear in the granular layer.

4.1.4 Determining the maximum traffic loading

Determining the upper traffic limits for granular pavements is likely to be outside the scope of this current research. The Transit New Zealand Supplement (2005) provides some guidance on limits traffic and research is continuing in this area.

4.1.5 That models used in design be based on observed performance

The continuous calibration of dTIMs with observed road performance data is already progressing towards this outcome. However, the successful use of these data to calibrate mechanistic design methods has not yet occurred.

4.1.6 Quantification of stabilisation performance

The quantification of the performance of stabilised pavements is desirable for a number of reasons and is currently receiving attention in an Opus Central Laboratories' Foundation for Research, Science and Technology project.

4.2 Pavement failure mechanisms

4.2.1 Definitions

AUSTROADS (2004a) does not define what is meant by pavement failure. For example it does not define failure levels for rutting. Earlier values for terminal roughness found in AUSTROADS (1992) were omitted from AUSTROADS (2004a). AUSTROADS (1992) states that the design procedure for flexible pavements assumes an initial roughness of 50 counts/km and a final roughness of 150 counts/km (110 counts/km for class 1 and 2 roads). This statement is not present in AUSTROADS (2004a) while the basic procedure for designing flexible pavements remains unchanged.

The document entitled *Technical Basis for AUSTROADS Pavement Design Guide* (AUSTROADS 2004b) states that implicit in the design procedure is a terminal condition of:

- an average rut depth of about 20 mm, and
- a terminal roughness of about three times the initial roughness.

It is not entirely clear if the road is considered failed with only one of the failure conditions existing or if both are necessary.

In New Zealand the reconstruction of pavements is driven by factors other than structural deterioration. Many of the reconstructed pavements have significant life remaining according to AUSTROADS and rutting and roughness are not generally the reasons for reconstruction (Bailey et al. 2004).

4.2.2 Rutting

Using the coefficients from dTIMS, developed for New Zealand conditions, it becomes apparent that, provided the equations are applicable, rutting should not play a significant factor in the failure of New Zealand roads for pavements designed according to 'Figure 8.4' of AUSTROADS (2004a). Under traffic loading of 1×10^7 ESA and ignoring cracking within the pavement the maximum rut depth predicted is 4.7 mm. The implicit assumption made by ignoring cracking is that the road surface is adequately maintained, which is reasonable if the maintenance programme is effective. Conversely, a pavement designed according to AUSTROADS' 'Figure 8.4' requires a traffic loading of 1×10^{10} ESA to produce a rut of 10 mm and 1×10^{12} ESA to produce a rut of 20 mm. Thus, assuming the model coefficients are calibrated correctly for New Zealand conditions and that roads are designed according to the Design Guide, the implication is that mechanisms other than rutting are responsible for prompting pavement rehabilitation.

4.2.3 Roughness

Cenek & Patrick (1991) working with a HDM model of roughness evolution presented Equation 4.1 to describe the progression of roughness based on the Structural Number and the traffic loading. The equation has the advantage that it can be used to back calculate the roughness at construction, assuming it is unknown.

$$RN(Y_2) = RN(Y_1)e^{0.0153(t_2-t_1)} + 5.7(1+SNC)^{-4.99}ESA(t_2-t_1)e^{0.0153t_2} \quad (4.1)$$

Where:
- $RN(Y_2)$ = roughness data at period 2 (units are NAASRA),
- $RN(Y_1)$ = earliest roughness data (units are NAASRA),
- Y_2 = year of predicted roughness,
- Y_1 = year of earliest roughness data,
- T_1 = Y_1-Y_0 (years),
- T_2 = Y_2-Y_0 (years),
- t_2-t_1 = Y_2-Y_1 (years),
- ESA = equivalent standard axles (units per day per lane) passing over the pavement being modelled, and
- SNC = modified structural number as defined in Equation 4.2.

When the HDM models are used to predict the evolution of roughness with time it is sometimes necessary to assume an initial roughness. Figure 4.1 was generated using a slightly simpler form of Equation 4.1, namely Equation 4.2.

$$RN(t) = [RN_0 + 18995(1+SNC)^{-4.99}300ESA \times t \times 10^{-6}]e^{0.0153t}, \quad (4.2)$$

Where:
- $RN(t)$ = roughness at time t (units are NAASRA),
- RN_0 = roughness at construction (units NAASRA),
- ESA = the units of ESA, in this case 10^6 ESA per year per lane,
- t = time in years, and
- SNC = modified Structural Number as defined in Equation 4.2.

The multiplication of the ESA variable by 300 assumes that this is the number of days working year. This assumption needs to be checked for specific roads as some have high weekend traffic levels.

The roughness evolution model from Equation 4.2 was used to model the roughness of a number of pavements for which the pavement thickness was determined according to Equation 2.3. For the purpose of calculating the Structural Number, it was assumed that the pavement contained a single structural layer, i.e. the basecourse, the modulus of which was assumed to be 500 MPa. These results are displayed in Figure 4.1 which clearly demonstrates the evolution of roughness with time, with the rate of roughness accumulation being greater for smaller CBR values. Figure 4.1 was generated assuming that the initial roughness was 60 NAASRA and the pavement thickness was determined using the granular thickness dictated by AUSTROADS (2004a) for the indicated traffic loading. The implication of Figure 4.1 is that despite being designed according to AUSTROADS' 'Figure 8.4' for the same design traffic the pavements perform differently.

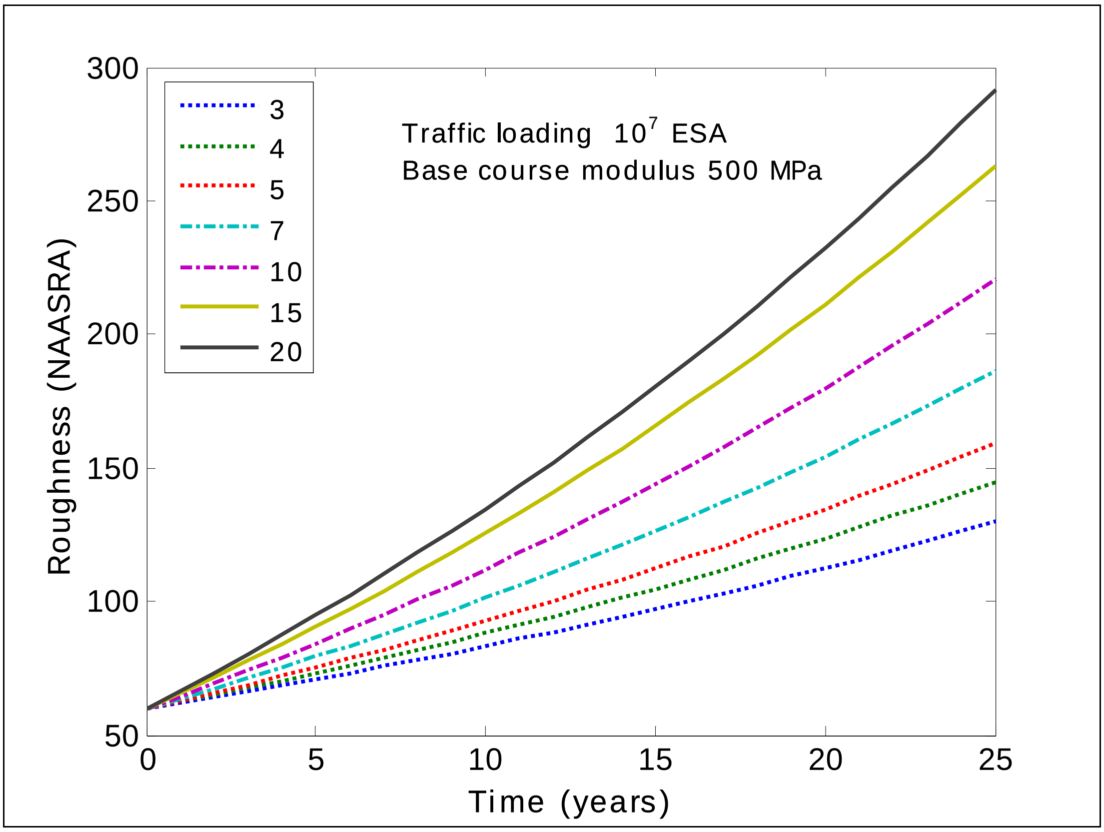

Figure 4.1 Plot of evolution of roughness with time for various subgrade CBR values (see legend).

If a pavement is untrafficked then Equation 4.2 reduces to $RN(t) = RN(0)e^{0.0153t}$, which effectively gives a measure of the evolution of roughness due to surrounding environmental conditions, such as weathering, for an untrafficked pavement.

Tonkin & Taylor (1998) examined the use of Equation 4.1 and commented that it appeared to produce excessively optimistic residual life predictions. The dTims system uses incremental forms of Equation 4.1 with the coefficients continuously being calibrated for New Zealand conditions so it could be expected that this system would produce results that would tend to reflect pavement performance. Research by Bailey et al. (2004) actually indicates the opposite is the case, namely that pavements are being rehabilitated before rutting and roughness become factors.

4.2.4 Cracking and fatigue

Fatigue cracking is the predominant failure mechanism of thin bituminous surfacings. Failure due to cracking was defined by de Pont et al. (1999) as cracking exceeding 5 m/m^2 over 50% of the trafficked area. Cracking is a factor in the HDM models that are used in dTims for modelling the roughness and rutting evolution.

4.3 Comparison of AUSTROADS and dTIMS

In 1998 New Zealand adopted the software platform dTIMS from Deighton Associates for the predictive modelling of pavement deterioration. The basic models used in dTIMS are derived from the Highway Design and Maintenance Standard Series (HDM) models.

Presented in AUSTROADS (2004a) is 'Figure 8.4' which for a given subgrade and design traffic prescribes the thickness of a granular overlay. The coefficients used in the HDM III roughness model are the same as those used in the New Zealand dTIMS roughness models. Therefore, the HDM III roughness model can be used as a substitute for that of dTIMS.

By setting a failure roughness, the HDM model for roughness can be used to determine a pavement structure for the variables of design traffic and subgrade CBR. The methods used to determine this pavement structure are discussed in the following paragraphs and displayed schematically in Figure 4.2.

For the HDM model, in keeping with AUSTROADS design methodology, the basecourse thickness is given by Equation 4.2 using a CBR = 30%, while the remainder of the granular thickness is composed of a sub-base with a modulus equal to 250 MPa. In the design method based on HDM the basecourse has a modulus of 500 MPa, which is the value of a presumptive basecourse modulus from AUSTROADS. It should be noted that the mechanistic method from AUSTROADS, for design of new flexible pavements, requires a basecourse modulus of 350 MPa to produce design agreement with 'Figure 8.4' of AUSTROADS. The initial pavement roughness is 60 NAASRA and the terminal roughness has been assumed as 150 NAASRA.

Figure 4.3 presents the total granular design thicknesses for both AUSTROADS' 'Figure 8.4' and the design method that uses the HDM roughness progression model to determine the pavement structure. This combined thickness includes both a basecourse component (shaded background) and sub-base component (clear background).

Assume initial roughness of 60 NAASRA.
Assume terminal roughness of 150 NAASRA.
Assume design life of 25 years.

Determine *SNC* by using the above assumptions and solving equation 4.2 for a range of design traffic levels from 10^5 to 10^7 ESA.

Determine basecourse thicknesses, T_{BC} for various design traffic levels, from 10^5 to 10^7 ESA, using the equation

$$T_{BC} = \left[219 - 211.\log 30 + 58\log^2 30\right]\log(DESA/120)$$

Assuming a pavement structure of a basecourse (BC), a sub-base (SB), and a subgrade the structural number is given by

$$SNC = \frac{0.14}{25.4}T_{BC} + \frac{0.12}{25.4}T_{SB} + SN_{SG}$$

Where T_{SB} is the thickness of the sub-base. With both the structural numbers and the T_{BC} having already been determined, then the above equation can be solved for sub-base thickness

$$T_{SB} = \frac{25.4}{0.12}\left(SNC - SN_{SG}\right) + \frac{0.14}{0.12}T_{BC}$$

Since the value of SN_{SG} is given by equation 3.6 this equation can be evaluated for the various subgrade CBR values.

The pavement design thickness is then the sum of T_{BC} and T_{SB}.

Figure 4.2 Process for evaluating the pavement thickness using the initial and terminal roughness, from the HDM roughness progression model, as the design criteria.

From Figure 4.3 it can be seen that for a design traffic of 10^5 ESA AUSTROADS predicts greater granular overlay than that required by the design according to the design based on HDM roughness described in Figure 4.2. In fact, for this design traffic, except for a subgrade CBR of 3, the basecourse thickness of approximately 99 mm is considered by the HDM model to be sufficient without a sub-base component.

If the design traffic is calculated from the HDM III roughness model assuming the AUSTROADS' design thicknesses for design traffic of 10^5 ESA, then, for all CBR values greater than 5% the HDM model predicts traffic greater than 10^6 ESA. Assuming a design life of 20 years for a DESA of 10^5 ESA then it could be presumed that if the achieved traffic life is an order of magnitude greater then the life, in years, should be an order of magnitude greater.

Assuming that the HDM model is providing a realistic prediction of pavement performance, it follows that Figure 4.3 is consistent with the statement made at the pavement forum that AUSTROADS is overly conservative for traffic levels of 10^5 ESA.

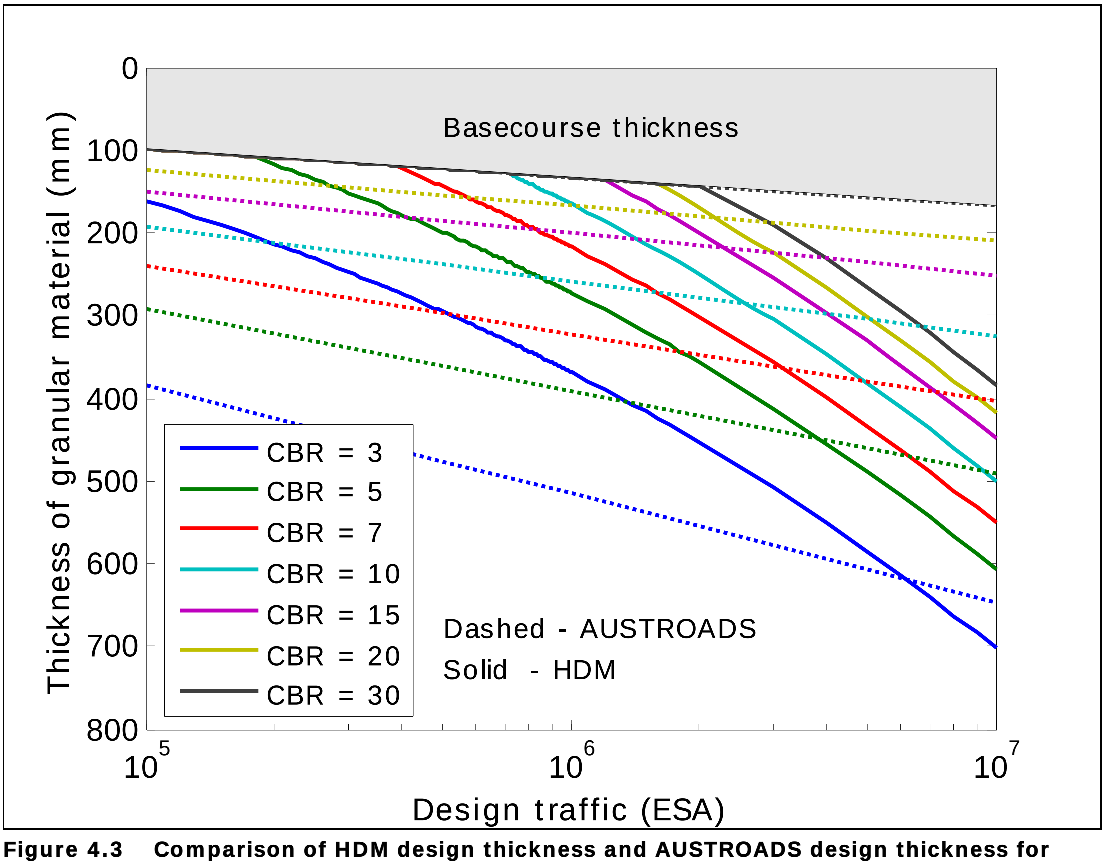

Figure 4.3 Comparison of HDM design thickness and AUSTROADS design thickness for various subgrade CBR values and design traffic levels.

For high design traffic of 10^7 ESA AUSTROADS predicts a thinner pavement than the HDM model which is the opposite of predictions for lower trafficked roads. The range of 2×10^6 to 7×10^7 ESA is the region where there is the closest agreement between AUSTROADS and the pavement designed according to the HDM roughness model. Within this range the point of agreement between the different methodologies shifts to lower design traffic for increasing CBR.

If the terminal roughness is reduced in the models the effect is to increase the pavement thickness for a given design traffic value; in effect, reducing the terminal roughness shifts the HDM curves to the left of Figure 4.3. As might be expected, an increase in the sub-base modulus results in a reduction in required sub-base thickness while a reduction results in an increased sub-base thickness.

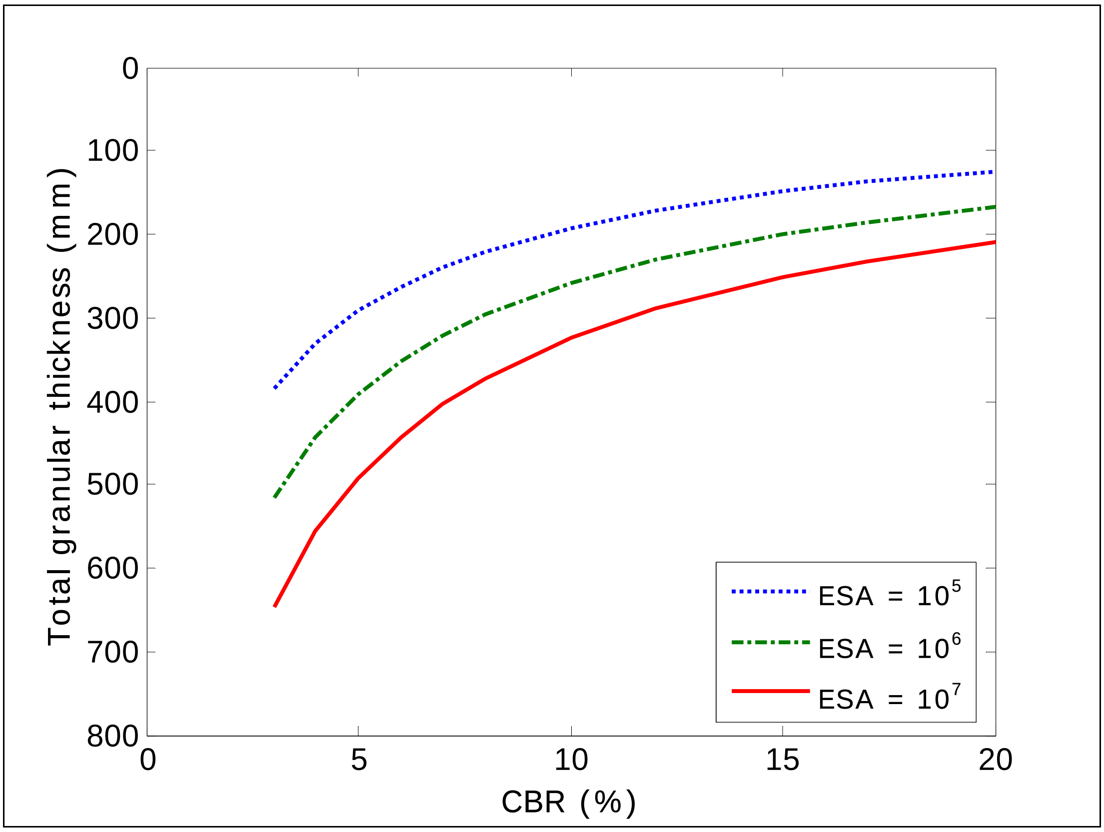

Figure 4.4 Sensitivity of AUSTROADS' flexible pavement design to CBR.

The AUSTROADS' 'Figure 8.4' is redrawn in Figure 4.4 with the horizontal axis being the subgrade CBR rather than design traffic. The figure emphasises the sensitivity of the design to CBR rather than to the design traffic. From Figure 4.4, it can be seen that for CBR = 5%, an increase in design traffic from 10^5 to 10^7 ESA can be accommodated by an approximate thickness increase of 200 mm. At a CBR of 20% a change of thickness of 100 mm is sufficient.

4.4 Subgrade strain criteria

4.4.1 General

As mentioned earlier, care must be taken when comparing subgrade strain criteria (Wardle et al., 2003). That being noted various subgrade criteria are discussed and compared in Chapters 4.4.2 to 4.4.6. Furthermore, these equations have been assumed to be invertible; however, given that they have been derived using regression analysis the validity of using the equations in this form needs to be treated with caution.

4.4.2 AUSTROADS

The subgrade strain criterion from AUSTROADS (2004a), Equation 2.9, can be rearranged such that:

$$\varepsilon = 9.3 \times 10^{-3} N^{-1/7} \tag{4.3}$$

In this context ε, in units of strain, can be regarded as the limiting design strain for a total design traffic loading of N. For Equation 4.3, the maximum allowable subgrade strains, ε_{SG}, are 0.0018, 0.0013 and 0.0009 for traffic levels of 10^5, 10^6 and 10^7 ESA respectively.

4.4.3 Strain criterion of Pidwerbesky et al.

Pidwerbesky et al. (1997) proposed an alternate subgrade strain criterion, namely Equation 4.4 where units are the normal units of strain.

$$\varepsilon = 0.012 N^{-0.145} \qquad (4.4).$$

Rearranging this equation gives the number of allowable loads in terms of the subgrade strain:

$$N = \left(\frac{0.012}{\varepsilon}\right)^{6.8966} \qquad (4.5).$$

When Equation 4.5 is used with the software package CIRCLY as a subgrade performance criterion it results in pavement thicknesses as displayed in Table 4.2.

Table 4.2 Comparison of pavement design thickness for two subgrade strain criteria.

Pavement 1: E_{BC} = 350 MPa, Subgrade CBR = 10%			
Traffic (ESA)	Design thickness (mm)		Difference (mm)
	AUSTROADS	Pidwerbesky et al.	
10^5	200.5	172.9	27.6
10^6	248.8	214.5	34.3
10^7	318.1	267.9	49.5
Pavement 2: E_{BC} = 350 MPa, Subgrade CBR = 5%			
Traffic (ESA)	Design thickness (mm)		Difference (mm)
	AUSTROADS	Pidwerbesky et al.	
10^5	229.5	200.0	29.5
10^6	284.4	245.1	39.3
10^7	374.0	309.5	64.5

So for traffic volume designs of 10^5, 10^6 and 10^7 ESA, the strain criteria proposed by Pidwerbesky et al*.* indicate that thinner pavements can be used when compared to the normal AUSTROADS subgrade strain performance criteria.

4.4.4 Salt & Stevens' subgrade soil strain

Salt & Stevens (2001) have suggested that subgrade strain criteria for volcanic soils may be conservative by at least a factor of 1.5 and in some soils by up to a factor of 2. The potential increase in allowable traffic was mentioned but the other potential consequence of higher allowable subgrade strains is the reduction of granular overlay for a given design traffic level. Obviously the construction of thinner pavements would increase the strain transmitted to the subgrade but if the subgrade could sustain higher strains there are potential economic benefits.

Assuming the allowed strain in Equation 2.9 is increased by a factor of 1.5 it follows that the equation becomes

$$N = \left[\frac{13.95 \times 10^{-3}}{\varepsilon_{Salt}}\right]^7 \qquad (4.6)$$

This can be rearranged to give the equation

$$\varepsilon_{Salt} = 0.01395 N^{-1/7} \qquad (4.7).$$

Equation 4.7, where ε_{Salt} is in units of strain, has been used to generate the curve labelled 'Salt' in Figure 4.6. If used as a performance criterion with CIRCLY, Equation 4.7 also predicts thinner pavements for volcanic subgrades.

4.4.5 Strain criteria of Hallett & McIlroy

Concurrent to the derivation of Equation 4.5, Hallett & McIlroy (2003) came up with a very similar equation based on rut performance of roads from the Western Bay of Plenty.

$$N = \left(\frac{0.012}{\varepsilon}\right)^{7.14} \qquad (4.8).$$

It should be remembered that Equations 4.7 and 4.8 were derived for specific subgrade conditions and are not material independent.

4.4.6 HDM equivalent subgrade soil strain

An effective subgrade strain experienced by pavements designed according to the HDM models was evaluated. This was compared with the subgrade strain criterion of AUSTROADS (2004a) and with the criteria from Pidwerbesky et al. (1997), Hallett & McIlroy (2003), and Tonkin & Taylor (1998).

A failure roughness of 120 NAASRA was determined from Transit's target roughness values for pavements trafficked by between 4000 and 10 000 vehicles per day. An initial roughness of 60 NAASRA was selected as the initial pavement roughness resulting in a roughness progression of 60 NAASRA over the 25 year design life of the pavement. For the modelling it was assumed that roughness evolution determined the pavement life since HDM predicts very little rut progression for New Zealand roads. A Structural Number was determined that would produce such roughness evolution, according to HDM roughness progression models, for the three traffic loads of 10^7, 10^6, and 10^5 ESA. A pavement was then designed that produced the required Structural Number. Design assumed a single layer of basecourse on top of a subgrade. Various subgrade CBR values were designed for and a pavement thickness was determined assuming a basecourse modulus of 350 MPa.

Using CIRCLY the subgrade strain was determined for the pavements designed according to the above methods. A critical subgrade strain (ε_c), according to HDM models, was determined for the various traffic loadings and subgrade CBR values. Results are displayed in Figure 4.5a, which demonstrates that the subgrade strain is not constant for a particular traffic level. Furthermore, for a traffic loading of 10^5 ESAs it can be seen that the subgrade behaviour is quite different to that for the 10^6 ESA and 10^7 ESA curves. It is considered likely that this is due to the thin basecourse layers required for the HDM model thicknesses, for these thicknesses the CIRCLY modelling breaks down. Taking representative strains for each of these traffic loadings, namely 6×10^{-3}, 1.5×10^{-3}, and 0.5×10^{-3} for loadings of 10^5 ESA, 10^6 ESA, and 10^7 ESA respectively, and plotting these points with the various strain criteria it can be seen in both Figure 4.5b and Figure 4.6 that HDM allows quite different subgrade strains. The equation that describes the HDM subgrade strains is presented as Equation 4.9.

$$\varepsilon = 2.949N^{-0.542} \tag{4.9}$$

Where: units of ε are strain and N are ESA.

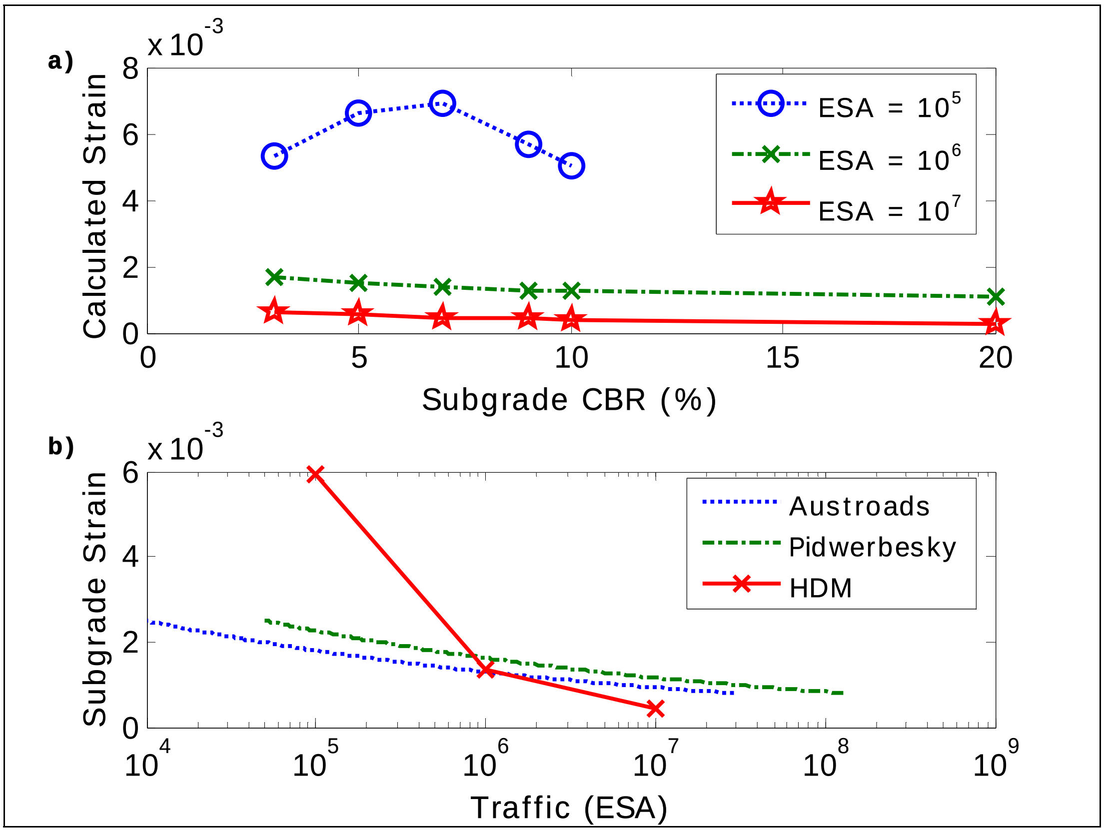

Figure 4.5 a) Calculated subgrade strain, for single standard axle, assuming construction governed by HDM; b) Various subgrade strain criteria including the average subgrade strain from HDM governed construction.

Figure 4.5(b) has been redrawn in Figure 4.6 as a log-log plot. Also included are additional curves: the curve derived from Salt & Stevens (Salt), Hallett & McIlroy's (Hallett) and a curve obtained from the South African (SA) design methodology for a pavement failing with a 20 mm rut. The subgrade strain criteria derived from the HDM roughness model behaves dramatically differently compared to the other curves. Furthermore, it can be seen that the South African slope is different to that of AUSTROADS, Pidwerbesky and the equation obtained from Salt & Steven's statement. The South African subgrade strain criteria is more conservative than all the other criteria except for the derived HDM criteria for traffic greater than 2×10^6 ESA.

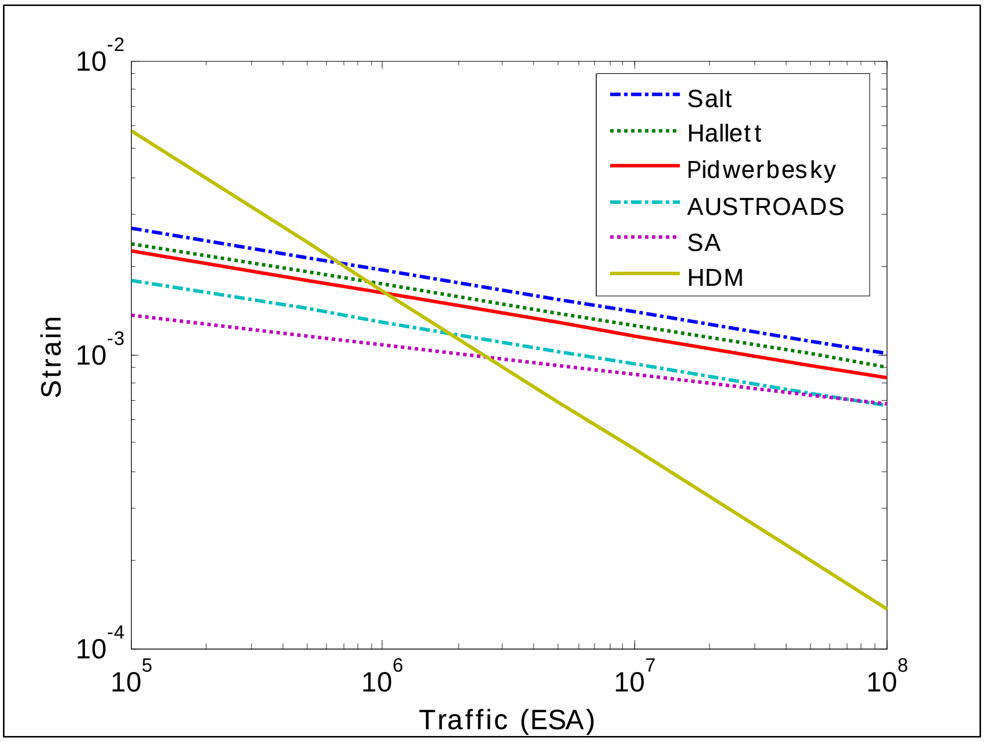

Figure 4.6 Various strain criteria as functions of the design traffic.

4.5 Sensitivity analysis

For pavements designed according to AUSTROADS the pavement performance is sensitive to the subgrade CBR; an error in the measurement of the *in situ* CBR could result in an over- or under-designed pavement.

Central Laboratories have produced a report (Patrick & Dongal, 2001) that documents the precision errors for measuring pavement thickness, CBR, and FWD deflection measurements. The sources of error have been obtained from various references and are displayed in Table 4.3. In terms of 'Figure 8.4′ the important errors are ±1.5 for measured CBR values less than 10% and ±2.5 for measured CBR between 10 and 30%.

Table 4.3 Error associated with *in situ* measurement of CBR.

Measured CBR (%)	Measurement error
<10	±1.5
Between 10 and 30	±2.5
Between 30 and 60	±5,
Greater than 60	±12.5

Displayed in Table 4.4 are the life expectancies predicted by CIRCLY for a pavement designed according to 'Figure 8.4′ from AUSTROADS (2004a). Total pavement thickness has been determined through the use of Equation 2.3 for the design CBR. Basecourse (BC) thickness is determined using Equation 2.3 and assuming the sub-base (SB) provides a CBR of 30% and the sub-base making up the remaining thickness.

Table 4.4 Sensitivity of DESA to *in situ* CBR.

Design CBR	DESA	Design thickness (mm) AUSTROADS	BC thickness (mm)	SB thickness (mm)	Error in *in situ* CBR (%)	Actual CBR (%)	Loading life according to CIRCLY (ESA)
5	1 000 000	391.5	132.8	258.7	-1.5	3.5	9.80E+04
					0	5	1.19E+06
					+1.5	6.5	6.25E+06
10	1 000 000	258.8	132.8	125.9	-2.5	7.5	2.13E+05
					0	10.0	1.68E+06
					+2.5	12.5	7.70E+06

Notes to Table 4.4:

(a) Pavement is assumed to have E_{BC} = 500 MPa and E_{SB} = 250 MPa with both layers having a Poisson's Ratio of 0.35.

(b) Poisson's Ratio is 0.45 for the subgrade and it is assumed that E_{SG} = 10 × CBR (MPa).

For a design CBR = 5 ± 1.5%, CIRCLY predicts a design traffic life that ranges by greater than a factor of 60. Similarly, for design CBR = 10 ± 2.5% the lower predicted traffic life is 36 times smaller than the greater life. These great variations in predicted traffic life are, according to ASTM (1984), the product of typical field measurement errors. These measurements of precision are not present in ASTM (2004), which states that no method presently exists to evaluate the precision of a group of non-repetitive plate load tests on soils and flexible pavement components because of the variability of these materials. It should be noted that the errors quoted here are for individual CBR measurements; in practice the error will be reduced by taking multiple measurements prior to any pavement design.

The same pavements examined in Table 4.4 were analysed with the South African software package mePADs, the results are displayed in Table 4.5. The climatic setting was set to moderate and all other settings were equivalent to New Zealand conditions. When pavements designed according to the AUSTROADS design guide are analysed using the mePADs software package the South African predicted lives, in ESA, are greater than the AUSTROADS design lives. Furthermore, with one exception, the predicted lives are greater than those calculated using CIRCLY, sometimes significantly so. Interestingly, for the South African software, the climatic setting does not affect the bearing capacity of the subgrade. Instead, these settings have the greatest influence on the sub-base and a reduced effect on the basecourse bearing capacity. As can been seen in the table, mePADs only predicts the subgrade layer to determine the life of the pavement for the weakest pavement; the lives of all the other pavements are determined by the sub-base. This is in contrast to the design assumptions of AUSTROADS (2004a) where the accumulation of subgrade strain determines the life of a pavement.

Table 4.5 Comparison between calculated life using CIRCLY and mePADS using the same pavements as presented in Table 4.4.

Total thickness (mm)	BC thickness (mm)	SB thickness (mm)	Subgrade CBR (%)	CIRCLY life (ESA)	mePADS life(ESA)
391.5	132.8	258.7	3.5	9.80E+04	3.00E+07 (SG)*
391.5	132.8	258.7	5	1.19E+06	1.30E+08 (SB)
391.5	132.8	258.7	6.5	6.25E+06	1.60E+08 (SB)
258.8	132.8	125.9	7.5	2.13E+05	4.00E+06 (SB)
258.8	132.8	125.9	10.0	1.68E+06	5.00E+06 (SB)
258.8	132.8	125.9	12.5	7.70E+06	6.00E+06 (SB)

*The bracketed expressions in the mePADS life column represent the subgrade (SG) and sub-base (SB) and denote the pavement layer that, according to mePADS, determines the life of the pavement.

For typical measurement errors in the CBR, CIRCLY predicts a dramatic variation in achieved lives. In contrast, the variation in predicted traffic life according to mePADS changes by relatively small amounts for the same CBR variations.

Displayed in Figure 4.7 is the Cumulative Damage Factor (CDF) calculated by CIRCLY for the pavements displayed in Table 4.6. The pavement thickness was determined using 'Figure 8.4' from AUSTROADS (2004a) and it is assumed that the basecourse modulus is 350 MPa. The life is calculated according to Equation 2.9 based on the subgrade strain. Despite the design thicknesses being calculated from AUSTROADS' 'Figure 8.4' but assuming a single structural layer with a modulus of 350 MPa it can be seen that the life is not 10^7, as would be expected from the design procedure, but instead ranges from 91% to 205% of the design life. If a basecourse and sub-base are assumed with the sub-base having a modulus of 250 MPa, then the critical strain and damage factors both reduce and the life increases.

Of particular interest, for low subgrade CBR the cumulative damage factor is low and the calculated life is high. Assuming the subgrade strain criteria from AUSTROADS (2004a) are appropriate the implication is that for a low CBR 'Figure 8.4' is highly conservative. When a traffic multiplier of 1.2 is used, as per Chapter 8.3.1 of AUSTROADS (2004a), while the damage factor changes the life does not. This is because the design life is determined by the subgrade strain according to Equation 2.9.

Table 4.6 Damage factors and pavement lives according to CIRCLY for thickness designs based on AUSTROADS (2004a).

AUSTROADS design traffic = 10^7 ESA			Traffic multiplier = 1	Traffic multiplier = 1.2	
CBR (%)	**Thickness (mm)**	**Critical strain ($\times10^6$)**	**Damage factor**	**Damage factor**	**CIRCLY life (ESA$\times10^7$)**
20	210	915.8	0.897	1.0774	1.11
15	251	905.1	0.827	0.9925	1.21
10	325	904.7	0.824	0.9892	1.21
7	404	943.4	1.106	1.3260	0.91
5	491	934.2	1.032	1.2381	0.97
4	556	911.4	0.868	1.0416	1.15
3	647	878.8	0.673	0.8070	1.49
2	791	839.5	0.489	0.5861	2.05

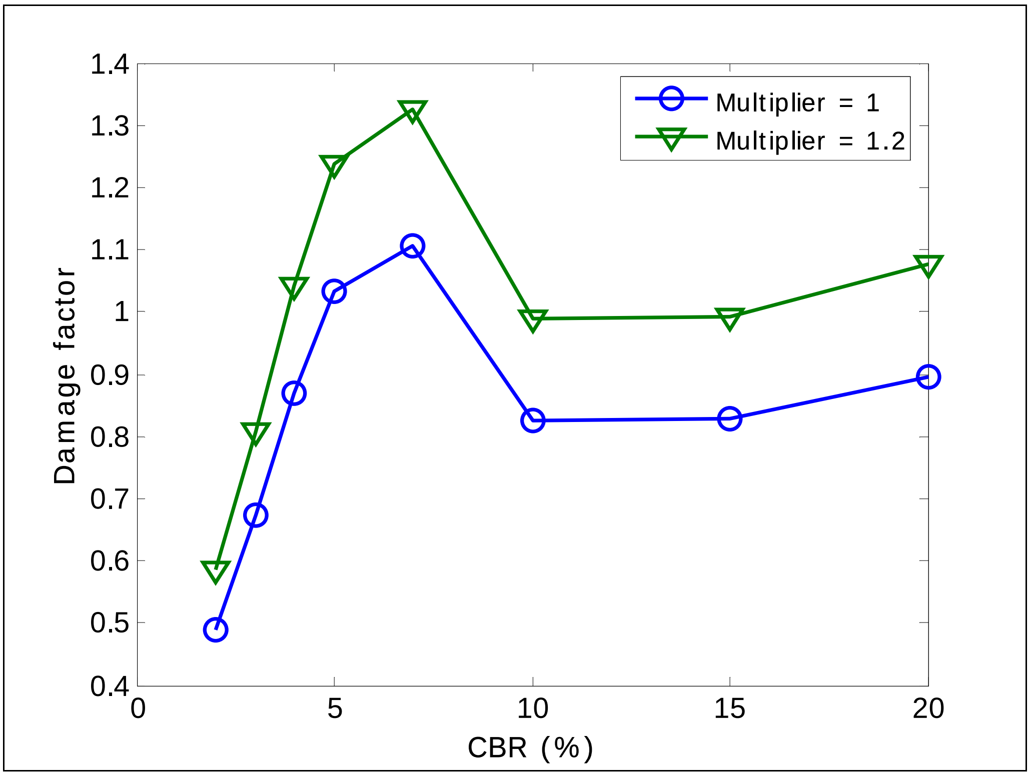

Figure 4.7 Plot of CBR versus Cumulative Damage Factor for data from Table 4.6.

4.6 Summary

Adherence to the design methodology presented in AUSTROADS (2004a) produces roads that generally perform satisfactorily. AUSTROADS, however, assumes that plastic strain in the subgrade is the factor that determines the life of the pavement. Numerous examples exist that demonstrate that, in addition to the subgrade, plastic deformation occurs in the basecourse and sub-base.

AUSTROADS design methodology assumes that the vertical elastic strain applied to the subgrade dictates the allowable traffic. Alternative subgrade strain criteria have been suggested, each criterion being less conservative than that presented in AUSTROADS (2004a). A significant factor in New Zealand pavement design is the performance of volcanic subgrades, which can sustain relatively high elastic strains with little plastic strain accumulation.

As mentioned earlier, the resilient modulus is not closely related to permanent deformation (Alabaster 2000) and this relationship is influenced by moisture content (Dodds et al. 1999, and Khogali & Mohamed 2004). AUSTROADS assumes a relationship between the elastic strain of the subgrade and the life of a pavement, with the accumulation of permanent strain causing the failure of the pavement. Given the uncoupling of plastic and elastic strain observed by the above researchers and the distribution of plastic strain though the pavement, a better characterisation of the plastic behaviour of pavement materials would assist in pavement modelling.

When the design curves of 'Figure 8.4' from AUSTROADS (2004a) are compared to design curves obtained from the HDM roughness model it appears that AUSTROADS over-designs for traffic levels of 10^5 ESA but under-designs for 10^7 ESA. The two methods approximately agree for design traffic between 2×10^6 and 4×10^6 ESA.

Using the AUSTROADS subgrade strain criteria it has been shown that the CIRCLY package predicts a large variation in pavement lives for changes in subgrade CBR that are representative of errors in CBR measurement. The South African package mePADS is not as sensitive to variations in subgrade CBR and, for the roads examined, generally predicts failure in the sub-base instead of the subgrade.

With the South African design method, pavement design choice is dictated by the performance of individual layers. According to the South African design methodology the pavement life is highly sensitive to the choice of climate. Similarly to AUSTROADS, the bearing capacity of the subgrade is empirically determined based on the vertical strain experienced at the sub-base–subgrade interface. Pavements designed according to AUSTROADS were analysed using the mePADs package; this suggested that the performance of these pavements would be better than that predicted by AUSTROADS. Assuming that the climatic choice is appropriate for New Zealand, this might help explain the observations that AUSTROADS under-predicts the pavement life by between 5 and 20 times (Bailey et al.2004).

It is felt that, especially for pavements with a DESA $=10^5$, pavements constructed 50 years ago in New Zealand have shaken down into a stable stage. What appears to happen is that some sort of age-related failure mechanism that is not fully understood has been causing these pavements to fail.

5 Conclusions

AUSTROADS does not specifically design for plastic deformation in the basecourse. However, both experiments and field observations demonstrate that, with sufficient traffic loading, plastic deformation accumulates in the basecourse, sub-base and the subgrade.

Currently, pavement design in New Zealand is critically dependent on subgrade CBR. Both of the subgrade strain criteria proposed by Pidwerbesky et al. (1997) and derived from Salt & Stevens' (2001) statements allow greater, when compared with AUSTROADS, subgrade strains for New Zealand conditions, the latter albeit is only valid for volcanic soils. While the use of these relaxed subgrade strain criteria might be useful in the short term, ultimately the linking of plastic strain with resilient modulus should be discontinued. Moisture influences plastic deformation much more dramatically than the resilient modulus (Khogali & Mohamed 2004). A better characterisation of the plastic behaviour of pavement materials would assist in pavement modelling and ultimately in characterising pavement performance.

The roughness model from HDM III indicates that for design traffic levels of 10^5 ESA AUSTROADS' subgrade strain criteria is highly conservative, while for design traffic levels of 10^7 ESA AUSTROADS is not conservative enough. From another perspective the HDM model indicates that AUSTROADS pavement thickness is excessive for low traffic of 10^5 ESA while the 10^7 ESA traffic levels require greater aggregate cover. The results for design traffic between 10^5 and 10^6 ESA might help explain the observation that lives greater than 50 years are being achieved in New Zealand (Bailey et al*.* 2004) since, assuming the modelling is correct, in effect, these roads have been over-designed.

Indications are that for a pavement designed according to AUSTROADS the maximum possible traffic loadings are significantly greater than the DESA of 10^5. However, further calibration is recommended.

Given that the AUSTROADS (2004a) design method only allows for pavement failure in the subgrade it is suggested that the incorporation of the South African design methodology, which utilises empirically determined layer performance models for the unbound granular layers as well as the subgrade, into New Zealand design is worth investigation, particularly given the similarity in the road construction practices of the two countries. Any investigation would require the incorporation of material characterisation from the laboratory, CAPTIF trial, and field trials, and would also require the accurate modelling of pavement behaviour.

6 Recommendations

Most roading data are for traffic levels less than 10^6 ESA and there are little data for 10^7 ESA or greater. Therefore, it is recommended that the subgrade strain criteria derived from the HDM roughness model be considered for adoption for traffic levels less than and equal to 10^6 ESA. Prior to any implementation, the validity of the proposed subgrade strain criteria should be determined by examining the performance of actual roads. For traffic levels greater than 10^6 ESA, verification is recommended.

Research on predicting rutting in the aggregate layers of pavements should be incorporated in pavement design. Dr Greg Arnold is currently conducting a study which may provide a methodology to predict the rutting potential of aggregate types.

From the literature it appears that the predominant failure indicator is roughness. What are not clear, however, are the physical mechanisms that drive the increase in roughness. Two potential mechanisms are shallow shear and differential settling because of inconsistent compaction. It would be beneficial to determine which of these two, if either, is the dominant mechanism in roughness evolution.

The literature indicates that quality control is potentially an area where improved pavement performance could be obtained. In the New Zealand context what is unclear is whether pavements are being constructed to a reasonable quality level or if the construction specifications are inadequate. Monitoring of quality control during pavement construction should be conducted. If this indicates potential for improving process quality control then there is potential to positively influence pavement construction quality and thereby the achieved pavement life. In doing so the top ranked issue of minimising post-construction rutting, as per the Industry Forum, would likely be addressed. Furthermore, the potential exists for the use of initial high rates of accumulation of permanent deformation to be used as an indicator of inadequate construction quality.

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Appendix A Industry Pavement Forum: comments from 1st December 2004 meeting.

A1 Participants

On the 1st of December 2004 an industry forum was conducted at the Brentwood Hotel in Wellington. The attendees were:

- Bruce Steven (University of Canterbury)
- Mofreh Saleh (University of Canterbury)
- Frank Bartley (Bartley Consultants)
- Ross Peploe (Bartley Consultants)
- John Hallett (Beca)
- Ken Hudson (Duffil, Watts and Tse)
- Graham Salt (Tonkin & Taylor)
- Bryan Pidwerbesky (Fulton Hogan)
- Dave Hutchison (Works Infrastructure)
- Allen Browne (Opus International Consultants)
- Martin Gribble (Opus International Consultants)
- John Patrick (Opus International Consultants)
- Clarence Morkel (NZIHT)
- Dave Alabaster (Transit New Zealand)
- Greg Arnold (Transit New Zealand)
- Norm Major (retired at time of conference[1])

This meeting fulfilled one of the tasks for the research programme. To give an idea of the scope of this forum, this Appendix contains the outcomes that were identified and discussed on the day and notes made from each speaker's presentation.

A2 Outcomes

A list of outcomes was distributed for ranking by the participants. The following are the outcomes as decided by the Forum including short descriptions.

1. Default values for modulus, fatigue etc.

This indicated that there is a need to characterise modulus values, fatigue behaviour and other pavement characteristics for New Zealand pavements to be used in design. Significant work in these areas has already been conducted; the publication of these results in a single source is perhaps justified but the results of laboratory tests should not be used directly in design.

2. Shear in the granular layer

Shallow shear in the granular layers was identified as a significant failure mechanism that justifies further research.

[1] Norm Major died in 2006 while this report was being prepared for press.

3. Models be based on observed performance

Models used in pavement design need to be further calibrated against observed performance in the field.

4. Stopping post-construction rutting

Rutting occurring soon after roads are opened to traffic, especially on Greenfield sites, was discussed as a significant problem that justified further attention.

5. Good relationship between theory and measured response in CAPTIF

Pavement models need to describe the measured pavement response to loading at CAPTIF.

6. CAPTIF 40-60% of surface deformation in subgrade

Not all pavement deformation occurs in the subgrade. Field observations, CAPTIF experiments and finite element models all support this statement. What is to be done with this knowledge?

7. Age effects

Many pavements appear to 'die' when about 50 years old. Does significant degradation of materials that contribute to this?

8. Rehabilitation use of curvature

It was suggested that an understanding of what drives the rehabilitation of pavements would be desirable. One methodology suggested was the use of the pavement curvature under FWD loading which could gave an indication of performance appropriate to both granular as well as bound pavements. There are also a number of other techniques available to indicate the need for rehabilitation.

9. Isotropic/anisotropic

AUSTROADS' use of anisotropic moduli contrasts with design methodologies used elsewhere. The proposal was tabled that the disadvantages of using anisotropic moduli outweigh the advantages.

10. Cost of alternatives

The whole-of-life cost of alternative materials should be used in evaluating alternative materials for use in pavement construction.

11. Modelling of thin asphaltic cement, shift factors, and fatigue life

Further information was considered desirable.

12. Reliability factors

Guidance in the concepts and use of reliability factors was considered necessary.

13. Stabilisation

Further research is needed in the use of subgrade and basecourse stabilisation to produce long lasting pavements, particularly with respect to substandard materials.

14. Design into construction practice

The implementation of pavement design specifications into actual construction was identified as an area of concern.

15. Risk management

The incorporation of risk management techniques into pavement design was identified as a desirable component of pavement design.

16. Maximum traffic loading possible on granular pavement coupled with intensity

Where is the limit for granular pavements with respect to maximum load and maximum traffic numbers?

17. What to do about wearing courses?

It was suggested that the greatest needs were in:

- Construction control (especially for Greenfield sites),
- material shear (rehabilitation) for modelling basecourse rutting,
- material durability, and
- alternative materials characterisation and incorporation into design.

The current design method was acknowledged as being primarily acceptable. This is provisional on the acknowledgement of the limitations of assumptions. There was general acceptance that subgrade cover is about right.

The ranking of the 17 Outcomes was received from a number of attendees and these rankings are displayed in Table A1.

Table A.1 Forum participants' rankings of issues facing pavement design in New Zealand.

Ranking	Original order ranking	Description	Norm Major	Frank Bartley	Allen Browne	Greg Arnold	Graham Salt	John Patrick	Ken Hudson	John Hallet	Martin Gribble	Weighting
1	4	Stopping post-construction rutting	5	1	2	1	1	3	–	3	3	**2.4**
2	2	Shear in the granular layer	3	5	1	2	1	2		4	4	**2.8**
3	16	Maximum traffic loading possible on granular pavement coupled with intensity	2	7	4	3	–	9	1	7	6	**4.9**
4	3	Models be based on observed performance	1	2	12	12	–	1	–	9	1	**5.4**
5	13	Stabilisation	8	6	17	5	3	6	1	2	7	**6.1**
6	14	Design into construction practice	4	3	3	7	–	13	–	11	5	**6.6**
7	5	Good relationship between theory and measured response in CAPTIF	11	10	5	–	–	4	–	13	2	**7.5**
8	15	Risk management	12	8	10	4	–	7	–	7	–	**8.0**
9	1	Default values for modulus fatigue etc.	6	12	8	14	5	12	1	8	–	**8.3**
10	11	Modelling of thin AC shift factor, fatigue life	15	4	7	8	–	10	–	6	–	**8.3**
11	9	Isotropic/anisotropic	7	15	13	13	3	15	–	1	–	**9.6**
12	7	Age effects	14	12	11	10	–	5	–	15	–	**11.2**
13	12	Reliability factors	9	9	15	15	–	8	–	12	–	**11.3**
14	6	CAPTIF 40-60% of surface deformation in subgrade	10	11	6	–	–	17	–	14	–	**11.6**
15	17	What to do about wearing courses	16	17	9	9	–	16	–	5	–	**12.0**
16	8	Rehabilitation (use of curvature as one possible example)	15	12	16	11	–	10	–	10	–	**12.3**
17	10	Cost of alternatives	17	16	14	6	–	13	–	16	–	**13.7**

A3 Comments

The following section includes the comments that were made by various attendees at the forum.

A3.1 David Hutchison

As requested, the main points that Hutchison wished to be considered were:

- that pressing issues are 'near surface' ones (shallow shear and effect of water);
- consequently, effort on refining subgrade strain criteria should be left alone;
- there is a need to review the basis for design of structural asphalt; and
- there is a need to promote the alternative of an alternative pavement design (compared with the 'special study' approach admitted for structures in NZS4203).

With regard to the first bullet point, Hutchison suggested during subsequent discussion that a pavement design guide needs to consider all modes of failure, as structural codes do (present emphasis is on subgrade compressive strain and asphalt flexural strain).

A3.2 Norman Major

- Pavement design is concerned with predicting when unacceptable pavement distress will occur.
- Deterioration rates vary from point to point in a pavement. The worst small portion will determine what is an acceptable overall state for the pavement section.
- 1% of distressed pavement surface is a large (1 square metre) chunk of distress on average every 12 metres along the section. This looks (and is) a large area of distress by pavement standards.
- Because extremes govern overall performance, uniformity (by reducing deviation of the bad bits from the average) will allow a pavement with poorer average competence to give satisfactory service.
- A practitioner design method can be simply use of a formula, but is better if it appears to be a physical model against which the user can measure observed field performance. To be effective, the design method must provide for possible modes of significant distress. Provision for some modes may be very simple. For example, provision against weakening of the upper subgrade by saturation is commonly provided for by having no ponding areas on the upper surface of the subgrade.
- A practitioner design method should be based on observed behaviour of constructions of the type and materials used in the signed-off design. Limits within which the design procedure is valid (e.g. material properties, layer dimensions, material compatibilities, conditions necessary for satisfactory construction, etc) are essential.
- Some conceptually possible modes of distress may be avoided or ameliorated by construction techniques (e.g. minimising in-service densification of upper granular layers by trafficking before finally trimming the surface shape and sealing).
- There is no virtue in more exact design if no benefits accrue from it. For example, in a central urban street, reinstatement of a trench opening may be better carried out by using an overthick replacement pavement which can be

placed quickly, than by carefully constructing the backfill or sub-base to a level leads to the pavement thickness being within 10mm of too thin.

- The effect of design on field performance cannot be determined if as built properties and dimensions are not available.
- Makers of lenses for spectacles do not need the corpuscular theory of light (CTL) to design new shapes and with new materials, even though the people who provided them with the design procedures knew all about CTL.

A3.3 Bryan Pidwerbesky

- Eighteen years ago, more engineering was used in pavement design. The swing is now too scientific, too much of a blackbox.
- The CBR vs. modulus relationship can be anywhere from 5–20 MPa and this should be remembered when using the AUSTROAD modulus = 10CBR.
- Subgrade strain criteria are not valid in practice.
- Pidwerbesky has never observed subgrade failures except when the pavement subgrade is wet.
- Relating design to construction, these two processes are inseparable.
- Key factors for unbound granular pavements:
  - material properties,
  - compaction inclusive of density,
  - deflection of each layer,
  - uniformity/variation,
  - design thickness.

A3.4 Allen Browne

Conventional granular pavement design/subgrade failure criterion:
Browne is uncomfortable with the current AUSTROADS flexible pavement design approach for determining the design life and subsequent failure of thin surfaced granular pavements. How confident can we be that terminal rut development is only due to cumulative subgrade strains and plastic deformation as is the basis of AUSTROADS? Brown understand that ‘Figure 8.4’ is based on the interpolation of the performance of a relatively small amount of roads where failure comprises surface rutting – rather than subgrade rutting. It is likely that the contribution of the aggregate towards surface deformation is a function of a number of parameters (principally total granular thickness) but also loading, material strength, grading and compacted density. Is the use of specifications such as M/4 and B/2 (Transit, 2005) to ensure that the aggregate acts as an inert durable load spreading device sufficient to provide confidence that subgrade failure criteria alone will represent overall pavement performance and deterioration for the design life.

We have ‛Figure 8.4′ and the CIRCLY software telling us that the addition of another 15 to 20 mm can double the fatigue life of a pavement based on a reduction in cumulative subgrade strain. This seems to make some assumptions over aggregate performance but this is the methodology by which we are expected to undertake design.

Brown has been involved in the assessment of a number of rutted pavements over last six years for both adequate life and prematurely failure. In many cases, where layer profile is mapped through to the subgrade, there is often a significant component of aggregate densification/ rutting in the aggregate. Unfortunately, while this has been documented in several cases, it is very difficult to isolate the root cause of rutting despite extensive investigation.

Recently, we have seen numerous contracts where rutting of new pavements is occurring because of densification. As a result various alternatives have been proposed to M/4, B/2 and construction methodology to mitigate this. Should we be considering the implementation of further variations to materials and construction specifications, design methodology and/or introducing some form of aggregate failure criterion? Alternatively, do we introduce more rigorous specifications for greater than 1×10^7 ESA pavements?

Modelling of non-structural AC surfacing

Should pavement designers be modelling mechanistically for a non-structural surfacing or wearing course? AUSTROADS notes that AC layers less than 40 mm are unreliable and cautions against modelling such layers. A variety of approaches are possible for AC surfacing over a granular base in New Zealand, including: ranging from modelling mechanistically as per AUSTROADS – but doing so for thicknesses <40 mm, modelling moduli properties but without a failure criterion, modelling as an equivalent thickness of aggregate, or excluding the AC surfacing altogether from design profile.

AUSTROADS (2004a) lists a number of concerns in Chapter 8.2.5 and appears to recommend that modelling is not undertaken for non-structural AC wearing course on a granular base. The alternative is modelling the wearing course as an equivalent thickness of aggregate. Designers are limited to selecting a wearing course based on experience and industry standards.

Is modelling the wearing course or thin AC on a flexible base in the 'too hard' basket with no mechanistic (or other) checks on performance except for a particular job mix history or is there another option for the designer?

Structural AC design modulus

Some confusion has arisen over confirmation of which resilient modulus designer use for designs incorporating structural AC. This was discussed at a Roading New Zealand Seminar. As a designer trying to develop representative mix properties based on Bands and published modulus ranges, Brown believed it was important to agree on a best practice method of using mix production test results to provide, ideally, a reference modulus or, at worst, a reasonable envelope.

AC shift factor

The shift factor of 5 introduced in the earlier AUSTROADS draft seems to have disappeared again. While there are adjustments for project reliability in AC fatigue to represent a shift factor and favourable SAR/ESA ratio relative to granular pavement this seems to be a long way from the shift factors proposed by the Shell (1978) manuals.

Super singles
The applied footprint pressure of a super single is greater than that assumed in AUSTROADS (2004a), which can have a big impact on pavement design life/performance. At present super singles are not significant components of traffic profile. Should the effects be ignored, particularly if larger proportions are added to truck fleets?

Conditioning
If M/4 and B/2 are providing a pavement ready to traffic then why are we getting rutting, and why is 'conditioning' having such a profound effect on mitigating early rutting? This would suggest that these specifications are not sufficient to implement AUSTROADS assumptions regarding design to on-the-road performance.

A3.5 Greg Arnold

The pavement design process should not be focused solely on pavement depth. Research at CAPTIF found that increasing pavement depth by 70 mm did not increase the pavement life, although the AUSTROADS design guide indicated a tenfold increase in life. Arnold's Ph.D. thesis which modelled deformation in pavement layers found a similar conclusion. In some cases, increasing the granular thickness will increase the amount of rutting. Pavement designers should simply look up a minimum cover for the subgrade (independent of traffic loading), then determine the amount of traffic loading to obtain a certain rut depth in the granular layers (say 10 mm?) based on permanent strain data from RLT testing or presumptive values. HDM models could also be used to determine the traffic loading to reach a terminal amount of roughness.

Designers should also compare the actual traffic loading with the design traffic loading to determine the frequency at which smoothing treatments will be required. They should then use whole of life costs to compare alternatives like structural asphalt pavements. Using this approach, if the designer wishes to increase the number of traffic loads before a certain rut depth or roughness then the designer could consider stabilising/modifying the aggregate and then testing in a multi-stage permanent strain triaxial test and re-modelling to predict the number of loads to a certain rut depth. Another approach is to stabilise the sub-base to form a working platform for better compaction of the overlying aggregate (i.e. the rut depth model could incorporate a compaction component) as this would extend the life of the pavement because of less rutting as the aggregate is compacted and there will be less aggregate to rut.

If there is a fundamental shift away from pavement depth as a design parameter, then the arguments over modulus values, isotropic versus anisotropic are trivial as their effect is to change pavement depth which will ultimately make very little difference to the pavement life.

Arnold expressed concern for over roads with greater than 1×10^7 axle passes as some granular materials and roads will become rough or rutted before the end of the design life. The design process which he has suggested should guard against this.
When designing a pavement a range of pavement types need to be considered; examples include structural asphalt, concrete, and stabilised sub-base with granular overlay. The best option should be based on risk management and whole of life costs that include road

user costs incurred during construction, maintenance, rehabilitation and a component which includes the risk of premature failure.

Interpretation of the AUSTROADS design guide, presumptive values, and all the other assumptions need to be consistent. A working group needs to form a consensus on such things as wearing courses, foam bitumen stabilisation, modified versus bound.

A3.6 Ken Hudson

Hudson considers that AUSTROADS has some flawed concepts for unbound chip sealed pavements but the method is capable of providing good pavement design if the subgrade CBR is greater than 3. Hudson has concerns about AUSTROADS design process for subgrade CBR values less than 3. He considers pavement design remains partially an art.

For unbound chip sealed pavements the subgrade strains are not the issue. For 8 tonne axles and subgrade CBR >3 then the pavement thickness design is fairly idiot resistant. However, Hudson has concerns about aggregate selection and aggregate specification. He considers that New Zealand has done enough research so it is known how to define, evaluate and select a good aggregate. There is a lot of information already existing and a lot of expertise available regarding basecourse aggregates. So the New Zealand problem is education, using what is already known.

Wander of heavies across the pavement is beneficial and New Zealand should consider 4 metre wide lanes on heavily trafficked roads. Consequently research into vehicle wander within 3.5 and 4 metre wide lanes recommended.

Today our challenge is to look to the future.

- We should look to future; so far we have concentrated on today's problems.
- Future will be more trucks.
- Future will be more congestion.
- Future will be fewer quarries.
- Want to know limits of unbound aggregate pavements.
- Want to know limits of modified aggregate pavements.
- Research into asphalt and concrete pavements are for other countries.
- New Zealand should conduct further research on:
  - lime stabilisation,
  - cement stabilisation,
  - Durabind™ stabilisation,
  - high impact bitumen stabilisation,
  - foamed bitumen stabilisation,
  - geotextiles as separators over weak subgrades,
  - grids over weak subgrades.

Pavement designers should be aware that the relationship of Modulus = 10 CBR not always correct. More information and choices are needed for determining the subgrade modulus behaviour. Nor is Hudson convinced that the use of reliability factors is correct, or that they are valid for New Zealand conditions.

A4 Forum notes

This part of the appendix contains notes taken during the forum by Martin Gribble and John Patrick. Participants were asked to contact the author if they felt that the notes were not representative of what they said; none did.

Frank Bartley

Stopping at 20 mm rut depth is too early. In practice, deformation is not a long term problem. Heaves are a major problem, however, and are connected to shallow shear failure. Bartley felt that it was important for local conditions to be properly understood in order to apply AUSTROADS. Effort needed to ensure designs are actually carried out in practice. Effort should be specified more tightly.

Clarence Morkel

Presumptive values should be published for people working in the field in an easily accessible form. The South African approach assumes elasto-plastic granular materials. (Graham Salt commented that FWD shows that AUSTROADS presumptive values are good).

John Hallett

Hallett commented that for the low traffic volumes (10^5) AUSTROADS is conservative. Hallett felt that subgrade strain criteria should be flagged as an issue. Heavier vehicles produce plastic deformation. He observed in the field that at Drury there was no rut in the subgrade. For cement treated bases with precracked 200 mm a 100 mm overlay works, while AUSTROADS recommends too much. Hallett desires a correlation between FWD and CIRCLY.

Dave Hutchison

- Shallow shear should be a primary focus in unbound granular pavement.
- Asphalt failure, water removal from the basecourse, and special studies for the New Zealand supplement also need attention.

Bryan Pidwerbesky

Pavement design should utilise common sense. We need a new value system. Subgrade strain criteria are useless (Pidwerbesky was quoting from a 1962 paper which stated that we should be limiting vertical compressive stress). Subgrade deformation only happens when things get wet. We cannot separate design from construction. Material properties, compaction and uniformity are all important while design thickness is the least important.

Norm Major

Care should be taken when using the term failure in the public forum. A failed road is one that you cannot negotiate with a four-wheel-drive vehicle. When communicating with laypeople the term distress should be used as an alternate. Models need to be based on observed performance. In Major's opinion a primary design factor is drainage. In producing design, procedures should determine what simple components of the complicated system should be provided to the practitioner.

David Alabaster

It should be remembered that AUSTROADS is a guide. Alabaster feels that we should be looking at the mechanisms that prompt maintenance. In practice, Transit does not see a lot of rutting.

Greg Arnold

Depth is not the only answer to pavement design. In practice, Arnold has observed two pavements which had the same achieved life but theory indicated that one should have a tenfold greater life. Total deformation is the integration of all deformations through the pavement. A solid working platform is necessary for pavement construction. If traffic levels are high, then rutting can be significant. A better method should be used to characterise granular distress versus permanent deformation. We should concentrate on using AC for heavy traffic pavements.

Bruce Steven

Whilst talking about CAPTIF, Steven observed that surface cracks did not allow water intrusion since the facility is indoors. He also commented that, despite tight construction control, density and deflection values still varied. Post-construction compaction was observed, followed by approximately linear deformation. Rutting was spread between subgrade and basecourse.

Would you expect ε_v to be coupled across depth? The vertical strain is linear with load at a particular depth. Between 40–60% of surface deformation is in the granular layers. The plastic strain rate can be calculated from elastic strains.

Ross Peploe

Texture problems drive rehabilitation. Most pavements are structurally sound, with soft spots repaired over time. Structural problems are not the real problem. AUSTROADS does not deal with asphalt well.

Other points that Peploe made were:

- local modified aggregate is better than M/4,
- modification helps remove some of the problems, such as plastic behaviour,.
- subgrade strain is not a major problem, and
- with rehabilitation it is ideal to stabilise the sub-base.

Graham Salt

- Curvature seems to be a good indicator of rehabilitation requirements.
- New Zealand should go back to isotropic moduli.
- A pulse wave should be used for analysis rather than static.

Allen Browne

- Brown has observed deformation in aggregate layer in the field.
- Our aim should be to prevent rutting early in built life.
- Asphaltic concrete – clarify how *in situ* and design moduli are related.
- AUSTROADS warns against non structural AC – how can thin AC layers be modelled?

Mofreh Saleh

- Failure need to be defined for pavements.
- Develop models for different mixes.

Ken Hudson

- Pavement design is an art.

- Aggregate selection is a problem, local knowledge is extensive but some is being lost.
- Heavy duty traffic should allow more wander; therefore damage is extended.
- What is the limit of very good unbound aggregate in terms of axial load and repetitions?
- Further investigate the utilisation of lime stabilisation, geotextiles and other methods to enhance pavement life.
- He feels a lack of confidence in the reliability factors.

Binh Vuong

- Design life is different from remaining life.
- We lack procedures to establish modulus values.
- Pavement production standards should be devised and enforced.
- Rutting is a de facto indicator for a number of failure mechanisms.

Appendix B Glossary

B1 Abbreviations and acronyms

σ_c	Vertical compressive stress on subgrade
σ_z	Vertical stress
$\mu\varepsilon$	Microstrain (10^{-6})
%HV	Percentage Heavy Vehicles
AADT	Annual Average Daily Traffic
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphaltic Cement
CAPTIF	Canterbury Accelerated Pavement Testing Indoor Facility
CBR	California Bearing Ratio
CGF	Cumulative Growth Factor
DF	Direction Factor
ESA	Equivalent Standard Axle
ESAL	Equivalent Single Axle Load
ESAL-km	Product of ESAL and kilometres travelled
FWD	Falling Weight Deflectometer
HDM	Highway Design and Maintenance Standard Series
HVAG	Heavy Vehicle Axle Groups
LDF	Lane Distribution Factor
LEF	Load Equivalency Factor
MDD	Maximum Dry Density
msa	Million standard axles
N	Allowable number of ESA loadings
$\mathbf{N_{DT}}$	Cumulative number of Heavy Vehicle Axle Groups
$\mathbf{N_{HVAG}}$	Average number of Heavy Vehicle Axle Groups
OMC	Optimum Moisture Content
OWC	Optimal Water Content
RLT	Repeated Load Triaxial
SADT	Single axle with dual tyres
SAR	Standard Axle Repetition
UGM	Unbound Granular Material

B2 Glossary

NAASRA roughness meter	A standard mechanical device used extensively in Australia and New Zealand since the 1970s for measuring road roughness by recording the upward vertical movement of the rear axle of a standard station sedan relative to the vehicle's body as the vehicle travels at a standard speed along the road being tested. A cumulative upward vertical movement of 15.2 mm corresponds to one NAASRA Roughness Count (1 NRM/km).

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