NEW ZEALAND SUPPLEMENT TO THE DOCUMENT,
Pavement Design – A Guide to the Structural Design of Road Pavements (AUSTROADS, 2004)

2007

NEW ZEALAND SUPPLEMENT TO THE DOCUMENT, Pavement Design – A Guide to the Structural Design of Road Pavements (AUSTROADS, 2004)

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FORWARD TO 2007 EDITION

The 2007 New Zealand Supplement (NZ Supp.) will replace all earlier versions of the NZ Supp. This NZ Supp. includes additional guidelines for the Engineer in applying the AUSTROADS design procedures resulting from research results and experienced gained in New Zealand. The aim is to minimise confusion and promote consistency in design assumptions applied in New Zealand.

Transit New Zealand (Transit) is an active member of AUSTROADS (the Association of State, Territory and Federal Road and Traffic Authorities in Australia) and has decided to contribute to and utilise, wherever possible and practical, the practices of that organisation. Therefore Transit has adopted the AUSTROADS pavement design procedures with variation as detailed in this NZ Supp. This provides a consistent approach for taking full advantage of the knowledge and experience of the roading fraternities in both New Zealand and Australia.

Most of the state roading authorities in Australia have their own supplementary document to the AUSTROADS Guide to integrate the standard design procedures with their unique material types and environmental conditions. This New Zealand Supplement (NZ Supp.) has been produced to facilitate the adoption of the AUSTROADS Guide in New Zealand by addressing the issues which are unique to New Zealand conditions.

Other state roading authorities in Australia place restrictions on the types of pavements that can be used in relation to traffic volumes. For example, it is common practice to use a structural asphalt pavement for urban motorways in Australia. To maximise the use of low cost thin-surfaced unbound pavements in New Zealand a risk based approach has been introduced to choose the most appropriate pavement type to reduce the risk of early failure.

Roading technology is continually being researched and changed. For this reason, both the AUSTROADS Guide and this NZ Supp. are intended to be living documents, i.e. they will be amended as new research findings come to light.

Rick Van Barneveld
Chief Executive

TABLE OF CONTENTS

Record of Amendments

Amendment N^{o}	**Subject**	**Effective Date**	**Updated By**
1	Added Section 3.2.1 on Water Flowing Within the Pavement Layers Updated Section 3.4.3 on Open-graded Asphalt Added Section 3.4.3.1 on Membrane Seals Beneath OGPA Layers Added Section 6.3.1 on Modified Local Aggregate Parameters Updated Table 6.5.4 on Surface Noise Added Section 6.5.4 on Asphaltic Concrete Fatigue Criteria Added Section 8.1.2.1 on Structural Asphalt Pavement Terminology Added Section 8.2.4 on the Design Approach for Foamed Bitumen Stablisied Layers Added Section 8.2.5 on Considerations for Pavement Design at Intersections Added Section 8.3 on Considerations for Pavement Widenings	01/02/07	David Alabaster

Manual Management Plan New Zealand Supplement to APDG, 2004

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1. Purpose

This is the Manual Management Plan for the New Zealand Supplement to the Document, Pavement Design – A Guide to the Structural Design of Road Pavements (AUSTROADS, 2004)

2. Document Information

Manual Name	New Zealand Supplement to the Document, Pavement Design – A Guide to the Structural Design of Road Pavements (AUSTROADS, 2004)
Manual No.	N/A
Availability	This manual is located in electronic format on the Transit website at: Www.transit.govt.nz
Manual Owner	CAPTIF Manager / Roading Engineer, National Office
Manual Sponsor	General Manager Network Operations
Review Team	Members of the Transit Document Review Team (DRT) Dr Greg Arnold and David Alabaster

3. Amendment and Review Strategy

All Corrective Action/Improvement Requests (CAIRs) suggesting changes will be acknowledged by the manual owner.

	Comments	**Frequency**
Amendments (of a minor nature)	Updates to information on documents listed in the guide will be incorporated as they occur.	As required
Review (major changes)	Amendments fundamentally changing the content or structure of the manual will be incorporated as soon as practicable. They may require coordinating with the DRT timetable.	Annually
Notification	All users that have registered their interest by email to specs@transit.govt.nz will be advised by email of amendments and updates.	Immediately

4. Other Information (at Manual Owners discretion)

There will be occasions, depending on the subject matter, when amendments will need to be worked through by the Review Team before the amendment is actioned. This may cause some variation to the above noted time frames.

5. Distribution

Copies of this Manual Management Plan are to be included on Interchange at the next opportunity and sent to:

Manual Sponsor	Review Team Members	Document Manager
Manual Owner	Head Office file AU2-0015	

CHAPTER 1 INTRODUCTION

1.1 Scope

This supplement provides a New Zealand context for the document “Pavement Design – A Guide to the Structural Design of Road Pavements” (Austroads 2004), herein after referred to as the *Austroads Guide or APDG*.

The section numbers used in the supplement generally correspond to the section numbers used in the Austroads Guide. New section numbers are used where additional information is provided for the benefit of the New Zealand pavement design fraternity and where a different numbering to the Austroads Guide is used the appropriate Austroads Guide section is listed in brackets.

The 2004 version of the Austroads Guide incorporates a number of updates from the previous 1992 version, however, the underlying design philosophy has not changed. The empirical Austroads design chart (Figure 8.4) has been retained for granular pavements with thin bituminous surfacings. The empirical – mechanistic approach using multi-layer elastic theory has also been retained although there have been a number of improvements, e.g.

- Updated sub-layering scheme for unbound layers;
- Use of a full standard axle in the design model;
- Revised subgrade strain performance criterion; and,
- Incorporation of a Project Reliability factor to reflect appropriate levels of design risk;

Designers are encouraged to review Research Report ARR 292 *“Origins of Austroads design procedures for granular pavements”* Jameson (1996) and *“Technical Basis for the 2001 Austroads Pavement Design Guide”* (Jameson, 2001).

This supplement should be applied in conjunction with relevant Transit New Zealand and Land Transport New Zealand standards.

CHAPTER 2 PAVEMENT DESIGN SYSTEMS

2.1 Project Reliability *(refer APDG 2.2.1.2)*

The desired project reliability is chosen by Transit New Zealand or the road designer. Typical reliability levels are given in Table 2.1.

Table 2.1 Typical project reliability levels.

Road Type[1]	Definition	Project Reliability (%)
Motorway	Designated as Motorway.	95 – 97.5
Urban Arterial	Arterial & collector roads within urban areas carrying > 7000 vpd.	90 – 97.5
Urban Other	Other urban roads carrying < 7000 vpd.	85 – 95
Rural Strategic	Arterial & collector roads connecting main centres of population carrying > 2500 vpd.	90 – 97.5
Rural Other	Other roads outside urban areas.	80 - 90

Note 1: Extension of road types presented in Project Evaluation Manual (Transit New Zealand 2004).

CHAPTER 3 CONSTRUCTION AND MAINTENANCE CONSIDERATIONS

3.1 General

The Engineer is reminded of the objectives of the Land Transport Management Act (LTMA) and the Resource Management Act (RMA), in particular the promotion of sustainable management of natural and physical resources. This may mean, for example, that local or recycled materials (with suitable improvement) could be appropriate for use in pavement construction. The RMA also obliges the organisation promoting any development to consult with interested parties and to obtain Resource Consents for activities that affect waterways or involve earthworks. The reader is referred to the document *"Planning Policy Manual, SP/M/001"* (Transit New Zealand, 1999) for more information.

3.2 Extent and Type of Drainage (*refer APDG 4.2*)

A large proportion of premature pavement distress can be attributable to excess water in the pavement structure. Therefore, careful consideration of pavement drainage is required and the presealing saturation requirements of TNZ B/2:2005 must be met.

Drainage design is essential and needs to consider:

1. Surface drainage, which is the drainage of water from the road surface and surrounding land and is essential:

   a. To prevent flooding which obstructs traffic;

   b. To prevent aqua-planing by minimising water film depth;

   c. To minimise the percolation of water into the pavement; and

   d. To intercept water which flows towards the road from the surface of land adjoining the road.

2. Underground drainage, to remove water that percolates from the surface into the pavement structural aggregates either from above or from the sides. Drainage of the pavement structural aggregates is called sub-pavement drainage; and

3. Subsurface foundation drainage, to control potential fluctuations in underground water level from natural or perched watertables or flows, to ensure optimum conditions in the subgrade and sound foundations for the road structure.

The installation of drainage features should never be considered as improving subgrade conditions for design but rather as maintaining them. Some unbound basecourse aggregates have shown from experience and in the repeated load triaxial test that they perform poorly when saturated. There needs to be provision for water to escape quickly from such moisture sensitive layers and drainage systems must operate effectively for the design life of the pavement.

3.2.1 Water Flowing Within the Pavement Layers

Pavement designers must be aware of the potential for water to flow either longitudinally or laterally (or both) within pavement layers. This is a common occurrence on slopes, in sag curves and on superelevated curves.

Water can enter the pavement structure from the top through defects in the seal, or even through the intact seal. It can also enter from the side where there are permeable shoulder or berm surfaces. Research shows that water can move laterally approximately 1 m without the benefit of gravity.

Therefore any permeable shoulder can be a significant source of water. This effect is exacerbated on the high side of superelevated curves where water can enter the pavement structure and flow through the pavement under the influence of gravity. Therefore, subgrade crossfall on the high side of curves should be graded away from the pavement in areas outside the seal extent.

Once water is flowing within a pavement it will follow the path of least resistance. The flow will continue until it reaches an area of higher pressure or becomes restricted by zones of relatively low permeability. Such zones could take the form of increased density under the wheel tracks or previous repairs that may have introduced stabilized aggregate or asphalt patches.

The pressure exerted by the water can result in significant deterioration of the mechanical properties of the pavement structure. In many cases water will be visible exiting the pavement surface, possibly pumping fine material with it.

The temptation is generally to carry out a digout repair of the affected area, however the resulting patch is likely to simply act as a bigger “dam” and the problem will be translated sideways or further back up the slope.

The best method of ensuring that water does not flow within the pavement layers is to install transverse cut-off drains to intercept longitudinal flow and to ensure that the shoulder is sealed on the high side of superelevated curves.

Note that a stabilized subbase layer can exacerbate this issues as water will tend to be trapped in the base layer and flow within the layer rather than draining out from the subbase. A stabilized base layer can be beneficial as it helps to keep water out of the pavement structure and the base material itself is generally less susceptible to the effects of water.

3.3 Use of Boxed Construction

Boxed construction is generally not appropriate for use in New Zealand as it can result in water being trapped in the pavement structure. This can result in reduced shear strength of aggregate materials and softening of subgrade soils.

3.4 Acceptable Risk *(refer APDG 3.14)*

3.4.1 Soft Subgrades *(refer APDG 3.15.1)*

The Austroads design procedure does not specifically take into account the improvement in mechanical properties obtained from chemical stabilisation of the subgrade. In New Zealand there is sufficient evidence to suggest that improvement in subgrade properties achieved by chemical stabilisation is reliable in the long term. Therefore, the increased stiffness of a stabilised subgrade layer can be included in the design analysis provided that reactivity of the additive has been verified by laboratory or field testing.

Where a mechanistic design approach is used, the stabilised subgrade layer should be considered to be anisotropic and sublayered. The sublayering should be carried out in accordance with the “selected subgrade materials” criteria, i.e. Eqns 8.1 and 8.2, of the Austroads Guide. This sets maximum top layer modulus values and suitable sublayering to ensure that the design model is representative of the conditions that can be expected in the field.

It must be noted that CBR values obtained in the laboratory for stabilised subgrade soils are generally much higher than the corresponding values achieved in the field. This is due to the superior compaction and mixing conditions inherent in the laboratory test procedures, eg confinement of the sample in the compaction mould.

The design subgrade CBR adopted should be checked during construction to verify the design value has been achieved (refer Chapter 11, APDG).

3.4.2 Sprayed Seals *(refer APDG 3.16.1)*

Primer seals are not considered to be appropriate for use in New Zealand.

3.4.3 Open-graded Asphalt *(refer APDG 3.16.3)*

Open-graded porous asphalt (OGPA) is used extensively in New Zealand, particularly on motorway and urban arterial roads. As OGPA is porous, it must be placed on an impermeable membrane and either, drainage from the edge of the OGPA allowed or, a detail provided which would allow for drainage of the OGPA without comprising the pavement.

OGPA has generally been considered to have a reasonably low stiffness and a high tolerance for deflection. Unfortunately it has a low tolerance for deformation of underlying layers and early deformation of Greenfield unbound granular pavements is nearly impossible to limit. Using current standards it is advisable to allow 3 months of normal loading prior to applying OGPA. However, recent research at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) suggests OGPA can be applied immediately if the basecourse's degree of saturation is below 60% prior to sealing. This effectively requires sealing to be undertaken in summer.

Recent test data also suggests that the stiffness of an aged OGPA layer can be very high, e.g. around 5 GPa or more. This may have implications on surfacing performance, especially where new OGPA layers are placed without first removing the existing OGPA surface and this construction technique should continue to be monitored.

While high elastic modulus values may be measured for aged OGPA layers there is no guarantee that the layers will not crack in the long term, in which case the modulus would reduce accordingly. Therefore, accounting for OGPA layers in design models should be achieved as follows:

- OGPA layer(s) over granular base – treat the OGPA layer as an extension of the base layer;
- OGPA layer(s) over structural asphalt base – include in the design model with an elastic modulus in the range 500 – 1,000 MPa depending on the speed and temperature environment.

3.4.3.1 Membrane Seals Beneath OGPA Layers

It is important that an effective seal is placed beneath an OGPA layer given its operating environment. General guidelines for membrane seals are as follows:

- Placing OGPA over an existing OGPA layer :– 1 l/m^2 CQ-60 emulsion (or equivalent) plus a Grade 5 chip.
- Placing OGPA over an existing asphaltic concrete layer :- 0.2 - 0.4 l/m^2 CQ-60 emulsion (or equivalent) plus a Grade 5 chip.
- Placing OGPA over an existing granular layer :- conventionally designed two-coat chip seal. If the chip seal is trafficked then a tack coat will be necessary prior to placing the OGPA. In high demand areas it is preferable to substitute the two-coat seal with an asphaltic concrete layer.

CHAPTER 4 ENVIRONMENT

4.1 General

The effect of freeze / thaw conditions on the performance of unbound granular layers is not addressed in the Austroads Guide. These conditions regularly occur in regions of New Zealand such as the central North Island and central and southern areas of the South Island.

Whenever the temperature of the pavement structure may fall below 0°C, all aggregates used must not be susceptible to freeze / thaw effects. Good drainage must also be provided to minimise the quantity of water that can enter the pavement and subsequently freeze (see Cheung & Dongol, 1996).

CHAPTER 5 SUBGRADE EVALUATION

5.1 Deflection Testing *(refer APDG 5.5.3)*

The volcanic soils of the central North Island exhibit a relatively resilient response to loading compared with non-volcanic soils. Therefore, the E = 10(CBR) relationship for subgrade elastic modulus can be inappropriate in many locations.

Transfund Research Report 213 (Bailey and Patrick, 2001) concludes that there is a range of constants that can be used in the E versus CBR relationship depending on the origin of the soil in question. The research showed that in the equation, E = k(CBR), k took the following values for anisotropic conditions:

- k = 1.5 : pumice / sandy soils;
- k = 4.5 : mixture of silty soils and brown ashes;
- k = 15 : typically clayey, ash soils.

It is vital to the performance of pavements on volcanic soils that their unconventional response is considered in the evaluation of subgrade properties for design (refer Section 5.3).

5.2 Subgrade Testing (*refer APDG 5.6.2*)

Soaked laboratory CBRs are generally appropriate for Greenfield sites where it is difficult to establish appropriate equilibrium moisture contents. Scala Penetrometer testing and/or Falling Weight Deflectometer (FWD) testing is generally sufficient on rehabilitation sites with the exceptions noted below.

Soaked laboratory CBRs are appropriate whenever the groundwater level may reach within one metre of the top of the subgrade, or the pavement could be subject to inundation by flooding. Note that the use of soaked subgrade parameters does not make a pavement exempt from moisture problems and the provision of an effective drainage system is always necessary. Scala penetrometer testing should also always be used to identify any weak layers to a depth of at least 1m below the top of the subgrade.

Care should be taken assessing silty and sensitive subgrades. They can be significantly weakened by the inappropriate use of construction equipment and this should be noted in the contract documents.

5.3 Limiting Subgrade Strain Criterion *(refer APDG 5.8)*

The subgrade strain criterion adopted in the Austroads Guide provides a reasonable relationship between elastic subgrade strain and expected service life for “conventional” subgrade soils. Experience shows that volcanic soils are able to tolerate much higher strain levels and should therefore be considered differently from non-volcanic soils.

For design of thin surface granular pavements:

- Use Figure 8.4 of the Austroads Guide using a measured subgrade CBR for design shall be used.

For design of pavements with one or more bound layers:

- Consider the subgrade cover requirements using Ev(sg) = 10(CBR) (anistropic) in the CIRCLY model.

- Consider the bound layer performance using a subgrade modulus that has been obtained from deflection tests, measured in the repeated load triaxial test or obtained using the findings of Transfund Research Report 213, as detailed in Section 5.1.

CHAPTER 6 PAVEMENT MATERIALS

6.1 General

Unbound aggregate pavements with chip seal or thin asphalt surfacing have been used extensively in New Zealand, generally with great success. However, with increasing wheel loads, higher tyre pressures, narrower lane widths and rising traffic volumes, the designer should consider the use of other options. Other options can include modified granular materials and/or structural asphalt layers as detailed in Section 8.1 to reduce the risk of premature rutting on “green fields” projects.

6.2 Unbound Granular Materials

The requirements for unbound granular basecourse materials are given in Transit New Zealand Specification TNZ M/4. Aggregate conforming to the M/4 specification would generally correspond to the “high standard crushed rock” material referred to in the Austroads Guide.

Transit New Zealand allows modified local aggregates to be used as a substitute for conventional M/4 aggregates provided these materials comply with TNZ M/22 (Notes). The requirements for such materials are described in Section 6.3 of the Austroads Guide, and in further detail in TNZ M/22 (Notes).

6.3 Modified Granular Materials

Experience indicates that some aggregates that do not comply with the Transit M/4 specification for premium basecourse can be improved by the addition of a chemical modifying agent. Typically this will involve mitigating the effect of deleterious swelling clay minerals in the parent material, as well as providing a low level of interparticle binding. The result is generally deemed to provide a level of performance exceeding that of conventional M/4 basecourse.

Note that in the modification process there is no intention to produce a cemented basecourse product. Cemented basecourse layers are susceptible to shrinkage and fatigue cracking and are unlikely to be acceptable in the upper part of the pavement unless they are specifically designed for this. They also require a strategy to mitigate the effect of early shrinkage cracking and eventual fatigue cracking. Vorobieff (2004) reports that a material must have a 28-day Unconfined Compressive Strength (UCS) in the range 0.7 – 1.5 MPa to qualify as a modified aggregate.

The approach listed above is consistent with the approach used by Queensland Department of Main Roads (Jones and Bell, 2004). Jones and Bell also suggest that structural asphalt pavements are the only viable option for urban roads with a first year traffic loading exceeding 3.4×10^5 ESA (25-year loading of approximately 1.4×10^7 ESA). They suggest that structural asphalt pavements are viable options for urban roads with a first year loading in the range 2.3×10^5 to 3.4×10^5 ESA and rural roads with a first year loading exceeding 4.6×10^5 ESA.

The use of suitable modified local aggregates has shown significant benefits in many areas of New Zealand. This approach generally results in basecourse layers that perform at least as well as those constructed using M/4 aggregate, with the additional advantages of reduced cost, expeditious construction and environmental benefits.

Modified aggregates may provide improved rut resistance in the context of “green fields” projects.

There are two methods to add chemical binder to aggregate for modification; either add the binder at a stationary plant such as a pugmilll or stabilise insitu with a hoe. Stabilising insitu has an effect on the aggregate grading - particularly 'softer' rock. A better result will be achieved with plant mixed material with the plant located remotely or at the site. Another advantage of using plant mixed materials is the ability to control moisture content, modifying agent and grading throughout the pavement layer - this will have flow on benefits in achieving the compaction requirements. However the designer must be aware that some additives may oxidise or leach out if the mixed material is stockpiled.

The action of a hoe may have a significant effect on the grading of an aggregate. The amount of particle breakdown under a hoe is dependent upon a number of factors, e.g. particle strength, hoe speed, hoe tip configuration, etc. Typically a reduction of one particle size category occurs, i.e. an AP65 converts to an AP40, an AP40 converts to an AP20, etc.

Suitable laboratory tests should be carried out to establish the optimum type and quantity of modifying agent. This should include tests to ensure mitigation of swelling clay fines and to verify appropriate strength gains. While the distinction between modified and cemented behaviour is difficult to define, a maximum 28-day UCS of 1 MPa (7-day UCS of 0.7 MPa) is considered to be a reasonable guideline for modified material.

Aggregate modification is generally achieved using small quantities of additive. While the use of minimal amounts of additive are desirable from a technical and economic viewpoint, there are practical limitations in terms of achieving accurate and uniform distribution of the additive. Materials with very high reactivity are very sensitive to the amount of additive. A modified material of highly variable properties can result because of the practical limitations on the fineness of control of additive in the field. In such circumstances, the modifying agent should either be added in a diluted form or an alternative agent sought. The designer is referred to the Austroads 1998 Guide to Stabilisation in Roadworks (AP-60/98) for detailed information regarding stabilisation issues, however a brief description of stabilising additives commonly used in New Zealand is presented in Table 6.4.1

6.3.1 Modified Local Aggregate Parameters

This document allows modified local aggregates to be used in place of traditional TNZ M/4 basecourse aggregates.

There are a number of issues that the designer must consider when specifying modified local aggregates. These include:

- The proposed material application, e.g. basecourse or subbase;
- The proposed pavement configuration, e.g. is the modified local aggregate layer part of a deep strength structural asphalt pavement?
- The envisaged traffic loading;
- The local experience with respect to aggregate performance and construction procedures;
- The nature of local aggregates available;

Material parameters and acceptance criteria should be established once due consideration has been paid with regard to the above issues. Appropriate material parameters will largely be dependent on the nature of the local aggregates. However, it is envisaged that most applications will require the modified material to achieve the acceptance criteria specified in TNZ M/4. This means that the appropriate modification additive type and quantity must be established so that the modified material is virtually equivalent to an M/4 basecourse. Alternatively the requirements of Transit's M/22 specification can be adopted. The latter may be more appropriate in low volume road applications.

It is recognized that modified local aggregates can achieve a somewhat higher elastic modulus than an untreated M/4 basecourse, however considerable care and experience is required when selecting modulus values for design. The designer should ensure that adequate quality control measures are in place during construction and that FWD testing is carried out at an appropriate time interval to verify that the design assumptions are substantiated.

Confirmation of the suitability of the modified local aggregate prior to construction is essential as it will be too late to change a design once the modified local aggregate layer is placed and tested. One way of achieving this is to obtain test information from similar materials that have been in place for a suitable period of time and to translate the back-calculated elastic modulus values to the proposed design. Alternatively, the proposed aggregate can be subjected to repeated load triaxial testing.

6.4 Cemented Materials

It is unlikely that cemented basecourse materials will be appropriate for the materials and loading conditions typically found in New Zealand. They are prone to cracking due to fatigue and shrinkage, and without an unbound cover layer or a suitably thick or resistant surface layer, any cracks will reflect through to the pavement surface. This will allow water to enter the pavement structure resulting in reduced layer stiffness and strength. Erosion of the subgrade may also occur through the action of “pumping”.

Cemented materials are likely to be better suited to the subbase layer where the additional strength and stiffness can provide a superior “anvil” for the compaction of the overlying layer(s) while also maximising the load-spreading ability of the subbase layer. If the cemented subbase layer does crack, the resulting decrease in stiffness can be accounted for in the design analysis and it is unlikely that the crack will be reflected through the basecourse layer to the pavement surface.

When constructing a cemented subbase layer, care should be taken to avoid hoeing through the layer and bring up significant quantities of subgrade soil into the subbase layer. An excessive quantity of fine soil particles can reduce particle interlock and inter-particle friction, therefore significantly decreasing the shear strength of the layer.

The designer is referred to the Austroads 1998 Guide to Stabilisation in Roadworks (AP-60/98) for detailed information regarding stabilisation issues, however a brief description of stabilising additives commonly used in New Zealand is presented in Table 6.4.1

Table 6.4.1 Application of common stabilising agents.

Additive	Process	Effects	Applications
Cement	Cementitious interparticle bonds are formed. Reactions are temperature dependent.	Low cement content – decrease susceptibility to moisture changes, produces modified to lightly cemented material. High cement content – significant increase in strength & modulus, produces cemented material.	Not limited, apart from aggregates containing deleterious components (organics, sulpahtes, etc). Suitable for granular soils & aggregate, inefficient in single sized material & heavy clays.
Lime	Cementitious interparticle bonds are formed but rate of development is slower than for cement. Reactions are temperature dependent & require clay minerals to be present.	Improves handling properties of cohesive materials. Low lime content – decreases susceptibility to moisture changes, produces modified to lightly cemented material. High lime content – significant increase in strength & modulus, produces cemented material.	Suitable for cohesive soils or aggregate with plastic clay minerals. Organic material will retard reactions. Extra caution required in allophone soils.
KOBM	Cementitious interparticle bonds are formed but rate of development is slower than for cement. Reactions are temperature dependent & require clay minerals to be present.	Low KOBM content – decrease susceptibility to moisture changes, produces modified to lightly cemented material. High KOBM content – significant increase in strength & modulus, produces cemented material.	Suitable for aggregate with plastic clay minerals. Organic material will retard reactions. Often used in conjunction with cement.
Durabind	Cementitious interparticle bonds develop similar to a low additive lime/cement reaction. Reactions are temperature dependent.	Low Durabind content – decrease susceptibility to moisture changes, produces modified to lightly cemented material. High Durabind content – significant increase in strength & modulus, produces cemented material.	Suitable for aggregate with plastic clay minerals. Organic material will retard reactions.
Bitumen (foamed bitumen,emulsion)	Agglomeration of fine particles.	Low bitumen content decreases permeability & provides cohesion & some strength increase. High Bitumen content decreases moisture sensitivity by coating fines & significantly increases strength.	Applicable to granular materials with suitable grading, low cohesion & low plasticity.

6.4.2 Determination of Design Modulus *(refer APDG 6.4.3)*

6.4.2.1 Alternative Methods *(refer APDG 6.4.3.2)*

Caution should be used when considering back-calculated elastic modulus values for cemented pavement layers. The very low deflection associated with cemented layers can cause significant variation in results due to the accuracy of the layer thickness data and limitations of accuracy and repeatability of the measuring equipment.

6.5 Asphalt

6.5.1 Introduction

The bituminous binders referred to in the Austroads Guide are classified in accordance with the mid-point of their viscosity range at 60° C in Pa.s. The binders used in New Zealand are classified in terms of their penetration grade. There are no direct correlations between the New Zealand and Australian bitumen classifications, however Table 6.5.1 provides a guide to approximately equivalent binders.

Table 6.5.1 Equivalent (approx) binder classifications.

Australian Binder	New Zealand Binder
Class 170	80 / 100
Class 320	40 / 50 and 60 / 70
Class 600	No Equivalent

6.5.2 Rate of Loading

The response of asphalt layers to traffic loading is dependent on the rate of loading, i.e. the speed of the traffic. VicRoads (2004) suggests Pavement Design Speeds as shown in Table 6.5.2 below.

Table 6.5.2 Pavement design speeds for asphalt layer design *(refer APDG Table 6.5.2.4)*

Designated Speed Limit (km/hr)	Pavement Design Speed (km/hr)
V >= 100	80
60 <= V < 100	60
40 <= V < 60	40
Signalised Intersections or Roundabouts	10

6.5.3 Typical Asphalt Mix Characteristics *(refer APDG 6.5.9)*

The designer is referred to Austroads 2002, Austroads Framework for Specifying Asphalt for information regarding asphalt mix design and characteristics, however Table 6.5.3 provides a brief summary of applications and characteristics for a range of common New Zealand asphalt mixes.

Table 6.5.4 provides information on the acoustic performance of a number of materials relative to dense asphalt. Note that where RMA consent conditions are in place for a project a specialist noise consultant should be consulted prior to finalising the choice of surfacing. Most of the surfaces listed in Table 6.5.4 are subject of ongoing performance monitoring and future guides will contain information on the long-term performance of the various surfaces. Future guides will also address whether cleaning of porous systems is needed to maintain their acoustic properties.

Table 6.5.3 Summary of typical asphalt mix applications and characteristics.

Asphalt Mix	Typical Application	Characteristics
Mix 6	Surfacing for lightly trafficked urban roads.	Good surface for braking and turning traffic in low speed environments. Low permeability, high water spray & moderate tyre noise. These mixes often have a higher binder contents & consequently have good crack resistance but can be more prone to rutting relative to AC mixes
TNZ Mix 10	Surfacing for light to moderate duty urban roads.	
TNZ Mix 15	Surfacing for heavily trafficked urban roads.	Good shear resistance for braking & turning traffic in low speed environments. Relatively stiff mix which, depending on the type and volume of binder can be prone to cracking with pavement deformation. Low permeability, high water spray & moderate tyre noise.
TNZ Mix 20, 25 & 40	Structural asphalt pavement layers.	Caution required to ensure large stone mixes are placed in appropriate lifts to achieve adequate compaction without segregation. These mixes often have a higher binder contents & consequently have good crack resistance but can be more prone to rutting relative to AC mixes
AC 14,20,28	Australian mix denoted by the prefix "AC".	Similar functional performance to TNZ mixes however these mixes often have a lower binder content & consequently have good rut resistance but can be more prone to cracking relative to TNZ mixes.
Open Graded Porous Asphalt (OGPA)	Surfacing on high speed urban arterials & motorways.	Permeable surface providing good tyre noise reduction, reduced glare & low water spray. Has good skid resistance but only fair to poor shear resistance for braking & sharp turning traffic. Increasing the aggregate size increases water drainage & storage capability. Must be placed on membrane seal. Susceptible to blockage and breakdown with age.
Stone Mastic Asphalt (SMA)	Surfacing for heavily trafficked urban arterial / motorway applications with high surface stresses.	Good skid resistance & shear resistance for braking & sharp turning traffic. Good rut resistance & durability, low permeability, moderate tyre noise & moderate water spray.
Fine Gap Graded Asphalt	Surfacing for lightly trafficked, low speed urban applications.	Very durable surface with high fatigue resistance. Greater flexibility than dense graded asphalt but has lower skid & shear resistance. Low rut resistance, low permeability & high water spray.
Slurry	Maintenance treatment with fine textured surface.	Reasonably permeable bituminous slurry surface with fine texture proving good skid resistance at low speeds. Poor flexibility, moderate tyre noise & high water spray.

Table 6.5.4: Effect of road surface types on traffic noise relative to asphaltic concrete (Mix 10) and the effect of the proportion of heavy vehicles (dbA).

Surfacing Material	Vehicle Type		% Heavy Vehicles			
	Light / car	Truck / heavy	0	3	10	20
Asphaltic Concrete – Mix 10	0	0	0	0	0	0
OGPA – TNZ P/11, 20% voids	0	-2	0	-0.3	-0.8	-1.2
High Void OGPA, 30% voids	-2	-3	-2.0	-2.1	-2.3	-2.6
Two Layer OGPA Systems	-2	-4	-2.0	-2.3	-2.7	-3.1
Ultra Thin AC (1)	+3	0	+3.0	+2.5	+1.8	+1.2
Macadam (1)	+3	0	+3.0	+2.5	+1.8	+1.2
Fine Chip – Grade 4, 5	+3	-2	+3.0	+2.4	+1.8	+0.4
Med Chip – Grade 3	+4	+1	+4.0	+3.5	+2.8	+2.2
Coarse Chip – Grade 2, Two Coats	+6	+1	+6.0	+5.5	+4.5	+3.5

Note (1) based on limited data.

6.5.4 Asphaltic Concrete Fatigue Criteria

The Reliability Factors (RFs) suggested in Table 6.13 of the Austroads Pavement Design Guide incorporate a combination of reliability and shift factor components. The latter is included to account for the improved performance that asphalt materials provide in the field compared with asphalt specimens tested for their fatigue properties in the laboratory.

Some industry experts maintain that the Austroads RF values are overly conservative. While this may be true, there is insufficient research or analysis currently available to verify this view. Further research into this issue will be conducted in the next update of the NZ Supplement.

In the interim, the current Austroads RF values stand. However, designers could consider relaxing the highest level of project reliability (97.5%) to the next level down, i.e. 95%. This would remove the most conservative RF value (0.67 for asphalt) and it would be consistent with the level of reliability that has been used in New Zealand in the past.

CHAPTER 7 DESIGN TRAFFIC

7.1 General

This section of the Supplement has been developed largely using weigh-in-motion (WIM) data obtained from Transit New Zealand WIM installations. It must be noted that there is a limited number of WIM sites throughout New Zealand. Therefore, the pavement designer should recognize the inaccuracies that are inherent in the data and apply a suitable degree of engineering judgment when applying the traffic loading data.

The Austroads Guide makes a distinction between urban and rural traffic distributions whereas this Supplement adopts a typical rural traffic distribution only. This is considered to be appropriate given the small number of WIM sites in New Zealand and the fact that the sites are typically located in rural areas.

The designer should also determine whether predicted traffic growth is likely to be geometric or linear and whether the design lane is likely to reach capacity for heavy vehicles sooner than the design life for the pavement.

7.2 Procedure for Determining Total Heavy Vehicle Axle Groups (*refer APDG 7.4*)

One of the fundamental parameters used in a typical design traffic analysis is the Average Annual Daily Traffic (AADT). Transit New Zealand has a number of automated classified count stations on the state highway network that may provide suitable data. However, locations that are remote from the count stations may require specific traffic counts to be carried out.

Generally it is not practical to undertake a traffic count for a full year therefore, reduced period counts are undertaken and the resulting data is scaled up by an appropriate factor to approximate the full year data. The importance of the project and the associated budgetary constraints will dictate the type, number and duration of counts that will be possible. At the high end of the range of investigation procedures an automated counter may be set up at the site for a period of one or more weeks. The results can then be used to determine the full year traffic, although consideration of seasonal influences may be necessary.

The low end of the investigation range may involve one or more 24-hour traffic counts. The 24-hour counts are preferable over the eight-hour counts as they identify all traffic using the site on a typical day.

Transfund New Zealand Research Report No. 96 examined weekly patterns of traffic, by day and by hour. A set of multipliers resulted for application to shorter counts to infer AADT.

7.2.1 Estimating Axle Groups Per Heavy Vehicle *(refer APDG 7.4.6)*

The presumptive number of axle groups per heavy vehicle (N_{HVAG}) for New Zealand state highways is 2.4. This value has been derived from mean WIM data from 2002 to 2004.

7.3 Estimation of Traffic Load Distribution *(refer APDG 7.5)*

A presumptive traffic load distribution for New Zealand state highways has been derived from WIM data from 2002 to 2004. The presumptive distribution is presented in Appendix 7.4 of this Supplement. Note that the data is considered to be appropriate for rural state highways. There is insufficient information to establish a corresponding distribution for urban state highways.

7.4 Pavement Damage in Terms of Standard Axle Repetitions *(refer APDG 7.6.2)*

Presumptive Damage Index parameters have been determined from New Zealand WIM data (see Table 7.8).

Table 7.8 Presumptive Damage Index parameters for New Zealand traffic loading conditions.

Damage Type	Unit	Value	Damage Index	Value
Overall Damage	ESA/HVAG	0.6	N/a	N/a
Asphalt Fatigue	SAR_a/HVAG	0.6	SAR_a/ESA	1.0
Subgrade Rutting	SAR_s/HVAG	0.8	SAR_s/ESA	1.2
Cemented Layer Fatigue	SAR_c/HVAG	2.2	SAR_c/ESA	3.6

CHAPTER 8 DESIGN OF NEW FLEXIBLE PAVEMENTS

8.1 General – (Pavement Type)

To reduce the risk of premature pavement rutting and provide a level of consistency with overseas best practice (Jameson, 2005), the pavement type should be selected to ensure a low risk of pre-mature rutting and major rehabilitation occurring before the end of the design life. For each pavement type the risk of pre-mature rutting can be reduced by implementing options for best practice as detailed in Sections 8.1.1 to 8.1.6.

The final pavement type may be restricted by financial constraints, although for a high profile pavement like an expressway or urban motorway a medium or higher level of risk must be noted in the design, as it may politically be unacceptable. Transit New Zealand's Risk Management Process Manual was followed to determine a starting point for designers in choosing the most appropriate pavement type (see Tables 8.1 and 8.2). The final choice of pavement type requires designers to assess pavement type options as per the Risk Management Process Manual along with the calculation of Net Present Value (NPV) of whole of life costs detailed in Appendix 1(includes road user travel delays due to maintenance, capital and road maintenance costs). Initial analysis for structural asphalt and concrete pavements has shown for high traffic volumes (approx. > $3x10^7$ ESAs in 20 years) the NPV whole of life costs are approximately the same but this is project specific and whole of life costs are required for each project.

Table 8.1 Probability of Failure for Pavement Types

	[1]Tendency to rut pre-maturely (ie. pavement requiring smoothing before the design life).
Pavement Type	
8.1.1 [2]CRCP - Continuously Reinforced Concrete Pavement	↑ Higher rut resistance Lower probability of failure Higher Initial Capital Cost Lower Maintenance Costs[3]
8.1.2 SA - Structural Asphalt	
8.1.3 [2]MABSS - Modified Aggregate Base and Stabilised Sub-base	
8.1.4 [2]MAB - Modified Aggregate Base only	
8.1.5 UABAC - Unbound Aggregate Base 3 month delay before AC	
8.1.6 UAB - Unbound Aggregate Base	

1 Probability of failure can be reduced through use of recognised best practice described in the following sections, for comparison a similar table should be produced that reflects local experience (i.e. if there are many rutting failures with the traditional unbound aggregate pavements then this should be noted);

2 Probabilities of failure should be increased where there has not been experience with the construction of these types of pavements;

3 Also reduced frequency of maintenance will result in less traffic delays caused by lane closures and thus reduced road user costs.

Table 8.2 Guidance for the Selection of Pavement Type

	20 year Design Traffic Volume (ESAs)			
Noise Restrictions for Maintenance at night?	$< 5 \times 10^6$	5×10^6 to 1×10^7	1×10^7 to 5×10^7	$> 5 \times 10^7$
Yes	**8.1.1 CRCP** **8.1.2 SA** **8.1.3 MABSS** **8.1.4 MAB** **8.1.5 UABAC** **8.1.6 UAB**			
No	**8.1.1 CRCP** **8.1.2 SA** **8.1.3 MABSS** **8.1.4 MAB** **8.1.5 UABAC** **8.1.6 UAB**			

Notes: The choices of pavement type above are for guidance only as the final choice of pavement type will depend on whole of life cost analysis (Appendix 1), including capital, maintenance and road user costs and the level of risk Transit (as per Transit's Risk Management Process Manual (AC/Man/1)) is prepared to take to keep within capital budget restraints.

8.1.1 CRCP - Continuously Reinforced Concrete Pavement

A Continuously Reinforced Concrete Pavement by its very nature eliminates rutting. Overseas practice is tending towards the use of rigid concrete pavements for very high traffic volumes where lives of 40 years or greater are required and ideally nil maintenance is required. For jointed concrete pavements joint maintenance is required and for all concrete pavements surfacing maintenance or replacement is likely required if skid resistance drops below threshold levels. Rigid concrete pavements are often economic when low discount rates are used in economic analysis, such as the 3.5% used in the UK. The low discount rates favour longer life pavements with lower maintenance costs.

Most new motorways and expressways constructed by RTA NSW are rigid concrete pavements (Jameson, 2005).

Recent economic analysis shows the Continuously Reinforced Concrete Pavements may be a viable option in New Zealand. However there are a number of requirements to consider including the current need to surface with Open Graded Porous Asphalt (OGPA) which reduces the economic benefits and that modern concrete pavement construction is a new technique not yet trialled in New Zealand.

8.1.2 SA - Structural Asphalt

A pavement with a structural asphalt layer reduces the rate of rutting significantly. A test at CAPTIF with 90mm of structural asphalt over an unbound granular base shows the rate of rutting is 0.8 mm per 1 Million passes (Figure 8.2).

Jones and Bell (2004) from the Queensland Department of Main Roads suggest that structural asphalt pavements are the only viable option for urban roads with first year loadings exceeding 3.4 x 10^5 ESA (25-year loading of approximately 1.4 x 10^7 ESA). They also suggest that structural asphalt pavements are viable options for urban roads with a first year loading in the range 2.3 x 10^5 to 3.4 x 10^5 ESA and rural roads with a first year loading exceeding 4.6 x 10^5 ESA.

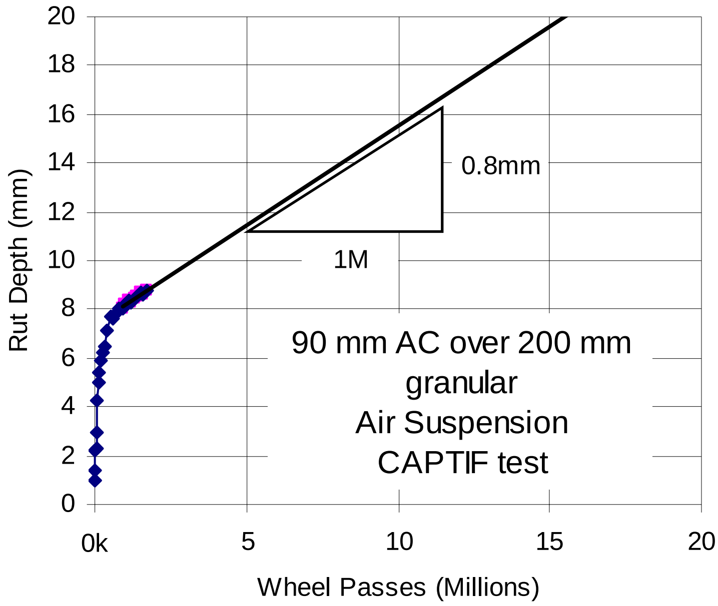

Figure 8.1 Rate of rutting for granular pavement with 90mm of structural AC.

Methods of designing pavements with structural asphalt layers are well covered in the Austroads Guide. However, asphalt layer thicknesses using the earlier State Highway Pavement and Rehabilitation Design Manual which was based on the Shell design method are 30% thinner than those required using the Austroads Guide. This greatly affects the economics of using structural Asphalt pavements in New Zealand. Two thirds of the Wellington and Auckland motorway network are constructed with structural asphalt having being designed using the earlier method and are performing well past their design lives with minimal structural maintenance required. Transit New Zealand is currently considering adjustments to the Reliability Factors for asphalt fatigue (Table 6.13, APDG) to match the experience of asphalt pavements found in New Zealand.

Best Practice

Techniques that reduce the probability of failure are simply good practice and should be considered, examples are:

- The asphalt mix is designed appropriately for its application especially for areas prone to rutting where heavy vehicles are moving slowly or are stopped, eg, bus stops, intersections;
- The asphalt mix is designed using APRG 18 and is appropriate for the vehicle speed and environment, with wheel track rutting test to verify asphalt mix performance;
- Performance based specifications are used with a warranty period of at least 2 years;
- Quality control is audited.

8.1.2.1 Structural Asphalt Pavement Terminology

A number of overseas references report the use of Full Depth and Deep Strength structural asphalt pavements. A brief description of the two pavement configurations follows:

Full Depth Structural Asphalt – the asphalt layer(s) are founded directly on the subgrade, or improved subgrade layer. While relatively rare, this type of structure has been used successfully in New Zealand. It is important that the subgrade is relatively robust to ensure that it is capable of supporting the construction traffic and provide a suitable anvil for the compaction of the asphalt layers. This may require some level of subgrade improvement, e.g. lime and/or cement stabilisation. In addition, suitable drainage provisions must be provided to ensure that the integrity of the subgrade is maintained throughout the design life of the pavement.

Deep Strength Structural Asphalt – the asphalt layer(s) are founded on an unbound or stabilized aggregate layer that provides additional strength for the pavement structure. While the asphalt layer(s) provide the majority of the pavement’s structural strength, the underlying layer also has a structural role.

Perpetual Pavements – refers to a sequence of asphalt layers that are optimized to provide (at least in theory) the greatest pavement life using the least overall thickness of asphalt. In the ideal situation the perpetual pavement has a virtually unlimited (structural) life and the only requirement is that the properties of the surface layer must be maintained. This will generally involve periodic milling and replacement of the surface layer only.

In general the perpetual pavement will comprise a sequence of three asphalt layers. From top to bottom these layers are: surface, intermediate and base.

The surface layer provides the specialized properties required of the pavement surface. These properties will be dependent upon the application, but will generally include skid resistance, rut resistance, shear strength, spray abatement, etc.

The intermediate layer must be durable and provide a high level of rut resistance. The intermediate layer may need to be placed in two lifts to ensure that the required density Is achieved. It may be divided into two layers of differing mixes in some instances.

The base layer provides a high degree of fatigue resistance. Accordingly, this layer is susceptible to stability issues and therefore the thickness of the base layer must be limited. In addition, the base layer should not be trafficked.

It must be noted that some Australian road authorities have encountered problems with the perpetual pavement configuration. This is thought to be a result of water entering the upper pavement layers and being trapped by the lower permeability base layer. This has caused durability issues, although there are no instances of this occurring in New Zealand to date.

8.1.3 MABSS - Modified Aggregate Base and Stabilised Sub-base

Figure 8.3 details two components of rutting that occur within an unbound granular layer, they are densification and shear related. A stabilised sub-base on top of a stabilised subgrade provides an anvil for improved compaction of the upper base as well as increasing the shear strength of the materials to reduce rutting. A Modified Aggregate Base (Section 6.3) is recommended on top of the stabilised sub-base to reduce the rate of shear related rutting (which is needed for high traffic roads) and to reduce the risk of moisture related rutting due to trapped water on top of the stabilised sub-base. It is also recommended that the stabilised sub-base be of higher porosity than the overlying base material.

Best Practice

Techniques which reduce the probability of failure are simply good practice and should be considered, examples are:

- A thorough laboratory study has been undertaken to determine the: most appropriate stabilising agent and amount; target densities and moisture content. (see Austroads Guide to Stabilisation and Triaxial testing as detailed in TNZ M22);
- Gradings with larger stones are used (e.g. GAP 65) where modified and stabilised aggregates are achieved insitu using a hoe;
- Modified and stabilised aggregates are plant mixed using a pugmil at the correct moisture content and gradings;
- Performance based specifications are used with a warranty period of at least 2 years;
- Quality control is audited.

8.1.4 MAB - Modified Aggregate Base Only

A Modified Aggregate Base (Section 6.3) is considered to reduce rutting (Figure 8.3). Research is programmed at CAPTIF that aims to quantify this reduction in rutting. This will ensure that the design life is reached on high volume roads before a 20mm rut depth is developed. Other benefits of a Modified Aggregate Base are expedience of construction and the performance is not affected by moisture to the same extent as an unbound material. Western Australia use Modified Aggregate Bases mixed in a pugmill for their high volume roads to prevent pre-mature rutting failures that were occurring with unmodified unbound aggregate (Jameson, 2005).

Best Practice (see above in Section 8.1.3)

8.1.5 UABAC - Unbound Aggregate Base, 3 month delay before OGPA

The compaction related rutting (Figure 8.3) occurs during approximately the first hundred thousand heavy axle passes (ESAs). This initial rutting cannot be eliminated with a few hundred passes of rollers and/or a water cart. A recent test at CAPTIF and failure of an expressway near Tauranga identified the OGPA surface layer cracked because of its inflexibility to mould into the shape of the early rutting that occurred in the pavement. This cracking can be prevented and the pavement life extended if the application of the OGPA is delayed by initially only applying a chip-seal surface. As a chip seal surface is noisier than a Asphalt surface a publicity plan maybe necessary to explain the necessity of applying the quieter asphalt surface at a later date or consideration given to lowering the traffic speed during the bedding in period.

Best Practice

Techniques which reduce the probability of failure are simply good practise and should be considered, examples are:

- Choose a different aggregate source or use modified aggregates (Section 6.3) if early rutting or shoving failures have occurred on other road projects with the aggregate concerned;
- Ensure the unbound base aggregate passes the Repeated Load Triaxial test detailed in TNZ M22;
- Use performance based specifications with a warranty period of at least 2 years;
- Increase and audit quality control;
- Seal in Summer;
- Reduce the degree of saturation of the basecourse to below 60% prior to sealing.

Granular Pavement Limitations

Granular pavements can fail in shear and very large ruts occur (Figure 8.2) along with shoving due to weakness in the aggregate, generally when wet. Delaying the application of an asphalt surface is not a remedy for this pavement weakness. To solve this problem, Modified Aggregates (Section 6.3) should be used in place of unbound aggregates or another source of aggregate of known good performance should be used.

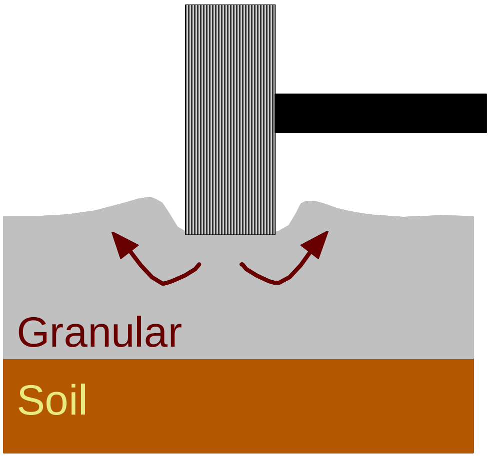

Figure 8.2 Shear failure within granular material usually caused by water and aggregates that reduce in strength if wet.

Recent research at Transit's accelerated pavement testing facility (CAPTIF) and modelling rutting of granular pavements (Arnold, 2004) suggests that thick unbound aggregate layers can be prone to rutting irrespective of the density of the layer. While many other factors are involved, rutting may occur simply as a result of shear displacement of aggregate particles under repeated loading (Figure 8.3). Therefore, heavily loaded pavements require a degree of shear strength that unbound aggregates may not be capable of providing. Table 8.1 reflects this finding by assigning lower probability of rutting to modified and bound materials.

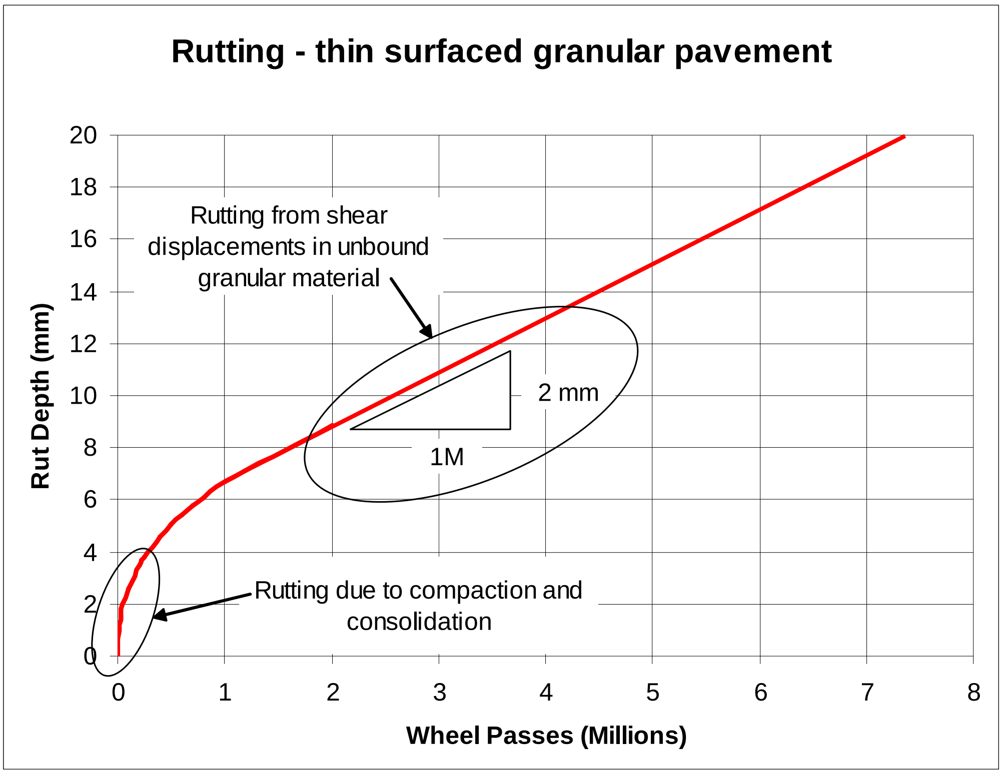

Figure 8.3 Typical rate of rutting in thin surfaced unbound granular pavement (from CAPTIF tests).

Several major thin surfaced unbound granular roads owned by Auckland and Manakau City Councils have shown significant rutting (rut depths >15mm over > 10% of its length) after 7 years in use and 7 million wheel passes (ESAs). This progression of rutting is similar to the CAPTIF result shown in Figure 8.3.

8.1.6 UAB - Unbound Aggregate Base

Thin-surfaced unbound aggregate bases are the most commonly used pavements in New Zealand. Generally, they perform satisfactorily for design traffic volumes up to 10 Million ESAs. Higher traffic volumes are possible but initial early rutting can be expected and is nearly impossible to eliminate, thus the risk of premature cracking failure of any planned asphalt surfacing should be managed by delaying the application of the asphalt layer.

Best Practice and Granular Pavement Limitations (see above in Section 8.1.5)

8.2 Mechanistic Procedure

8.2.1 Design of a Pavement with Soft Subgrades

Subgrades that have, at the time of construction, a measured in-situ CBR <= 3 require a working platform or reinforcement. This is to enable adequate compaction to be achieved in the overlying granular layers and to ensure that fines do not intrude into the pavement structure.

A number of options are available to establish a working platform; they are listed below in Transit's preferred order of use:

- Having established that the subgrade soil is suitably reactive, and that stabilisation is practically viable, stabilise the subgrade to a depth of at least 150 mm and refer to Section 3.4.1 (NZ Suppl) for design criteria.
- Allow for a reinforcing geosynthetic to be placed between the subgrade and the subbase (and elsewhere in the pavement structure if required). Design the overlaying pavement layers in accordance with Section 8.2.3 (NZ Suppl). The provision of a separate or integrated geotextile fabric may be considered necessary to prevent migration of fines from the subgrade into the pavement structure. Note that geotextile fabrics should be selected in accordance with the requirements of TNZ F/7 Specification for Geotextiles.
- Allow a sacrificial depth of 150 mm of granular material and design the pavement assuming no improvement to the subgrade CBR or modulus.

It should be noted that the presence of excess water can be a major contributing factor where there are poor subgrade conditions. Therefore, the pavement designer must ensure that suitable drainage provisions are specified. Effective drainage may eventually result in improved subgrade conditions, however this improvement should not be anticipated in the design.

8.2.2 Design of a Pavement Incorporating a Cement Stabilised Subbase *(refer APDG 8.2.4)*

New Zealand experience suggests that placing a minimum depth of 100mm of unbound basecourse over a cement stabilised subbase should be sufficient to prevent reflection cracking during the post cracking phase of the pavements life.

8.2.3 Design of a Pavement Incorporating Geosynthetic Reinforcement *(refer APDG 8.2.6)*

When appropriate geosynthetic materials are provided at the interface of the subbase and subgrade, increased pavement life or reduced pavement thickness can be achieved from any one, or a combination of, the following four mechanisms (Perkins et. al. 1998):

- Resistance to lateral spreading of the subbase aggregate as vertical loads are applied at the pavement surface.
- Increased confinement afforded to the subbase causing an increase in the lateral stress in that layer and correspondingly an increase in the elastic modulus of the subbase (and base) layers.
- Improved distribution of stress to the subgrade which generally results in the subgrade layer achieving a higher elastic modulus.

- Reduced shear stresses being transferred to the subgrade resulting in lower vertical strains being mobilised in the subgrade.

The various mechanisms of reinforcement described above are specific to the type and configuration of geosynthetic used. The majority of the New Zealand geosynthetic market comprises geogrid type products that confine the aggregate particles within the apertures of the product. The alternative tension membrane products are rarely used in New Zealand and are not discussed further in this document. To mobilise the tensile benefits of such geosynthetics vertical deflection of the pavement must occur. This deflection may be significant for some overlying pavement materials such as thin asphalt surfacings

A review of geosynthetic reinforced pavements has been published by CROW (2004). The review states that there is insufficient information available to establish a reliable design procedure for geosynthetic reinforced pavements. However, manufacturers claim that a pavement thickness saving of up to one third of the equivalent unreinforced pavement thickness is appropriate. It is up to the geosynthetic supplier to provide relevant and credible evidence that such savings are applicable for the particular product in question. The saving in pavement thickness must not exceed the lesser of 150 mm or one third of the pavement thickness irrespective of the design process used. In addition, the reduced depth must be realised in the subbase layer and not the basecourse layer.

8.2.4 Design Approach for Foamed Bitumen Stabilised Layers

Foamed bitumen stabilized (FBS) layers are designed using an approach that is similar to that used for cemented materials, i.e. using a two-phase design life, i.e.: (Wirtgen, 2004)

Phase One (Seating-in Phase) - the FBS layer achieves a relatively high elastic modulus as the material cures and the water content reaches an equilibrium state.

Phase Two (Steady State Phase) – The FBS layer stiffness reduces until it reaches a steady state condition, sometimes referred to as an “equivalent granular state”. The elastic modulus that is achieved in the steady state phase is dependent on factors such as aggregate quality, bitumen dosage, bitumen stiffness, lime / cement dosage, supporting structure, drainage factors, tensile strength ratio, etc.

While it is possible to analyse the “seating-in” phase using the Austroads hot mix asphalt performance criterion, it is unclear how appropriate the criterion is for FBS materials. Given this uncertainty, it is generally appropriate to design the FBS layer for the steady-state condition only.

As a general guide, the following material parameters are considered to be appropriate, however the actual parameters will be dependent on the factors stated above:

- Elastic modulus of the order of 800 MPa;
- Poisson’s Ratio = 0.3;
- Anisotropic layer;
- No sub-layering.

Care should be taken to ensure that cracking is not a primary mode of failure by limiting the application of cemetitious additives.

At this stage, the majority of the expertise in the field of FBS is held by the contracting industry. Therefore, designers should seek assistance from the industry regarding both the mix design and the layer thickness analysis.

8.2.5 Considerations for Pavement Design at Intersections

Pavements at intersections and roundabouts are subject to loading conditions that are significantly more demanding than those occurring on general highway pavements. In addition, safety requirements dictate specific criteria for geometric and surfacing design. The latter includes premium skid resistance, spray reduction and visibility. Designers should refer to Transit documents T/10 “Skid Resistance Investigation and Treatment Selection” (2002) and NetO 1/05 “Macrotexture Requirements for Surfacings” (2005) for detailed information regarding skid resistance.

It is relatively common to observe rutting, heaving, and sometimes corrugations in the wheel tracks of intersections. Water can pond in the depressions and skid resistance is severely compromised. The deformation of the surface layer can also induce cracking which allows water to enter the pavement structure and weaken the supporting layers.

The relatively high lateral stresses occurring at intersections necessitates that the surface course / basecourse interface receives a high level of attention to detail in both the design and construction stages.

Structural adequacy

The thickness and configuration of the pavement layers must be sufficient to achieve the critical strain criteria. Note that the middle of the intersection can receive loading from more than one approach leg and this should be accounted for in the evaluation of the design traffic parameter.

If the intersection has a hot mix asphalt surface, elastic deflections must be kept at an acceptable level to ensure that fatigue cracking is minimized. As a general guide, Benkelman Beam deflections should be less than approximately 1 mm. Slightly higher deflections may be tolerable if the compliance properties of the surface material have been addressed, e.g. by polymer modification.

Deformation resistance

Relatively high levels of shear stress are imposed on the pavement surface and upper pavement layers as a result of vehicles braking, accelerating and cornering at the intersection. Therefore, the upper pavement materials must have a high level of shear strength to achieve appropriate performance. The use of structural asphalt, concrete or modified aggregate materials should be considered. In most parts of New Zealand, SMA has shown very good performance in terms of shear resistance and favourable surface properties. Another option is to use dense asphaltic concrete mixes with the surface being grooved to provide drainage and enhanced skid resistance.

Deformation by way of plastic flow of bituminous materials is a common problem at intersections. The visco-elastic properties of hot mix asphalt make this material somewhat susceptible to deformation under slow-moving or stationary loads. Walker and Buncher (1999) reported a ten-fold decrease in asphalt mix stiffness as loading duration increases from 0.1 s to 100 s. Therefore, asphalt mixes that perform well on high speed applications may not perform well in low speed environments. Heat emitted from vehicle exhaust pipes can also affect the properties of the mix.

The effect of these factors can be reduced by specifying appropriate mixes, particularly at the surface, (eg. SMA). The material must have a stable mineral component and a suitable grade of binder. It may be beneficial to change to a higher stiffness bituminous binder in the area of the intersection (FPCWV, 2000). In addition, the volumetrics of the mix should be such that adequate air voids are maintained in the material.

The designer must ensure that the intersection design extends into the approach legs by an appropriate distance. This will generally be dictated by the expected queue length.

Construction Considerations

The construction or rehabilitation of pavements at intersections requires an increased attention to detail, e.g. (TAPA, 2006):

- Thoroughly clean milled surfaces;
- Avoid segregation during production, transportation and placing;
- Ensure proper joint construction;
- Achieve target densities in all layers.

8.3 Considerations for Pavement Widenings

A common practice in the design and construction of carriageway widenings has been to simply excavate the existing shoulder and bring the new pavement up to level using compacted subbase and basecourse aggregates.

This practice results in a discontinuity of materials and layer performance in the area of the interface between the old and the new pavement. The discontinuity can be attributed to a number of factors, most notably:

- segregation of the new aggregate;
- reduced layer stiffness as a result of removing the lateral restraint provided by the shoulder;
- difficulties associated with compacting layers with a narrow or irregular shape.

The majority of widening failures involve a mechanism that starts with differential movement of the pavement surface at the interface of the old and new pavements. The differential movement results in rupture of the surface seal which allows water to enter the pavement structure. Consequently the pavement structure deteriorates and the distress spreads and accelerates. High water pressures can force material out of the basecourse, further reducing the surface waterproofing and promoting the formation of potholes.

One practical solution for widening issues is to ensure that there is homogeneity of base materials across the widening interface. This can be achieved by modifying the upper materials to half or full width of the carriageway. The depth of modification should be of the order of 200 mm and appropriate additive (hydraulic and/or bituminous) should be used to improve the base layer properties. There should be no intention to establish a very stiff, cemented base layer that may be susceptible to fatigue cracking.

Other fundamentals of pavement materials and construction must be observed in widening projects as they would be in any other high-quality pavement construction project, e.g:

- provide adequate drainage;
- step layer interfaces;
- keep widening interfaces away from wheel paths;
- use appropriate materials and additives
- provide suitable compaction, etc.

CHAPTER 9 COMPARISON OF DESIGNS

9.1 General

Whole of life costing incorporating the probability of failure (Appendix 1) along with road user costs should be undertaken to compare designs. However, in some cases the particular pavement type should be chosen based on technical reasons, particularly where other pavement types have failed early in the past. Transit may allow higher risk options where there are restraints on the capital budget.

CHAPTER 10 IMPLEMENTATION OF DESIGN & COLLECTION OF FEEDBACK

10.1 Implementation of Design

Pavement design using the Austroads mechanistic procedure involves the use of three sets of direct inputs to the Circly program, i.e:

- h1, h2, h3, ..., hn : thickness of each layer;
- E1, E2, E3 ......, En : elastic modulus of each layer; and,
- ν1, ν2, ν3, ..., νn : Poisson's Ratio of each layer.

It has been established that Poisson's Ratio does not have a significant effect on the multi-layer elastic analyses and therefore it is reasonable to adopt an accepted presumptive value. However, both h and E can have a significant influence on the calculated pavement life, and consequently, these parameters need to be substantiated at all stages of the construction process. The theoretical life of most pavements is extremely sensitive to the subgrade modulus (E_{sg}) parameter and therefore establishing and verifying E_{sg} in an appropriate fashion is extremely important.

It is vitally important that the values used in design are re-evaluated during construction to ensure that the pavement layer dimensions have been derived from truly representative soil data. Where an elastic modulus value or layer thickness is found to vary from the design value, the cause of the variation should be investigated and the significance that the variation has on the theoretical pavement life determined. If the effect on the pavement life cannot be accommodated then a mitigation strategy should be developed.

10.1.1 Elastic Modulus Testing

Substantiation of the elastic modulus values for the subgrade and various pavement layers can be carried out using a range of test methods, e.g. Scala penetrometer, in situ CBR, portable or conventional FWD, Benkelman Beam, etc. However, the elastic modulus values that are derived from such tests can be highly dependent on the test itself as well as somewhat tenuous correlations. Factors such as:

- Non-linear stress / strain responses;
- Anisotropic stress / strain responses;
- Temperature or loading rate conditions;
- Variations in material composition and quality;
- Variations in construction quality;
- Moisture conditions; and,
- Statistical significance of testing frequency,

should all be considered in the analysis of the test results. A small disparity between the design and implementation conditions for any of the above factors can have a major influence on the pavement modelling process.

The water content of the materials needs to be taken into account separately from quality control issues and two scenarios should be considered. In the situation where the water content of the pavement layer is greater than that on which the pavement was designed, this may represent the worst-case scenario, which may warrant a revision of the design. A water content less than that adopted in the design may deceive the designer as dry, open structures can readily collapse if the water content subsequently increases.

Where laboratory tests are used to determine or substantiate elastic modulus parameters it is essential that the test specimens and loading configurations accurately reflect the conditions in the field. The most important factors in this regard are:

- Specimen density;
- Specimen composition and grading;
- Specimen water content; and,
- Test stress conditions.

In addition, a reasonable number of tests should be carried out to ensure the reliability of the results. Isolated test results should not be considered to be conclusive.

Substantiation of design parameters should be treated separately from to the quality assurance testing of materials and construction. There are a number of factors other than those considered in the Circly model that have an influence on pavement performance.

10.2 Collection of Feedback Data

A corollary to the substantiation of design parameters is the feedback obtained by designers regarding the magnitude and consistency of elastic modulus parameters for various materials, construction techniques and environmental conditions. This gives designers a valuable source of information that would supplement the data obtained from subsequent or adjacent site investigations.

REFERENCES

Arnold G. (2004) Rutting of Granular Pavements. PhD Thesis, University of Nottingham, Nottingham, UK

Bailey, R. & Patrick, J. (2001) “Determination of the Structural Number of Pavements on Volcanic Subgrades”, Transfund New Zealand Research Report 213, Transfund New Zealand

Cheung & Dongol (1996) “Freeze-Thaw Effects in New Zealand Pavements”, Transit New Zealand Research Report 51, Transfund New Zealand

CROW (2004) “Thin Asphalt Pavements: Design and Re-design”, CROW Publication 157 – English Translation, Netherlands

FPCWV (2000) “Intersections Present Design Challenges”, Centerline, Flexible Pavements Council of West Virginia, USA.

Jameson. G.W. (2005) "Summary of Australian Motorway Pavement Designs” ARRB Contract Report VE71170 for Transit New Zealand, ARRB Transport Research Ltd, Victoria, Australia

Jameson. G.W. (2001) “Technical Basis for the 2001 Austroads Pavement Design Guide (Flexible Pavements), APRG Document 01/07(PD), ARRB Transport Research Ltd, Victoria, Australia

Jameson G.W. (1996) “Origins of Austroads Design Procedures for Granular Pavements”, Research Report ARR 292, ARRB Transport Research Ltd, Victoria, Australia

Jones A. & Bell, A, (2004) “A New Approach to Making Decisions About the Type of Pavement to be Adopted”, Road System & Engineering Technology Forum, Queensland, Australia

Kvatch I, Patrick J.E, Dravitzki V, (2005) “Low Noise Surface Performance Monitoring”, Central Laboratories Report 5-21087.00, Opus International Consultants, Wellington.

Mara M K (xxx): “Traffic Stream Data, Road Categories, Traffic Counting Strategies.” Transfund New Zealand Research Report No. 96, Transfund New Zealand

TAPA (2006) “Heavy Duty Hot Mix Asphalt Intersections”, Texas Asphalt Pavement Association Technical Articles, www.txhotmix.org/publicarticles.

Vorobieff, G. (2004) “Modification Versus Bound Pavements”, NZIHT Stabilisation of Road Pavements Seminar, Auckland, New Zealand

Vicroads (2004) “Code of Practice for Selection and Design of Pavements and Surfacings”, Vicroads Geopave Materials Technology RC 500.22, Victoria, Australia

Walker, D. & Buncher, M. (1999) “How You Can Build Better Intersections”, Better Roads Magazine, www.betterroads.com/articles/braug99a.htm.

Wirtgen (2004) “Wirtgen Cold Recycling Manual”, Wirtgen GmbH, Germany.

APPENDIX 1 Procedures for the comparison of pavement design.

Introduction

This appendix provides a procedure to compare alternative pavement designs. This approach has been implemented in a spreadsheet, which is also available for download.

Background

This procedure is designed to evaluate a base pavement design option against an alternative option using the Land Transport New Zealand Economic Evaluation Manual (EEM) methodology. There are a number of approaches to this type of analysis, all requiring varying degrees of data. The approach developed is risk based and allows implementation in a spreadsheet using data that is readily available.

A risk based approach allows for the fact that unbound granular chipseal construction is perceived to be less reliable in terms of its early performance than say full depth asphalt but that well constructed unbound pavements can work very well.

The Federal Highways Administration's (FHWA) approach is similar in concept to that proposed however the amount of data required makes it impractical for New Zealand; they themselves cut their process down to just agency costs and travel time savings in their "Realcost" software, ignoring vehicle-operating costs. The Australian Asphalt Pavement Association (AAPA) approach is simpler than the FHWA procedure, effectively concentrating on design and construction risks however suitable data is still not available and while simpler it appears open to bias, they also suggest road user costs are likely to be similar and thus can generally be ignored. The approach taken is simpler still but considers the risks in a more global sense, it considers the probability and consequences of five generic failure scenarios (early to late failure). The probabilities for these generic scenarios have been set through discussion with design and construction experts from industry.

The comparison of alternative pavement designs can also be undertaken using more advanced models in dTIMS or HDM4 however the Land Transport New Zealand's PEM has a number of additional requirements which are not considered in these systems. The PEM considers Noise, Safety and Travel time and these would need to be considered separately and structure of the Evaluation Spreadsheet followed to include these items.

Alternative Pavement Design Evaluation Spreadsheet

The Evaluation spreadsheet effectively has four parts:

1. An initial data sheet, containing the relevant project data, lane kms, discount rates etc.
2. Two sheets for undertaking the economic analysis of the base and alternative options respectively. These contain summaries of maintenance costs, roughness costs etc.
3. A summary sheet to compare the options.
4. And finally a number of subsequent sheets to assist in the develop of the construction and maintenance strategies and their deterioration curves, which are input into the economic analysis sheets in Part 2.

Spreadsheet Summary

The Evaluation Spreadsheet provides the structure for a relatively full comparison of two pavement options according to Land Transport New Zealand's Project Evaluation Manual. Basic guidance is provided on deterioration models and likely lives of typically used materials in New Zealand. Further work is required on modelling the performance of Full depth AC and Concrete options.

The use of advanced deterioration modelling in packages such as dTIMS and HDM4 should be considered as should the development of a more advanced risk based approach.

Spreadsheet Structure

The spreadsheet contains a number of individual sheets to evaluate a base pavement design option against an alternative option. The data that needs to be supplied for each project is highlighted in yellow. The data that needs to be changed as a result of changes in Land Transport New Zealand's PEM is highlighted in green.

Initial Data Sheet

The initial data sheet contains the common project data:

- The Construction Options (Base and Alternative)
  - Option names
  - Surfacing type
  - Construction cost
- Project Data
  - Pavement area
  - Lane km's
  - Number of Households effected by Noise
  - Traffic volume (AADT)
  - Lanes (total)
  - % HCV's
  - Vehicle km's/yr
  - Traffic composition (urban, rural etc)
  - Traffic growth percentage
- Land Transport New Zealand Data
  - Discount rate
  - Transfund cost parameters
    - Roughness
    - Pavement elastic deflection
    - Texture
    - CO_2
    - Noise

Base Option and Alternative Option Sheets

The "Base Option" and "Alternative Option" sheets contain the formula for doing the economic comparison of the options.

The Option spreadsheets require the user to input;

- A construction and maintenance strategy
- The treatment costs for the strategy
- The annual maintenance costs
- The progression of:
  - Roughness
  - Texture
  - Deflection
  - Noise
- Noise change is from the base option to the alternative (this assumes the alternative has already been checked against any RMA requirements). Hence, the base option should start at zero.
- If applicable, safety costs and travel time cost should be included if savings can be made between options.

The prediction of the condition of the pavement could be made using TNZ's HDM models and compared with previous experience indicated by RAMM and additional sheets are provided to assist this.

The output of the base and alternative sheets is compared on the "NPV Summary" sheet.

The remaining sheets are used to assist in building the base and alternative option sheets.

NPV Summary Sheet

The output of the base and alternative sheets is compared on the "NPV Summary" sheet. The option with the lower NPV value is the preferred option.

Additional Assistance Sheet - Maintenance Cost

The "Maintenance Cost" sheet provides annual maintenance costs for New Zealand pavements separated into region and pavement types. The data was obtained from the 2001 dTIMS Model special study, "National Calibration of Maintenance Cost Index", by Opus Central Laboratories. (noted that dTIMS no longer uses a generic Maintenance Cost Model).

The study suggested that maintenance costs were constant with age once maintenance was required.

Additional overseas data is required for concrete pavements and this must be from road controlling authority sources.

Additional Assistance Sheet - Roughness

The "Roughness" sheet predicts roughness progression on thin surfaced unbound granular pavements.

Models are required for structural asphaltic concrete pavements and concrete pavement. The model in this sheet is from Central Laboratories report 91-29301, “Prediction of Road Roughness Progression”.

The spreadsheet uses the HDM3 roughness progression model rather than the HDM4 model as it does not need to be in an incremental form and thus it is simpler to implement.

Additional Assistance Sheet - Texture Model

The “Texture” sheet predicts texture loss progression and is based on the default resealing lives from RAMM and the texture model in “Implementation of dTIMS to New Zealand: Final Report Phase 1”.

Coefficients for model have been obtained from a memo from Peter Cenek of Opus Central Laboratories to Sean Rainsford (MWH New Zealand Ltd) noting an error in the then current dTIMS setup dated 14th April 2003.

The texture of concrete pavements is assumed to remain constant.

The default resealing lives should be used to define the maintenance strategy for surfacing the options. The texture model can then be used to predict the texture during the life of the surfacing.

Additional Assistance Sheet - Deflection and Noise

Deflection is assumed to stay constant for all options and is based on the design deflection. Adjustments for future overlays may be made.

The table provided in this sheet provides indicative noise values referenced from a TNZ P/11 mix. It also provides an indication of possible problems with low texture should various options be considered.

Until further research, currently being undertaken by Transit, clarifies the situation it is assumed that noise remains constant over time.

Additional Assistance Sheet - Delay Costs

Modelling of Delay Costs is based on a method of slices approach implemented in Federal Highways Administrators Real Cost Life Cycle Analysis spreadsheet. Note that realistic estimates of when maintenance can be carried out during the day and night must be made. Estimates are required for:

- Duration of activity
- Work zone capacity

APPENDIX 7.4 PRESUMPTIVE TRAFFIC LOAD DISTRIBUTION

Transit New Zealand's WIM sites are all located on rural state highways, therefore a single presumptive traffic load distribution has been established (see Table 7.4.1).

Table 7.4.1 Presumptive traffic load distribution for New Zealand (rural) state highways.

Load (kN)	Axle Group Type				
	SAST (%)	SADT (%)	TAST (%)	TADT (%)	TRDT (%)
10	0.611	1.783	0.007	0.106	0.000
20	13.478	11.672	0.143	0.841	0.045
30	16.815	17.773	0.253	2.718	0.177
40	14.411	18.999	0.829	4.804	0.513
50	31.016	20.810	1.742	5.950	1.571
60	20.520	11.624	5.431	6.832	3.449
70	2.843	7.855	17.583	6.831	4.521
80	0.222	5.848	22.432	6.907	5.364
90	0.048	2.612	21.742	8.049	5.974
100	0.023	0.733	19.555	9.528	6.681
110	0.008	0.193	8.283	10.857	7.622
120		0.067	1.497	10.630	8.751
130		0.018	0.251	9.182	9.458
140		0.009	0.112	7.295	9.582
150		0.006	0.067	4.956	9.446
160			0.043	2.629	8.458
170			0.029	1.230	6.980
180			0.022	0.466	5.033
190			0.006	0.156	3.154
200				0.047	1.690
210				0.014	0.870
220					0.388
230					0.175
240					0.071
250					0.038
260					0.014
270					0.011
280					0.006
290					0.005
300					0.005
Sum	**100.00**	**100.00**	**100.03**	**100.03**	**100.05**
Proportion	**0.334**	**0.105**	**0.093**	**0.412**	**0.056**

Damage Index	Value
N_{HVAG}	2.4
ESA / HVAG	0.6
ESA / HV	1.4
SAR_a / ESA	1.0
SAR_s / ESA	1.2
SAR_c / ESA	3.6