New Zealand guide to pavement structural design

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More information

Record of Amendments			
Amendment Number	**Subject**	**Effective Date**	**Updated by**
1.0	Document issued	29/6/2017	Martin Gribble
1.1	Added missing table on asphalt design speeds. Corrected some referencing errors.	12/4/2018	Martin Gribble
1.2	Updated references to latest Austroads Part 2.	12/4/2018	Martin Gribble

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Foreword

The 2017 *New Zealand Guide to Pavement Structural Design* replaces the *New Zealand Supplement to the Document, Pavement Design – A guide to the structural design of road Pavements (Austroads 2004)* written by Transit in 2007. This Guide includes additional guidelines for the Engineer in applying the Austroads design procedures resulting from research results and experience gained in New Zealand. The aim is to minimise confusion and promote consistency in design assumptions applied in New Zealand.

The New Zealand Transport Agency is an active member of Austroads and has decided to contribute to and utilise, wherever possible and practical, the practices of that organisation. Therefore the NZ Transport Agency has adopted the Austroads pavement design procedures with variation as detailed in this Guide. 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 *Guide* has been produced to facilitate the use of the Austroads document, Guide to Pavement Technology Part 2: Pavement Structural Design (Austroads 2017) 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.

Pavement technology is continually being researched and changed. For this reason, both Austroads (2017) and this Guide are intended to be living documents and will be regularly amended as new research findings come to light.
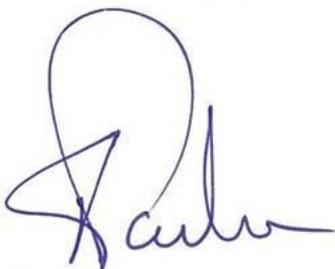
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Abbreviations

BB	Benkelman Beam
CBR	California Bearing Ratio
CV	Coefficient of Variation
DCP	Dynamic Cone Penetrometer
DESA	Design Equivalent Standard Axles
ESA	Equivalent Standard Axles
FBS	Foamed Bitumen Stabilisation
FWD	Falling Weight Deflectometer
FWP	Forward Work Plan
ITS	Indirect Tensile Strength
MESA	Millions of Equivalent Standard Axels
MSG	Maximum Specific Gravity
OGPA	Open Graded Porous Asphalt
OWC	Optimum Water Content
PI	Plasticity Index
PLM	Pavement Life Multipliers
RLT	Repeated Load Triaxial
RMT	Resilient Modulus Test
TAS	Thin Asphaltic Surfacing
TSD	Traffic Speed Deflectometer
UCS	Unconfined Compressive Strength

1 Introduction

1.1 SCOPE

This NZ Guide to Pavement Structural Design provides a New Zealand context for the document "Guide to Pavement Technology Part 2: Pavement Structural Design" (Austroads 2017). It replaces the NZ Supplement published by Transit (Transit 2007).

The section numbers used in the supplement generally correspond to the section numbers used in the Austroads Guide Part 2. New section numbers are used where additional information is provided for the benefit of the New Zealand pavement design community and where different numbering to the Austroads Guide Part 2 is used the appropriate Austroads Guide Part 2 section is referenced in the text.

The Austroads Guide Part 2 is one of ten documents published by Austroads as a comprehensive set of design guides addressing a range of pavement design, materials and construction topics. The Austroads Guide Part 2 incorporates a number of updates from previous Austroads Pavement Design Guide documents, 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.

Designers are encouraged to review Research Report ARR 292 *Origins of Austroads design procedures for granular pavements* Jameson (1996) and the Technical Basis for the 2001 *Austroads Pavement Design Guide* (Jameson, 2001). Adaptation of the Austroads Pavement Design Guide for New Zealand Conditions (Gribble and Patrick 2008) also provides some New Zealand context.

This supplement should be applied in conjunction with relevant NZ Transport Agency materials and construction specifications and standards.

1.2 SAFETY IN DESIGN

Safety in design requires the designer to consider the risks associated with the construction, operation, maintenance and rehabilitation of the pavement and to eliminate or minimise any risks.

2 Pavement design systems

2.1 PROJECT RELIABILITY

Project reliability is discussed in Austroads (2017).

The desired project reliability is chosen by the NZ Transport Agency or the road designer, depending on the project procurement process for any given project.

Minimum project reliability values are detailed in Table 1 for the various the NZ Transport Agency One Network Road Classifications (ONRC)[1].

Table 1 Minimum project reliability as a function of road classification.

One Network Road Classification[1]	Minimum Project Reliability (%)
National (High Volume)	97.5
National	95
Regional	90
Arterial	
Primary Collector	
Secondary Collector	
Access	80
Access (Low Volume)	

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[1] ONRC definitions are available at: *https://www.the NZ Transport Agency.govt.nz/assets/Road-Efficiency-Group/docs/functional-classification.pdf*

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* (The NZ Transport Agency, 2007), for more information.

3.2 EXTENT AND TYPE OF DRAINAGE

Austroads (2017) discusses drainage in section 4.2 where is acknowledges that moisture has a major influence on pavement performance.

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 B2:2005 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, with the following objectives:
   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;
   d. To intercept water which flows towards the road from the surface of land adjoining the road; and
   e. To prevent concentrated flows across the carriageway as a result of water running along the high-side lip of OGPA surfacing.
2. Pavement drainage, to remove water that enters the pavement's structural layers either from above or from the sides. Care is required to provide suitable drainage in areas where there are contrasts in layer permeability, for example. at repair areas, widenings, medians on super-elevated sections, etc.; and
3. Subsurface foundation drainage, to control potential fluctuations in underground water level from natural or perched water tables 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.

Where pavement drainage is achieved by "daylighting" granular layers to the edge of the carriageway, the daylighted layers should be fully exposed and not covered with topsoil unless it can be established that the topsoil will not impede the flow of water, even in a consolidated state.

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, particularly under pressure from vehicle tyres. 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 cemented subbase layer can exacerbate this issue as water will tend to be trapped in the base layer and flow within the layer rather than draining out from the subbase. A cemented subbase must be graded to a pavement drain or the overlying layer(s) must be daylighted at the edge of the carriageway to allow water to escape from the base layer.

Care should be taken to ensure that, where a cemented subbase layer is not continuous across the full width of the carriageway, it does not trap water at the interface of the cemented and untreated (or modified) aggregate. Similarly, cemented layers must not be constructed over the top of subsoil drain trenches.

Where layer modification is discontinued within the pavement cross-section (typically) there should not be a significant contrast in permeability between the modified and untreated aggregate and therefore the drainage properties through the full width of the layer can be considered to be continuous. An exception to this situation could occur if the aggregate is prone to significant breakdown under the action of the stabilising hoe. In this case the modified material could be considerably less permeable than the untreated material and suitable drainage provisions must be included in the design. Permeability testing of the aggregate with and without additive, and with appropriate

conditioning to simulate the hoeing action, should be undertaken for large projects, or whenever a potential contrast in pre versus post modification permeability is suspected.

3.2.2   Use of boxed construction

Boxed construction is allowable as long as suitable drainage is provided to ensure that any water that enters the pavement structure has an opportunity to escape and cannot be trapped within the pavement layers.

3.3   IMPROVED SUBGRADES

Information on improved subgrades may be found in Austroads (2017) section 3.14.

3.4   SOFT SUBGRADES

Soft subgrades are discussed in section 3.14 of Austroads (2017).

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.

Mix design testing in the laboratory for lime stabilised subgrade samples is undertaken using hydrated lime (for safety reasons) whereas lime oxide is typically used in construction. Hydrated lime contains a lower proportion of active calcium compared with lime oxide, therefore the application rate used in construction is generally overstated by approximately 20% to 30%. This is not considered to be an issue as a small amount of overdosing provides an allowance for the superior mixing and compaction conditions associated with laboratory-prepared test specimens. In addition, nominal overdosing of a stabilised subgrade layer is unlikely to have deleterious performance effects.

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. equations 39 and 40, of Austroads (2017). 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, e.g. 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, Austroads (2017)).

Section 8 of this Guide provides additional information with respect to pavement design for soft subgrades.

3.5   SPRAYED SEALS

Refer Austroads (2017) section 3.15.1 and Chipsealing in New Zealand (Transit 2005) for information on designing and constructing sprayed seals.

Primer seals, with high levels of cutter, are not considered to be appropriate for use in New Zealand.

3.6 OPEN-GRADED POROUS ASPHALT

Open-graded porous asphalt (OGPA) is used extensively in New Zealand, particularly on motorways and free-flowing urban arterial roads. As OGPA is porous, it must be placed on an impermeable membrane with either drainage from the edge of the OGPA or, a detail must be provided which allows for drainage of the OGPA without compromising 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 newly constructed unbound granular pavements is nearly impossible to limit. Using current standards it is advisable to allow three months of normal loading prior to applying OGPA. However if delaying the OGPA application is not possible, 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 of 500 MPa irrespective of the speed and temperature environment.

3.6.1 Epoxy modified open graded porous asphalt

The NZ Transport Agency draft specification P11E was introduced in 2014 to specify the properties of Epoxy Modified Open Graded Porous Asphalt (EMOGPA). OGPA manufactured with epoxy modified binders has been found to be extremely resistant to oxidation and is expected to produce much longer lives than those achievable with conventional or polymer modified binders.

Dilution of commercially available epoxy bitumen by a factor of up to four (i.e. 25% epoxy bitumen) with standard 80/100 penetration grade bitumen is recommended. Dilution reduces costs whilst still achieving significant gains in the ravelling performance of the EMOGPA, however, no significant rheological improvements are generated at this level of dilution.

Epoxy bitumen is a two part system consisting of epoxy resin and a hardener-bitumen blend. As with standard epoxy materials the two components are mixed just prior to use and curing takes place over time. Epoxy modified bitumen uses a slow curing chemistry so that it can be handled and applied at high temperatures (between 100 and 150 °C) without excessive curing. Epoxy modified bitumen differs from other polymer modifiers in that it is thermosetting (i.e. it will not melt once cured), and cures to a flexible rubbery consistency.

The long lives expected for EMOGPA means that the material is not suitable for use in higher stress situations where the EMOGPA aggregate will polish and eventually result in poor skid resistance, however, the expectation is that EMOGPA will be used, where appropriate, on Roads of National Significance.

Where EMOGPA is proposed to be placed then the underlying pavement should be assessed to ensure that it has sufficient remaining life for the expected long life of the surfacing. Furthermore, the stiffness of the pavement should be assessed to ensure that the EMOGPA is not subjected to strains that will compromise the life of the surfacing.

While a number of projects have been constructed using EMOGPA the design expertise lies with David Alabaster, Principal Pavement Engineer, New Zealand Transport Agency, and he should be contacted to provide advice on materials and construction techniques.

3.6.2   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 :- 1 l/m^2 CQ-60 emulsion (or equivalent) plus a Grade 5 chip.

- Placing OGPA over an existing granular layer: - establish a minimum of 3 l/m^2 (residual) binder to provide suitable waterproofing. This is likely to involve constructing a prime coat and a racked in or single coat seal, and then a reseal prior to placing the OGPA.

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 and Dongol, 1996).

5 Subgrade evaluation

5.1 GENERAL

The pavement design process is highly dependent on the ability to adequately characterise the properties of the subgrade, as well as materials proposed for use in subgrade improvement, subbase, base and surfacing layers. To achieve this, site investigations must be thorough and well-considered. The ability to stage site investigations is beneficial as more targeted testing can be carried out depending on the results of initial tests. In particular, site investigations should be developed to include consideration of factors such as the effects of moisture, compactive effort, temperature, loading, construction methodology, introduction of additives, etc.

Designers are referred to Section 4 of the New Zealand Guide to Pavement Evaluation and Treatment Design (The NZ Transport Agency, 2017) for information regarding best practice for site investigations. While the above reference is primarily focused on pavement maintenance and rehabilitation, most of the site investigation considerations can be applied to green-fields applications.

5.2 DEFLECTION TESTING

Refer Austroads (2017) section 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).

5.3 SUBGRADE TESTING

Information on subgrade testing can be found in Austroads (2017) section 5.6.

Soaked laboratory CBRs are generally appropriate for green-field 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.

Establishing a correlation between soaked CBR and field test parameters, such as in situ CBR, Scala Penetrometer, etc, is difficult as there are a number of variables inherent in the various test conditions and procedures. The recommended methodology for verifying the soaked CBR of subgrade and subgrade improvement layers (SIL) during construction is to take representative samples from the field for laboratory soaked CBR testing. Samples of cohesive soils can be obtained using thin-walled sampling tubes, while samples of non-cohesive soils can be taken and reconstituted in the laboratory to match the in situ water content and density. Note that the latter may require scalping of the top particle sizes which could affect the response of the material. This type of testing can only realistically be carried out at a relatively low frequency due to programme and cost constraints. Therefore proof rolling and deflection testing (where applicable) is recommended (in addition to CBR testing) for achieving suitable test coverage of subgrade and SIL areas.

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.

Subgrade sensitivity can be assessed by carrying out shear vane tests and recording the peak and residual shear strength of the soil. The Shear Strength Ratio is defined as the ratio of the undisturbed (peak) shear strength to the remoulded (residual) shear strength. The greater the Shear Strength Ratio, the greater the risk that the subgrade soil could lose strength as a result of disturbance from construction activities.

The NZ Geotechnical Society (2005) defines levels of soil sensitivity as shown in Table 2. Where a subgrade is described as being "sensitive" (or worse), i.e. having a Shear Strength Ratio of four or higher, the pavement design should address the potential for loss of subgrade strength during construction.

Managing sensitive subgrades could involve:

- assessing the subgrade strength based on its remoulded strength rather than its undisturbed strength;

- limiting the types of plant that can traffic the subgrade;

- not allowing vibratory compaction on the lower lifts of the pavement;

- bridging the subgrade using a cemented (or otherwise bound) form of subbase; and,

- setting a minimum vertical clearance to the sensitive soil layer.

Table 2 Definition of soil sensitivity levels (NZ Geotechnical Society, 2005).

Definition	Shear Strength Ratio
Insensitive	< 2
Moderately Sensitive	2 – 4
Sensitive	4 – 8
Extra Sensitive	8 – 16
Quick	>16

5.4   LIMITING SUBGRADE STRAIN CRITERION

Refer Austroads (2017) section 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:

- 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 $E_{v(sg)}$ = 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.

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 M4. Aggregate conforming to the M4 specification would generally correspond to the "high standard crushed rock" material referred to in the Austroads Guide.

The NZ Transport Agency T15 (2014) specifies the Repeat Load Triaxial (RLT) test as a means to assess the expected performance of aggregates in terms of rutting resistance. Aggregates that do not meet the NZ Transport Agency M4 specification can be tested using the RLT to assess their suitability for use at low traffic sites.

Conversely, the RLT (to the NZ Transport Agency T/15) can be used to assess M/4 compliant aggregates that may be proposed for relatively highly trafficked applications. RLT testing will identify premium aggregates and therefore reduce the risk of early failure that a lesser quality material may attract.

6.3 MODIFIED GRANULAR MATERIALS

Experience indicates that some aggregates that do not comply with the Transit M4 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 the unmodified basecourse. As described above, the RLT can be used to assist in the assessment an aggregate's performance in terms of rut resistance, this process requires RLT testing of the unmodified aggregate and ITS testing of the modified aggregates. The RLT test results and ITS results are combined to estimate the rutting performance of the modified material. Details of the approach can be found in New Zealand Transport Agency Research Report 498 (Alabaster *et al.* 2013).

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, however when using these limits care is required to use the testing methods including sample preparation, mould sizes and compaction techniques.New Zealand typically controls the maximum amount of binder and further advice is given in T19 (NZTA 2017).

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. However, local research indicates that modified aggregates that may initially be lightly cemented will tend to revert to effectively premium unbound aggregate in a short to medium timeframe. Therefore, modified base layers should be assigned an EV value (in Circly models) not exceeding 500 MPa.

There are two methods to add chemical binder to aggregate for modification; either add the binder at a stationary plant such as a pugmill or stabilise in situ with a hoe. An 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 terms of achieving compaction requirements.

The action of a stabilising 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 forward speed, drum rpm, hoe tip configuration, stabiliser gate setting, 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, guidelines regarding mix design criteria are presented in The NZ Transport Agency T/19 (Notes).

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 Austroad's guide on stabilised materials (Austroads 2006) for detailed information regarding stabilisation issues, however a brief description of stabilising additives commonly used in New Zealand is presented in Table 3.

6.4   CEMENTED MATERIALS

Cemented basecourse materials, with the exception of HiLab, are not appropriate for use on state highways in New Zealand. Cemented layers are prone to cracking due to fatigue and shrinkage, and without a suitable thickness of unbound granular, modified granular or structural asphalt cover, any cracks that form in the cemented layer 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 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 as long as there is a suitable depth of cover, it is unlikely that the crack will be reflected through to the pavement surface.

When constructing a cemented subbase layer, care should be taken to avoid hoeing through the layer and incorporating 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. Fines may also consume a proportion of the binder and reduce the efficacy of the cementing process.

The designer is referred to the Austroads (2006) 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.

Designers can consider "pre-cracking" a cemented subbase by exposing the layer to passes of a vibrating roller after approximately 24 to 48 hours after stabilisation, i.e. before full strength gain has been achieved. The objective of this is to create a series of micro-cracks that relieve shrinkage stresses and avoid the formation of large cracks while maintaining a reasonable level of layer strength and stiffness. Both research and experience indicates that the success of pre-cracking is somewhat varied, with some sites showing significant healing of the micro-cracks, while others show a significant degree of strength loss in the layer which is not recovered. The current recommendation is not to micro-crack a cemented subbase layer unless the designer considers that there are compelling reasons to the contrary and these are discussed and agreed with the Road Controlling Authority.

Consideration should be given to the potential for a cemented subbase to be cracked by construction traffic placing and compacting the overlying base layer. Factors that will affect the potential for cracking include:

- the cement and water content of the cemented subbase;

- the curing conditions and duration;

- the number of passes and axle weights of the construction traffic; and,

- the strength of the layer supporting the CTSB, etc.

Table 3 Application of common stabilising agents

Additive	Process	Effects	Applications
Cement	Cementitious inter–particle 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 and modulus, produces cemented material.	Not limited, apart from aggregates containing deleterious components (organics, sulphates, etc.). Suitable for granular soils & aggregate, inefficient in single sized material & heavy clays.
Lime	Cementitious inter–particle 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 inter–particle 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 and modulus, produces cemented material.	Suitable for aggregate with plastic clay minerals. Organic material will retard reactions. Often used in conjunction with cement.
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.

6.4.1   Determination of design modulus

Refer Austroads (2017) section 6.3.

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 asphalt production in New Zealand have traditionally been classified in terms of penetration grade. As of 2016 asphalt binders are characterised in terms of performance parameters as specified in The NZ Transport Agency M/01-A (2016).

There are no direct correlations between the New Zealand and Australian bitumen classifications, however Table 4 provides a guide to approximately equivalent binders.

Table 4 Approximately equivalent binder classifications

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

Compacted layer thickness limits for asphalt are typically slightly greater in New Zealand compared to Australia. Designers should adopt the minimum layer thickness recommendations presented in the NZ Transport Agency M10 specification (NZTA 2014).

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 5 below.

Table 5 Pavement design speeds for asphalt layer design

Designated Speed Limit V (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 Austroads (2017) 6.5.

The designer is referred to Austroads (2009) for information regarding asphalt mix design and characteristics, however Table 6 provides a brief summary of applications and characteristics for a range of common New Zealand asphalt mixes.

PMB mixes are used in a number of applications to provide superior performance over conventional binder mixes. Austroads (2014) provides information regarding the use of PMB in asphalt mixes and characterisation of the material for the purpose of design modelling.

Table 6 Summary of typical asphalt mix applications and characteristics

Asphalt Mix	Typical Application	Characteristics
DG7	Surfacing for light to moderate duty urban roads. Levelling mix prior to resurfacing – often designed with a PMB	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 content & consequently have good crack resistance but can be more prone to rutting
DG10, DG14	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.
DG20	Structural asphalt pavement layers.	Durable mix for base layers, good fatigue resistance but can be prone to rutting.
AC10, AC14,	Surfacing for heavily trafficked urban roads. AC 14 can be used as a structural layer or a high fatigue layer (with increased binder content).	Excellent rut resistance but slightly lower bitumen content can result in increased permeability, as well as reduced durability and fatigue resistance.
AC20, AC28	Structural layer.	

7 Design traffic

7.1 GENERAL

This section of the Supplement has been developed largely using weigh-in-motion (WIM) data obtained from the NZ Transport Agency 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 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.

Design traffic shall be determined according to the methodologies described in (2017) section 7.

One of the fundamental parameters used in a typical design traffic analysis is the Annual Average Daily Traffic (AADT). The NZ Transport Agency has a number of automated classified count stations on the state highway network that provide suitable data. Count information data from these stations is available on the NZ Transport Agency web site.

8   Design of new pavements

8.1   GENERAL

The pavement type should be selected to ensure a low risk of premature distress and major rehabilitation occurring before the end of the design life. For each pavement type the risk of premature distress can be reduced by implementing options for best practice as detailed in Sections 8.1.1 to 8.1.9.

Pavement design is a risk-based process. In this context, risk is defined as being equal to the probability of failure multiplied by the consequence of that failure. For a particular site, the consequence of premature failure cannot be easily changed. Risk can therefore only be managed by reducing the probability of failure. This is achieved by accurate characterisation of material properties, adopting lower risk pavement designs and a focused attention on the quality of the construction process. At the same time, however, the One Network Road Classification needs to be considered to ensure that the level of risk adopted is appropriate. For example, the use of marginal materials with higher shearing potential may be appropriate on lower classification roads. The intent of this Supplement is to assist designers to appropriately manage the risk of premature pavement failures in New Zealand and, in particular, minimise the risk of failure through informed design.

Assuming appropriate material characterisation and construction methodology the relative performance risk profiles of typical pavement configurations are detailed in Table 7.

Table 7 Risk of failure for pavement types

25 year design traffic volume (ESAs)	Less than 5×10^6	Between 5×10^6 and 1×10^7	Between 1×10^7 to 5×10^7	Greater than 5×10^7
Continuously Reinforced Concrete Pavement	Unlikely to be economic	Unlikely to be economic	Unlikely to be economic	Low risk
Structural Asphalt	Unlikely to be economic	Unlikely to be economic	Low risk	Low risk
Modified aggregate overlay basecourse and bound subbase	Unlikely to be economic	Low risk	Low risk	Medium risk
Foamed bitumen basecourse	Low risk	Low risk	Low risk	Medium risk
Modified aggregate base only	Low risk	Low risk	Medium risk	High risk
Unbound aggregate overlay	Low risk	Medium risk	High risk	High risk

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. The NZ Transport Agency's Risk Management Process Manual was followed to determine a starting point for designers in choosing the most appropriate pavement type. 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).

8.1.1   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 no maintenance is required. For jointed concrete pavements joint maintenance is required and for all concrete pavements surfacing maintenance or replacement is 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.

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 Structural asphalt

Experience from major the NZ Transport Agency and Territorial Local Authorities pavement projects shows that structural asphalt provides a low-risk solution for heavy traffic applications, albeit at a relatively high initial cost. Structural asphalt pavements provide a number of benefits including durability, construction expedience, smoothness, rut resistance, water-proofing, etc.

Success of structural asphalt pavements is dependent on the pavement design and asphalt mix designs being well-considered, as well as the construction methodology and materials production meeting the specified levels of quality.

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 3.4×10^5 ESA (25-year loading of approximately 1.4×10^7 ESA). They also suggest that structural asphalt pavements are viable options for urban roads with a first year loading in the range 2.3×10^5 to 3.4×10^5 ESA and rural roads with a first year loading exceeding 4.6×10^5 ESA.

Methods of designing pavements with structural asphalt layers are well covered in the Austroads (2017). 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.

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, for example at bus stops, intersections, and on tight curves.

- The asphalt mix is designed using the NZTA M10 specification and is appropriate for the vehicle speed and environment, with wheel track rutting testing where appropriate to verify asphalt mix performance;

- Compaction temperatures are appropriate;

- Asphalt mix size is compatible with the proposed lift thickness (or vice-versa);

- Quality control is audited.

8.1.2.1 Structural asphalt 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. In such cases the upper intermediate layer will generally comprise a smaller stone size to promote both water-proofing and ride quality.

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   HiLab pavements

The High strength Low fines Aggregate Base, known as Hi-Lab, has been in use by the agency as part of maintenance rehabilitation work and in trial sections over the past eight years.

Hi-Lab is a pavement system modelled on the original Macadam pavement design philosophy, pioneered by Scottish engineer John Loudon McAdam around 1820. McAdam developed a road pavement design that was comprised of clean broken stone, mixed and laid so it become bound by its angularity into a firm and compact mass.

Hi-Lab is constructed from coarse-graded large stone aggregate, combined with a slurry of cement and finer stones to fill some of the remaining air voids, the grading limits are quite tight compared to traditional basecourse and subbase particle size distribution requirements. The structural strength of the pavement is derived by larger stones interlocking ensuring load transfer through direct stone-on-stone contact. Modern construction techniques are used to ensure the pavement layers are laid and compacted to the appropriate density with the same productivity as traditional construction methods.

Field trial pavements constructed using Hi-Lab materials have shown a lot of promise. The observed performance in testing performed at the Canterbury Accelerated Pavement Testing Indoor Facility in Christchurch has exceeded expectations. The trial sections are performing well after four million load repetitions which is the equivalent of a full pavement design life for a high volume road.

The Hi-Lab material is very stiff and therefore offering a high degree of rut resistance and low deflection under loading, to achieve good long term performance it requires a sound construction platform.

While a number of projects have been constructed using Hi-Lab materials the design expertise lies with Gerhard Van Blerk, Principal Technical Advisor (Pavement), New Zealand Transport Agency, and he should be contacted to provide advice on materials and construction techniques. A draft specification is also available.

8.1.4   Foamed bitumen stabilised base

Foamed Bitumen Stabilisation involves mixing a suitable aggregate with small quantities of bitumen, water, cement and other modifying agents to bind the materials to give increased strength.

Although, considered a low risk option by the roading industry there has recently been some early failures. In some circumstances the foaming process has not achieved the classic properties of a foamed bitumen stabilised material. Ideally very small spots of bitumen are dispersed throughout the sample. However, on occasion "stringers" can occur which indicates poor dispersion.

After investigating failures and consultation with the New Zealand Industry the NZ Transport Agency has decided that foamed bitumen stabilised pavements shall be considered as lightly modified pavements. Therefore, the Portland cement content will be ideally limited to 1% although, where justified, a maximum of 1.25% will be permitted. More emphasis is required for testing the quality and rut resistance of the source aggregate.

In situations, due to very high traffic and road user delays by detours, where the road has to be opened to traffic immediately after construction, the limitations on cement content to 1.25% may render the fresh foamed bitumen layer susceptible to premature rutting by traffic. In these cases the use of foamed bitumen stabilisation may not be practical and another design solution may be required.

A maximum PI of 10 is generally considered to be appropriate for an aggregate that is to be treated with foamed bitumen. Pre-treatment using lime or KOBM may be required for highly plastic materials.

Care should be taken when designing bitumen application rates for chip seals over FBS base layers as the foamed bitumen in the base layer can be drawn into the seal and cause flushing. Therefore, bitumen application rates for chip seals over FBS bases should be reduced by typically 10 to 20%.

Foamed bitumen mix design testing is carried out according to the procedures described in the NZTA T19 specification and the associated notes.

8.1.5   Modified aggregate base and cemented subbase

Figure 2 details two components of rutting that occur within an unbound granular layer, they are densification and shear related. A cemented subbase 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.

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;

* Gradings with larger stones are used (e.g. GAP 65) where modified and stabilised aggregates are achieved in situ using a hoe;

* Modified and cemented aggregates are plant mixed using a pugmill at the correct moisture content and gradings;

* Quality control is audited.

8.1.6 Modified aggregate base only

A Modified Aggregate Base (Section 6.3) is considered to reduce rutting (Figure 2). Research has been completed at CAPTIF that aimed to quantify this reduction in rutting. Following the advice in by Alabaster *et al.* (2013) will ensure that the design life is reached on high volume roads before a 20 mm 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.

Comments in Section 8.1.4 regarding good practice also apply to the Modified Aggregate Base pavement configuration.

## 8.1.7 Unbound aggregate base - delayed OGPA application

The compaction related rutting shown in Figure 1 occurs during approximately the first hundred thousand heavy axle passes. This initial rutting cannot be eliminated with a few hundred passes of rollers and/or a water cart. Testing 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 an asphalt surface a publicity plan may be 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.

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 NZTA T15;

- Increase and audit quality control;

- Seal in summer;

- Reduce the degree of saturation of the basecourse to below 60% prior to sealing.

8.1.8 Granular Pavement Limitations

Granular pavements can fail in shear and very large ruts occur as shown in Figure 1, 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.
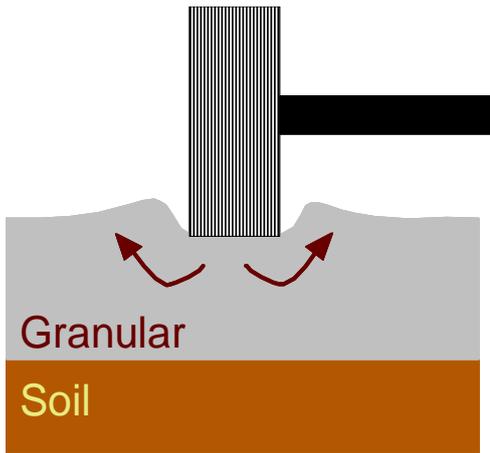
Figure 1 Shear failure within granular material usually caused by water and aggregates that reduce in strength if wet

Research at the NZ Transport Agency'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 2. Therefore, heavily loaded pavements require a degree of resistance to plastic strain that unbound aggregates may not be capable of providing. This is recognised by assigning a lower probability of rutting to modified and bound materials compared to unbound granular materials.
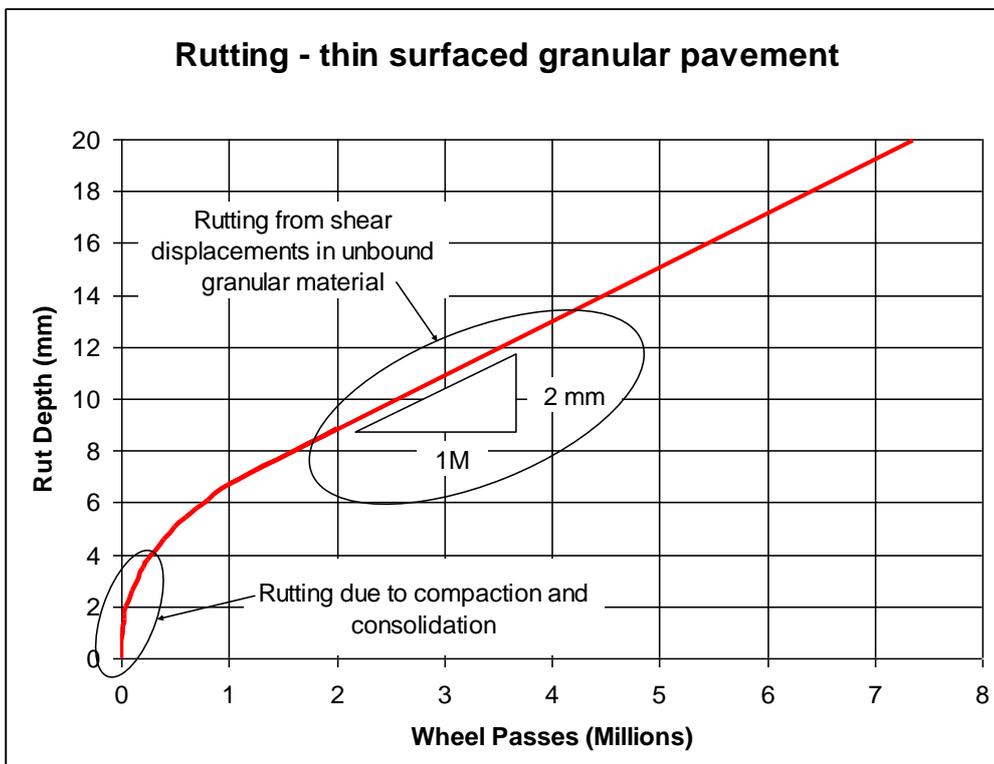
Figure 2 Typical rate of rutting in thin surfaced unbound granular pavement (from CAPTIF tests).

Several major thin surfaced unbound granular roads owned by Auckland Transport have shown significant rutting (rut depths > 15 mm over for more than 10% of its length) after seven years of service (approximately 7 MESA). This progression of rutting is similar to the CAPTIF result shown in Figure 8.3.

8.1.9   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 5 MESAs. 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. Alternatively, a more robust pavement configuration should be adopted.

Best practice and granular pavement limitations are described above. (see Section 8.1.6)

9   Mechanistic procedure

9.1   THIN ASPHALT SURFACINGS

Mechanistic pavement design shall not rely on the stiffness of a thin asphalt surfacing being any greater than the stiffness of the underlying layer (or top sub-layer where applicable). In addition, OGPA and SMA surfacing moduli are subject to the following limits:

- $E_{OGPA} \leq 500$ MPa

- $E_{SMA} \leq 1250$ MPa

The above stiffness criteria can be adopted without adjustment for speed or temperature.

9.2   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 the NZ Transport Agency'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 180 mm and refer to Section 3.4.1 of this Guide 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 9.4 of this guide. 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.

9.3 DESIGN OF A PAVEMENT INCORPORATING A CEMENTED SUBBASE

When designing a pavement with a cemented subbase section 8.2.4 of Austroads (2017) should be considered.

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

There is an option to use the limiting tensile stress method for designing uncracked cemented subbase layers. This involves configuring the thickness of the cemented subbase, as well as the type and thickness of the overlying base layer, such that the tensile stress at the bottom of the cemented subbase is no more than 50% of the flexural beam strength of the cemented material. This produces a non-failure equilibrium situation where, as long as the axle loads are not increased significantly, the cemented layer effectively has an indefinite fatigue life.

The tensile stress at the bottom of the cemented subbase layer is determined by analysing the proposed pavement model using Circly with the cemented layer assigned an elastic modulus value in the range 3,000 to 10,000 MPa.

The flexural strength is determined using the flexural beam test with the test specimens prepared at 95% of the material's maximum dry density. Alternatively, the ITS test can be used in place of the flexural beam test, again with the test specimens prepared at 95% of MDD. If this approach is used, the flexural strength is generally approximated as being twice the ITS value. A further approximation is that the elastic modulus of the cemented layer is, $E_{CTSB} = 10,000 \times ITS$.

As for any pavement configuration, factors such as quality of materials and construction, curing conditions, drainage and water-proofing must be suitably specified to ensure that the cemented layer achieves its optimum performance properties.

9.4 DESIGN OF A PAVEMENT INCORPORATING GEOSYNTHETIC REINFORCEMENT

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.

9.5 DESIGN APPROACH FOR FOAMED BITUMEN STABILISED LAYERS

All NZ Transport Agency projects using bitumen stabilised materials shall follow the applicable standards and specifications for design and construction, in particular the NZ Transport Agency specifications B6 (2012) and B7 (2012), with the exception of clauses detailed in this part of the Guide, which have been taken from the Asphalt Academy's document Technical Guideline: Bitumen stabilised materials, colloquially known as TG2 (2009) as well as recent NZ Transport Agency findings.

The mix design for the bitumen stabilised base or subbase aggregate will not be accepted unless the criteria of this part of the Guide are met. If the proposed mix design does not meet this Guide then the pavement design and/or the stabilised mix design shall be changed until the requirements of this Guide are met.

The pavement layers under the foamed bitumen layer need to follow the methodologies of Austroads (2017), particularly in regard to modulus gain for different material types over subgrade.

The achieved modulus of the foamed bitumen layer shall be limited to five times the underlying modulus up to a maximum of 800 MPa. The foamed bitumen layer shall not be sublayered unless this is required to meet the requirement that the achieved modulus is less than five times the underlying modulus. The underlying support shall have a stiffness greater than 100 MPa with a thickness greater than 100 mm; construction on a less stiff subbase shall not be permitted.

If sublayering is required and the Foamed Bitumen Stabilised (FBS) layer thickness is 220 mm or greater, then the FBS layer can be split into two sublayers with the lower layer having a modulus of 400 MPa and the upper layer having a modulus of 800 MPa. Sublayering of foamed thicknesses of less than 220 mm shall not be permitted.

The modulus of foamed bitumen material under asphalt greater than 60 mm in thickness shall meet the requirements for a premium aggregate as detailed in Table 6.5 in Austroads (2017). Asphalt thicknesses greater than 40 mm shall be modelled for fatigue performance. Asphalt thickness 40 mm or less shall be treated as detailed in Section 7.2.9 above.

Poisson's Ratio shall be equal to 0.3.

The horizontal modulus is half the vertical modulus, meaning that the degree of anisotropy is 2.

Foamed bitumen layer shall be equal to or thicker than the basecourse thickness required by Austroads figure 8.4 (Austroads 2017).

9.5.10 Material properties

The aggregate to be stabilised shall have a plasticity index less than 10. Where the PI is greater than 10 then the plasticity shall be treated with lime, KOBM or another such product to reduce the plasticity to less than 10.

For construction where virgin aggregate is to be bitumen stabilised the Particle Size Distribution of the source aggregate shall meet the requirements in TG2 (2009) in the ideal category and all other requirements of the NZ Transport Agency M4. For pavement materials that are to be recycled while it would be preferable to have an ideal grading, however, the less suitable grading could be considered. Table 8 details the grading requirements with these values being obtained from TG2 (2009). The NZ Transport Agency B5 (NZTA 2008) notes provide guidance on requirements for imported aggregates used to blend with recycled aggregates to improve their performance.

Table 8 Particle size distribution requirements

	Percentage passing			
	BSM emulsion		BSM foam	
Sieve size (mm)	Ideal	Less suitable	Ideal	Less suitable
50	100		100	
37.5	87-100		87-100	
26.5	77-100	100	77-100	100
19.5	66-99	99-100	66-99	99-100
13.2	67-87	87-100	67-87	87-100
9.6	49-74	74-100	49-74	74-100
6.7	40-62	62-100	40-62	62-100
4.75	35-56	56-95	35-56	56-95
2.36	25-42	42-78	25-42	42-78
1.18	18-33	33-65	18-33	33-65
0.6	12-27	27-54	14-28	28-54
0.425	10-24	24-50	12-26	26-50
0.3	8-21	21-43	10-24	24-43
0.15	3-16	16-30	7-17	17-30
0.075	2-9	9-20	4-10	10-20

The reactivity of the proposed foamed bitumen and cement additives with the aggregate shall be tested according to the TG2 (2009) using a two phase design life but with the modifications from Wertgen (2004) reproduced as equations 9.1 and 9.2 below.

$$MR_{Phase\ 1} = \left(\log ITS_{equ} \times 3950 - 7000 \right) \times TSR \times F_{drainage} \qquad 9.1$$

$$MR_{Phase\ 2} = \frac{MR_{Phase\ 1} \times TSR}{(0.5 \times UCS_{equ}) + 0.7} \qquad 9.2$$

Where

$MR_{Phase\ 1}$	= Resilient Modulus during Phase 1	MPa
ITS_{equ}	= ITS at equilibrium moisture content	kPa
TSR	= Tensile strength retained (ratio of soaked to unsoaked ITS values)	
$F_{drainage}$	= Drainage factor from Table 9 below	
$MR_{Phase\ 2}$	= Steady state resilient modulus	MPa
UCS_{equ}	= UCS at equilibrium moisture content	MPa

Table 9 Drainage factors, Fdrainage, for estimating field stiffness values of bitumen stabilised material

	Mean annual rainfall (mm)			
Drainage quality	< 200	200 to 600	600 to 1000	>1000
Very good	1.4	1.3	1.2	1.1
Good	1.3	1.2	1.1	1.0
Fair	1.2	1.1	1.0	0.9
Poor	1.1	1.0	0.9	0.8
Very Poor	1.0	0.9	0.8	0.7

The $MR_{Phase\ 2}$ value shall be greater than 800 MPa, however, values significantly greater than 800 MPa will not be considered acceptable either.

As described in Section 6.3, the RLT can be used to assist in the assessment an aggregate's performance in terms of rut resistance, this process requires RLT testing of the Unmodified aggregate and ITS testing of the modified aggregates. The RLT test results and ITS results are combined to estimate the rutting performance of the modified material. Details of the approach can be found in New Zealand Transport Agency Research Report 498 (Alabaster *et al.* 2013)

9.6 DESIGN OF HILAB PAVEMENTS

HiLab pavements involve the use of tightly specified materials and construction criteria, as presented in the NZ Transport Agency HiLab Draft Specification (2017). The design expertise for HiLab pavements lies with Gerhard Van Blerk, Principal Technical Advisor (Pavement), New Zealand Transport Agency, and he should be contacted to provide advice on design, materials and construction techniques.

9.6.11   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 the NZ Transport Agency documents T/10 "Skid Resistance Investigation and Treatment Selection" (2013) 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 pavement design and construction receives a high level of attention to detail.

The NZ Transport Agency Technical Advice Note 17-01 *Asphalt depths at high stress locations for new pavements and renewals* provides[2] details for pavement requirements at intersections. Note 17-01 calls for structural asphalt (or concrete) pavements to be used at intersections with certain loading conditions. The minimum asphalt thickness of 125 mm stated in Note 17-01 is only there as an assurance that structural asphalt is adopted, but it is not a default asphalt thickness. The structural asphalt pavement must be designed according to the requirements of the Austroads Part 2 Guide and this Guide.

The construction 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.

9.7   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

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[2] https://www.nzta.govt.nz/roads-and-rail/highways-information-portal/technical-advice-note/

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 preferably the 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, for example:

- provide adequate drainage, paying particular attention to the continuity of drainage across the interface of the new and existing pavements;

- step layer interfaces;

- keep widening interfaces away from wheel paths;

- consider spanning the widening interface using a reinforcing grid;

- use appropriate materials and additives; and,

- provide suitable compaction.

10 Comparison of designs

10.1 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. The NZ Transport Agency may allow higher risk options where there are restraints on the capital budget.

11 Implementation of design and collection of feedback

11.1   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,

- $\nu1, \nu2, \nu3, ..., \nu n$: 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.

Deflection testing using a Falling Weight Deflectometer (FWD), a Traffic Speed Deflectometer or a Benkleman Beam (BB) is a valuable tool for monitoring the stiffness response of pavement layers during, and at the end of construction. However, due consideration should be given to both the accuracy of the measuring process and the precision of any deflection criteria specified for a given pavement layer. In addition, a suitable number of test results should be collected to ensure that the data is statistically representative of a given pavement area.

Generally, deflection tests, using the FWD and the BB, should be carried out at 10 to 20 m centres in each wheel track, however the test frequency could be increased or decreased depending on the objectives of the testing and the consistency of the pavement response. Irrespective of the test frequency adopted, a minimum of 30 data points is considered to be appropriate for any given pavement area under consideration.

While the FWD generally provides accurate and repeatable data, the Benkelman Beam has been shown (in some instances) to have significant issues in terms of repeatability and reproducibility of test results. This can be attributed to inconsistencies in the equipment, particularly the configuration and operation of the test truck, as well as the operation of the beam itself. Automated beam equipment may alleviate some of these issues, but inconsistencies related to the test truck set-up remain.

Caution should be exercised when specifying deflection criteria as the complexity of pavement material responses cannot be modelled with true precision given the assumptions and simplifications that are adopted in most pavement

models. Therefore, deflection criteria should be seen as targets that are used for identifying areas that may require additional investigation, rather than being used as hard and fast, pass/fail acceptance criteria.

11.1.12 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;

- test stress conditions; and,

- test loading frequency.

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.

11.2 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.

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12 Appendix 1 Procedures for the comparison of pavement design.

12.1 INTRODUCTION

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

12.2 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.

12.3 ALTERNATIVE PAVEMENT DESIGN EVALUATION SPREADSHEET

The Evaluation spreadsheet effectively has four parts:

1. An initial data sheet, containing the relevant project data, lane kilometres, 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 kilometers

  ➢ Number of households effected by noise

  ➢ Traffic volume (AADT)

  ➢ Lanes (total)

  ➢ % HCVs

  ➢ Vehicle kms/yr

    ➢   Traffic composition (urban, rural etc.)

    ➢   Traffic growth percentage

- Economic evaluation manual data

    ➢   Discount rate

    ➢   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 calibrated 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 P11 mix. It also provides an indication of possible problems with low texture should various options be considered.

Until further research, 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