NEW ZEALAND SUPPLEMENT TO THE DOCUMENT, Pavement Design – A Guide to the Structural Design of Road Pavements (AUSTROADS, 1992)

2000

ISBN ??????????

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Transit New Zealand 2000. New Zealand Supplement to the Document, *Pavement Design – A Guide to the Structural Design of Road Pavements* (AUSTROADS, 1992)

Keywords: pavement, design, road, rehabilitation

FORWARD TO 2000 EDITION

The 2000 New Zealand Supplement (NZ Supp.) will replace all earlier versions of the NZ Supp. This NZ Supp. includes additional guidelines for the Engineer in applying the AUSTROADS design procedures resulting from research results and experienced gained in New Zealand. The aim is to minimise confusion and promote consistency in design assumptions applied in New Zealand.

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

The AUSTROADS document *Pavement Design - A Guide to the Structural Design of Road Pavements* (1992), subsequently referred to as the AUSTROADS Guide, has after study been found to be well suited for use in New Zealand. It has superseded the previous New Zealand pavement design standard, viz. the State Highway Pavement Design and Rehabilitation Manual (Transit 1989).

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

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

R J Dunlop
General Manager

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1 INTRODUCTION

1.1 SCOPE

For ease of use, the section numbers used in this supplement correspond to the section numbers used in the AUSTROADS Pavement Design Guide (1992) under the relevant topic areas. Those sections in the AUSTROADS Guide that did not require amending and/or clarification for use in New Zealand have not been included in this NZ Supp. Consequently, there are numerical gaps in the sequence of section numbers in this supplement. Section 10 (Rehabilitation Design) is a new section, written specifically for NZ conditions and replaces Section 10 (Overlay Design) of the AUSTROADS Guide.

Section numbers marked with "(AUSTROADS Guide)" are linked directly with the same section in the document: *Pavement Design – A Guide to the Structural Design of Road Pavements* (AUSTROADS, 1992) hereinafter referred to as the "AUSTROADS Guide". Conversely those sections with no reference to the AUSTROADS Guide are unique stand alone sections in the NZ Supp. to provide additional information to the Engineer.

Engineers are encouraged to review Research Report ARR 292 *Origins of AUSTROADS design procedures for granular pavements* by G.W. Jameson, ARRB Transport Research Ltd. This will ensure the background to the development of the AUSTROADS Guide is better understood and not misapplied. Also, if you have not already done so please register your name as a holder of the AUSTROADS Guide. This will ensure that you receive updates to the AUSTROADS Guide directly.

The AUSTROADS Guide is based on the philosophy that the design engineer must have a good understanding of the design process and the mechanics of pavement behaviour. The engineer is encouraged to develop the design from first principles with the use of a computer program (e.g. CIRCLY, Wardle 1980) for calculating stresses and strains in multi-layered elastic media. The procedure allows a degree of judgement and experience to be included with an understanding of the mechanisms involved. Where a number of designs are required the engineer can develop a set of design charts for the materials and loadings appropriate to the particular project. A set of example design charts is included in Section 8 of the AUSTROADS Guide.

The pavement design computer program aligned with the AUSTROADS Pavement Design Guide is CIRCLY. However, other pavement design computer programs (e.g. ELMOD, ELSYM5, finite element methods, or spreadsheets using the Odemark-Boussinesq equations) can be used provided sufficient comparative analysis have been carried out to show that the difference in pavement layer thicknesses are not significant in practice to those computed using CIRCLY (Tonkin and Taylor 1998).

Pavement design computer programs provide an estimate of the stress, strain and deflection at any point in the pavement under known loading conditions. The AUSTROADS Guide uses the vertical compressive strain at the top of the subgrade and the horizontal tensile strain at the bottom of the bound layers as an indication of pavement life. These relationships are empirical and the concept was developed from the AASHO Road Test carried out in the 1950s.

This document should be used in conjunction with all relevant Transit and Transfund New Zealand standards.

1.2 DESIGN OF NEW PAVEMENTS FOR LIGHT TRAFFIC

Austroads have produced a document, *A guide to the design of new pavements for light traffic – A Supplement to Austroads Pavement Design. APRG Report No. 21.* This document uses the same design principles as the Austroads Guide and therefore it's appropriate use is acceptable

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3 CONSTRUCTION AND MAINTENANCE CONSIDERATIONS

3.1 GENERAL (AUSTROADS Guide)

The Engineer is reminded of the objectives of the Resource Management Act, including that of promoting the 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 Act also obliges the organisation promoting any development to consult with interested parties and to obtain resource consents for activities which affect waterways or involve earthworks. For further information the reader is referred to Highway Planning Under the Resource Management Act 1991 (Transit New Zealand 1994).

3.2 EXTENT AND TYPE OF DRAINAGE (AUSTROADS Guide)

Engineers are reminded that most pre-mature failures of pavements can be attributed to water. Careful consideration of pavement drainage is required. Some unbound granular basecourses have shown from experience and in the Repeat Load Triaxial apparatus to have poor performance when saturated. For these moisture susceptible materials provision needs to be provided in the pavement to allow water once entered to escape relatively quickly from the basecourse.

3.3 USE OF BOXED CONSTRUCTION (AUSTROADS Guide)

Boxed construction is not appropriate for use in New Zealand as it can lead to the bath-tub effect (Cedergren 1974). This effect is caused when water enters the pavement structure but cannot escape, therefore leading to saturation of the pavement materials and softening of the subgrade.

3.6 USE OF STABILISATION (AUSTROADS Guide

Some parts of this section are now outdated. For relevant details of stabilisation refer to the 1998 AUSTROADS Guide to Stabilisation in Roadworks (AUSTROADS Publication NO. AP 60/98).

4 ENVIRONMENT

4.1 GENERAL (AUSTROADS Guide)

Freeze/thaw conditions do occur in some regions of New Zealand, e.g. in the central North Island and in the South Island. Specific construction techniques and materials are required when freeze/thaw conditions prevail, but there is no change to the pavement design philosophy.

4.2 MOISTURE ENVIRONMENT (AUSTROADS Guide)

See Section 3.2 of this NZ Supp.

4.3 TEMPERATURE ENVIRONMENT (AUSTROADS Guide)

In regions where the temperature of the pavement structure may fall below 0° C, all aggregates used must not be susceptible to freeze/thaw collapse. Also, good drainage must be provided to minimise the quantity of water which can enter the pavement and subsequently freeze (see Freeze-Thaw Effects in New Zealand Pavements, Cheung & Dongol 1996).

5 SUBGRADE EVALUATION

5.2 MEASURES OF SUBGRADE SUPPORT (AUSTROADS Guide)

Use of the modulus of subgrade reaction (k) is required for the design of rigid pavements.

5.3 FACTORS TO BE CONSIDERED IN ESTIMATING SUBGRADE SUPPORT (AUSTROADS Guide)

The relative values of subgrade support presented in Tables 5.2, 5.3 and 5.4 (AUSTROADS Guide) may not be applicable to New Zealand soils. Consideration should be given to measuring the ratio of CBR_{OMC} (California Bearing Ratio at Optimum Moisture Content) to CBR_{SOAKED} (California Bearing Ratio after 3 days soaked in water)

5.4 METHODS OF ESTIMATING SUBGRADE SUPPORT VALUES (AUSTROADS Guide)

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 a soaked subgrade CBR does not make a pavement exempt from moisture problems and the provision of an effective drainage system is always required.

5.5 FIELD DETERMINATION OF SUBGRADE CBR (AUSTROADS Guide)

5.5.1 Insitu CBR Test (AUSTROADS Guide)

Insitu CBR tests should be performed according to the procedures specified in NZS 4402:1986 (Test 6.1.3).

5.5.2 Cone Penetrometers (AUSTROADS Guide)

Dynamic cone penetrometer tests should be performed according to the procedures specified in NZS 4402:1988 (Test 6.5.2). Note the requirement to establish a correlation with local soils.

5.5.3 Elastic Modulus

Insitu subgrade CBR can be estimated from moduli back-calculated from deflection bowl measurements (e.g. from Falling Weight Deflectometer or Benkelman Beam) provided a correlation between moduli and CBR for local soils is established. Benkelman Beam deflection bowls should be used with caution as the accuracy of the device is questionable at low values of deflection.

5.7 LABORATORY DETERMINATION OF SUBGRADE CBR AND ELASTIC PARAMETERS (AUSTROADS Guide)

Laboratory CBR tests on remoulded soils should be performed according to the procedures specified in NZS 4402:1986 (Test 6.1.1) and NZS 4407:1991 (Test 3.15).

5.9 SUBGRADE FAILURE CRITERION (AUSTROADS Guide)

Full scale testing using the Canterbury Accelerated Pavement Testing Indoor Facility reported by Pidwerbesky (1994) showed that the AUSTROADS subgrade strain criterion gave a better characterisation of pavement performance than that of the previous Transit subgrade strain criterion. This was also evident in the work of Steven (1993) who measured strains in forestry roads.

The subgrade strain criterion was derived from the thickness design chart Figure 8.4 (AUSTROADS Guide). There was no field testing and instead several assumptions were made on how to convert a pavement from the chart to a multi-layer linear elastic model for analysis using CIRCLY. One significant assumption was that the vertical modulus of the subgrade = 10CBR and the horizontal modulus is half this value, i.e. anisotropic material parameters. It should be noted that these relationships are equivalent to the isotropic modulus of the subgrade = 6.7CBR. For further details see Tonkin and Taylor (1998).

The intention in deriving a subgrade strain criterion was to ensure that when CIRCLY was used the unbound granular thickness for a thin surfaced pavement would be the same to that obtained from the chart (Figure 8.4, AUSTROADS Guide).

Experience has shown in Australia that pavements designed using Figure 8.4 using actual measured subgrade CBR values generally achieve their design life. This was also shown to be true when investigating the performance of pavements in the Wanganui region over volcanic ash subgrades.

The volcanic ash soils of the central North Island typically demonstrate a relatively high bearing capacity, reflected by high CBR test results, but a relatively low stiffness, reflected by low elastic modulus values. This means that the relationship $E_v = 10CBR$ is generally not appropriate for these materials. There is evidence of volcanic ash soils achieving a CBR of 9% while only producing an elastic modulus of 20 MPa. In addition, pumice soils may show CBR values in excess of 30% while only achieving design elastic modulus values of around 50 MPa.

When volcanic subgrade soils are encountered which show the performance tendencies described above, an alternative material characterisation should be used, i.e.

- For design of thin surfaced unbound granular pavements either:
  - use Figure 8.4 (AUSTROADS Guide) using a measured subgrade CBR for design; or
  - model the proposed pavement using CIRCLY assuming the vertical modulus of the subgrade = 10CBR and the horizontal modulus is half this value.
- For the design of pavements with one or more bound layers:
  - to check for adequate cover/protection of the subgrade, model the pavement using CIRCLY assuming the vertical modulus of the subgrade = 10CBR and the horizontal modulus is half this value; and
  - to check the fatigue life of any bound materials model the subgrade preferably using a modulus value that has either been inferred from deflection tests or measured in the repeated load triaxial apparatus.

6 PAVEMENT MATERIALS

6.2 GRANULAR MATERIALS (AUSTROADS Guide)

An interim guideline for the comparative assessment of unbound granular materials is given in TNZ M/22 Notes. This method uses results from the repeated load triaxial permanent deformation tests to estimate a rut depth within the granular material being assessed. The estimated rut depth is only one parameter considered when assessing compliance of a basecourse as per Transit New Zealand's *Notes For The Evaluation Of Unbound Road Base and Sub-Base Aggregates* (TNZ M/22 (notes): 2000).

6.2.1 Introduction (AUSTROADS Guide)

The requirements for granular basecourse materials are given in Transit New Zealand Specification M/4. Note the use of the terms "high quality crushed rock and base quality gravel". The relationship for New Zealand aggregates needs to be established by the pavement designer.

6.2.2 Modulus and Poisson's Ratio (AUSTROADS Guide)

Laboratory (or in-situ) tests should be conducted on basecourses to determine the appropriate design modulus to use. Suitable test methods are outlined in Table 6.3 (AUSTROADS Guide), however the use of the laboratory Loadman test should also be considered. The designer should always be mindful of the stress dependent and anisotropic response of unbound aggregate materials when analysing test data. The modulus of the basecourse will therefore be influenced by the material's proximity to the applied wheel load as well as the thickness and modulus of any overlying layers.

As with all testing, it is essential that the density and water content of the test specimens, and the configuration of the test procedure provide an accurate representation of the in-service conditions. Isolated test results should also be viewed with caution and multiple test results should be obtained wherever possible.

In the absence of test data, presumptive values of E_v should be taken from Tables 6.4(a) and 6.6 (AUSTROADS Guide).

The modulus achievable in practice is strongly related to the stiffness of the underlying materials. Therefore, the sub-layering requirements in Section 8.2.2 (NZ Supp.) are to be used. Also, it should be recognised that moduli can increase in the early stages of post-construction trafficking.

6.3 CEMENTED MATERIALS (AUSTROADS Guide)

6.3.1 Definition

Cemented materials are those defined in Section 6.6 of the NZ supplement as lightly bound or heavily bound stabilised aggregates.

6.3.3 Determination of the Fatigue Characteristics of Cemented Materials (AUSTROADS Guide)

AUSTROADS has adopted the Queensland fatigue relationships for cemented materials. The equations for these relationships are detailed in the October 1997 updates of the AUSTROADS Pavement Design Guide. A single fatigue equation that is not in the AUSTROADS Guide that can be used is detailed as follows:

$$N = \left(\frac{11266 / E^{0.804} + 190.7}{\mu \in} \right)^{12}$$

where:

N = number of Equivalent Standard Axles (ESA) to failure;

$\mu\varepsilon$ = applied or generated (micro) strain under 1 ESA;

E = modulus of cemented material (>1,000 MPa).

6.3.4 Fatigue Characteristics (AUSTROADS Guide)

6.3.4.4 Cracking (AUSTROADS Guide)

As noted in the AUSTROADS Guide, thermal and shrinkage stresses cause cracks in cemented materials. These cracks will reflect through to the surface prematurely unless the surfacing has been designed appropriately to cope with the stresses and strains involved. Regardless of the surfacing design it is prudent to allow for ongoing remedial treatment for any cracks that may reflect through to the surface.

6.4 ASPHALT (AUSTROADS Guide)

Typical values of elastic modulus for asphalt (see Table 6.4(b) in AUSTROADS Guide) are likely to be in the range 1,500 MPa to over 4,000 MPa depending on the mix characteristics, rate of loading, and temperature. Elastic modulus values for design should be determined by laboratory testing. Nomographs, such as those in Figures 6.6 and 6.7 (AUSTROADS Guide) and appropriate software packages may be used to estimate a modulus for asphalt, but only if laboratory test results cannot be obtained.

6.4.6 Suggested Fatigue Criteria (AUSTROADS Guide)

Now included in updates of the AUSTROADS Guide.

6.6 STABILISED AGGREGATES

6.6.1 Definition

Hydraulic stabilising agents (e.g. lime, cement, KOBM, etc) are often added to an aggregate to improve one or more of an aggregate's properties. This process has been shown to provide considerable benefits in terms of both pavement performance and cost effectiveness. However, pavement designers must be clear regarding their objectives when using hydraulic stabilising agents as their misuse can result in severe pavement distress which can be difficult and costly to rectify. In particular, the potential for cracking is a major issue to consider.

The use of hydraulic stabilising agents with aggregates has been separated into three categories corresponding with the definitions described in Table 4.1 of the 1998 Austroads Guide to Stabilisation in Roadworks (reproduced as Table 6.1 below), i.e.

- modified aggregates;
- lightly bound aggregates; and,
- heavily bound aggregates.

Table 6.1 – Typical Properties of Modified, Lightly Bound and Heavily Bound Materials.

Material Type	Layer Thickness (mm)	[1]Design UCS (MPa)	Design Modulus (MPa)
Modified	Applicable for any thickness	< 1	500 – 1,500
Lightly Bound	Generally $\leq$ 250	1 – 3	1,500 – [2]5,000
Heavily Bound	Generally > 250	> 3	[2]5,000 –20,000

[1] 28 day test results
[2] 2000 MPa is the criteria in the 1998 Austroads *Guide to Stabilisation in Roadworks.*

Modified Aggregates

Modified aggregates generally contain the minimum dosage of stabilising agent required to rectify a shortcoming in the physical properties of the original material. This will typically involve the addition of lime or a similar additive to an aggregate containing excess and/or reactive fines with the objective of reducing the plasticity of the fine fraction and increasing the aggregate's strength when wet (e.g. soaked CBR).

Plasticity reduction can often be achieved with the addition of a very small quantity of stabilising agent. While the use of a minimum dosage is desirable from a technical and economic standpoint, there are practical limitations with respect to achieving a uniform distribution of additive. This, along with other quality assurance issues will be addressed in a separate paper.

With modified aggregates there is no intention to provide significant tensile strength. Therefore, an upper limit of 1 MPa for the 28-day Unconfined Compressive Strength (UCS) (0.8 MPa for 7 days moist curing) is considered to be appropriate. It is recognised that the tensile and compressive strengths for representative specimens will not coincide, however the UCS is often used as it is a readily obtainable parameter. Given the low strength criterion, the modified aggregate should be considered to be completely unbound. Therefore, the design procedure for a pavement with a modified aggregate layer should be no different to that for an untreated, unbound pavement. This includes factors such as sub-layering, anisotropy, maximum modulus criteria and performance criteria.

Finely graded gravels, clayey gravels, silty sands (> 50% passing 425 µm sieve) and other materials which do not achieve significant particle interlock are not suitable for use as base materials when modified, as they may deteriorate rapidly with use. The life of these materials will generally be short and rapid disintegration of the pavement may occur upon the onset of cracking.

Lightly and Heavily Bound Aggregates

When sufficient hydraulic stabilising agent is added to an aggregate to produce significant tensile strength, i.e. 28-day UCS > 1 MPa (UCS at 7 days moist curing > 0.8 MPa) the material can be considered to be *bound*. To fall into the *lightly bound* category the material should have an elastic modulus in the range 1,500 MPa to 5,000 MPa. The *heavily bound* category applies to materials with a 28-day UCS > 3 MPa and a modulus value > 5000 MPa.

When a cemented state has been achieved, the potential for cracking is an issue which must be considered in design. The 1998 Austroads Guide to Stabilisation in Roadworks states that cement bound materials are susceptible to two classes of cracking, i.e. *shrinkage* and *fatigue*. Caution must be exercised when designing and specifying highly cemented materials.

6.6.2 Elastic Parameters

Test methods for determining elastic parameters for cemented materials are described in Table 6.3 (AUSTROADS Guide). It should be noted that few laboratories would have the equipment or experience to carry out all of the stated test methods. Indirect tension and direct compression tests are most likely to be available, however flexure tests are deemed to be the most reliable as the loading configuration best matches that occurring in-service.

While it is possible to back-calculate elastic modulus values from deflection tests, i.e. FWD or Benkelman Beam, this method ***must be used with caution***. The very low deflections associated with cemented pavement layers can cause significant variation in results simply as a result of the limitations of accuracy and repeatability of the measuring equipment.

Another option is simply to use presumptive values based on the data in Table 6.4(a) (AUSTROADS Guide) and the guidance of experienced practitioners.

6.7 STABILISED SUBGRADE SOILS

This section describes the procedure to be used for the design of pavements incorporating a stabilised subgrade soil layer. For the purposes of this procedure, stabilised subgrade soils should be considered to fall into one of three categories. These categories are: modified, lightly cemented and moderately to highly cemented.

To reduce the risk of cracking caused by shrinkage and hydration the amount of stabilising agent introduced to the soil should be limited to the minimum quantity required to achieve the desired effect but still achieve effective mixing.

6.7.1 Modified Subgrade Soils

To qualify as a modified subgrade soil the stabilised soil must have a soaked CBR less than 30% or tensile strength less than 80 kPa after 7 days curing at 20°C.

The objective of soil modification is to improve the properties of the original subgrade by reducing the plasticity and/or moisture content. This provides improved workability and a modest increase in soil strength and resilient modulus. Soil modification does not develop sufficient tensile strength for the layer to act as a slab.

The resilient modulus of the modified soil can be determined using a repeated load triaxial apparatus using conditions (moisture, loading and confinement) similar to those expected in the field. Alternatively, one may use correlation’s between other test parameters and the resilient modulus, e.g. Ev = 10(CBR). Laboratory tests should be carried out using specimens cured for 7 days at 20°C. The laboratory Loadman test is also a valid evaluation method. As with all soil testing, the designer must be mindful of the stress dependence of the material.

6.7.2 Lightly Cemented Subgrade Soils

To qualify as a lightly cemented subgrade soil the stabilised soil should have a tensile strength greater than 80 kPa but a resilient modulus of 5,000 MPa or less. It will generally be treated with additional stabilising agent over that required for simple subgrade soil modification (see Section 6.7.1 NZ Supp.).

6.7.3 Moderately to Highly Cemented Subgrade Soils

To qualify as a moderately to highly cemented subgrade soil the stabilised soil should have a resilient modulus greater than 5,000 MPa.

The cemented subgrade layer is designed to carry a relatively large proportion of the applied load by the development of slab action. For slab action to occur mobilisation of significant tensile stresses at the underside of the cemented layer is required. Therefore the layer must be homogeneous throughout its depth with respect to both binder distribution and material density. Caution must be exercised when designing and specifying highly cemented materials.

7 DESIGN TRAFFIC (AUSTROADS Guide)

7.5 DESIGN TRAFFIC FOR FLEXIBLE PAVEMENTS CONTAINING ONE OR MORE BOUND LAYERS (AUSTROADS Guide)

7.5.2 For Traffic in Terms of Annual Average Daily Number of Axles by Type (AUSTROADS Guide)

The calculation of design traffic loading using method 2 (described in Appendix E, AUSTROADS Guide) requires knowledge of the mean number of ESA loads per axle group, as presented for Australian traffic conditions in Table E4 (AUSTROADS Guide). Average ESA per axle group recorded at five weigh-in-motion sites in New Zealand are summarised in Table D1 (Appendix D, NZ Supp.). A heavy commercial vehicle (HCV) is defined as any vehicle with a gross weight exceeding 3.5 tonnes.

Due to the adoption of the Queensland fatigue relationships for cemented materials, NSC as defined in Section 7.5.1 (AUSTROADS Guide) is now calculated as:

$$NSC = 10.0NE$$

7.5.3 For Traffic in Terms of Annual Average Daily Traffic (AADT) and Percentage of Commercial Vehicles (AUSTROADS Guide)

The calculation of design traffic loading using method 3 (described in Appendix E, AUSTROADS Guide) requires knowledge of the mean number of ESA loads per heavy commercial vehicle (HCV), as presented for Australian traffic conditions in Table E5 (AUSTROADS Guide). Average ESA per heavy commercial vehicle recorded at five weigh-in-motion sites in New Zealand are summarised in Table D3 (Appendix D, NZ Supp.). An average ESA of 1.0 per HCV (where both directions are averaged) is considered representative for typical vehicle mixes on State Highways throughout New Zealand. However, the pavement designer should determine an appropriate value of average ESA per HCV for each project considered.

Additional, weigh-in-motion data on mean ESA per 2, 3, 4, 5, 6, 7, and 8 axle vehicles and percentage of HCV at each vehicle classifier site are given in Transfund New Zealand Research Report, Appropriate Design Traffic Data for the New Zealand Supplement to the AUSTROADS Pavement Design Guide. This information will assist the pavement designer in determining the appropriate mean ESA per HCV to use.

Unless otherwise known 50% of the vehicles classified in the 5.5m to 11m category and all vehicles >11m wheelbase shall be classed as HCVs (i.e. >3.5 tonnes).

7.6 DESIGN TRAFFIC FOR FLEXIBLE PAVEMENTS CONSITING OF UNBOUND GRANULAR MATERIALS AND OVERLAYS FOR FLEXIBLE PAVEMENTS (AUSTROADS Guide)

7.6.2 For Traffic in Terms of Annual Average Daily Number of Axles by Type (AUSTROADS Guide)

See 7.5.2 (NZ Supp.)

7.6.3 For Traffic in Terms of Annual Average Daily Traffic (AADT) and Percentage Commercial Vehicles (AUSTROADS Guide)

See 7.5.3 (NZ Supp.)

7.8 INITIAL AND TERMINAL PAVEMENT CONDITIONS (AUSTROADS Guide)

Table 7.4 (AUSTROADS Guide) should be used as a rough guide only for indicative values. The pavement designer should choose and document the appropriate level of terminal roughness (in terms of NAASRA counts per kilometre) for the particular pavement being designed.

Note that the standard procedure is based on a terminal roughness that is three times the initial roughness as typical for Australian roads. There is provision for the designer to vary the standard ratio of initial roughness to terminal roughness for New Zealand roads.

7.9 MODIFICATION OF DESIGN TRAFFIC TO IMPROVE RELIABILITY OF DESIGN (AUSTROADS Guide)

Attention is drawn to the statement "one significant source of uncertainty is associated with the prediction of loads on axle groups". The designer should confirm the appropriate parameters to be used.

8 DESIGN OF NEW FLEXIBLE PAVEMENTS

8.2 MECHANISTIC PROCEDURE (AUSTROADS Guide)

The contact stress is related to tyre pressure and shall be taken as 750 kPa for an equivalent standard axle (circular contact radius = 92 mm). This change corresponds to the October 1997 changes in the AUSTROADS Guide.

Section 5.9 gives guidelines for assigning appropriate design values to the subgrade.

Section 6.6 (NZ Supp.) gives design guidelines for pavements incorporating a stabilised subgrade soil layer.

Section 8.2.4 (NZ Supp.) gives design guidelines for pavements incorporating geosynthetic reinforcement. The geosynthetic supplier should be consulted in conjunction with reading the guidelines provided in this section.

8.2.2 Procedure for Elastic Characterisation of Granular Materials (AUSTROADS Guide)

In addition to the sub-layering requirements for unbound granular material the vertical modulus of the top sub-layer shall not exceed $E_{v \text{ top of granular}}$ as defined in the formula below:

$$E_{v \text{ top of granular}} = E_{v \text{ subgrade}} \times 2^{(\text{total granular thickness}/125)}$$

where: total granular thickness = thickness of unbound base + subbase;
$E_{v \text{ subgrade}}$ = elastic modulus of the subgrade.

This formula has been developed by AUSTROADS to account for the influence of subgrade stiffness on the modulus achievable in overlying unbound granular materials. The exception to this rule is when a geogrid is used, see section 8.2.4 of this NZ Supplement.

8.2.3 Subgrades of CBR ≤ 3

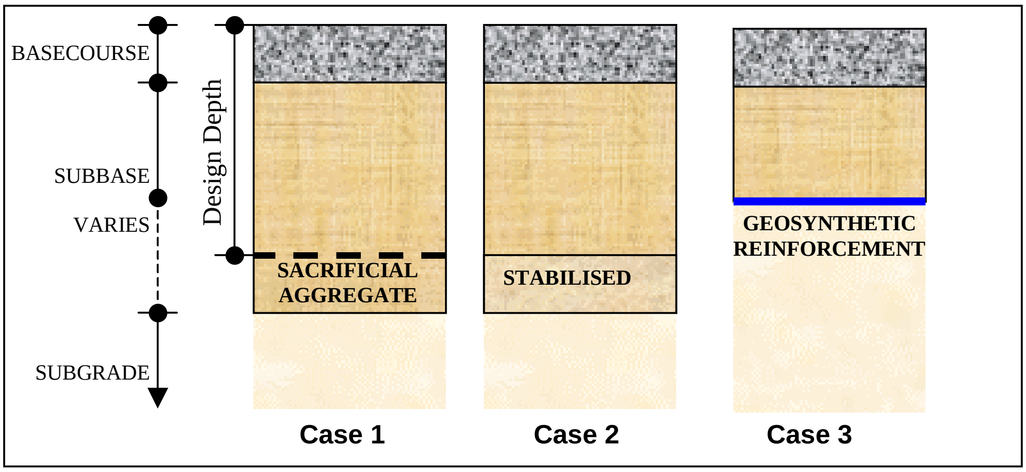
Subgrades that have, at the time of construction, a measured in situ CBR ≤ 3 (i.e. not inferred by modulus or deflection tests) require a working platform. A working platform is needed to enable adequate compaction of the overlying granular layers to be achieved and to ensure that subgrade fines do not intrude into the pavement structure. Several options are available to form a working platform, including:

i. allow a sacrificial depth of 150 mm of granular material and design the overlying pavement (i.e. the pavement thickness above the sacrificial 150 mm depth of granular material) assuming no improvement to the subgrade CBR or modulus; or

ii. having established that the subgrade soil is suitably reactive and that stabilisation is practically viable, stabilise the subgrade to a depth of at least 150 mm and refer to Section 6.6 (NZ Supp.) for design requirements; or

iii. allow for a reinforcing geosynthetic to be placed between the subgrade and the subbase (and elsewhere in the aggregate layer if required) and design the overlying pavement layers in accordance with Section 8.2.4 (NZ Supp.). The provision of a separate or integrated geotextile fabric (Refer to Note 1) may also be considered necessary to prevent migration of clay fines from the subgrade into the pavement structure.

The above four cases can be diagrammatically represented as follows:



It should be noted that the presence of water is almost always the cause of poor subgrade conditions in cohesive soils. Therefore, the pavement designer must ensure that suitable drainage provisions are specified. Effective drainage may eventually result in an improvement in subgrade conditions, however this improvement should not necessarily be anticipated in the design.

Note 1*: The geotextile fabric shall be selected in accordance with the requirements of the Transit NZ Specification for Geotextile (TNZ F/7: 2000) except that the strength class shall be one grade lower than otherwise required due to the presence of the reinforcing geosynthetic.*

8.2.4 Design of a Pavement Incorporating Geosynthetic Reinforcement

When appropriate geosynthetic materials are provided at the interface of the subbase and subgrade, reinforcement can be achieved from any one, or 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 state of 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. To allow for the different reinforcement mechanisms, two design procedures are suggested, one based on a ***confinement*** approach and the other based on a ***tension membrane*** approach. In both of the suggested procedures, any reduction in pavement depth achieved by the use of geosynthetics must be applied to the ***subbase*** layer only. The design should also be carried out using the in situ subgrade CBR to quantify the reduction in pavement depth realised by incorporating the geosynthetic reinforcement.

It is the responsibility of the pavement designer to determine which design procedure is best suited to the type of geosynthetic in question. The geosynthetic supplier should provide sufficient evidence of performance to the designer by way of case history data and/or supporting research to allow this determination to be made. Alternative design procedures can be used provided that they are suitably justified and documented.

The magnitude of the pavement depth reduction for both the confinement and tension membrane approaches should generally ***not exceed 150 mm*** without specific site testing and case history data to support the higher saving.

8.2.4.1 Design Procedure for Geosynthetic Reinforcement Using the *Confinement* Approach

The designer should use the standard design procedures for a conventional unreinforced, unbound pavement, except the depth of aggregate can be modified based on empirical data. It is claimed that reductions of ***up to one third of the total pavement depth*** can be achieved when a geosynthetic reinforcement layer is placed at the subgrade / subbase interface.

When the reinforced pavement thickness exceeds 400 mm, a second geosynthetic reinforcement layer is required at mid-height in the aggregate layer.

The geosynthetic supplier must provide relevant and credible case history data to the designer to support the use of this procedure for the particular product in question.

8.2.4.2 Design Procedure for Geosynthetic Reinforcement Using the *Tension Membrane* Approach

The original subgrade CBR is transformed to an *equivalent reinforced subgrade CBR* to reflect the higher level of support afforded to the subgrade by the geosynthetic. This transformation is achieved using a Subgrade Improvement Factor (SIF) determined using *tension membrane* theory based on full-scale trials by Giroud and Noiray (1981) and described in Meyer and Elias (1999). Note that the subgrade must deform for the tension membrane effect to be mobilised.

Once the original subgrade CBR has been factored up by the relevant SIF to obtain the reinforced subgrade CBR the pavement is designed using the conventional AUSTROADS mechanistic procedure.

A tabulation of SIF values is presented in Table 8.1 (NZ Supp.). Note that the SIF data presented is based on a geosynthetic with a 2% secant modulus value of 1,000 MN/m width. If a geosynthetic with a different 2% secant modulus value is proposed, the designer should seek guidance from the supplier regarding the applicability of the data in Table 8.1.

Table 8.1 SIF values for geotextile reinforcement design.

In Situ CBR (%)	SIF Value
≤ 1.0	2.4
1.5	2.3
2.0	2.2
2.5	1.9
3.0	1.7
3.5	1.4
4.0	1.2
4.5	1.1
≤5.0	1.0

The geosynthetic supplier must provide relevant and credible case history data to the designer to support the use of this procedure for the particular product in question.

8.2.5 Design of Pavements Incorporating Bound Aggregate Layers

When sufficient hydraulic stabilising agent is added to an aggregate to produce significant tensile strength (i.e. UCS at 7 days moist curing > 0.8 MPa) the material can be considered to be *bound*. To fall into the *lightly bound* category the material should have an elastic modulus in the range 1,500 MPa to 5,000 MPa. The *heavily bound* category applies to materials with a modulus value > 5,000 MPa.

All bound aggregate layers are susceptible to two cracking mechanisms, shrinkage and fatigue.

Shrinkage cracking is caused by thermal contraction and shrinkage associated with the hydration of the cementing agent. If shrinkage cracking is properly managed it need not have a significant detrimental effect on the performance of the layer. Shrinkage crack management will generally take the form of pre-cracking to ensure that the cracks are closely spaced and the crack apertures are narrow. The cracks should be sufficiently narrow that shear stresses can be transferred from one side of the crack to the other. If shrinkage cracking is not properly managed the result is likely to be transverse cracking of the cemented layer.

A combination of high shrinkage potential and high tensile strength in cementitiously bound bases can cause widely spaced (0.5 to 5.0 m) transverse and/or block cracking to occur. While this may reduce the ride quality of the pavement it usually does not lead to serious structural problems if the cracks are sealed or a polymer modified or scrap rubber binder wearing surface is applied and provided crack widths do not exceed 2 mm. Not sealing the cracks will lead to moisture entry into the pavement, which may lead to pumping of fines from erosion and rapid deterioration of the pavement under the action of traffic.

Fatigue cracking is attributable to fatigue failure of the cemented aggregate due to the repeated application of wheel loads. Fatigue cracking results in an alligator pattern of cracks in the wheelpaths. To design against fatigue cracking of the bound layer the pavement layer materials and thicknesses should be arranged so that the horizontal tensile strains occurring at the bottom of the cemented layer are limited to an acceptable level. This acceptable level is defined by the tensile fatigue performance criterion defined in the AUSTROADS Pavement Design Guide for cemented materials. For design purposes the cemented material should be considered to be isotropic with no sub-layering.

Even if significant fatigue cracking does occur, provided moisture ingress into the pavement can be prevented through re-sealing and re-seal patches, there may still be considerable life left in the pavement in the post cracking phase. In this situation the cracked bound layer effectively acts as an unbound or modified layer. This post cracking phase life can be quantified using the design procedures in the AUSTROADS Guide to the Structural Design of Road Pavements (1992) where the pavement is assumed to be unbound.

The likelihood of pumping of fines from the base, subbase and subgrade can be minimised by ensuring that an adequate binder content is used in the stabilised layer(s) to achieve the appropriate erodability limits. (Howard 1990, Wong 1992). (See Figure 4.3, 1998 Austroads Guide to Stabilisation in Roadworks)

For low traffic roads (design traffic < 1 x 10^6 ESA) it is often uneconomic to fully design against fatigue cracking and the post-cracking life is utilised. In this situation the cement treated aggregate may not be covered by an unbound granular material. This type of pavement will likely have increased maintenance requirements in terms of sealing cracks (shrinkage and fatigue) as they reflect through the surfacing.

A quick method of design for low traffic roads is to simply use the same thickness of pavement as needed for an unbound granular pavement using Figure 8.4 of the Austroads Pavement Design Guide. The minimum thickness of the bound aggregate or CTB (Cement Treated Base) is the base thickness required as detailed in Figure 8.4 of the Austroads Pavement Design Guide. Using CIRCLY and the appropriate tensile fatigue criterion, the CTB thickness and/or underlying support is adjusted until an acceptable level of reseal patching and re-sealing is obtained over the life of the pavement. To determine the expected level of reseal patching and re-sealing required the following procedure can be used.

The fatigue life of a bound layer is very dependent on the level of support provided by the underlying layer(s). Fatigue cracking will occur first (typically within two years) in the weakest underlying areas before gradually spreading to other areas of the pavement. However, it is likely that areas with the highest level of underlying support may never fail by fatigue of the bound layer. To predict the extent of fatigue cracking each year over the pavement's life a CIRCLY or similar linear elastic analysis can be undertaken for the different levels of subgrade support over the length of the pavement.

An example analysis has been undertaken by determining the tensile fatigue life of a CTB layer over 70 cross-sections back-analysed from Falling Weight Deflectometer (FWD) deflection bowls on an existing unbound granular pavement. The resulting charts shown in Figures 8.1 and 8.2 (NZ Supp.) can be scaled up or down depending on the year that fatigue cracks are predicted to first show through to the surface and the percentile cross-section used for determining this value. The tensile fatigue life plus the time for the crack to reflect through the surfacing (Table 8.1) is used to determine the year for predicted fatigue cracking.

A starting point to estimate the time for a crack to reflect through the surface is given in Table 8.1. More research is required in this area and consideration should also be given to the HDM crack prediction models in NZ dTIMMS plus any other sources.

Table 8.1 Time estimates for reflective cracking through various types of surface layers.

Seal Type	Approximate length of time for crack to reflect through the seal
Single coat seal	2 years
Two coat seal	4 years
PMB seal	4 years
Geotextile seal	15 years

To illustrate the use of this data, consider a pavement designed with layer properties and thicknesses such that fatigue cracking is expected after two years. If a single coat seal is used then the first cracks should start to show at the surface two years later, i.e. four years after construction. Figure 8.1 (NZ Supp) shows the percentage of pavement expected to show fatigue cracking over a range of time periods and design percentiles. Assuming a 10 percentile design cross-section for this example, 10 percent of the area would be cracked after four years, approximately 18% after 8 years, approximately 22 % after 12 years, and so on.

After the scales on the charts (Figures 8.1 and 8.2, NZ Supp.) have been adjusted to align with the predicted onset of fatigue cracking a maintenance regime in terms of reseal patches, resealing and digouts should be determined. This will allow life-cycle costs to be calculated for comparing alternative pavement structures/materials. The maintenance regime is primarily aimed at keeping the water from penetrating the cracks to prevent erosion at the joints by undertaking re-seal repairs as cracking reflects through the surface. Figure 8.3 (NZ Supp.) indicates the early stage of fatigue cracking that should be sealed within a year to prevent erosion at the joints as pictured in Figure 8.4 (NZ Supp.).

Any fatigue cracking that occurs in the first 2 years will likely be due to the lack of underlying support. These areas will have a subgrade strength less than the design value (e.g. 10 percentile) and will therefore likely rut before the end of the design life. Ideally these areas of weak support should be dug out and strengthened.

Any cracking less than 2 mm in width without significant rutting can simply be re-sealed. A digout, asphalt smoothing treatment and/or in situ stabilisation repair is not required until rutting and/or erosion at the edge of the alligator cracking is significant (Figure 8.4, NZ Supp.). Shrinkage cracks are usually transverse and can be crack sealed to prevent joint erosion and pumping of the underlying support (i.e. subgrade fines).

8.2.6 Pavement Design for Stabilised Subgrades

As described in Section 6.6 of the NZ Supp, stabilised subgrade layers can be categorised into the following categories:

modified;
lightly cemented; and
moderately to highly cemented.

The design procedure for each category of material is described as follows.

8.2.6.1 Modified Subgrades

The modified layer should be modelled as a series of sub-layers as required in Section 8.2.2 of the AUSTROADS Guide. This is done to recognise the inevitable density gradation through the layer. The (vertical) elastic modulus assigned to the top sub-layer should be no more than three times the resilient modulus of the original subgrade. The modified soil should be modelled as being anisotropic and having the same Poisson's ratio as that assigned to the original subgrade soil.

The multi-layer elastic design analysis should check the level of vertical compressive strain under standard loading conditions at both the top of the modified layer and the top of the original subgrade. The AUSTROADS subgrade performance criterion (i.e. Equation 5.1, AUSTROADS Guide) should be used in both instances.

Alternatively, Figure 8.4 (AUSTROADS Guide) can be used to determine the appropriate cover required over both the stabilised and existing subgrade soil. Section 8.3.2 Pavement Composition (AUSTROADS Guide) gives guidelines on using Figure 8.4 (AUSTROADS Guide) for this purpose (i.e. the stabilised subgrade can be classed as a sub-base course with a known design CBR).

8.2.6.2 Lightly Cemented Subgrades

By producing a layer with a reasonably high resilient modulus the designer must appreciate that the layer will attract a relatively large proportion of the applied load and may suffer from fatigue. In addition, the thermal stresses associated with hydration of the binder, makes the moderately cemented subgrade soil layer prone to cracking. The extent and severity of cracking is mainly influenced by the proportion and type of binder, the presence of swelling clay minerals and the curing conditions. To avoid the somewhat uncertain issues of when and where the layer will crack, pre-cracking during construction should be employed.

The pre-cracked layer should be designed as if it was equivalent to the same thickness of unbound sub-base. Therefore, in the design analysis the pre-cracked layer should be treated as being part of the total unbound cover and sub-layered as required to in Section 8.2.2 of the AUSTROADS Guide.

The designer should be satisfied that the resilient modulus of the pre-cracked material achieved in the field meets or exceeds the resilient modulus adopted in the design model.

To ensure cracks in the moderately cemented subgrade soil layer do not reflect to the surface an appropriate cover is required.

8.2.6.3 Moderately to Highly Cemented Subgrades

In the design analysis the cemented layer should be modelled as an isotropic material. The cemented layer and the overlying aggregate cover do not require sub-layering. The critical part of the design analysis is likely to be horizontal tensile strain occurring at the bottom of the cemented layer. The fatigue life of the layer is determined using the equations presented in Section 6.3.3 of this Supplement. Although, the vertical compressive strain at the top of the subgrade is unlikely to be critical in the design it should still be checked.

The concept of a "life after death" for the cemented layer may be considered based on the discussion in Section 8.5 of the AUSTROADS Guide. In simple terms, the life of the cemented layer can be modelled over two periods. In the first period the layer remains intact until it has consumed its fatigue life. In the second period its elastic properties revert to those of an equivalent thickness of unbound aggregate. Therefore the layer provides an extended period of protection for the subgrade albeit at a reduced resilient modulus (the same as an unbound granular aggregate).
To ensure cracks in the cemented subgrade soil layer do not reflect to the surface, an appropriate thickness of covering material (not less than 150 mm) is required.

8.3 GRANULAR PAVEMENTS WITH THIN BITUMINOUS SURFACING

8.3.2 Pavement Composition (AUSTROADS Guide)

The aggregate used in the base layer of the pavement should comply with the requirements of the Transit New Zealand Specification M/4. Where alternative materials are being considered the designer is referred to the requirements of Transit New Zealand Specifications B/3 and M/22.

In addition, it is important that the Engineer check that each pavement layer has sufficient cover as per the requirements of Figure 8.4 (AUSTROADS Guide) based on the design subgrade CBR and traffic values.

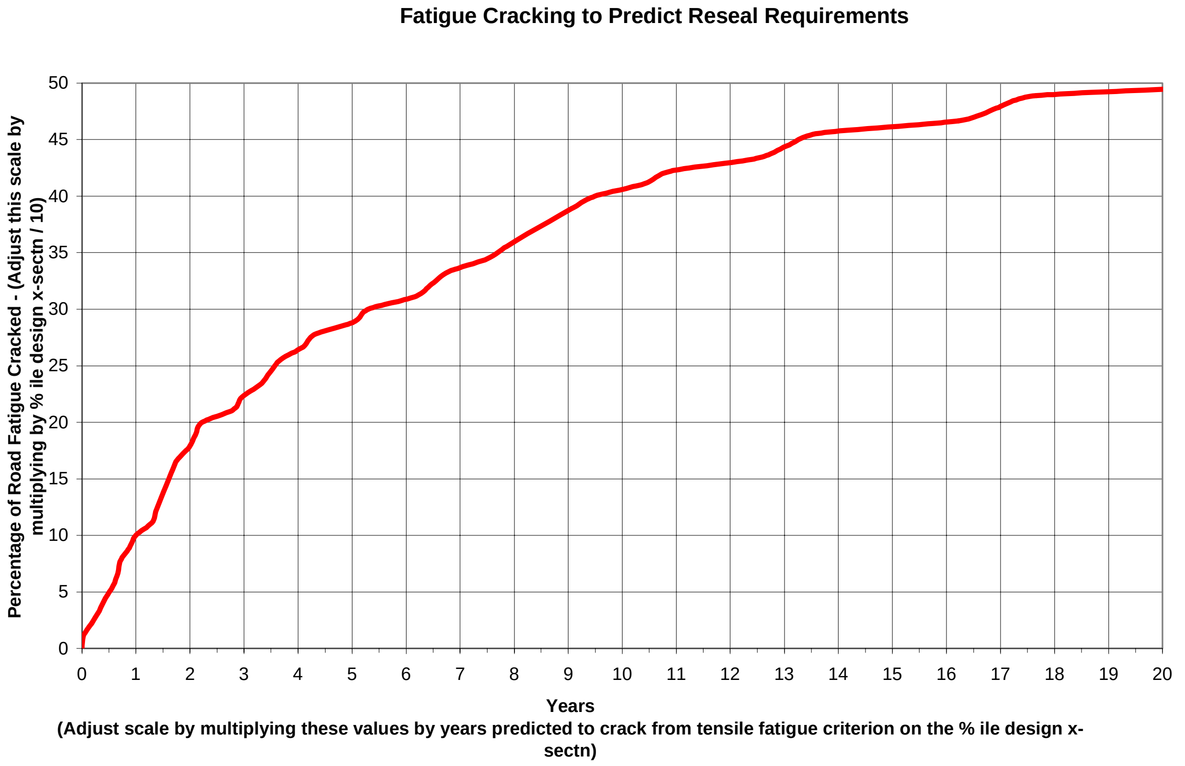


Figure 8.1 Example of expected fatigue cracking percentage for reseal prediction.

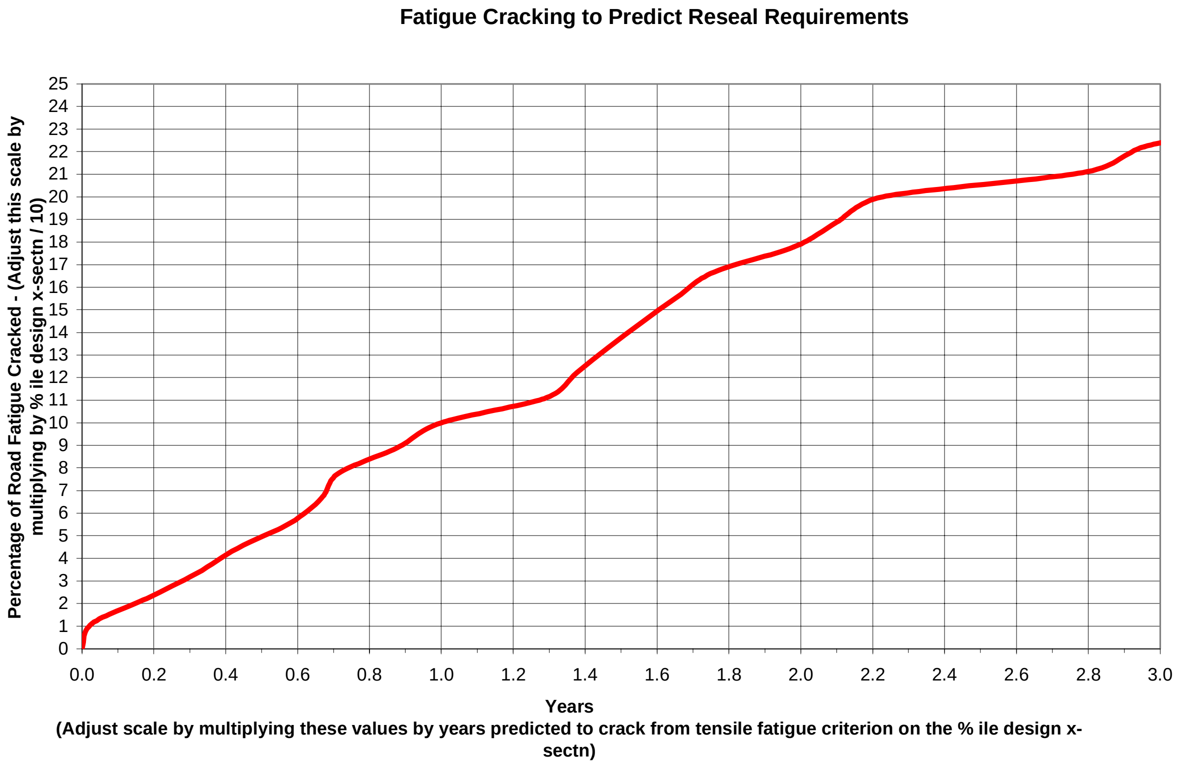


Figure 8.2 Example of expected fatigue cracking percentage for reseal prediction (compressed scale).

Figure 8.3 Onset of fatigue cracking reflecting through the surface.

Figure 8.4 Onset of erosion at the crack edges.

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10 REHABILITATION DESIGN

This section replaces Section 10 Overlay Design of the AUSTROADS Guide for design of rehabilitation treatments for New Zealand roads. The term "rehabilitation" has been used as this reflects New Zealand use. In some cases, a smoothing treatment or modification of the existing pavement layers rather than an overlay will be the most cost-effective treatment.

This New Zealand Supplement covers only the rehabilitation of thin surfaced granular pavements. Future revisions will also include procedures for bound pavements, including asphalt overlays.

10.1 GENERAL

The purpose of rehabilitation design is to determine an appropriate treatment whereby the ride, waterproofness, integrity and strength of the existing pavement can be restored or improved to meet the demands of the future traffic over the required design period. One of the most important aspects of the rehabilitation design system is the identification of the pavement's deficiencies, and needs. Sufficient investigation must be carried out to determine the state of the existing pavement and its past performance, so that the various rehabilitation options appropriate to that state can be considered relative to their economic values. Charts A1, A2, A3 and A4 in Appendix A (NZ Supp.) give guidance on the level of investigation required and appropriate rehabilitation treatments.

On some pavements a succession of chipseal applications over many years may result in a significant increased depth of surfacing, e.g. 50 to 100 mm. This layer is likely to comprise single sized aggregate and excess bitumen causing unstable behaviour under heavy traffic loading. In such circumstances the existing surface layer should either be removed or appropriately stabilised before an overlay is constructed. Alternatively, an overlay of sufficient thickness to compensate for the unstable layers of chipseals may be appropriate.

An adequate transfer of shear forces generated by traffic must occur at the interface between the new overlay and the existing pavement. If the existing seal is smooth, then this may require a ripping treatment to provide sufficient texture to bond with the new overlay.

For any rehabilitation treatment the surfacing should restore or improve waterproofness and skid resistance. A mechanistic design procedure is required for the design of pavement rehabilitation in place of other procedures. Mechanistic design procedures allow a range of rehabilitation treatments to be investigated with the selected option being normally based on life cycle economic considerations.

10.2 CHARACTERISATION OF EXISTING PAVEMENT STRUCTURE

An existing layered pavement structure (Figure 10.1 NZ Supp.) is required for mechanistic design analysis using a pavement design program (e.g. CIRCLY). The modulus values can be estimated using either direct measurement, back analysis from deflection bowls or presumptive values as detailed in Sections 5 and 6 (AUSTROADS Guide).

Reference should be made to Section 8.2.2 (AUSTROADS Guide) for determining how the unbound granular material should be layered for mechanistic design. Test pits and/or local knowledge will be required to determine the pavement material thicknesses.

It may be necessary to subdivide the road section requiring rehabilitation into representative subsections of the same pavement structure for separate analyses.

Where an asphalt overlay or stabilisation treatment is being considered it is important to determine appropriate elastic moduli in order that the tensile strain in these layers can be modelled. The life of the pavement is controlled by the tensile strains at the bottom of these layers.

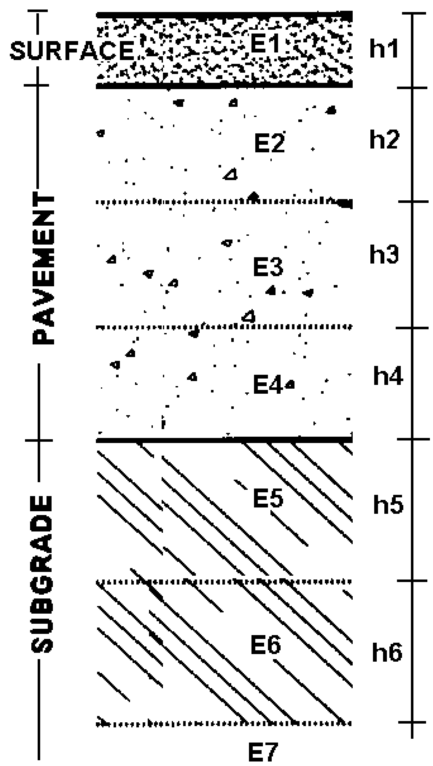

Figure 10.1 - Example of existing layered pavement structure (E1 to E7 are elastic moduli and h1 to h6 are layer thicknesses)

The modulus of elasticity (E) of unbound granular materials is stress dependent. Therefore, when designing a rehabilitation treatment moduli values assigned to the existing granular layers will need to be reduced by approximately 20% or as determined from the stress dependency relationship for the granular material derived from RLT (Repeat Load Triaxial) modulus tests.

The revised subgrade strain criterion (Section 10.4.2, NZ Supp.) will compensate for loss in strength due to seasonal variations. An incremental drop in subgrade strain from that which the pavement is currently experiencing will always occur (e.g. lower strains in the summer months).

10.3 PAVEMENT SHEAR STRENGTH

In the mechanistic design method the pavement surfacing and base layers are assumed to have sufficient shear strength to withstand the proposed traffic loading. Adequate shear strength is assured through the use of materials that comply with the appropriate Transit specifications. However, a pavement in need of rehabilitation may have layers that do not comply with the specifications and therefore may not have sufficient shear strength for the proposed traffic loading.

If, in the assessment of the existing pavement, it is considered failure has occurred in the basecourse, e.g. shallow shear, then the shear strength should be checked. In many cases, a visual assessment by an experienced roading engineer will confirm that the base material is contaminated or inadequate.

Rehabilitation options would include removal, stabilisation or construction of an overlay with sufficient thickness, so that the shear stresses within the layer are consequently limited to prevent possible shear failure.

Checking for shear strength is recommended only where shallow shear was the primary mode of failure or the existing pavement layers are suspected to have insufficient strength to support future traffic loads.

10.3.1 Prevention of Shear Failure

The minimum thickness of good quality base material (which may be a combination of old plus new material) for the rehabilitated pavement shall comply with Figure 8.4 (AUSTROADS Guide). Adequacy of the existing basecourse (in relation to minimum requirements for M/4) can be checked using Sand Equivalent and/or Clay Index tests.

The pavement designer shall ensure that the shear strength of the pavement material is greater than the shear stress imposed by traffic as defined by Equation 10.1 (NZ Supp.)

$S_z \geq \tau_{max, z}$ [Equation 10.1]

where:

Sz = repeated load shear strength of the pavement material, located at depth z below the surface.
$\tau_{max, z}$ = maximum shear stress at depth z below the surface under a standard wheel load (ESA).

The maximum shear stress ($\tau_{max, z}$) is determined using a computer program such as CIRCLY and will occur outside the tyre pavement contact area. The pavement designer needs to ensure that stresses are computed at sufficient distance from the load to determine the maximum shear stress. CIRCLY computes the shear stress automatically, although the user has to find the maximum value by scrolling through the results.

10.3.2 Evaluation of Shear Strength in Unbound Pavement Materials

Determining the repeated load shear strength (Sz) of the pavement material can be costly. Ideally, the pavement material should be tested at a range of loads using machines such as the repetitive load triaxial apparatus. In this machine the shear strength will be related to the vertical load and confinement applied, where the permanent deformation at the end of the test is at an acceptable level. The pavement material is usually saturated and tested for 10^5 load cycles.

A simplified procedure to evaluate the adequacy of the shear strength is outlined below. However, this procedure is unproven for New Zealand materials and should only be used as a guide.

From research (Uzan et al 1980) a relationship has been identified between the shear strength of unbound granular materials and the CBR (California Bearing Ratio). A minimum CBR of 80% specified in M/4 for basecourse results in sufficient shear strength at the pavement surface to resist the maximum shear stress under a standard wheel load (750 kPa contact pressure) of 230 kPa. Based on this, the repeated load shear strength (S_z) of unbound granular material is found to be equal to 2.9 times the CBR (%).

$S_z = 2.9 \times CBR$ [Equation 10.2]

where:

S_z = repeated load shear strength (kPa).
CBR = California Bearing Ratio (%).

Using the above relationship, the minimum CBR required to prevent shear failure at a range of different depths below the surface has been calculated for a typical pavement structure and are given in Figure 10.2 (NZ Supp.).

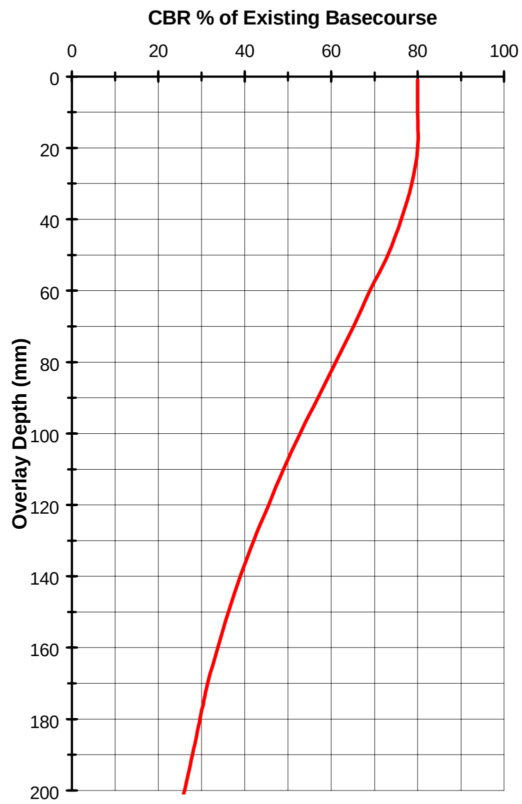

Figure 10.2 - Overlay depth required to prevent shallow shear vs CBR % of existing basecourse.

Figure 10.2 indicates that if the CBR of the existing basecourse is less than 80% then an overlay or stabilisation of the basecourse layer is required. If the basecourse was a material with a CBR of 40% the depth of overlay would be a minimum of 130 mm. This may often be greater than the design thickness required to reduce the subgrade strain to the design level as defined in Section 10.4.2 (NZ Supp.). CBR values should be measured directly in the field or the laboratory.

Overlay thicknesses derived originally from Falling Weight Deflectometer (FWD) or Benkelman Beam deflection measurements (Section 10.4.2 NZ Supp.) may automatically compensate for any weaknesses in the existing pavement by calculating a thicker overlay. Weaknesses in the existing basecourse may also be detected, where vertical compressive strains computed in the existing basecourse layers are higher than the computed strains in the subgrade.

10.4 MECHANISTIC DESIGN FOR REHABILITATION

In General Mechanistic Procedures, the reaction of different pavement rehabilitation designs under a standard wheel load (1 ESA) are analysed using a computer program such as CIRCLY. Strains at various critical layers (Figure 8.2, AUSTROADS Guide) are computed for each design being considered. The designs which are acceptable are those which meet or exceed the performance criteria detailed in Sections 6.4.6 (AUSTROADS Guide) and 6.3.3 (NZ Supp.). The performance criterion for the subgrade is given by Equation 10.3 (Section 10.4.2 NZ Supp.) and now replaces Equation 5.1 (Section 5.9, AUSTROADS Guide) for pavement rehabilitation design. However, pavement designers can choose to use the AUSTROADS subgrade strain criterion where a major change in road use is expected (e.g. a rural road that has only previously carried light traffic is upgraded to a standard capable of supporting heavy vehicles), or where estimating the ratio of future to past traffic is extremely difficult, or where the primary distress mode is not related to permanent strain in the subgrade.

A range of pavement designs for rehabilitation can be considered including: strengthening the existing pavement layers (stabilisation or other means); granular overlay; asphalt overlay; or any combination taking into account any minimum requirements determined previously to prevent shear failure of the base layers (Section 10.3, NZ Supp.). This design process is similar to the design of new pavements as detailed in Section 8.2 (AUSTROADS Guide).

10.4.1 Design of Unbound Granular Overlays

The unbound granular overlay thickness shall not be less than:

2.5 times the maximum particle size;
the thickness required to reduce the subgrade strain to the design level as determined in section 10.4.2 (NZ Supp.); and if applicable,
the thickness determined in section 10.3 (NZ Supp) to prevent shear failure of the pavement base layers.

The material used for thin unbound granular overlays shall comply with the Specification for Basecourse Aggregate (M/4).

Mechanistic analysis of an unbound granular overlay involves modelling a trial overlay thickness over the existing pavement structure, as determined in Section 10.2 (NZ Supp.). The modulus assumed for the unbound granular overlay shall not exceed the modulus estimated for the top unbound granular layer (E2) of the existing pavement (Figure 10.1 NZ Supp.) unless greater values can be substantiated. In addition, the moduli assigned to the existing pavement structure will need to be reduced slightly to account for the stress dependency of unbound granular materials. This is a conservative estimate of modulus compared with the guidelines given in Section 8.2.2 (AUSTROADS Guide). However, at this stage it is considered appropriate.

10.4.2 Design For Subgrade Deformation

For rehabilitation treatments, the vertical compressive strain computed at the top of the subgrade (point 3, Figure 8.2, AUSTROADS Guide), shall not exceed the design strain (ε_{des}) as defined by Equation 10.3 (NZ Supp.):

$$\varepsilon_{des} = \varepsilon_{cvs} (N_F/N_P)^{-0.23} \quad \text{[Equation 10.3]}$$

where:

N_F/N_P = ratio of future traffic to past traffic,
N_F = future design traffic in ESAs (Section 7, AUSTROADS Guide).
N_P = estimated traffic in ESA carried by the pavement since it was last strengthened or smoothed (or since it was constructed if not strengthened or smoothed), and until the level of serviceability for that class of road justified rehabilitation.
ε_{cvs} = existing vertical compressive strain at the top of the subgrade, when modelling the existing pavement structure in accordance with Section 10.2 (NZ Supp.) using a standard wheel load (1 ESA). Alternatively use Equation 10.4, where several layered pavement structures have been developed from back analysis of deflection bowl measurements and where it is expected that differing depths of overlay are likely to be required:

$$\varepsilon_{cvs} = \alpha - fs \quad \text{[Equation 10.4]}$$

where:

α = the mean of the existing vertical compressive strains at the top of the subgrade computed for all the layered existing pavement structures developed with similar subgrade soil types.
s = the standard deviation of the existing vertical compressive strains at the top of the subgrade computed for all the layered pavement structures developed with similar subgrade soil types.
f = pavement condition factor and shall be 0 unless it is considered that more or less than 50% of the road section has "failed" (e.g. rut depth from subgrade deformation >20 mm) in which case the appropriate value of f (derived from the normal distribution) is given in Table 10.1 (NZ Supp.):

Table 10.1 - Appropriate value of pavement condition factor (f) for Equation 10.4 (NZ Supp.)

f	% of the Road Section Considered to Have "Failed"[1]
-0.254	40%
0	50%
0.254	60%
0.525	70%
0.843	80%

1 Failed means subgrade deformation causing rutting

The use of Equation 10.4 (NZ Supp.) for calculating the design strain (Equation 10.3, NZ Supp.) will calculate one design vertical compressive strain (ε_{des}) that will apply to the design of a rehabilitation treatment for all existing pavement structures determined by back analysis of deflection bowl measurements. Consequently, a thicker overlay will be computed over existing pavement structures that are less stiff or weaker than average. Conversely, a thinner overlay will be computed over existing pavement structures that are more stiff or stronger than average. This approach, appears to compensate for any weaknesses in the existing pavement structure and may reduce the number of destructive tests required as inferred in Section 10.3.2 (NZ Supp.).

The use of Equation 10.3 (NZ Supp.) for calculating the design strain without the use of Equation 10.4 (NZ Supp.) will calculate a different design vertical compressive strain (ε_{des}) for each individual existing pavement structures determined by back analysis of deflection bowl measurements. If the future traffic is more than the past traffic then the design vertical compressive strain at the top of the subgrade will always be less than the existing subgrade strain regardless of the strength (stiffness) of the existing pavement. Consequently, the thickness of an unbound granular overlay computed will be similar for all the pavement structures analysed. This approach to overlay design is the same as used in the State Highway Pavement Design and Rehabilitation Manual (SHPDRM Transit New Zealand, 1989) where the strength of the existing basecourse does not alter the overlay thickness required.

Pavement designers should carefully review any overlay thicknesses originally derived from interpretation of FWD or Benkelman Beam Deflections. Comparisons should be made between the thicknesses calculated in Section 10.3 (NZ Supp.) and those calculated using the existing subgrade strain per individual point and the mean subgrade strain (Equation 10.4, NZ Supp.).

This subgrade strain criterion (Equation 10.3, NZ Supp.) considers the past performance of the pavement and is based on the design assumptions used in the State Highway Pavement Design and Rehabilitation Manual (SHPDRM Transit New Zealand, 1989). The exponent (-0.23) used in Equation 10.3 (NZ Supp.) is derived from the vertical compressive strain criterion used to develop the pavement design charts in the SHPDRM. Use of this exponent is considered appropriate as slightly thicker overlays are calculated compared with those determined when using the exponent (-0.14) adopted for the AUSTROADS subgrade strain criterion (Equation 5.1, AUSTROADS Guide) are computed. This conservative approach may also compensate for any basecourse degradation in strength.

10.5 DESIGN OF ASPHALT OVERLAYS

This section is new and until there is experience in its use designers should use this design methodology cautiously and rely on their own judgement when deciding on asphalt overlay depths.

10.5.1 Asphalt Overlay on a Chip Seal Pavement: Design Criteria

The asphalt overlay thickness shall not be less than the thickness required to:

Criterion 1 - reduce the subgrade strain to the design level as determined in section 10.4.2 (New Zealand Supp.), and

Criterion 2 - limit the horizontal tensile strain at the bottom of the asphalt overlay to a value not exceeding the future traffic design strain as determined by the performance (fatigue) relationship given in AUSTROADS (1992), section 6.4.6.

To account for stress dependency, the moduli assigned to unbound materials in the existing pavement structure shall be adjusted using the procedures described in section 10.5.4 (New Zealand Supp.).

10.5.2 Stress Dependent Materials

The moduli of unbound granular base materials increase when the confining stress increases. The moduli of fine-grained (cohesive) subgrades decrease with increasing stress level until a certain value is reached, after which it tends to increase slightly (AUSTROADS 1992, section 6.2.2.2).

After application of the overlay the stress levels under load in the existing pavement structure decrease. The magnitude of this change in stress level depends on the thickness of the applied overlay as well as the stiffness (moduli and thickness) of the existing pavement structure. For a structure comprising an asphalt layer overlying a granular base on a cohesive subgrade, stress dependency adjustments to the moduli of the base and subgrade can be calculated using the procedure described in section 10.5.4 (New Zealand Supp.). This procedure, which was developed from the application of a general mechanistic analysis of typical pavement configurations using a range of typical moduli, uses the "*transformed pavement structure*" principle which is described in section 10.5.3 (NZ Supp.)

10.5.3 Transformed Pavement Principle

The multi-layered pavement structure is transformed to an equivalent single-layered granular system of the same stiffness. The equivalent granular thickness (t_e), and hence the transformed pavement thickness (h_e), are calculated using the following expressions:

$$t_e = t_o[(E_{AC})^{1/3}/(E_{GR} \text{ x } E_{SG})^{1/6}]$$

$$h_e = h + t_e$$

The symbols are as defined in Figure 10.3 (NZ Supp.).

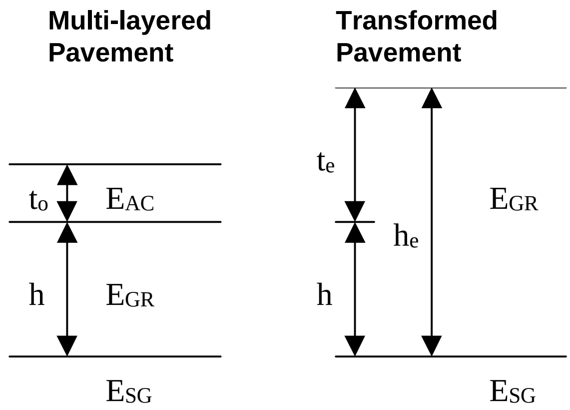

Figure 10.3 – Transforming a double-layered structure into a single layer structure of similar stiffness

10.5.4 Stress Dependency Adjustments

The adjusted modulus for the unbound granular base ($E_{GR(adj)}$) is calculated using Equation 10.5.

$$E_{GR(adj)} = E_{SG} \text{ x } (E_{GR}/E_{SG})^{h/he} \quad \text{[Equation 10.5]}$$

The symbols are defined in Figure 10.3.

The adjusted modulus for an unbound cohesive subgrade ($E_{SG(adj)}$) is calculated using Equation 10.6.

$$E_{SG(adj)} = F_{SG} \text{ x } E_{SG} \quad \text{[Equation 10.6]}$$

F_{SG} is the subgrade adjustment factor selected from Table 1(NZ Supp, Appendix E). Subgrade adjustment factors for pavement configurations not listed in Table E1 can be obtained by linear interpolation.

For F_{SG} values smaller than 1.3, the influence of the subgrade modulus adjustment becomes negligible, and can therefore be ignored. From an analysis of many pavement configurations, it is evident that F_{SG} is relatively small (< 1.3) in pavement configurations where the existing subgrade is relatively stiff (i.e. where E_{SG} > 70 MPa). Subgrade modulus adjustments are therefore considered to be unwarranted if the modulus assigned to the existing subgrade exceeds 70 MPa.

10.5.5 Asphalt Overlay on an Asphalt Pavement

An asphalt pavement structure is considered to have failed when the total cumulative damage, either due to fatigue (asphalt distress) or due to rutting (subgrade distress), reaches 100%.

The cumulative damage factor (CDF) is defined as:

$CDF = N_{app}/N_{allow}$ [Equation 10.7]
where:
N_{app} = applied traffic load in ESAs;
N_{allow} = allowable traffic load in ESAs as determined from the appropriate performance relationship.

The asphalt cumulative damage factor in the existing asphalt due to past traffic ($CDF_{(AC:old)past}$) is determined using Equation 10.7 with the tensile strain at the bottom of the existing (old) asphalt layer. The remaining life damage factor for the existing asphalt ($CDF_{(AC:old)RL}$) is given by:

$CDF_{(AC:old)RL} = 1 - CDF_{(AC:old)past}$ [Equation 10.8]

When assessing the service life of an asphalt overlay on an existing asphalt surface, two cases are considered:

Case 1:

If the cumulative damage factor in the existing asphalt due to the past traffic ($CDF_{(AC:old)past}$) exceeds unity, the existing asphalt life has been totally consumed by the past traffic, and the existing asphalt layer is considered to have no remaining life. The existing asphalt is, for the purpose of further analysis, modelled as unbound with a reasonable low modulus, in the range 150 - 500 MPa. (Bayomi et al. 1997). A modulus similar to the value assigned to the underlying granular base is considered to be appropriate for this purpose unless a different value can be substantiated. The asphalt overlay thickness provided shall be sufficient to satisfy the two criteria specified in Section 10.5.1 (NZ Supp.).

Case 2:

If the cumulative damage factor in the existing asphalt due to the past traffic, ($CDF_{(AC:old)past}$) does not exceed unity, the existing asphalt layer has remaining life. The asphalt overlay thickness provided shall be sufficient to satisfy the two criteria specified in section 10.5.1 (NZ Supp.), as well as the following criterion:

$CDF_{AC(old)} \leq CDF_{(AC:old)RL}$ (*Criterion 3*)

where:
$CDF_{AC(old)}$ = the cumulative damage factor calculated in the old asphalt under the future (design) traffic, and

$CDF_{(AC:old)RL}$ = the remaining life damage factor (Equation 10.8, NZ Supp.)

10.5.6 "Life after death" analysis

The objective of *Criterion 3* is to control the remaining life in the existing asphalt. Meeting the requirements of *Criterion 3* would ensure that the existing asphalt will not reach the end of its fatigue life under the future (design) traffic load.

If the trial overlaid pavement configuration being analysed meets the two criteria specified in Section 10.5.1 (NZ Supp.), but fails to satisfy *Criterion 3*, it means that the remaining life in the existing asphalt layer is consumed part-way through the future design traffic. It is, however, possible that the pavement may have sufficient "life after the death" of the existing asphalt to accommodate the remaining future traffic (N_{RFT}). The latter can be calculated from Equation 10.9:

$N_{RFT} = N_F - N_{(AC:old)RL}$ [Equation 10.9]

where:
N_{RFT} = remaining future traffic, in ESA's, after the end of life of the existing asphalt,
N_F = future (design) traffic, in ESA's, and
$N_{(AC:old)RL}$ = the life remaining in the existing asphalt at the time of rehabilitation, in ESA's.

To analyse the "life after death" capacity of the pavement, adopt a pavement configuration in which the existing asphalt is modelled as unbound with a modulus similar to that of the underlying granular material. The overlay thickness provided shall be adequate to:

- limit the strain in the subgrade to a design level as determined in section 10.4.2 (NZ Supp.) using the future remaining traffic (NRFT) as the design traffic parameter, and
- limit the horizontal tensile strain at the bottom of the asphalt overlay to a value not exceeding the future remaining traffic design strain as determined by the performance (fatigue) relationship given in Section 6.4.6 (Austroads Guide)

10.5.7 Characterisation of Existing Asphalt

The modulus of asphalt is dependent upon both temperature and the rate of loading. In the mechanistic model, the asphalt overlay must be characterised at the "Weighted Mean Average Pavement Temperature" (WMAPT) for the locality. Values of WMAPT for selected locations throughout New Zealand are listed in Table C1 (Appendix C, NZ Supp.).

Characterisation of the existing asphalt depends on level of distress. If, for the design period, the existing asphalt is considered to be cracked, select the asphalt stiffness for the appropriate speed of loading and WMAPT from Table 1(Appendix F, NZ Supp.). If the existing asphalt is in a sound condition, it is recommended that its modulus be back-calculated from deflection test results.

Asphalt moduli back-calculated from deflection test results require a temperature adjustment if the tests were conducted at a pavement temperature other than the WMAPT for the locality. If, for the design period, the asphalt is considered to be sound, determine the in-service asphalt modulus, E_{AC} , as follows:

$E_{AC} = E_{BC} \times (a / b)$ [Equation 10.10]
where:
E_{BC} = back-calculated asphalt modulus,
a = relative asphalt stiffness, from Table 1 (Appendix G, NZ Supp.), for the appropriate speed of loading at the WMAPT of the location, and
b = relative asphalt stiffness, from Table 1 (Appendix G, NZ Supp.), for the relevant test device at the measurement temperature.

10.6 EXAMPLES OF MECHANISTIC PROCEDURE FOR REHABILITATION DESIGN

Examples of the mechanistic procedure for rehabilitation design are contained in Appendix A (NZ Supp.).

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APPENDIX A : REHABILITATION DESIGN FLOW CHARTS AND EXAMPLES

The State Highway Pavement Design and Rehabilitation Manual (Transit, 1989) was written in a format to enable relatively inexperienced staff to follow through the methodologies set out and to arrive at a suitable design. In the 1992 AUSTROADS Guide all investigation and design of pavements, including rehabilitation is assumed to be undertaken by or under the close direction of an experienced professional pavement designer. This is the approach taken in this NZ Supp.

The experienced designer must be involved in all field and laboratory testing and derivation of data for input into the mechanistic models. This includes the selection of appropriate test methodologies, and the visual assessment of the pavement condition.

The input data for the analysis must be representative of the pavement structure and condition and must take into account variability along the road section and environmental effects that may affect the pavement in the long term.

Pavement rehabilitation requires a clear understanding of what is happening to the existing pavement, its history and environment. Once the relevant parameters are established the same procedures employed in the design of new pavements should be applied. The design parameters can be input into CIRCLY (or a similar computer program) for mechanistic analysis, as documented in Section 10 (NZ Supp.) to produce an appropriate rehabilitation treatment.

As an aid to the rehabilitation process four flow charts and examples have been developed in Appendix A:
Chart A1 Guidelines for Determining Appropriate Rehabilitation Treatments.
Chart A2 Primary and Secondary Condition Assessment.
Chart A3 Field and Laboratory Investigation.
Chart A4 Pavement Rehabilitation Design.
Example 1 : Unbound granular overlay design procedure
Example 2 : Light stabilisation (modification) of a relatively thin granular pavement
Example 3 : Moderate cementation of the upper part of a relatively thick granular pavement
Example 4 : Asphalt overlay on a thin-surfaced (e.g. chip seal) granular pavement.
Example 5 : Asphalt overlay on an existing asphalt surface which is cracked and consequently without remaining life.
Example 6 : Asphalt overlay on an existing asphalt surface which is still considered to be in a sound condition with some remaining life.
Example 7 : Asphalt overlay on existing asphalt (in SOUND condition) using “life after death” analysis of existing asphalt layer.

Chart A1 : Guidelines for Determining Appropriate Rehabilitation Treatments.

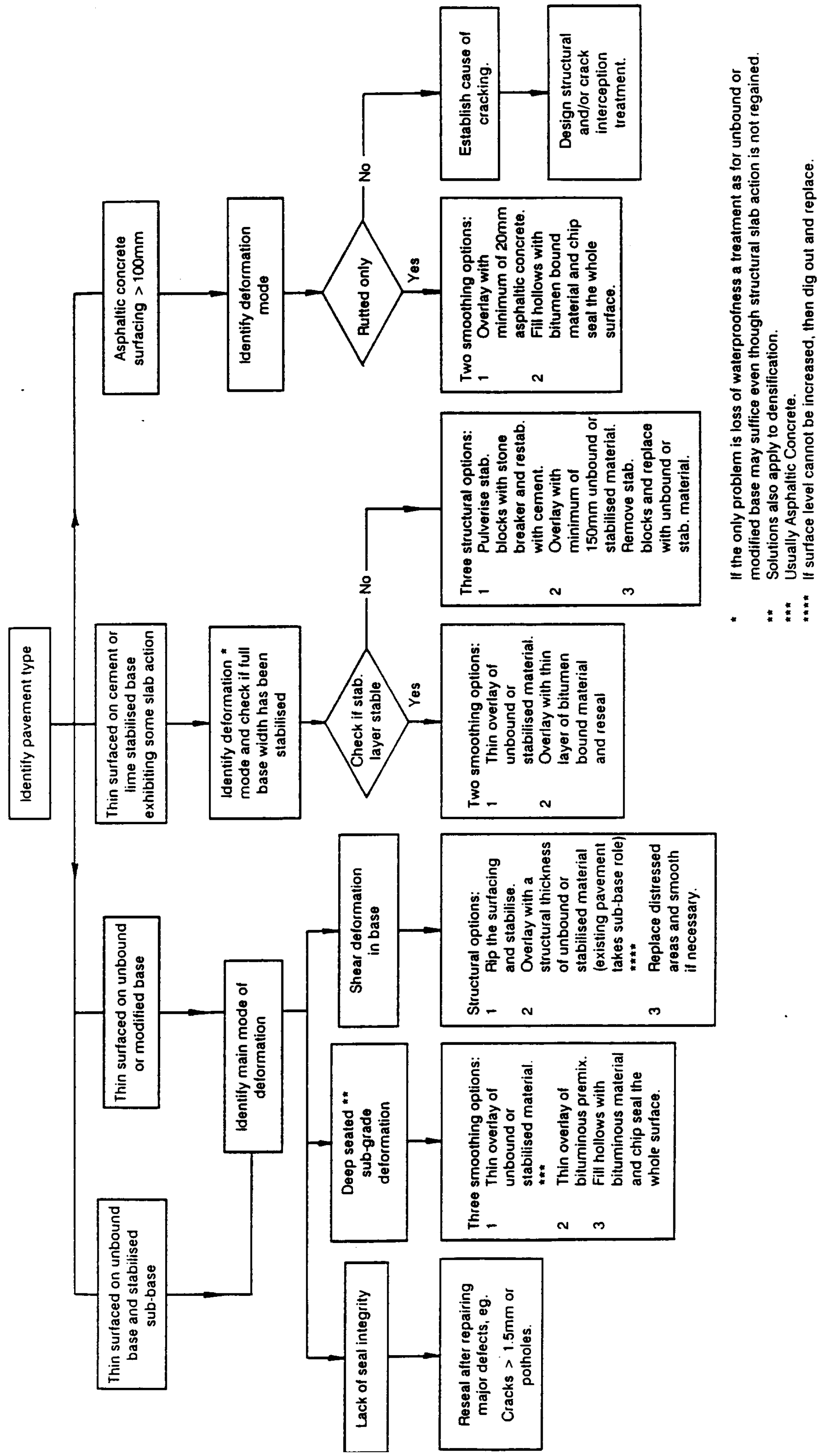


Chart A2 : Primary and Secondary Condition Assessment.

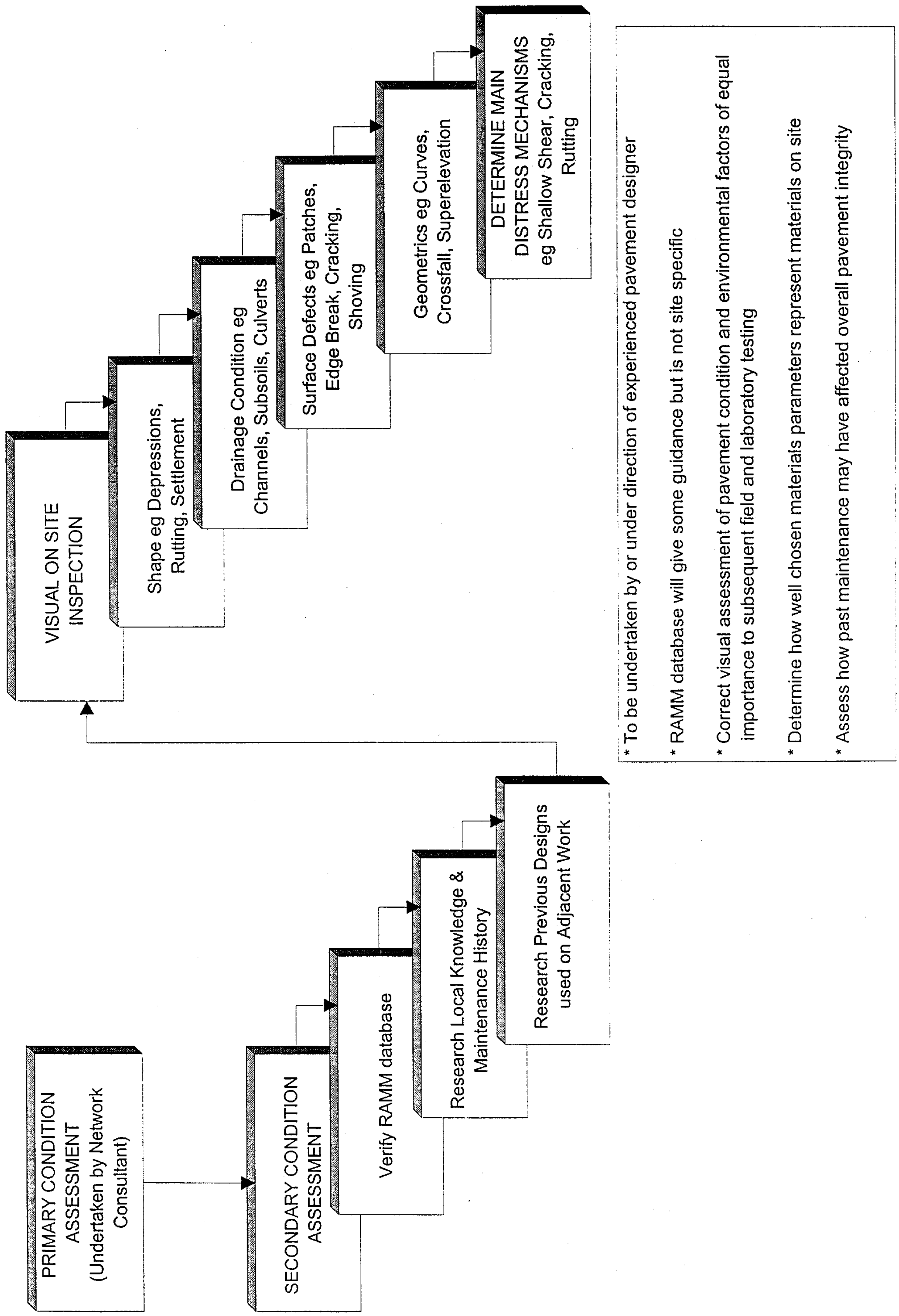
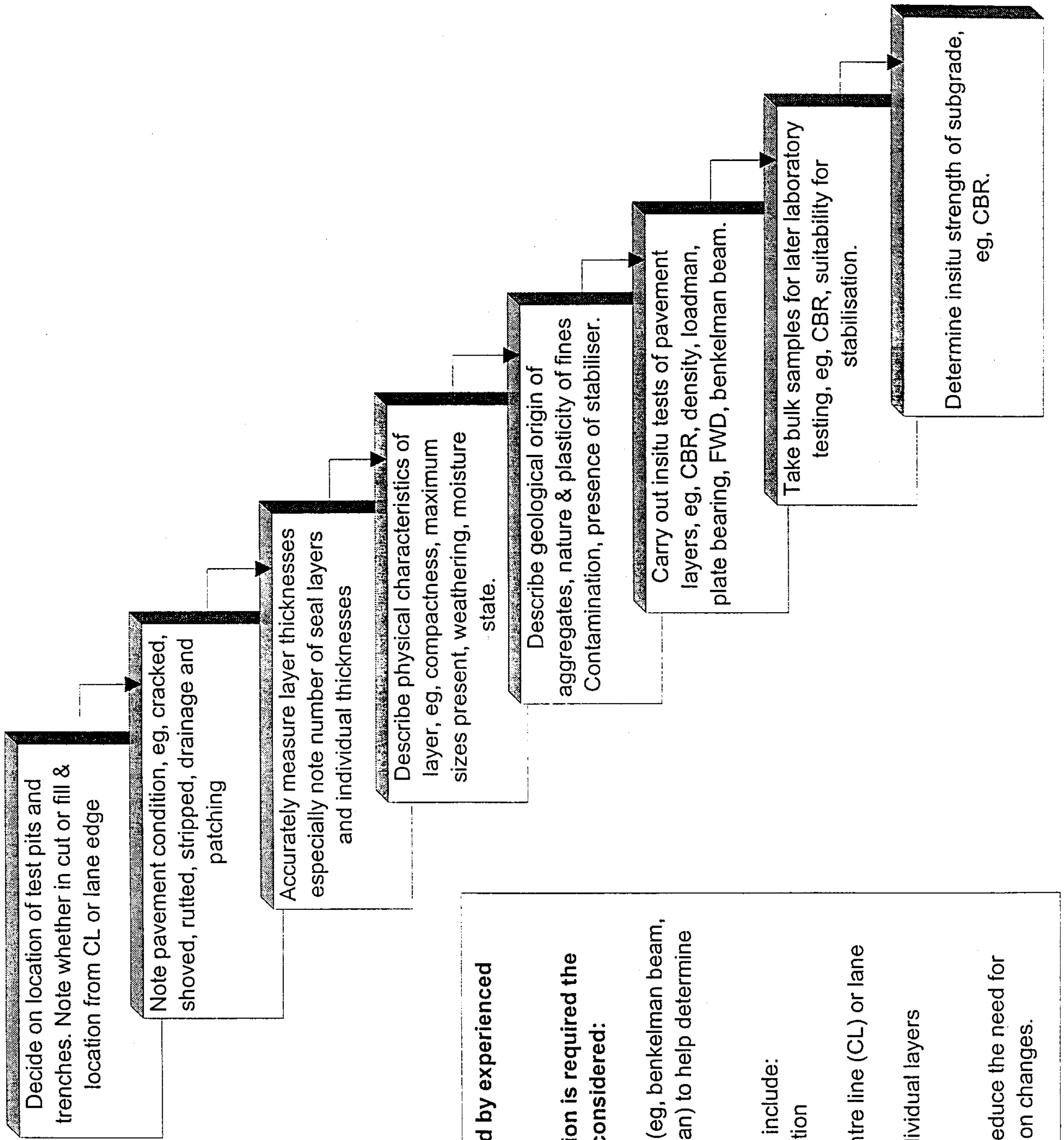


Chart A3 : Field and Laboratory Investigations.



Investigation scope to be determined by experienced pavement design engineer.

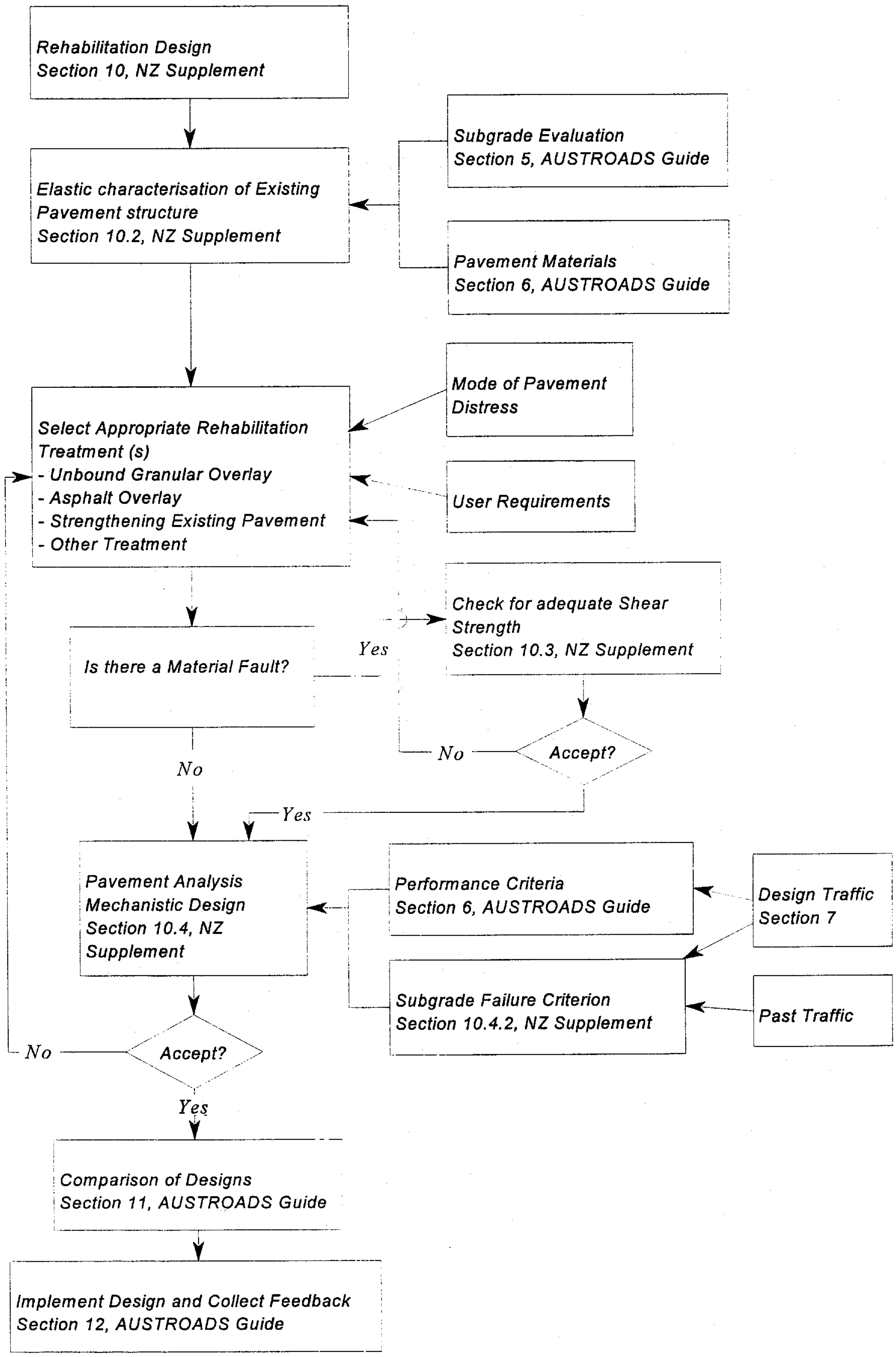
Where detailed pavement investigation is required the following sequence of tests maybe considered:

* Carry out non destructive testing first (eg, benkelman beam, falling weight deflectometer or loadman) to help determine test pit location.

* Field data to be collected in report to include:
  - description of pavement condition
  - pavement history if known
  - test pit location offset from centre line (CL) or lane edge
  - test pit logs with depths on individual layers referenced to datum

Careful recording of all data will often reduce the need for repeat testing if proposed design solution changes.

Chart A4 : Pavement Rehabilitation Design.



Example 1 : Unbound granular overlay design procedure

Procedure to determine the overlay thickness required to achieve a specified design traffic target:

1. Model the existing pavement in CIRCLY. Use the following parameters:
   - *Traffic count* : Past traffic (N_P)
   - *Subgrade strain criterion*: $\varepsilon = 0.021N^{-0.23}$

2. Run the CIRCLY analysis and record the subgrade cumulative damage factor due to the past traffic ($CDF_{(SG)past}$).

3. Postulate a trial overlay thickness (t_0).

4. Modify the thickness of the existing granular layer in CIRCLY to include the overlay thickness. Use the same modulus for the overlay as the value assigned to the existing granular layer in step 1, unless greater values can be substantiated (see section 10.4.1, NZ Supp.).
   - Change the traffic count value to future traffic (N_F).

5. Re-run the CIRCLY analysis on the overlaid configuration and note the subgrade cumulative damage factor in the trial configuration due to the future traffic ($CDF_{(SG)trial}$).
   - Subgrade strain requirements are satisfied when $CDF_{(SG)trial} \leq CDF_{(SG)past}$.
   - Repeat the procedure from step 3 (if necessary) until this requirement is met.

6. Check if the calculated overlay thickness also satisfies shear requirements in cases where the existing granular material is deemed to be of a lesser quality than M/4. This is done in accordance with section 10.3 (NZ Supp.).

7. The overlay thickness selected is the larger of the values calculated to satisfy subgrade strain and shear requirements respectively.

Given:	Existing pavement: 350mm U/B granular:	Estimated E_{GR}	= 200 MPa
	Quality of existing base material is suspect:	Estimated CBR	= 60%
	Existing subgrade: Fine-grained, cohesive:	Estimated E_{SG}	= 70 MPa
	Past traffic:	Estimated N_P	= 1×10^5 ESA

Find: Granular overlay thickness for a future (design) traffic of $N_F = 5 \times 10^5$ ESA

Characterisation:

EXISTING PAVEMENT

350mm	E_{gr} = 200 MPa (sub-layered)
	E_{sg} = 70 MPa

OVERLAID CONFIGURATION

	t_o = 125mm	E_{gr} = 200 MPa (sub-layered)
475mm	350mm	
		E_{sg} = 70 MPa

Solution: Example 1 : Unbound Granular Overlay

	PROCEDURE	*Reference*	EXAMPLE 1
1	Model existing pavement in CIRCLY		Traffic count: (N_P): $1x10^5$ Layers: No 1: Gran200 - 350mm No 2: TNZ-SG70 - 0mm Performance criterion: Subgrade
2	Run CIRCLY analysis. Record subgrade cumulative damage factor under past traffic ($CDF_{(SG)past}$)		$CDF_{(SG)past} = 0.42$
3	Postulate a trial overlay thickness (t_o)		t_o = 125 mm
4	Change the traffic count to N_F . Modify pavement thickness in CIRCLY layered system (add t_o to existing granular thickness).		Traffic count: (N_P): $5x10^5$ Layers: No 1: Gran200 - 475mm No 2: TNZ-SG70 - 0mm Performance criterion: Subgrade
5	Run CIRCLY analysis. Note the $CDF_{(SG)trial}$ value for the trial configuration and check it against $CDF_{(SG)past}$. Repeat from step 3 until $CDF_{(SG)trial} \leq CDF_{(SG)past}$		For t_o = 125 mm, $CDF_{(SG)trial} = 0.42$ ∴125mm of overlay is required to satisfy subgrade strain requirement.
6	Check overlay thickness required to satisfy shear requirements.	NZ Supp. §10.3.2 Fig 10.2	For CBR=60%, the overlay depth to satisfy shear requirements = 80 mm
7	The overlay thickness selected is the larger of the values calculated to satisfy subgrade strain and shear requirements respectively.	NZ Supp. §10.4.1	Answer: Overlay thickness governed by subgrade strain = 125mm

Examples 2 & 3 : Stabilisation treatment of existing granular layers

Two stabilisation treatment examples are presented:

Example 2 : Light stabilisation (modification) of a relatively thin granular pavement
Example 3 : Moderate cementation of the upper part of a relatively thick granular pavement

Section 6.6 of the NZ Supp. describes the different categories of stabilisation.

Procedure to determine the improvement in the traffic load carrying capacity of an existing pavement if the strength (modulus) of the existing material is improved by stabilisation:

1. Model the existing pavement in CIRCLY. Use the following parameters:
   - Traffic count : Past traffic (N_P)
   - Subgrade strain criterion: $\varepsilon = 0.021N^{-0.23}$

2. Run the CIRCLY analysis and record the subgrade cumulative damage factor due to the past traffic $CDF_{(SG)past}$.

3. Change the modulus of the existing granular layer in CIRCLY to the improved (modified) value.

4. Postulate a trial traffic count (N_F) and check the subgrade cumulative damage factor due to the trial traffic count ($CDF_{(SG)trial}$). Repeat this step until $CDF_{(SG)trial} = CDF_{(SG)past}$.

Example 2 : Light stabilisation (modification) of a relatively thin granular pavement

Given: Existing pavement: 200mm U/B granular: Estimated E_{GR} = 200 MPa
Existing subgrade: Fine-grained, cohesive: Estimated E_{SG} = 50 MPa
Past traffic: Estimated $N_P = 1x10^4$ ESA

Treatment: Light, full depth stabilisation of existing granular material resulting in an estimated modulus of 600 MPa.

Find: The improvement in pavement life expectancy (i.e. N_F/N_P)

Note: The qualifications for *modified* soils that are described in section 6.6.1 of the *Supplement* pertain to subgrade soils. The modulus of low quality base or sub-base materials can readily undergo a three-fold increase through moderate stabilisation.

Characterisation:

EXISTING PAVEMENT

200mm	E_{gr} = 200 MPa (sub-layered)
	E_{sg} = 50 MPa

REHABILITATED CONFIGURATION

200mm	E_{gr} = 600 MPa (sub-layered)
	E_{sg} = 50 MPa

Solution - Example 2 : Light stabilisation (modification) of a relatively thin granular pavement

	PROCEDURE	Reference	EXAMPLE 2
1	Model existing pavement in CIRCLY		Traffic count: (N_P): $1x10^4$ Layers: No 1: Gran200 - 200mm No 2: TNZ-SG50 - 0mm Performance criterion: Subgrade
2	Run CIRCLY analysis. Record subgrade cumulative damage factor under past traffic ($CDF_{(SG)past}$)		$CDF_{(SG)past}$ = 3.4
3	Modify base top modulus in CIRCLY layered system. Postulate trial future traffic count (N_F).		Traffic count: (N_F): $2.5x10^4$ Layers: No 1: Gran600 - 200mm No 2: TNZ-SG50 - 0mm Performance criterion: Subgrade
4	Run CIRCLY analysis. Repeat by changing the traffic count until $CDF_{(SG)trial} = CDF_{(SG)past}$		$CDF_{(SG)trial}$ = 3.4 when the traffic count is $2.5x10^4$ N_F/N_P = 2.5/1.0 = 2.5

Example 3 : Moderate cementation of the upper part of a relatively thick granular pavement

Given:	Existing pavement: 450mm U/B granular:	Estimated E_{GR}	= 400 MPa
	Existing subgrade: Fine-grained, cohesive:	Estimated E_{SG}	= 70 MPa
	Past traffic:	Estimated N_P	= $1x10^6$ ESA

Treatment: Moderate, in-situ stabilisation of existing granular material to a depth of 200mm, resulting in an estimated modulus of 1200 MPa.

Find: The expected design life improvement (i.e. N_F/N_P).

Note: Moderately cemented materials with moduli in the range 1000 to 2000 MPa should be pre-cracked during construction and characterised as unbound (NZ Supp., section 6.6.2)

Characterisation:

EXISTING PAVEMENT

450mm	E_{gr} = 400 MPa (sub-layered)
	E_{sg} = 70 MPa

REHABILITATED CONFIGURATION

450mm	200mm	E_{gr} = 1200MPa stabilised (sub-layered)* layer
	250mm	
		E_{sg} = 70 MPa

Solution - Example 3 : Moderate cementation of the upper part of a relatively thick granular pavement

	PROCEDURE	Reference	EXAMPLE 3
1	Model existing pavement in CIRCLY		Traffic count: (N_P): $1x10^6$ Layers: No 1: Gran400 - 450mm No 2: TNZ-SG70 - 0mm Performance criterion: Subgrade
2	Run CIRCLY analysis. Record subgrade cumulative damage factor under past traffic ($CDF_{(SG)past}$)		$CDF_{(SG)past}$ = 0.6
3	Modify base top modulus in CIRCLY layered system. Postulate trial future traffic count (N_F).		Traffic count: (N_P): $2.7x10^6$ Layers: No 1: Gran1200 - 450mm No 2: TNZ-SG70 - 0mm Performance criterion: Subgrade
4	Run CIRCLY analysis. Repeat by changing the traffic count until $CDF_{(SG)trial} = CDF_{(SG)past}$		$CDF_{(SG)trial}$ = 0.6 when $N_F = 2.7x10^6$ i.e. N_F/N_P = 2.7/1.0 = 2.7

* NOTE: In this example the pavement is, after stabilising the top part, modelled as comprising a single granular layer. This is justified as long as the rehabilitated layer is characterised as an <u>unbound</u> layer. After the process of sub-layering has taken its course, there is little difference between a rigorous <u>multi-layer</u> model and the simpler <u>single layer</u> model. For instance, in the above example, the answer (N_F/N_P) for a rigorous multi-layered analysis is 2.5.

Example 4 : Asphalt overlay on a thin-surfaced (e.g. chip seal) granular pavement

1. Model the existing pavement in CIRCLY. Use the following parameters:
   - Traffic count : Past traffic (N_P).
   - Subgrade strain criterion: $\varepsilon = 0.021N^{-0.23}$

2. Run the CIRCLY analysis and record the subgrade cumulative damage factor due to the past traffic $CDF_{(SG)past}$.

3. Postulate a trial overlay thickness (t_0).

NOTE: Steps 4 to 6 provide for adjustment to the moduli of unbound materials to account for the stress dependent properties of these materials. To obtain a first order of magnitude answer, these steps can initially be ignored and only included in the final analysis to refine the answer.

4. To transform the system comprising a granular layer and an asphalt layer into an equivalent single granular layer, first calculate the equivalent granular thickness (t_e) and then the transformed pavement thickness (h_e), from:

$$t_e = t_0 [(E_{AC})^{1/3} / (E_{GR} \times E_{SG})^{1/6}]$$
$$h_e = h + t_e$$

   where: E_{AC} = Asphalt (overlay) modulus (MPa)
   E_{GR} = Granular base modulus (top) (MPa)
   E_{SG} = Subgrade modulus (MPa)
   t_0 = asphalt overlay thickness (mm)
   h = existing granular base thickness (mm)

5. Adjust modulus of the <u>granular</u> material: $E_{GR(adj)} = E_{SG} \times (E_{GR} / E_{SG})^{h/he}$

6. Adjust the modulus of the <u>subgrade:</u> Obtain the subgrade adjustment factor F_{SG} from Table 1 (NZ Supp., Appendix E). The effect of this adjustment is negligible and can be ignored if the adjustment factor F_{SG} is less than 1.3. Adjusted subgrade modulus: $E_{SG(adj)} = F_{SG} \times E_{SG}$

7. Change the layered system in CIRCLY to the overlaid configuration with adjusted moduli. Also change the traffic count to the future traffic value (N_F), and select the subgrade performance criterion.

8. Re-run the CIRCLY analysis on the overlaid configuration and determine the subgrade cumulative damage factor in the trial configuration due to the future traffic ($CDF_{(SG)trial}$). <u>Subgrade strain</u> requirements are satisfied when $CDF_{(SG)trial} \leq CDF_{(SG)past}$. Repeat the procedure from step 3 (if necessary) until this requirement is met.

9. Change the performance strain criterion to asphalt. Re-run the CIRCLY analysis on the overlaid configuration and determine the asphalt cumulative damage factor in the trial configuration due to the future traffic $CDF_{(AC)trial}$. <u>Asphalt strain</u> requirements are satisfied when $CDF_{(AC)trial} \leq 1$. Repeat the procedure from step 3 (if necessary) until this requirement is met.

Given: Existing pavement: 400mm U/B granular: E_{GR} = 450 MPa
Subgrade: Fine-grained, cohesive: E_{SG} = 70 MPa
Past traffic estimate: $N_P = 1 \times 10^6$ ESA

Treatment: ASPHALT overlay. Assume an asphalt with stiffness E_{AC} = 2100 MPa, and with a fatigue constant K= 4624 [in $N=(K/\mu\varepsilon)^5$]

Find: Asphalt overlay thickness for a future traffic $N_F = 5 \times 10^6$ ESA
Characterisation:

EXISTING PAVEMENT		OVERLAID CONFIGURATION	
		T_0=180mm	E_{AC} = 2100MPa
400mm	E_{GR} = 450Mpa (sublayered)	400mm	$E_{GR(adj.)}$ = 180MPa (sublayered)
	E_{SG} = 70MPa		$E_{SG(adj)}$ = 90MPa

Solution - Example 4 : Asphalt overlay on a thin-surfaced (e.g. chip seal) granular pavement.

	PROCEDURE	Reference	EXAMPLE 4	
1	Model existing pavement in CIRCLY		Traffic count: (N_P) 1 x 10^6 Layers: 1. Gran450 - 400mm 2. TNZ-SG70 - 0mm Performance criterion Subgrade	
2	Run CIRCLY analysis. Record subgrade cumulative damage factor under past traffic ($CDF_{(SG)past}$)		$CDF_{(SG)past}$ = 1.08	
3	Postulate a trial overlay thickness		t_o = 180 mm	
4	Transform overlaid pavement to an equivalent granular pavement. Transformed thickness $h_e = h + t_e$ where $t_e = t_o\ [(E_{AC})^{1/3}/(E_{GR} \times E_{SG})^{1/6}]$		$t_e = t_o\ [(E_{AC})^{1/3}/(E_{GR} \times E_{SG})^{1/6}]$ $= 180\ [2100^{1/3}/(450 \times 70)^{1/6}]$ = 180 [2.28] = 410 mm $h_e = h + t_e$ = 400+ 410 = 810 mm	
5	Stress dependency adjustment to unbound granular modulus (E_{GR})		$E_{GR(adj)} = E_{SG}\ (E_{GR} / E_{SG})^{h/he}$ $= 70\ (450/70)^{400/810}$ = 176 MPa ≈ 180 Mpa	
6	Subgrade stress dependency adjustment: Obtain F_{SG} from Table 1 E_{SG}(adjusted) = F_{SG} x E_{SG}(original)	*NZ Suppl.* Appendix E:	E_{gr} = 450 E_{SG} = 70 h_{gr} = 400 t_e = 410	⇒ F_{SG} = 1.3 E_{SG}(adj)= 1.3 x 70 ≈ 90 MPa
7	Adjust moduli in the overlaid configuration. Change traffic count to future traffic (N_F). Select subgrade performance criterion.		Traffic count: (N_F) 5 x 10^6 Layers: 1. Asph2100 - 180mm 2 .Gran180 - 400mm 3. TNZ-SG90 - 0mm Performance criterion: Subgrade	
8	Run CIRCLY analysis. Determine $CDF_{(SG)trial}$ and check against $CDF_{(SG)past}$. Repeat from step 3 until $CDF_{(SG)trial} \le CDF_{(SG)past}$		$CDF_{(SG)trial}$ = 0.043 < 1.08 (∴OK)	
9	Change Performance criterion to Asphalt: Run CIRCLY analysis. Check $CDF_{(AC)trial}$ against unity. Repeat from step 3 until $CDF_{(AC)trial} \le 1$		$CDF_{(AC)trial}$ = 0.93 < 1.0 (∴OK) ∴ 180 mm asphalt overlay acceptable	

Examples 5, 6 & 7: Asphalt overlay designs on an existing asphalt surface

1. Model the existing pavement in CIRCLY. Use the following parameters:

   - Traffic count : Past traffic (N_P)

   - Subgrade strain criterion: $\varepsilon = 0.021N^{-0.23}$

   Asphalt modulus:

   If, for the design period, the existing surface is considered to be cracked: Establish the WMAPT for the location from the NZ Supp. Appendix C, Table 1, and select the appropriate asphalt modulus from the NZ Supp. Appendix F, Table 1.

   If, for the design period, the existing surface is considered to be in a sound condition, the modulus can be presumed or back-calculated from deflection testing. In the latter case, establish the WMAPT for the location from the NZ Supp., Appendix C, Table 1, and calculate the asphalt modulus from:

   $$E_{AC} = E_{BC} \times (a/b)$$

   where:
   E_{BC} = back-calculated modulus (at test temperature),
   a = value from Appendix G (NZ Supp.) corresponding to the WMAPT for the location, and
   b = value from Appendix G (NZ Supp.) corresponding to the test temperature.

2. Run the CIRCLY analysis and record the subgrade cumulative damage factor due to the past traffic ($CDF_{(SG)past}$).

3. Change the performance criterion to asphalt. Re-run the CIRCLY analysis and record the asphalt cumulative damage factor due to the past traffic ($CDF_{(AC:old)past}$).

4. Validate the status of the existing asphalt and adjust the pavement model for further analysis (i.e. analysis of the expected future performance of the existing pavement):

   - Case 1: If $CDF_{(AC:old)past} \geq 1$, the existing asphalt has no remaining life left. The existing asphalt is modelled as unbound with a relatively low modulus (150-500MPa). A value equal to the modulus of the existing granular base is considered appropriate.

   - Case 2: If $CDF_{(AC:old)past} < 1$, the asphalt has some remaining life left. Model the existing asphalt as a separate layer with modulus as calculated in step 1.
   Calculate and record the available remaining life damage factor: $CDF_{(AC:old)RL} = 1 - CDF_{(AC:old)past}$

5. Postulate a trial overlay thickness (t_0).

   NOTE: Steps 6 to 8 which follow provide for adjustment to the moduli of unbound materials to account for the stress dependent properties of these materials. To obtain a first order of magnitude answer, these steps can initially be ignored and only included in the final analysis to refine the answer.

6. To transform the system comprising an asphalt layer over a granular layer into an equivalent single granular layer, first calculate the equivalent granular thickness (t_e) and then the transformed pavement thickness (h_e), from:

   $$t_e = t_0 [(E_{AC})^{1/3} / (E_{GR} \times E_{SG})^{1/6}]$$
   $$h_e = h + t_e$$

   where: E_{AC} = Asphalt (overlay) modulus (MPa)
   E_{GR} = Granular base modulus (top) (MPa)
   E_{SG} = Subgrade modulus (MPa)
   t_0 = asphalt overlay thickness (mm)
   h = existing granular base thickness (mm)

7. Adjust the modulus of the granular material: $E_{GR(adj)} = E_{SG} \times (E_{GR}/E_{SG})^{h/he}$

8. Adjust the modulus of the subgrade: Ascertain the subgrade adjustment factor F_{SG} from Appendix E (NZ Supp., Appendix E). The effect of this adjustment is negligible and can be ignored if the adjustment factor F_{SG} turns out to be less than 1.3. Adjusted subgrade modulus: $E_{SG(adj)} = F_{SG} \times E_{SG}$

9. Change the layered system in CIRCLY to the overlaid configuration with adjusted moduli. Also change the traffic count to the future traffic value (N_F), and select the subgrade performance criterion.

10. Re-run the CIRCLY analysis on the overlaid configuration and determine the subgrade cumulative damage factor in the trial configuration due to the future traffic ($CDF_{(SG)trial}$). Subgrade strain requirements are satisfied when $CDF_{(SG)trial} \le CDF_{(SG)past}$. Repeat the procedure from step 5 (if necessary) until this requirement is met.

11. Change the performance criterion to new (overlay) asphalt. Re-run the CIRCLY analysis on the overlaid configuration and determine the new asphalt cumulative damage factor in the trial configuration due to the future traffic $CDF_{(AC:O/L)trial}$. The asphalt strain requirements in the new asphalt layer are satisfied when $CDF_{(AC:O/L)trial} \le 1$. Repeat the procedure from step 5 (if necessary) until this requirement is met.

12. This step, and the rest of the procedure that follows, only apply to the Case 2 scenario (i.e. the existing asphalt has some remaining life). Change the performance criterion to old asphalt. Re-run the CIRCLY analysis on the overlaid configuration and determine the old asphalt cumulative damage factor in the trial configuration due to the future traffic $CDF_{(AC:old)trial}$. The remaining life asphalt strain requirements for the existing asphalt are satisfied when $CDF_{(AC:old)trial} \le CDF_{(AC:old)RL}$ (step 4). If this requirement is not met, it means that the life in the existing pavement is consumed part-way through the future traffic.

The remainder of the procedure (from step 13 onward) serves to check if there is sufficient "life after death" in the existing asphalt to accommodate the balance of the future traffic referred to as the Future Remaining Traffic (N_{FRT}).

13. Express the remaining life in the existing asphalt ($N_{(AC:old)RL}$) in terms of ESA's.

$$N_{(AC:old)RL} = \left[\frac{CDF_{(AC:old)RL}\ (\text{step 4})}{CDF_{(AC:old)trial}\ (\text{step12})}\right] \times N_F$$

14. Calculate the future remaining traffic (N_{FRT}) after the end of life of the existing asphalt in terms of ESA's
$N_{FRT} = N_F - N_{(AC:old)RL}$

15. Calculate the remaining life cumulative damage factor in the overlay after the end of life of the existing asphalt:

$$CDF_{(AC:O/L)RL} = 1 - \left[\frac{N_{(AC:old)RL}\ (\text{step13})}{N_F} \times CDF_{(AC:O/L)trial}\ (\text{step 11})\right]$$

16. Calculate the remaining life cumulative damage factor in the subgrade after the end of life of the existing asphalt:

$$CDF_{(SG)RL} = 1 - \left[\frac{N_{(AC:old)RL}\ (\text{step13})}{N_F} \times CDF_{(SG)trial}\ (\text{step 10})\right]$$

17. Adopt a new configuration (model) for analysis of the pavement after the end of life of the existing asphalt. The existing asphalt is now modelled as an unbound layer with modulus similar to that of the granular base. This pavement is then transformed to an equivalent granular pavement (as in step 6) in order to make the necessary stress dependency adjustments to the unbound base and subgrade materials.

18. Adjust the modulus of the unbound base material (as in step 7):

Adjusted base modulus: $E_{GR(adj)} = E_{SG} \times (E_{GR}/E_{SG})^{h/he}$

19. Adjust the modulus of the subgrade (as in step 8):

Ascertain the subgrade adjustment factor F_{SG} from Appendix E (NZ Supp., Appendix E). The effect of this adjustment is negligible and can be ignored if the adjustment factor F_{SG} turns out to be less than 1.3:

Adjusted subgrade modulus: $E_{SG(adj)} = F_{SG} \times E_{SG}$

20. Change the layered system in CIRCLY to the new model with adjusted moduli. Also change the traffic count to the future remaining traffic value (N_{FRT}) - (see step 14). Select the subgrade performance criterion.

21. 21. Re-run the CIRCLY analysis and determine the subgrade cumulative damage factor due to the future remaining traffic ($CDF_{(SG)FRT}$). Subgrade strain requirements are satisfied when $CDF_{(SG)FRT} \leq CDF_{(SG)RL}$ (step 16).

22. Change the performance criterion to new (overlay) asphalt. Re-run the CIRCLY analysis and determine the overlay asphalt cumulative damage factor due to the future remaining traffic $CDF_{(AC:O/L)FRT}$. The asphalt strain requirements are satisfied when $CDF_{(AC:O/L)FRT} \leq CDF_{(AC:O/L)RL}$ (step 15).

If any of the requirements of steps 21 and 22 are not met, repeat the procedure from step 5.

Example 5 : Asphalt overlay on an existing asphalt surface which is cracked and consequently without remaining life.

Given: Existing pavement:

80mm Asphalt in cracked condition
275mm U/B granular base: E_{GR} = 450 MPa
Subgrade: Fine-grained, cohesive: E_{SG} = 60 MPa
Past traffic estimate: N_P = 1 x 10^6 ESA
WMAPT for the location: 17 °C

Treatment: ASPHALT overlay over the existing AC. Assume an asphalt with stiffness E_{AC} = 2100 MPa, and the volume of bitumen in the mix, V_B = 11%

Find: Asphalt overlay thickness for a future traffic N_F = 2.15 x 10^6 ESA

Characterisation:

1. Existing asphalt stiffness:
   The existing asphalt being cracked, ascertain the stiffness value (modulus) from Appendix F in the NZ Supp. Assume the pavement is designed for highway traffic (at the design speed 80 km/h):
   For WMAPT = 17 °C, E_{AC} = 1500MPa

2. Fatigue constants for asphalt materials:
   From section 6.4.6 (Austroads Guide): K = [6918(0.856V_B + 1.08)/$(S_{mix})^{0.36}$]

Asphalt stiffness	Fatigue constant (K)
E_{AC} = 1500MPa	K = [6918(0.856x11+ 1.08)/$1500^{0.36}$] = 5219
E_{AC} = 2100MPa	K = [6918(0.856x11+ 1.08)/$2100^{0.36}$] = 4624

EXISTING

80mm	$E_{AC(old)}$ = 1500MPa
275mm	E_{GR} = 450MPa (sublayered)
	E_{SG} = 60MPa

OVERLAID

t_0 =160mm	$E_{AC(O/L)}$ = 2100Mpa
355mm	$E_{GR(adj.)}$ = 160Mpa (sublayered)
	$E_{SG(adj)}$ = 90Mpa

Solution - Example 5 : Asphalt overlay on existing asphalt (CRACKED)

	PROCEDURE	Reference	EXAMPLE 5
1	Model existing pavement in CIRCLY		Traffic count: 1×10^6 Layers: 1. Asph1500 - 80mm 2. Gran450 - 275mm 3. TNZ-SG60 - 0mm Performance criterion: Subgrade
2	Run CIRCLY analysis. Record subgrade cumulative damage factor under past traffic ($CDF_{(SG)past}$)		$CDF_{(SG)past} = 1.15$
3	Change performance criterion to Asphalt. Run CIRCLY analysis. Record asphalt cumulative damage factor under past traffic ($CDF_{(AC:old)past}$)		$CDF_{(AC:old)past} = 1.09$
4	Identify condition of existing asphalt: Since $CDF_{(AC:old)past} \geq 1$, it is a CASE 1 situation (no remaining life). For further analysis, remodel cracked asphalt as unbound with $E_{AC(exist)} = E_{GR}$		$CDF_{(AC)past} = 1.09 > 1$ Case 1: Pavement model adopted for further analysis: Layers: 1. Asph2100 - t_o mm 2. Gran450 - 355mm 3. TNZ-SG60 - 0mm
5	Postulate a trial overlay thickness		$t_o = 160$ mm
6	Transform overlaid pavement to an equivalent granular pavement. Transformed thickness $h_e = h + t_e$ $t_e = t_o \, [(E_{AC})^{1/3}/(E_{GR} \times E_{SG})^{1/6}]$		$t_e = t_o \, [(E_{AC})^{1/3}/(E_{GR} \times E_{SG})^{1/6}]$ $= 160 \, [(2100)^{1/3}/(450 \times 60)^{1/6}]$ $= 160 \, [2.34]$ $= 374$ mm $h_e = h + t_e = 355 + 374 = 729$ mm
7	Stress dependency adjustment to unbound granular modulus (E_{GR})		$E_{GR(adj)} = E_{SG} \, (E_{GR} / E_{SG})^{h/he}$ $= 60 \, (450/60)^{355/729}$ $= 160$ MPa
8	Subgrade stress dependency adjustment: Obtain F_{SG} from Table 1 E_{SG}(adjusted) = $F_{SG} \times E_{SG}$(original)	*NZ Suppl.* Appendix E:	$E_{gr} = 450$, $E_{SG} = 60$, $h_{gr} = 355$, $t_e = 374$ $\Rightarrow$ $F_{SG} = 1.5$ $E_{SG}(adj) = 1.5 \times 60$ ≈ 90 MPa
9	Adjust moduli in the overlaid configuration. Change traffic count to future traffic (N_F) Select subgrade performance criterion.		Traffic count: 2.15×10^6 Layers: 1. Asph2100 - 160mm 2. Gran160 - 355mm 3. TNZ-SG90 - 0mm Performance criterion: Subgrade
10	Run CIRCLY analysis. Ascertain $CDF_{(SG)trial}$ for the trial configuration and check against $CDF_{(SG)past}$. Repeat from step 5 until $CDF_{(SG)trial} \leq CDF_{(SG)past}$		$CDF_{(SG)trial} = 0.044$ < 1.15 ($\therefore$OK)
11	Change performance criterion to Asphalt: Run CIRCLY analysis. Ascertain $CDF_{(AC:O/L)trial}$ for the trial configuration and check against unity. Repeat from step 5 until $CDF_{(ACO/L)trial} \leq 1$		$CDF_{(AC:O/L)trial} = 0.99$ < 1.0 ($\therefore$OK) Conclude: 160 mm asphalt overlay acceptable

Example 6 : Asphalt overlay on an existing asphalt surface which is still considered to be in a sound condition with some remaining life

Given: Existing pavement:

80mm Asphalt in SOUND condition	
275mm U/B granular base:	E_{GR} = 450 MPa
Subgrade: Fine-grained, cohesive:	E_{SG} = 60 MPa
Past traffic estimate:	N_P = 0.5 x 10^6 ESA
WMAPT for the location:	15 °C

Treatment: ASPHALT overlay. Assume an asphalt stiffness with E_{AC} = 2100 MPa, and the volume of bitumen in the mix, V_B = 11%

Find: Asphalt overlay thickness for a future traffic N_F = 2.4 x 10^6 ESA

Characterisation:

1. Existing asphalt stiffness:
   The existing asphalt is sound. An FWD deflection test is performed when the pavement temperature is 20 °C and a modulus of 1200MPa is back-calculated. From Appendix G (NZ Supp.), select the relative stiffness values for temperatures of 15°C and 20°C respectively:

   $$E_{AC} = E_{BC} \times (a/b)$$
   $$= 1200 \times (6.35/4.42)$$
   $$\approx 1700\text{Mpa}$$

2. Fatigue constants for asphalt materials:
   From section 6.4.6 (*Austroads Guide*): $K = [6918(0.856V_B + 1.08)/(S_{mix})^{0.36}]$

Asphalt stiffness	Fatigue constant (K)
E_{AC} = 1700MPa	$K = [6918(0.856\text{x}11 + 1.08)/1700^{0.36}] = 4989$
E_{AC} = 2100MPa	$K = [6918(0.856\text{x}11 + 1.08)/2100^{0.36}] = 4624$

EXISTING

80mm	$E_{AC(old)}$ = 1700MPa
275mm	E_{GR} = 450Mpa (sublayered)
	E_{SG} = 60MPa

OVERLAID

t_0 = 125mm	$E_{AC(O/L)}$ = 2100MPa
80mm	$E_{AC(old)}$ = 1700MPa
275mm	$E_{GR(adj.)}$ = 125Mpa (sublayered)
	$E_{SG(adj)}$ = 100MPa

Solution: The model solution appears on the next page.

NOTE: In step 6, for the purpose of estimating the equivalent granular thickness for the old and new asphalt, consider them as one layer with modulus equal to the average of their respective E-values, in this instance Eac = (2100+1700)/2 = 1900MPa.

Solution - Example 6 : Asphalt overlay on an existing asphalt surface which is still considered to be in a sound condition with some remaining life.

	PROCEDURE	Reference	EXAMPLE 6
1	Model existing pavement in CIRCLY		Traffic count: $(N_P) = 0.5x10^6$ Layers: No.1: Asph1700 - 80mm No 2: Gran450 - 275mm No 3: TNZ-SG60 - 0mm Performance criterion; Subgrade
2	Run CIRCLY analysis. Record subgrade cumulative damage factor under past traffic $(CDF_{(SG)past})$		$CDF_{(SG)past} = 0.54$
3	Change performance criterion to Asphalt. Run CIRCLY analysis. Record asphalt cumulative damage factor under past traffic $(CDF_{(AC:old)past})$		$CDF_{(AC:old))past} = 0.62$
4	Identify condition of existing asphalt: Since $CDF_{(AC:old)past} <1$, it is a CASE 2 situation (some life left in asphalt). Calculate remaining life cumulative damage factor, $CDF_{(AC:old)RL} = 1- CDF_{(AC)past}$		$CDF_{(AC:old)past} = 0.62 < 1$ $CDF_{(AC:old)RL} = 1- 0.62 = 0.38$ Case 2: Pavement model adopted for further analysis Layers: 1. Asph2100 - t_o mm 2. Asph1700 - 80mm 3. Gran450 - 275mm 4. TNZ-SG60 - 0mm Performance criterion: Subgrade
5	Postulate a trial overlay thickness		$t_o = 125$ mm
6	Transform overlaid pavement to an equivalent granular pavement. Transformed thickness $h_e = h + t_e$ where $t_e = t_o\ [(E_{Ac})^{1/3}/(E_{GR} \times E_{SG})^{1/6}]$		$t_e = t_o\ [(E_{Ac})^{1/3}/(E_{GR} \times E_{SG})^{1/6}]$ $= (125+80)x\ [(1900)^{1/3}/(450x60)^{1/6}]$ $= 205 \times [2.26]$ $= 464$ mm $h_e = h + t_e = 275+ 464 = 739$ mm
7	Stress dependency adjustment to unbound granular modulus (E_{GR})		$E_{GR(adj)} = E_{SG} (E_{GR} / E_{SG})^{h/he}$ $= 60\ (450/60)^{275/739}$ $= 127$ MPa ≈ 125 MPa
8	Subgrade stress dependency adjustment: Obtain F_{SG} from Table 1 $E_{SG}(adjusted) = F_{SG} \times E_{SG}(original)$	*NZ Suppl.* Appendix E:	$E_{gr} = 450$ $E_{SG} = 60$ $h_{gr} = 275$ $t_e = 464$ $\Rightarrow F_{SG} = 1.7$ $E_{SG}(adj)= 1.7 \times 60$ ≈ 100 MPa
9	Adjust moduli in the overlaid configuration. Change traffic count to future traffic (N_F). Select subgrade performance criterion.		Traffic count: (N_F) 2.4×10^6 Layers: No1: Asph2100 - 125mm No 2: Asph1700 - 80mm No 3: Gran125 - 275mm No 4: TNZ-SG100 - 0mm Perf. crit: Subgrade
10	Run CIRCLY analysis. Determine $CDF_{(SG)trial}$ and check against $CDF_{(SG)past}$. Repeat from step 5 until $CDF_{(SG)trial} \le CDF_{(SG)past}$		$CDF_{(SG)trial} = 0.026$ < 0.54 ($\therefore$ OK)
11	Change Performance criterion to Asphalt(O/L): Run CIRCLY analysis. Determine $CDF_{(AC:O/L)trial}$ and check against unity. Repeat from step 5 until $CDF_{(AC:O/L)trial}$ ≤ 1		$CDF_{(AC:O/L)trial} = 0.01$ < 1.0 ($\therefore$OK)
12	Change Performance criterion to Asphalt(old): Run CIRCLY analysis. Determine $CDF_{(AC:old)trial}$ for the trial configuration and check against $CDF_{(AC:old)RL}$. Proceed to step 13 only if $CDF_{(AC:old)trial >} CDF_{(AC:old)RL}$		$CDF_{(AC:old)trial} = 0.36$ < 0.38 ($\therefore$OK) Conclude: 125mm asphalt overlay acceptable

Example 7 : Asphalt overlay on existing asphalt (in SOUND condition) using “life after death” analysis of existing asphalt layer

Given: Existing pavement:
80mm Asphalt in SOUND condition
275mm U/B granular base: E_{GR} = 450 MPa
Subgrade: Fine-grained, cohesive: E_{SG} = 60 MPa
Past traffic estimate: $N_P = 0.5 \times 10^6$ ESA
WMAPT for the location: 15 °C

Treatment: ASPHALT overlay. Assume an asphalt stiffness with E_{AC} = 2100 MPa, and the volume of bitumen in the mix, V_B = 11%

Find: Asphalt overlay thickness for a future traffic $N_F = 3.65 \times 10^6$ ESA

Characterisation:

1. Existing asphalt stiffness:
The existing asphalt is sound. An FWD deflection test is performed when the pavement temperature is 20°C and a modulus of 1200MPa is back-calculated. From Appendix G (NZ Supp.), select the relative stiffness values for temperatures of 15°C and 20°C respectively:

$$E_{AC} = E_{BC} \times (a/b)$$
$$= 1200 \times (6.35/4.42)$$
$$\approx 1700\text{MPa}$$

2. Fatigue constants for asphalt materials:
From section 6.4.6 (*Austroads Guide*): $K = [6918(0.856V_B + 1.08)/(S_{mix})^{0.36}]$

Asphalt stiffness	Fatigue constant (K)
E_{AC} = 1700MPa	$K = [6918(0.856\text{x}11 + 1.08)/1700^{0.36}] = 4989$
E_{AC} = 2100MPa	$K = [6918(0.856\text{x}11 + 1.08)/2100^{0.36}] = 4624$

EXISTING

80mm	$E_{AC(old)}$ = 1700MPa
275mm	E_{GR} = 450Mpa (sublayered)
	E_{SG} = 60MPa

OVERLAID

t_0 = 130mm	$E_{AC(O/L)}$ = 2100MPa
80mm	$E_{AC(old)}$ = 1700MPa
275mm	$E_{GR(adj.)}$ = 125MPa (sublayered)
	$E_{SG(adj)}$ = 100MPa

Solution: The model solution appears on the next page.

NOTE: In step 6, for the purpose of estimating the equivalent granular thickness for the old and new asphalt, consider them as one layer with modulus equal to the average of their respective E-values, in this instance Eac = (2100+1700)/2 = 1900MPa.

Solution: Example 7 : Asphalt overlay on existing asphalt (in SOUND condition) using "life after death" analysis of existing asphalt layer

	PROCEDURE	Reference	EXAMPLE 7
1	Model existing pavement in CIRCLY		Traffic count: (N_P) 0.5×10^6 Layers: No.1: Asph1700 - 80mm No 2: Gran450 - 275mm No 3: TNZ-SG60 - 0mm Performance criterion: Subgrade
2	Run CIRCLY analysis. Record subgrade cumulative damage factor under past traffic ($CDF_{(SG)past}$)		$CDF_{(SG)past}$ = 0.54
3	Change performance criterion to Asphalt. Run CIRCLY analysis. Record asphalt cumulative damage factor under past traffic ($CDF_{(AC:old)past}$)		$CDF_{(AC:old))past}$ = 0.62
4	Identify condition of existing asphalt: Since $CDF_{(AC:old)past}$ <1, it is a CASE 2 situation (some life left in asphalt). Calculate remaining life cumulative damage factor, $CDF_{(AC:old)RL}$ = 1- $CDF_{(AC)past}$		$CDF_{(AC:old)past}$ = 0.62 < 1 $CDF_{(AC:old)RL}$ = 1- 0.62 = 0.38 <u>Case 2</u>: Pavement model adopted for further analysis Layers: 1. Asph2100 - t_o mm 2. Asph1700 - 80mm 3. Gran450 - 275mm 4. TNZ-SG60 - 0mm Performance criterion: Subgrade
5	Postulate a trial overlay thickness		t_o = 130 mm
6	Transform overlaid pavement to an equivalent granular pavement. Transformed thickness $h_e = h + t_e$ where $t_e = t_o\ [(E_{Ac})^{1/3}/(E_{GR} \times E_{SG})^{1/6}]$		$t_e = t_o\ [(E_{Ac})^{1/3}/(E_{GR} \times E_{SG})^{1/6}]$ $= (130+80) \times [(1900)^{1/3}/(450 \times 60)^{1/6}]$ $= 210 \times [2.26]$ = 475 mm $h_e = h + t_e = 275 + 475 = 750$ mm
7	Stress dependency adjustment to unbound granular modulus (E_{GR})		$E_{GR(adj)} = E_{SG}\ (E_{GR} / E_{SG})^{h/he}$ $= 60\ (450/60)^{275/750}$ = 126 MPa ≈ 125 Mpa
8	Subgrade stress dependency adjustment: Obtain F_{SG} from Table 1 E_{SG}(adjusted) = $F_{SG} \times E_{SG}$(original)	*NZ Suppl.* Appendix E:	E_{gr} = 450 E_{SG} = 60 h_{gr} = 275 t_e = 475 ⇒ F_{SG} = 1.7 E_{SG}(adj)= 1.7 x 60 ≈ 100 MPa
9	Adjust moduli in the overlaid configuration. Change traffic count to future traffic (N_F). Select subgrade performance criterion.		Traffic count: (N_F) 2.4×10^6 Layers: No1: Asph2100 - 130mm No 2: Asph1700 - 80mm No 3: Gran125 - 275mm No 4: TNZ-SG100 - 0mm Perf. crit: Subgrade
10	Run CIRCLY analysis. Determine $CDF_{(SG)trial}$ and check against $CDF_{(SG)past}$. Repeat from step 5 until $CDF_{(SG)trial} \leq CDF_{(SG)past}$		$CDF_{(SG)trial}$ = 0.0353 < 0.54 (∴ OK)
11	Change Performance criterion to Asphalt(O/L): Run CIRCLY analysis. Determine $CDF_{(AC:O/L)trial}$ and check against unity. Repeat from step 5 until $CDF_{(AC:O/L)trial}$ ≤1		$CDF_{(AC:O/L)trial}$ = 0.0146 < 1.0 (∴OK)

	PROCEDURE	Reference	EXAMPLE 7 continued
12	Change Performance criterion to Asphalt(old): Run CIRCLY analysis. Determine $CDF_{(AC:old)trial}$ for the trial configuration and check against $CDF_{(AC:old)RL}$. The existing asphalt requirements are met if $CDF_{(AC:old)trial} < CDF_{(AC:old)RL}$ If not met, proceed to step 13.		$CDF_{(AC:old)trial} = 0.477$ > 0.38 (step 4) ($\therefore$ not OK) Conclude: Existing asphalt life consumed part-way through future traffic. Check "life after death". Proceed to step 13.
13	Determine remaining life in existing asphalt. $N_{(AC:old)RL} = \frac{CDF_{(AC:old)RL\ (step4)}}{CDF_{(AC:old)trial\ (step\ 12)}} xN_F$		$N_{(AC:old)RL} = (0.38/0.477)x(3.65x10^6)$ $= 2.91 \times 10^6$ ESA's
14	Calculate the future remaining traffic (N_{FRT}) after end of life of existing asphalt. $N_{FRT} = N_F - N_{(AC:old)RL}$		$N_{FRT} = (3.65 – 2.91)x10^6$ $= 0.74 x10^6$ ESA's
15	Calculate the remaining life cumulative damage factor in the overlay after the end of life of existing asphalt $CDF_{(AC:O/L)RL} = 1 - \left[\frac{N_{(AC:old)RL}\ (step13)}{N_F} x\ CDF_{(AC:O/L)trial}\ (step\ 11)\right]$		$CDF_{(AC:O/L)RL} = 1- [(2.91/3.65)x\ 0.0146]$ $= 1- 0.0116$ $= 0.988$
16	Calculate the remaining life cumulative damage factor in the subgrade after the end of life of existing asphalt $CDF_{(SG)RL} = 1 - \left[\frac{N_{(AC:old)RL}\ (step13)}{N_F} x\ CDF_{(SG)trial}\ (step\ 10)\right]$		$CDF_{(SG)RL} = 1- [(2.91/3.65)x\ 0.0353]$ $= 1- 0.028$ $= 0.972$
17	Adopt a new pavement configuration for further analysis. Calculate the equivalent granular pavement thickness for the transformed pavement. $t_e = t_o\ [(E_{Ac})^{1/3}/(E_{GR} \times E_{SG})^{1/6}]$ $h_e = h + t_e$		Pavement model adopted for further analysis Layers: 1. Asph2100 - 130mm 2. Gran450 - 355mm 3. TNZ-SG60 - 0mm $t_e = t_o\ [(E_{AC})^{1/3}/(E_{GR} \times E_{SG})^{1/6}]$ $= 130x[(2100)^{1/3}/(450x60)^{1/6}]$ $= 304$ mm $h_e = h + t_e = 355+ 304 = 659$ mm
18	Stress dependency adjustment to unbound granular modulus (E_{GR})		$E_{GR(adj)} = E_{SG}\ (E_{GR} / E_{SG})^{h/he}$ $= 60\ (450/60)^{355/659}$ $= 178$ MPa ≈ 180 MPa
19	Subgrade stress dependency adjustment: Obtain F_{SG} from Table 1 $E_{SG}(adjusted) = F_{SG} \times E_{SG}(original)$	*NZ Suppl. Appendix E:*	$E_{gr} = 450$ $E_{SG} = 60$ $h_{gr} = 355$ $t_e = 304$ $\Rightarrow F_{SG} = 1.45$ $E_{SG}(adj)= 1.45 \times 60$ $= 87$ MPa ≈ 90 MPa
20	Adjust moduli in the adopted configuration. Change traffic count to future remaining traffic (N_{FRT}) – see step 14. Select subgrade performance criterion.		Traffic count: (N_{FRT}) 0.74×10^6 Layers: No1: Asph2100 - 130mm No2: Gran180 - 355mm No3: TNZ-SG90 - 0mm Perf. crit: Subgrade
21	Run CIRCLY analysis. Determine $CDF_{(SG)FRT}$ and check if $CDF_{(SG)FRT} \leq CDF_{(SG)RL}$		$CDF_{(SG)FRT} = 0.033$ < 0.972 ($\therefore$ OK)
22	Change Performance criterion to Asphalt(O/L): Run CIRCLY analysis. Determine $CDF_{(AC:O/L)RFT}$ and check if $CDF_{(AC:O/L)RFT} \leq CDF_{(AC:O/L)RL}$		$CDF_{(AC:O/L)RFT} = 0.95$ < 0.988 ($\therefore$ OK) Conclude: 130mm overlay sufficient

APPENDIX B : PAVEMENT LIFE MULTIPLIERS

(NZ SUPPLEMENT FOR APPENDIX B IN AUSTROADS GUIDE)

Pavement Life Multipliers (PLM) for a number of urban centres throughout New Zealand have been calculated using the procedures reported by Youdale (1984). The climate data is from Metgen Meteorological Consultancy (1995). PLMD and PLMN values are presented in the following table.

TABLE B1 - PAVEMENT LIFE MULTIPLIERS FOR URBAN CENTRES THROUGHOUT NEW ZEALAND

	Asphalt Thickness					
	50 mm		75 mm		100 mm	
Centre	PLM_D	PLM_N	PLM_D	PLM_N	PLM_D	PLM_N
Whangarei	0.83	0.11	1.13	0.49	1.44	1.00
Auckland	0.73	0.14	1.05	0.53	1.38	1.00
Hamilton	0.58	0.07	0.99	0.45	1.36	1.00
Tauranga	0.63	0.11	0.98	0.50	1.32	1.00
Rotorua	0.39	0.07	0.83	0.46	1.23	1.00
Taupo	0.33	0.07	0.82	0.45	1.27	1.00
Gisborne	0.69	0.08	1.07	0.47	1.43	1.00
Napier	0.60	0.09	1.02	0.48	1.40	1.00
New Plymouth	0.47	0.10	0.82	0.48	1.17	1.00
Wanganui	0.47	0.09	0.86	0.48	1.22	1.00
Palmerston North	0.41	0.07	0.83	0.46	1.21	1.00
Masterton	0.40	0.07	0.86	0.45	1.29	1.00
Wellington	0.29	0.09	0.68	0.47	1.05	1.00
Nelson	0.43	0.09	0.85	0.48	1.23	1.00
Westport	0.35	0.08	0.71	0.46	1.05	1.00
Greymouth	0.30	0.08	0.69	0.46	1.05	1.00
Kaikoura	0.25	0.08	0.68	0.46	1.05	1.00
Christchurch	0.31	0.07	0.79	0.46	1.19	1.00
Timaru	0.27	0.06	0.72	0.44	1.11	1.00
Oamaru	0.24	0.06	0.66	0.44	1.05	1.00
Queenstown	0.19	0.06	0.71	0.44	1.18	1.00
Dunedin	0.22	0.07	0.62	0.45	1.00	1.00
Invercargill	0.20	0.06	0.61	0.44	1.00	1.00

APPENDIX C : WEIGHTED MEAN ANNUAL PAVEMENT TEMPERATURES

(NZ Supplement for Appendix C in AUSTROADS Guide)

Weighted Mean Annual Pavement Temperatures (WMAPTs) for a number of urban centres throughout New Zealand have been calculated using the weighting factors recommended by Edwards and Valkering (1974) and the air temperature - pavement temperature correlation reported by Youdale (1984). The climate data is from Metgen Meteorological Consultancy (1995). WMAPTs are presented in the following table.

TABLE C1 - WMAPTs FOR URBAN CENTRES THROUGHOUT NEW ZEALAND

Centre	WMAPT (^{0}C)
Whangarei	22
Auckland	23
Hamilton	21
Tauranga	21
Rotorua	20
Taupo	19
Gisborne	21
Napier	21
New Plymouth	21
Wanganui	21
Palmerston North	21
Masterton	20
Wellington	20
Nelson	21
Westport	19
Greymouth	19
Kaikoura	19
Christchurch	19
Timaru	19
Oamaru	18
Queenstown	18
Dunedin	18
Invercargill	17

APPENDIX D : METHODS FOR CHARACTERISING INITIAL DAILY TRAFFIC

(NZ Supplement for Appendix D in AUSTROADS Guide)

Table D1 - Average ESA per axle group recorded at the five weigh-in-motion sites.

Site	Transit Region	Year	Days of Record	Mean ESA Per Axle Group Type			
				SAST	SADT	TADT	TRDT
Sulphur Beach	Auckland	1995 1996	347 189	0.4 0.3	0.2 02	0.6 0.6	0.5 0.5
Drury SB	Auckland	1994 1995 1996	175 364 144	0.5 0.4 0.5	0.2 0.2 0.2	0.4 0.4 0.4	0.5 0.4 0.4
Ohakea	Wanganui	1995 1996	361 199	0.5 0.5	0.2 0.2	0.3 0.3	0.3 0.4
Pukerua Bay	Wellington	1994 1995 1996	81 255 121	0.3 0.4 0.4	0.2 0.2 0.2	0.3 0.3 0.3	0.3 0.4 0.4
Waipara	Christchurch	1994	126	0.4	0.3	0.3	0.5

Table D2 - Average ESA per axle group with respect to Transit New Zealand region.

Transit Region	Mean ESA Per Axle Group Type			
	SAST	SADT	TADT	TRDT
Auckland	0.4	0.2	0.5	0.5
Hamilton	-	-	-	-
Napier	-	-	-	-
Wanganui	0.5	0.2	0.3	0.4
Wellington	0.4	0.2	0.3	0.4
Christchurch	0.4	0.3	0.3	0.5
Dunedin	-	-	-	-

Table D3 - Mean ESA per HCV recorded at the five weigh-in-motion sites.

Site	Transit Region	Year	Days of Record	Mean ESA Per HCV
Sulphur Beach	Auckland	1995 1996	347 189	0.8 0.8
Drury SB	Auckland	1994 1995 1996	175 364 144	1.2 1.1 1.1
Ohakea	Wanganui	1995 1996	361 199	1.0 1.1
Pukerua Bay	Wellington	1994 1995 1996	81 255 121	0.7 0.9 0.8
Waipara	Christchurch	1994	126	1.1

Table D4 - Average ESA per HCV with respect to Transit New Zealand region.

Transit Region	Mean ESA Per HCV
Auckland	1.0
Hamilton	-
Napier	-
Wanganui	1.1
Wellington	0.8
Christchurch	1.1
Dunedin	-

APPENDIX E : SUBGRADE STRESS DEPENDENCY FACTORS

Esg(exist) = 30 MPa

E(gr) MPa	h(gr) mm	50	100	150	200	250	300	350	400	450	500
		Equivalent overlay thickness, t(e) - mm									
200	200	1.4	1.8	2.2	2.6	2.9	3.1	3.3	3.4	3.5	3.6
	250	1.3	1.6	1.9	2.2	2.4	2.6	2.8	2.9	2.9	3.0
	300	1.3	1.5	1.7	1.9	2.1	2.3	2.4	2.5	2.5	2.6
	350	1.2	1.4	1.6	1.7	1.9	2.0	2.1	2.2	2.2	2.3
	400	1.2	1.3	1.5	1.6	1.7	1.8	1.9	2.0	2.0	2.1
	450	1.1	1.3	1.4	1.5	1.6	1.7	1.8	1.8	1.8	1.9
	500	1.1	1.2	1.3	1.4	1.4	1.5	1.6	1.7	1.7	1.7
	550	1.1	1.1	1.2	1.3	1.3	1.4	1.5	1.5	1.5	1.6
	600	1.1	1.1	1.2	1.2	1.3	1.3	1.4	1.4	1.4	1.5
	650	1.1	1.1	1.1	1.1	1.2	1.2	1.3	1.3	1.3	1.4
400	200	1.4	1.7	2.1	2.4	2.7	2.8	3.0	3.1	3.2	3.3
	250	1.3	1.6	1.8	2.1	2.2	2.4	2.6	2.6	2.6	2.7
	300	1.2	1.4	1.6	1.8	2.0	2.1	2.2	2.3	2.3	2.3
	350	1.2	1.4	1.5	1.6	1.7	1.8	2.0	2.0	2.0	2.1
	400	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.8	1.8	1.9
	450	1.1	1.2	1.3	1.4	1.4	1.5	1.6	1.6	1.6	1.7
	500	1.1	1.2	1.2	1.3	1.3	1.4	1.5	1.5	1.5	1.6
	550	1.1	1.1	1.2	1.2	1.2	1.3	1.4	1.4	1.4	1.4
	600	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.3	1.2	1.3
600	200	1.3	1.7	2.0	2.3	2.5	2.7	2.9	3.0	3.0	3.1
	250	1.3	1.5	1.8	2.0	2.1	2.3	2.4	2.5	2.5	2.6
	300	1.2	1.4	1.6	1.7	1.9	2.0	2.1	2.1	2.2	2.2
	350	1.2	1.3	1.5	1.6	1.7	1.8	1.9	1.9	1.9	2.0
	400	1.1	1.3	1.3	1.4	1.5	1.6	1.7	1.7	1.7	1.8
	450	1.1	1.2	1.3	1.3	1.4	1.4	1.5	1.6	1.6	1.6
	500	1.1	1.1	1.2	1.2	1.3	1.3	1.4	1.4	1.4	1.5
	550	1.1	1.1	1.1	1.1	1.2	1.2	1.3	1.3	1.3	1.4
800	200	1.3	1.7	2.0	2.3	2.5	2.6	2.8	2.9	2.9	2.9
	250	1.3	1.5	1.7	1.9	2.1	2.2	2.3	8.0	2.4	2.5
	300	1.2	1.4	1.6	1.7	1.8	1.9	2.0	2.1	2.1	2.1
	350	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.8	1.8	1.9
	400	1.1	1.2	1.3	1.4	1.5	1.5	1.6	1.6	1.6	1.7
	450	1.1	1.2	1.2	1.3	1.3	1.4	1.5	1.5	1.5	1.5
	500	1.1	1.1	1.2	1.2	1.2	1.3	1.3	1.4	1.4	1.4

Esg(exist) = 50 MPa

E(gr) MPa	h(gr) mm	50	100	150	200	250	300	350	400	450	500
		Equivalent overlay thickness, t(e) - mm									
200	200	1.3	1.5	1.7	2.0	2.1	2.2	2.4	2.5	2.5	2.5
	250	1.2	1.4	1.6	1.8	1.9	2.0	2.1	2.2	2.2	2.3
	300	1.2	1.3	1.5	1.6	1.7	1.8	1.9	2.0	2.0	2.0
	350	1.1	1.3	1.4	1.5	1.6	1.7	1.8	1.8	1.8	1.9
	400	1.1	1.2	1.3	1.4	1.5	1.5	1.6	1.7	1.7	1.7
	450	1.1	1.2	1.3	1.3	1.4	1.5	1.5	1.6	1.6	1.6
	500	1.1	1.1	1.2	1.3	1.3	1.4	1.4	1.5	1.5	1.5
	550	1.1	1.1	1.2	1.2	1.3	1.3	1.4	1.4	1.4	1.5
	600	1.1	1.1	1.1	1.2	1.2	1.3	1.3	1.4	1.4	1.4
	650	1.0	1.1	1.1	1.1	1.2	1.2	1.3	1.3	1.3	1.3
400	200	1.2	1.5	1.7	1.9	2.0	2.1	2.2	2.3	2.3	2.3
	250	1.2	1.4	1.5	1.7	1.8	1.9	2.0	2.0	2.0	2.1
	300	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.8	1.9	1.9
	350	1.1	1.2	1.3	1.4	1.5	1.6	1.6	1.7	1.7	1.7
	400	1.1	1.2	1.3	1.3	1.4	1.5	1.5	1.6	1.6	1.6
	450	1.1	1.2	1.2	1.3	1.3	1.4	1.4	1.5	1.5	1.5
	500	1.1	1.1	1.2	1.2	1.3	1.3	1.4	1.4	1.4	1.4
	550	1.1	1.1	1.1	1.2	1.2	1.2	1.3	1.3	1.3	1.3
	600	1.0	1.1	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.2
600	200	1.2	1.4	1.6	1.8	1.9	2.0	2.1	2.2	2.2	2.2
	250	1.2	1.4	1.5	1.6	1.7	1.8	1.9	1.9	2.0	2.0
	300	1.1	1.3	1.4	1.5	1.6	1.6	1.7	1.8	1.8	1.8
	350	1.1	1.2	1.3	1.4	1.5	1.5	1.6	1.6	1.6	1.6
	400	1.1	1.2	1.3	1.3	1.4	1.4	1.5	1.5	1.5	1.5
	450	1.1	1.1	1.2	1.2	1.3	1.3	1.4	1.4	1.4	1.4
	500	1.1	1.1	1.2	1.2	1.2	1.3	1.3	1.3	1.3	1.3
	550	1.0	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2
800	200	1.2	1.4	1.6	1.8	1.9	2.0	2.1	2.1	2.1	2.2
	250	1.2	1.3	1.5	1.6	1.7	1.8	1.9	1.9	1.9	1.9
	300	1.1	1.3	1.4	1.5	1.5	1.6	1.7	1.7	1.7	1.7
	350	1.1	1.2	1.3	1.4	1.4	1.5	1.5	1.6	1.6	1.6
	400	1.1	1.2	1.2	1.3	1.3	1.4	1.4	1.5	1.5	1.5
	450	1.1	1.1	1.2	1.2	1.3	1.3	1.3	1.4	1.4	1.4
	500	1.1	1.1	1.1	1.2	1.2	1.2	1.3	1.3	1.3	1.3

Esg(exist) = 70 MPa

E(gr) MPa	h(gr) mm	50	100	150	200	250	300	350	400	450	500
		Equivalent overlay thickness, t(e) - mm									
200	200	1.1	1.2	1.3	1.4	1.5	1.5	1.6	1.6	1.6	1.6
	250	1.1	1.2	1.3	1.3	1.4	1.4	1.5	1.5	1.5	1.5
	300	1.1	1.2	1.2	1.3	1.3	1.4	1.4	1.4	1.4	1.5
	350	1.1	1.1	1.2	1.2	1.3	1.3	1.3	1.4	1.4	1.4
	400	1.1	1.1	1.2	1.2	1.2	1.3	1.3	1.3	1.3	1.3
	450	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.3	1.3	1.3
	500	1.0	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2
	550	1.0	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2	1.2
	600	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2
	650	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.2	1.2
400	200	1.1	1.2	1.3	1.4	1.4	1.5	1.5	1.5	1.6	1.6
	250	1.1	1.2	1.2	1.3	1.4	1.4	1.4	1.4	1.5	1.5
	300	1.1	1.1	1.2	1.2	1.3	1.3	1.3	1.4	1.4	1.4
	350	1.1	1.1	1.2	1.2	1.2	1.3	1.3	1.3	1.3	1.3
	400	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.3	1.3	1.3
	450	1.0	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2
	500	1.0	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2	1.2
	550	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2
	600	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1
600	200	1.1	1.2	1.3	1.4	1.4	1.4	1.5	1.5	1.5	1.5
	250	1.1	1.2	1.2	1.3	1.3	1.4	1.4	1.4	1.4	1.4
	300	1.1	1.1	1.2	1.2	1.3	1.3	1.3	1.3	1.4	1.4
	350	1.1	1.1	1.2	1.2	1.2	1.2	1.3	1.3	1.3	1.3
	400	1.0	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2
	450	1.0	1.1	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.2
	500	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2
	550	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
800	200	1.1	1.2	1.3	1.3	1.4	1.4	1.5	1.5	1.5	1.5
	250	1.1	1.2	1.2	1.3	1.3	1.3	1.4	1.4	1.4	1.4
	300	1.1	1.1	1.2	1.2	1.3	1.3	1.3	1.3	1.3	1.3
	350	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.3	1.3	1.3
	400	1.0	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2
	450	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2
	500	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1

APPENDIX F : STIFFNESS VALUES FOR CRACKED ASPHALT AS A FUNCTION OF TEMPERATURE AND LOADING SPEED

Asphalt Temperature (°C)	Asphalt stiffness (MPa) 5 km/h	80 km/h
12.5	1850	2200
15	1575	1850
17.5	1300	1500
20	1075	1325
22.5	850	1150
25	800	1035
27.5	750	920
30	700	835
32.5	650	755
35	600	670
37.5 +	550	585

APPENDIX G : RELATIVE STIFFNESS VALUES FOR SOUND ASPHALT AS A FUNCTION OF TEMPERATURE AND LOADING SPEED

Asphalt Temperature (°C)	Relative asphalt stiffness 5 km/h (Benkelman Beam)	80 km/h (FWD, highway traffic)
12.5	3.25	7.40
15	2.55	6.35
17.5	1.85	5.30
20	1.55	4.42
22.5	1.25	3.55
25	1.20	2.89
27.5	1.15	2.23
30	1.10	1.85
32.5	1.05	1.48
35	1.02	1.31
37.5 +	1	1.15