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1. **FOREWORD**


The guide provides pavement designers with information on evaluating a pavement and designing a suitable pavement rehabilitation treatment.

This Guide should be read in conjunction with the Guide to Pavement Technology Part 5: Pavement Evaluation and Treatment Design (Austroads 2011) and specifically addresses pavement issues which are unique to New Zealand conditions. It contains guidance on how New Zealand research can be incorporated into pavement design methods and reemphasises the precedent strain design methodology.

By applying the New Zealand Guide to Pavement Evaluation and Treatment Design together with Austroad’s Guide to Pavement Technology Part 5, we take full advantage of the knowledge and experience of the roading community in both New Zealand and Australia.

Tommy Parker

General Manager – System Design and Delivery
### Abbreviations

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

1.1 GENERAL

The 2017 New Zealand Guide to Pavement Evaluation and Treatment Design provides guidance for investigating, testing, designing and constructing pavement rehabilitation treatments in New Zealand.


It is envisaged subsequent editions will provide an avenue to implement outcomes of research projects in New Zealand where they benefit the pavement performance.

The Guide assumes a need to treat a site has been already identified through network inspections and forward works programming. However, it could be found after investigations are completed and interpreted that structural rehabilitation is not necessary as alternative treatments offer lower whole of life costs. The steps below outline the process needed to evaluate the most appropriate treatment and associated design assumptions:

1. Preliminary Visual and Desktop Investigation – Undertaking the initial visual inspection of pavement defects; considering historical information about pavement type, maintenance, high speed data (such as roughness, rutting and texture), performance, and past traffic; and then assembling information on the existing pavement. The road topography and geometry should be considered, particularly in regard to drainage. Site constraints such as those imposed by kerb and channel as well as underground utilities should also be considered. These data should be examined to identify any deficiencies or missing information;

2. Testing Pavement and in situ Materials – Testing the in situ pavement, including (but not limited to): deflection testing, test pits, Scala penetrometer and laboratory testing any extracted pavement materials;

3. Problem Definition – Conducting a root cause analysis by reviewing the results of the investigation to determine the failure modes and cause or causes of the pavement failure;

4. Preliminary Treatment Design Options – Reviewing the failure modes and cause or causes to identify appropriate rehabilitation treatments. Associated design assumptions should also be identified. A key assessment is the ability of in situ materials to resist the accumulation of plastic deformation. The adequacy of pavement depths and subgrade strength should also be established. Some initial treatment designs should be developed for comparison. These options could range from a simple patch repair and reseal, a full width stabilisation, or an overlay;

5. Comparison and Treatment Selection – Comparing treatment options to determine which is the most appropriate based on: engineering (design life expected); appetite for risk; cost and budget. Select three treatments for the next stage (steps 6 and 7 below);

6. Laboratory Mix Design for Chosen Treatment – If the recommended treatment requires materials to be modified or stabilised, consider the levels of risk for a particular pavement; undertake full laboratory mix designs and refine the design-based values inferred from the laboratory test results;

7. Finalise the Design – Material parameters used in the modelling are based on the derived laboratory mix design results. However, consideration should be given to the ability to construct the pavement to achieve the assumed properties. In particular, is there sufficient support and can densities at which the materials were tested be achieved in the field. The design should also be examined to ensure that it is constructible
and the resultant treatment does not create any additional problems. Safety in design requires the designer to consider risks associated the construction, operation, maintenance and rehabilitation of the pavement and to eliminate, isolate, minimise, inform or control these risks. As a consequence, some potential treatments might be rejected for safety reasons;

8. **Quality Assurance Testing** – Develop an inspection and testing plan to ensure design assumptions and relevant specification requirements are met during the construction process;

9. **Monitoring** – Documenting pavement design assumptions for monitoring purposes.

A design report is required to support the chosen rehabilitation treatment and design method. It should set out the evaluation process used to decide the recommended treatment using the methodology detailed above. The report should also detail how safety in design has been incorporated into the design process.

This Guide is structured to mimic rehabilitation design process. It begins with identifying the pavement distress, deciding on a testing plan, establishing design assumptions, and establishing the risk of each proposed pavement treatment.

The Guide is to be read in conjunction with the Austroads *Guide to Pavement Technology – Part 5: Pavement Evaluation and Treatment Design* (2011). In particular, the guidance on asphalt overlays thicker than 40 millimetres provided by Austroads Part 5 should be followed.

The design information in Austroads *Guide to Pavement Technology – Part 2: Pavement Structural Design* (2017) should also be considered in the development and characterisations of pavement designs.

Where the recommendations of the Austroads documents contradict this Guide then this document will take precedent.
2 Pavement evaluation and treatment design

2.1 EVALUATION AND TREATMENT DESIGN PROCESS

The processes involved in pavement evaluation and treatment design are shown in Figure 1 on the following page. The intended meanings of the actions are detailed in the following sections. The risk based treatment design referred to in Figure 1 should consider the details discussed in the following sections as well as the risks detailed in Table 4 below.
2.1.1 Identify site

To be a candidate for rehabilitation a site has to come to the attention of the pavement engineer. This will most probably be through increased maintenance costs and/or faults developing on the site.
2.1.2 Sites verified with Principal

The Principal should be consulted to ensure they agree the site requires rehabilitation. This should involve a site visit with the visual inspection being supplemented with the site history information.

2.1.3 Establish site history and context

What is known about the site, what is recorded in RAMM, what is known from the High Speed Data (HSD), Falling Weight Deflectometer (FWD), or the Traffic Speed Deflectometer (TSD); this is discussed in more detail in Chapter 3. People involved in the maintenance of the site should be interviewed to assess any local knowledge available.

2.1.4 Identify faults

How the observed faults are contributing to failure of the pavement should be assessed. This is discussed in more detail in Chapter 3. Moisture in the pavement is often a significant factor that causes pavement failure. The cause of any moisture ingress should be identified and any rehabilitation treatment should eliminate that source of moisture.

2.1.5 Identify causes of failure

Given the faults observed the cause of failure should be identified, this is discussed in more detail in Chapter 3

2.1.6 Confirm failure modes with testing

Various field and laboratory tests are available to help identify the causes of failure in a pavement. These are discussed in more detail in Chapter 4

2.1.7 Root cause analysis

This step integrates and interprets the various failure mechanisms, the existing pavement structure, and the traffic history to determine the root cause or causes of the pavement failure. This is detailed further in Chapter 5.

2.1.8 Risk based treatment selection

Preliminary treatment options should be selected to eliminate the root causes of the pavement failure.

In combination with the existing pavement structure, the current and future forecast traffic will somewhat dictate the pavement design options. Designs with unreasonable levels of risk should be eliminated from the list of possible treatments; however, a number of options should be examined to ensure a number of risk/benefit selections are considered.

Given the site characteristics, some treatment options can be eliminated on the basis of the traffic criticality. This is discussed in Chapter 6

Construction risks should also be considered. These are discussed in Section 11.3.

This Guide assumes a site has been identified as a candidate requiring rehabilitation. It is possible, however, that investigation will show structural rehabilitation is not necessary as other treatments offer lower whole of life costs.
2.1.9 Pavement and material testing

The root cause analysis may identify material deficiencies that have contributed to the pavement failure. Additional on site and laboratory tests can confirm the suitability/unsuitability of the material. These tests can be done in parallel with the mix design.

2.1.10 Mix design

If the existing in situ materials have deficiencies then these deficiencies may be treated with chemical or granular stabilisation. Where stabilisation is being considered, the expected improvements should be determined through laboratory testing. In this context, stabilisation is used in its pure sense where it references chemical or physical stabilisation, through addressing grading deficiencies of the material regardless of the level of stabilisation. Therefore, stabilisation can refer to granular stabilisation, chemical modification or the production of a heavily chemically bound material.

Guidance on granular (mechanical) stabilisation can be obtained from Austroads (2011). Generally it is expected that chemical modification will be achieved through the addition of cement, lime, or bitumen although other chemical modifiers are available and can achieve good success.

2.1.11 Drainage improvements

Drainage improvements will most likely be necessary as moisture ingress is often a cause of pavement deterioration. No pavement rehabilitation treatments should be undertaken without concurrent drainage improvements being considered.

2.1.12 Pavement design

Having identified a number of treatment selection options, they should be refined by the pavement designer through formal design using empirical or mechanistic design techniques where appropriate. Mechanistic design should be informed with mix design test results, where material modification is a possible treatment option, and through the laboratory material test results.

All the investigations and analysis should be compiled into a detailed design report. This should clearly identify: the faults that caused the failure; the results of the root cause analysis; the treatments considered; and the justification for the recommended treatment. An example of the expected content of a detailed design report is provided in Chapter 10.

The root cause analysis will have probably have identified moisture ingress as a significant cause of failure. Drainage improvements should always be considered as part of the pavement design; this can include subsoil replacement, pavement layer drains, and surface drainage improvements.

The pavement design purpose is to develop a pavement structure that: has a high probability of achieving an effective life equal to or greater than the design life; will, over a suitable analysis period, minimise maintenance costs and traffic delays. This is discussed in Section 2.3.

An integral component of the pavement design is the surfacing design. The surfacing design should minimise the ingress of water of moisture into the pavement. This will mean that a membrane seal will be necessary under asphalt surfacing less than 80 mm in thickness. A prime coat and a chipseal will be necessary under Open Graded Porous Asphalt (OGPA). A general rule of thumb is that three litres per square metre is required to achieve
waterproofing underneath an OGPA. First coat chipseals will require a second coat chipseal the following sealing season. Neither the first coat nor the second coat should be constructed outside of the sealing season.

All pavement rehabilitation designs shall be reviewed by a Chartered Professional Engineer (CPEng) or a person acceptable to the Principal.

2.1.13 Pavement construction

The pavement construction process should meet all the material design assumptions. This will include all of the relevant construction specifications being met. That the specifications have been met should be clearly documented with records kept of all the testing performed. An indicative pavement construction quality assurance process is discussed in Chapter 11.

2.1.14 Construction review and audit

The construction records are examined to ensure the pavement design has been constructed, the construction quality records are complete, and show the design assumptions have been met.

2.1.15 Post construction measures

The ongoing performance of the pavement should be verified through post construction measures. These could include monitoring rutting and roughness progression as well as other measures such as maintenance treatments.

2.2 BEST PRACTICE

The intent of this Guide is to document the best practice as known at the time of publication. The NZ Transport Agency is a member of Austroads and the current Austroads guides to pavement technology are endorsed by the NZ Transport Agency. However, inclusion in either the Austroads guides or New Zealand guides and specifications is not justification for disregarding new research that improves best practice.

It is the NZ Transport Agency’s desire to, where possible, continually improve on best practice. This improvement will require evidence based designs being performed with clearly documented design assumptions that are supported by published research. The NZ Transport Agency Pavement Group will need to be consulted on any proposed practice that is in variance to Austroads or the NZ Transport Agency’s Guides.

While not directly covered in this Guide, resource efficiency and environmental effects should be considered when constructing pavement rehabilitations. Some advice is given on the Highway Information Portal of the NZ Transport Agency’s website under the Technical Disciplines section.

2.3 CONSTRUCTION BASED RISK DESIGN

Pavement design is a risk-based process. High volume roads have associated higher levels of risk. In this context risk is defined as being equal to the probability of failure multiplied by the consequence of that failure as shown in equation 2.1.

\[ \text{Risk} = \text{Probability} \times \text{Consequence} \]
2.3.1 Probability

The probability of failure will depend primarily on suitability of the pavement design for the traffic environment, the ground conditions and the performance of the construction materials. A pavement design should have a probability equal to the project reliability of achieving its design life. The project reliability values are detailed in Table 1 below where the project reliability should be equal or better than the value stated for the various road classifications.

Austroads (2017) defines the project reliability is the probability that the pavement, when constructed to the chosen design, will outlast its design traffic before major rehabilitation is required. In regard to these reliability procedures, a project is defined as a portion from a uniformly designed and (nominally) uniformly constructed road pavement which is subsequently rehabilitated as an entity.

<table>
<thead>
<tr>
<th>Road Classification</th>
<th>Project Reliability (%)</th>
</tr>
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<tbody>
<tr>
<td>National (high volume)</td>
<td>97.5</td>
</tr>
<tr>
<td>National</td>
<td>95</td>
</tr>
<tr>
<td>Regional or Arterial</td>
<td>90</td>
</tr>
<tr>
<td>Primary Collector or Secondary Collector</td>
<td>90</td>
</tr>
<tr>
<td>Access and Access (low volume)</td>
<td>80</td>
</tr>
</tbody>
</table>

In some cases various pavement design solutions may satisfy the design requirements but may have different reliability factors. Some guidance in ranking the reliability of various treatments is detailed in Section 2.3.3.

2.3.2 Consequence

The consequence of failure is higher on high volume roads since:

- Treatment of a failed high volume road will require more materials, and
- Traffic management requirements and customer disruptions are greater on the high volume road.

2.3.3 Risk

A fault or a collection of faults on a low volume road are therefore less risk than the same fault or faults on a higher volume road.

For a particular rehabilitation the consequence of premature failure cannot be easily changed as the traffic levels are fixed. Risk can therefore only be managed by reducing the probability of failure. This is achieved by accurate characterisation of material properties, adopting lower risk pavement designs and focusing attention on the quality of the construction process. At the same time, however, the One Network Road Classification needs to be considered to ensure the adopted level of risk is appropriate. For example, using marginal materials with higher shearing potential may be appropriate on lower classification roads. The intent of this Guide is to assist designers to appropriately manage the risk of premature pavement rehabilitation failures in New Zealand and, in particular, minimise the risk of failure through informed design.
Assuming appropriate material characterisation and construction methodology, the relative performance risks of particular pavement rehabilitation techniques are detailed in Table 2 below.

Table 2  Risk of pavement rehabilitation techniques against traffic volume

<table>
<thead>
<tr>
<th>25 year design traffic volume (ESAs)</th>
<th>Less than $5 \times 10^6$</th>
<th>Between $5 \times 10^6$ and $1 \times 10^7$</th>
<th>Between $1 \times 10^7$ to $5 \times 10^7$</th>
<th>Greater than $5 \times 10^7$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Continuously Reinforced Concrete Pavement</td>
<td>Unlikely to be economic</td>
<td>Unlikely to be economic</td>
<td>Unlikely to be economic</td>
<td>Low risk</td>
</tr>
<tr>
<td>Structural Asphalt</td>
<td>Unlikely to be economic</td>
<td>Unlikely to be economic</td>
<td>Low risk</td>
<td>Low risk</td>
</tr>
<tr>
<td>Modified aggregate overlay basecourse and bound subbase</td>
<td>Unlikely to be economic</td>
<td>Low risk</td>
<td>Low risk</td>
<td>Medium risk</td>
</tr>
<tr>
<td>Foamed bitumen basecourse</td>
<td>Low risk</td>
<td>Low risk</td>
<td>Low risk</td>
<td>Medium risk</td>
</tr>
<tr>
<td>Modified aggregate base only</td>
<td>Low risk</td>
<td>Low risk</td>
<td>Medium risk</td>
<td>High risk</td>
</tr>
<tr>
<td>Unbound aggregate overlay</td>
<td>Low risk</td>
<td>Medium risk</td>
<td>High risk</td>
<td>High risk</td>
</tr>
</tbody>
</table>
3 Preliminary visual and desktop investigation

3.1 GENERAL

Pavement inspection and testing shall generally be as documented in the Austroads Guide to Pavement Technology Part 5: Pavement Evaluation and Treatment Design (Austroads 2011). However, for rehabilitation investigations in New Zealand the following provides guidelines on conducting both field and laboratory pavement investigation. These notes should be read in conjunction with the information given in Sections 5 and 6 of the Austroads Guide to Pavement Technology Part 2: Pavement Structural Design (Austroads 2017).

Broadly speaking, pavement treatment projects fall into two categories. In the first, a new or existing pavement is being constructed to accommodate an anticipated change in traffic usage, and is not normally associated with seeking solutions to existing pavement distress. In a project such as this, the investigative process needs to discover the likely impact the change in traffic will have on the existing foundation and pavement structures.

The second category of project work involves rehabilitating a pavement that is showing signs of distress. In this case, the investigative process will need to help the designer understand what has caused the observed distress, and then support the review of options to strengthen the pavement to address this distress and any future change in usage. In broad terms, it is based on an assessment of the actual pavement condition with respect to the required condition.

The nature and extent of the investigative process (inspection, investigation and testing) for a pavement project will be influenced by the purpose of the proposed works, the design complexity, and the level of risk in the project. The pavement designer must carefully consider such issues when planning the inspection, investigation and testing works required for a pavement project.

It is important to visually inspect the failed pavement and to review any historical information available from the RAMM database. This allows the designer to get an understanding of the reasons for failure such that the most appropriate investigation and testing plan can be developed along with appropriate rehabilitation treatment options and associated design assumptions. The design report should show how the chosen rehabilitation treatment targets the recognised problems causing the pavement distress.

3.2 HISTORICAL DATA

Where available and accurate the RAMM database and local knowledge should be used to obtain information on at least the following information:

- Pavement life in years achieved, by
  - adding the number of years from today’s date
  - plus the most recent first coat seal
  - less the number of years heavy maintenance was applied

NB: These maintenance records should be in RAMM;
• Past traffic during the pavement life, in terms of number of Equivalent Standard Axles (ESAs) – see Section 7 of Part 2 of Austroads (2017);
• Current mix of heavy traffic, to see if there is a loaded and an unloaded direction for calculating ESAs in each direction;
• Pavement cross-section information (thickness of surfacing and aggregate layers), although test pits are required to confirm the pavement material layer thicknesses;
• Past rehabilitation treatment types (e.g. granular overlay, cement stabilisation or asphalt overlay);
• Surfacing history.

The past performance of pavement treatments also needs to be assessed. If they have performed poorly, alternative treatments should be considered.

### 3.3 VISUAL CONDITION DATA

A visual inspection is very important and a mapping of visual distress is required. An example method is detailed in Figure 3.1 in the Austroads Guide to Pavement Technology – Part 5 Pavement Evaluation and Treatment Design (Austroads 2011). The design report shall use the visual inspection information to support the chosen design, design assumptions and treatment selection. Therefore, the pavement designer is expected to visit the site to understand the nature and extent of the distress.

Contextual information is important and not generally indicated in data per se. For example, a shaded road in a deep cutting may be contributing to pavement failure. General and close-up photos as well as a written description of the site should be recorded.

### 3.4 INITIAL ASSESSMENT OF TESTING, INVESTIGATION AND BEST TREATMENT

After the rehabilitation sites have been agreed, the pavement faults and their causes must be identified, the pavement structure analysed and its history understood. This information will guide the appropriate level of investigation and the correct treatment. Appendix A in Austroads (2011) contains information on identifying visual distresses in pavements and its causes and treatments. Chapter 5 of this Guide also gives some help with a root cause analysis of the pavement failure.

### 3.5 PAVEMENT RISK ASSESSMENT

Pavement risk is the probability of the pavement failure multiplied by the consequence of that failure. Table 4 below details the various risks associated with various overlay and stabilisation treatment options. It should be noted these risks are in the context of particular traffic volumes and are compared to specific treatments scaled to the traffic volumes. The relative costs of various treatments in a specific contract may thus change the risk of the selected treatment. The risks presented are therefore indicative and should be discussed and agreed within a specific contract. Having agreed to the relative risks, various treatments can be evaluated in terms of cost and risk.

Table 4 below provides a guide on the levels of risk associated with pavement treatments arising from various causes of distress. In all cases the risk of the failure between the longitudinal and transverse interface of the treated and the adjacent untreated pavement should be considered and the interface should be designed and constructed to minimise this risk.
There are three types of pavement distress related to aggregate rutting; *in situ* base aggregate has poor rut resistance when dry, *in situ* base aggregate has good rut resistance when dry, *in situ* base aggregate has poor rut resistance when wet, these are denoted as aggregate rutting A, B, and C in Table 4 respectively.

Indicative values of good, medium and poor performance, as inferred from the Repeated Load Triaxial (RLT), are indicated in Table 3

<table>
<thead>
<tr>
<th>Table 3 Indicative aggregate performance based on the RLT slopes</th>
</tr>
</thead>
<tbody>
<tr>
<td>NZTA T15 RLT slope first five stages (%/1M)</td>
</tr>
<tr>
<td>Poor</td>
</tr>
<tr>
<td>Medium</td>
</tr>
<tr>
<td>Good</td>
</tr>
</tbody>
</table>
Table 4 Guideline for Assessing Pavement Treatment Options

<table>
<thead>
<tr>
<th>Treatment</th>
<th>Subgrade Rutting</th>
<th>Aggregate Rutting A (poor when dry)(^1)</th>
<th>Aggregate Rutting B (good when dry)(^1)</th>
<th>Aggregate Rutting C (poor when wet)</th>
<th>Construction Risk</th>
<th>Maintenance risk</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unbound Granular Overlay (Design traffic less than 5 MESA)</td>
<td>Low Risk (unless experience in area with poor performing overlays) increase pavement depth designed specifically to protect subgrade</td>
<td>High Risk</td>
<td>Low Risk (provided drainage is improved and surface is waterproof).</td>
<td>Medium Risk</td>
<td>Risk of trapping water if existing seal is not broken up</td>
<td></td>
</tr>
<tr>
<td>In situ Stabilisation (Modified) – no overlay (Low Binder content to generate a UCS &lt; 1 MPa) (Design traffic 15 MESA)</td>
<td>High Risk (still effectively behaves as unbound with no increase in pavement depth)</td>
<td>High Risk (light modification only tidies up fines and improves wet rut resistance, dry rut resistance relies on stone on stone contact which is poor)</td>
<td>Low Risk (light modification only tidies up fines and improves wet rut resistance, dry rut resistance relies on stone on stone contact which is good)</td>
<td>High Risk (still effectively behaves as unbound with no increase in pavement depth to protect the potentially weak subbase)</td>
<td>Risk of shrinkage and block cracking depending on interaction between chemical modifier used, aggregate(s), surfacing included and water content added during process.</td>
<td></td>
</tr>
<tr>
<td>Treatment</td>
<td>Subgrade Rutting</td>
<td>Aggregate Rutting A (poor when dry)&lt;sup&gt;1&lt;/sup&gt;</td>
<td>Aggregate Rutting B (good when dry)&lt;sup&gt;1&lt;/sup&gt;</td>
<td>Aggregate Rutting C (poor when wet)</td>
<td>Construction Risk</td>
<td>Maintenance risk</td>
</tr>
<tr>
<td>-----------------------------------------------</td>
<td>------------------</td>
<td>-----------------------------------------------</td>
<td>-----------------------------------------------</td>
<td>-----------------------------------</td>
<td>--------------------</td>
<td>------------------</td>
</tr>
<tr>
<td>In situ Stabilisation (Modified) – with inclusion of an overlay hoed in (Low Binder content generate a UCS &lt; 1 MPa) (Design traffic 15 MESA) – (note that the overlay material may change the aggregate rutting to good performance)</td>
<td>Low Risk (increase pavement depth designed specifically to protect subgrade). Stabilised overlay reduces risk further compared with an unbound overlay.</td>
<td>High Risk (light modification only tidies up fines and improves wet rut resistance, dry rut resistance relies on stone on stone contact which is poor). This assumes the mixture of overlay and existing aggregate has been assessed as still showing poor performance.</td>
<td>Low Risk (light modification only tidies up fines and improves wet rut resistance, dry rut resistance relies on stone on stone contact which is good)</td>
<td>Medium Risk (the increase in pavement depth will give added protection for the weak subbase). Ideally overlay needs to be designed to protect the subbase.</td>
<td>Risk of shrinkage and block cracking depending on interaction between chemical modifier used, aggregate(s), surfacing included and water content added during process.</td>
<td></td>
</tr>
<tr>
<td>Treatment</td>
<td>Subgrade Rutting</td>
<td>Aggregate Rutting A (poor when dry)(^1)</td>
<td>Aggregate Rutting B (good when dry)(^1)</td>
<td>Aggregate Rutting C (poor when wet)</td>
<td>Construction Risk</td>
<td>Maintenance Risk</td>
</tr>
<tr>
<td>-----------</td>
<td>-----------------</td>
<td>------------------------------------------</td>
<td>------------------------------------------</td>
<td>-----------------------------------</td>
<td>-------------------</td>
<td>------------------</td>
</tr>
<tr>
<td><strong>In situ Stabilisation</strong> (bound) – with or without inclusion of an overlay hoed in as needed to meet the bound design rules (stress&lt;50% beam strength) (High Binder content to produce a bound material) (Design traffic 30 MESA)</td>
<td>Low Risk (provided stabilised bound layer meets the design assumptions when constructed as the bound stabilised layer bridges the weak subgrade). Risk is lowered further if the cover to the subgrade is the same as needed for an unbound granular pavement (Austroads, Figure 8.4)</td>
<td>Low Risk (provided stabilised bound layer meets the design assumptions when constructed as the bound stabilised layer should remain bound over its design life). As the aggregate is poor quality there will be a risk of shrinkage cracking.</td>
<td>Low Risk (provided stabilised bound layer meets the design assumptions when constructed)</td>
<td>Low Risk (provided stabilised bound layer meets the design assumptions when constructed as the bound stabilised layer bridges the weak subbase).</td>
<td>High risk of shrinkage and block cracking depending on interaction between chemical modifiers and aggregates. For this reason the use of cement bound basecourse pavements require approval from the NZ Transport Agency Pavements Team</td>
<td></td>
</tr>
</tbody>
</table>
## Cause of Pavement Distress

<table>
<thead>
<tr>
<th>Treatment</th>
<th>Subgrade Rutting</th>
<th>Aggregate Rutting A (poor when dry)¹</th>
<th>Aggregate Rutting B (good when dry)¹</th>
<th>Aggregate Rutting C (poor when wet)</th>
<th>Construction Risk</th>
<th>Maintenance risk</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mill out basecourse thickness (for final pavement treatment) and in situ</td>
<td>Low Risk (provided stabilised subbase bound layer meets the design assumptions when constructed as the bound stabilised subbase layer bridges the weak subgrade). Risk is lowered further if the cover to the subgrade is the same as needed for an unbound granular pavement (Austroads, Figure 8.4)</td>
<td>Not applicable – as basecourse is new imported high quality crushed rock.</td>
<td>Low Risk</td>
<td>Low Risk (provided stabilised subbase bound layer meets the design assumptions when constructed as the bound stabilised subbase layer fixes the weak subbase).</td>
<td>Risk of contamination of subbase from underlying subgrade</td>
<td>Risk of shrinkage and block cracking depending on interaction between chemical modifier used, aggregate(s), surfacing included and water content added during process.</td>
</tr>
<tr>
<td>stabilisation of the underlying subbase (strongly bound material) and then fill in with new imported good quality crushed rock basecourse (e.g. NZTA M4 AP40) and if needed lightly stabilise basecourse (a UCS &lt; 1 MPa) Design traffic 30 MESA</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1. **In situ** base aggregate is the required basecourse thickness of unmodified/unbound material required for a new rehabilitated pavement set out in Austroads (2017) from Figure 8.4. It could be either the **in situ** existing aggregate; a mixture of seal and existing aggregate (plus new overlay aggregate if used to either change the aggregate from poor to good); and/or extra cover to protect the subgrade or subbase.

2. It is assumed the treatments meet their design assumptions when constructed.
3. High Risk can be interpreted that, for the given traffic levels, the treatment is likely to achieve project reliability of the road classification two down from the design classification. For example, from Table 1, reliability will decrease from 97.5% to 90% for a National (high volume) road.

4. Medium Risk can be interpreted as, for the given traffic levels, the treatment is likely to achieve the project reliability of the next lower road classification design life.

5. Low Risk can be interpreted as, for the given traffic levels, the treatment is likely to achieve the project reliability for the design life.
4 Testing of pavement and *in situ* materials

4.1 INTRODUCTION

Investigation and testing the *in situ* pavement involves obtaining information to help decide the most appropriate rehabilitation treatment and pavement design assumptions. The main aims of the investigation and testing are to determine the quality of the *in situ* materials, the pavement depth and the characteristics of the subgrade material.

4.2 INVESTIGATIONS

Generally there are three levels of physical investigation. These are determined by the level of risk associated with a particular site. The level of risk has been associated with the One Network Road Classification and is detailed in Table 5:

**Table 5 Investigation level required for various One Network Road Classifications**

<table>
<thead>
<tr>
<th>One Network Road Classification</th>
<th>Investigation Level 1</th>
<th>Investigation Level 2</th>
<th>Investigation Level 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>National, National (high volume), Regional, or Arterial</td>
<td>Preliminary investigation only</td>
<td>Preliminary investigation only</td>
<td>Minimum requirements</td>
</tr>
<tr>
<td>Primary Collector or Secondary Collector</td>
<td>Preliminary investigation only</td>
<td>Minimum requirements</td>
<td>Possible as supplementary</td>
</tr>
<tr>
<td>Access and Access (low volume)</td>
<td>Minimum requirements</td>
<td>Possible as supplementary</td>
<td>Possible as supplementary</td>
</tr>
</tbody>
</table>

- Level 1 involves low or minimal investigation. The distress and reasons for this can be assessed visually along with RAMM data and local knowledge to obtain the information needed for design.
- Level 2 requires an average level of investigation; at least one test pit is undertaken to determine pavement depths and material quality.
• Level 3 investigation is a high level of investigation; there is significant distress and high traffic and/or recently constructed pavements have failed early, so more investigation is required to develop pavement solutions that will not fail early.

Typically investigations will be conducted in three stages: general inspection, detailed inspection, and field and laboratory testing. These stages may overlap or be entirely concurrent. The minimum field and laboratory investigation requirements are detailed more fully in Section 4.2.3.

4.2.1 General inspection by designer

The purpose of the walkover inspection by the designer is to view and record the condition of the existing pavement. This involves noting issues of potential consequence to the design such as topography, adjoining drainage conditions, vegetation etc. Records of the past performance of the pavement should be taken into the field, and used to correlate condition and distress types with location on the road. The visual inspection should note the type, severity and extent of any distress.

Any significant areas of distress in the existing pavement observed during the walkover should also be targeted in the subsequent investigations. This will highlight whether there are any special reasons for the observed failures which do not occur over the whole site. The results of the walkover inspection should be recorded in a systematic way and plotted on a plan of the site against an acknowledged reference system to support future investigations.

4.2.2 Detailed site inspection by technical staff

The purpose of the detailed inspection is to record and measure the condition of the existing pavement, based on the instructions prepared by the designer. On small projects this detailed inspection can probably be undertaken by the designer and incorporated in the walkover inspection. A crack and defect mapping sheet shall be completed whilst on site; an example of a mapping sheet is displayed in Figure 3.1 of Austroads (2011). Records of the past performance of the pavement should be taken into the field, and used to correlate condition and distress types with location on the road. The detailed site inspection should focus on the following performance indicators and influences:

• Pavement shape e.g. depressions, rutting, and/or settlement: The observed condition of the existing pavement can be supplemented with information from high speed data surveys. Alternatively a straight edge can be used to determine the size and extent of any rutting.
• Surface defects e.g. patches, edge break, cracking, shoving: The on-site inspection is the principal means of determining the primary distress mode or the combination of distress modes of the existing pavement. This can influence the pavement rehabilitation design options.
• Drainage conditions: The detailed inspection will often identify moisture-related distress in the existing pavement, the cause of which needs to be determined if any rehabilitation treatment or reconstruction work is to achieve its intended purpose. This information will assist the designer to consider whether drainage issues need to be addressed as part of the design. The inspector should check to: ensure there is adequate drainage in the existing road pavement design; how drainage maintenance, or possibly lack of maintenance has contributed to the observed defects; or whether there have been any apparent changes in the local groundwater or surface water regimes.
• Geometrics: e.g. curves, crossfall, super-elevation. Road geometry should be visually checked to establish whether the road meets acceptable standards. If the road does not appear to comply then possible alterations to the road geometry need to be considered when designing the rehabilitation treatment.
• Document and review pavement distress observations and records: The observed condition of the pavement should be carefully recorded. The nature of the distress observed in a pavement can provide guidance to the designer about the causes of and best treatments for the distress. A key question for the
designer is to see whether the distress modes observed in the pavement are due to shallow, near surface deterioration (shallow shear, attrition, shoving), or deep seated structural deformation. The recording and review of inspection records and historic performance data needs to support the informed review of the type, severity, and extent of pavement distress on a particular project site.

4.2.3 Field inspection and laboratory testing

Once the first two inspection steps have been completed, field and laboratory testing should be performed. Underground services should be located beforehand if the pavement is to be disturbed, for example with a digger or a Scala.

A specialist pavement designer will be required to specify the exact testing required. Testing requirements of the in situ ground condition or remoulded material will depend on the proposed alignment and the depth of any cut and fill.

As an example the material testing required from the NZTA M/4:2006 tests (NZTA 2006) includes: crushing resistance; weathering quality index; California Bearing Ratio, both in situ and laboratory based tests; quantity of fines including sand equivalent, clay index, and Plasticity Index (PI); broken face content; particle size distribution; and moisture content.

The detail of testing required depends on the project and an outline of different levels of investigation is given in Table 6. The required test levels are defined in Table 5 for the various One Network Road Classifications. The level of investigation relates to the risk of the pavement project and this is related to the level of traffic expected for the treatment site as well as the other factors that affect the One Network Road Classification for a particular road, for example criticality. The test requirements detailed in Table 6 are minimum requirements and it is expected that the pavement engineer will require additional testing should they consider this to be necessary. Where an item is recommended in Table 6, its exclusion from the test program will have to be justified in the design report.

In many cases at least Level 2 testing is required. Level 1 is normally only used for screening or preliminary investigation for estimating purposes. If there were any concerns over the quality of the basecourse layers or if there were only meagre information available then Level 3 testing would be required. This is illustrated schematically in Table 5 where the level of the investigation is determined by the road classification as defined by the One Network Road Classification1.

Table 6 Minimum testing programmes by level of investigation

<table>
<thead>
<tr>
<th>Test</th>
<th>Level 1</th>
<th>Level 2</th>
<th>Level 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Survey</td>
<td>Required</td>
<td>Required</td>
<td>Required</td>
</tr>
<tr>
<td>Condition Assessment</td>
<td>Optional</td>
<td>Recommended</td>
<td>Required</td>
</tr>
<tr>
<td>FWD/Deflection measurements</td>
<td>Required</td>
<td>Required</td>
<td>Required</td>
</tr>
<tr>
<td>RAMM Data (Historic performance including neighbouring treatments, plus pavement layer information)</td>
<td>Required</td>
<td>Required</td>
<td>Required</td>
</tr>
</tbody>
</table>

1 Further information can be found at https://www.nzta.govt.nz/roads-and-rail/road-efficiency-group/one-network-road-classification/key-documents/
### Table 7 Repeated load triaxial testing

<table>
<thead>
<tr>
<th>Test</th>
<th>Level 1</th>
<th>Level 2</th>
<th>Level 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>NZTA T/15 RLT Permanent Strain Test (Dry) - (note add appropriate proportions of seal and overlay aggregate if required)</td>
<td>Optional</td>
<td>Recommended</td>
<td>Required</td>
</tr>
</tbody>
</table>

The RLT test requirements are detailed in Table 7. The expectation is that a single Repeated Load Triaxial test will be used to characterise the basecourse materials for a site, unless there are significant changes in the materials. These changes will be based on a visual assessment including the material type and the partial size distribution. A RLT test is not required for every test pit.

### Table 8
details the test requirements to validate a mix design. Obviously these tests are only required when stabilisation is a valid treatment option. Furthermore, the materials tested should incorporate the appropriate

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2 These requirements are for the upper granular pavement, reduced testing is justified for subbase granular materials. For all subbase materials Level 1 tests become Optional and all Required Level 2 tests become Recommended while Level 3 test requirements relax to the Level 2 requirements.
proportions of seal and overlay aggregate if these will be incorporated into the final pavement construction. A single Indirect Tensile Strength suite of tests is expected to characterise the site unless there are significant changes in the materials throughout the site. A complete suite of Indirect Tensile Strength tests is not required for every test pit.

**Table 8 Mix Design for Stabilisation**

<table>
<thead>
<tr>
<th>Test</th>
<th>Level 1</th>
<th>Level 2</th>
<th>Level 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Indirect tensile strength (ITS) soaked tests according to NZTA T19 for a range of binder contents</td>
<td>Optional</td>
<td>Required</td>
<td>Required</td>
</tr>
<tr>
<td>ITS tests according to NZTA T19 to confirm target binder contents</td>
<td>Optional</td>
<td>Recommended</td>
<td>Required</td>
</tr>
<tr>
<td>Flexural Beam Strength Test at two or three different binder contents</td>
<td>Optional</td>
<td>Optional</td>
<td>Recommended</td>
</tr>
<tr>
<td>NZTA T15 RLT Permanent Strain Test (Dry) – test to get estimated rutting life of unbound dry source aggregate which is used to find the life of the stabilised mix based on its ITS for determining a traffic life multiplier</td>
<td>Optional</td>
<td>Recommended</td>
<td>Required</td>
</tr>
<tr>
<td>NZTA T15 RLT Permanent Strain Test (Wet/Soaked) - (recommended where cement content is 1% or lime is used)</td>
<td>Optional</td>
<td>Optional</td>
<td>Recommended</td>
</tr>
</tbody>
</table>

**4.3 TEST PITS**

All the data collected up to this stage of the investigation shall be examined by a person experienced in pavement design and investigations so they can determine the location and number of test pits. Test pits shall be located to avoid underground services.

Test pits should be targeted to represent homogeneous sections of the site. These should be identified by observed deterioration and, if available, deflection testing of the pavement. Austroads (2010) suggests a homogeneous section is where the Coefficient of Variation (CV) of the deflection is less than 25%. The CV is defined as the ratio of the standard deviation of a distribution to its arithmetic mean (CV= standard deviation/mean). For many sites a CV under 25% will not be obtained. In these cases the cumulative sum or a moving average technique should be used to determine where significant changes in the structural response occur.

The number of test pit excavations at locations depends on the budget constraints and relative risk. Sampling should be concentrated within or near the wheel track locations, and alternate between sides of the proposed road alignment.

The NRB (1989) *State Highway Pavement Design and Rehabilitation Manual* required at least five representative test pits to be sampled and tested within each section with similar properties. Because of the significant financial consequences of pavement failures the minimum number of sample test pits, for both urban or rural sites, shall be either:
• three test pits for treatment lengths up to 200 m, and
• every 100 m for treatment lengths 200 m or greater.

In a rural roading environment, the test pit frequency might be reduced to a minimum of one test pit every 200 metres provided there is evidence that demonstrates consistency of the pavement. Suitable evidence would be consistent FWD results or historic local knowledge and any reduced test frequency would have to be approved by the Client.

At least one test pit(s) shall be located as depicted in Figure 2 across the left hand wheelpath and extending towards the relatively un-trafficked shoulder. Other test pits should be located in the right hand wheelpath and in other locations to ensure that a good understanding of the pavement structure is established, particularly if the pavement has been widened in the past. Several test pits are required to understand pavement variability and to ensure sufficient material is excavated for future laboratory testing. Test pit locations should be positions so as to avoid damaging of any underground services.

![Figure 2 Test pit location](image)

### 4.3.1 Test pit size

The test pit needs to be big enough for the subgrade to be tested; the surface dimensions should be at least 400 mm × 1200 mm. A mechanical digger shall be used to construct a trench when excavating a test pit on state highways. For asphalt surfacing the existing asphalt shall be saw-cut around the perimeter of the test pit to ensure a straight edge for accurate excavation. These measures are to assist in the successful compaction of test pit backfill and quality surfacing reinstatement.

Coring the surface with a 150 mm diameter corer is not allowed for most investigations. It is only suitable when the top bound layers require analysis. This could for example, be for analysing aged open graded porous asphalt overlaying asphalitic concrete where it is necessary to establish if the distress is localised in only the top layer.
4.3.2 Logging layers

Logging of basecourse, subbase, and subgrade information should be done by trained staff who can classify the soils. Each material layer encountered should be sampled and the material described in terms of the Field Description of Soils and Rock (New Zealand Geotechnical Society 2005).

When back analysing FWD measurements, accurate layer thicknesses are required to assist in the interpretation of layer stiffness. A layer thickness accuracy of ±10 % or better is required.

As a general rule, for a 40 mm basecourse material a sample of approximately 35 kg should be taken. If extensive laboratory testing has been indicated or if larger materials are present, then at least two bags of sampled material should be obtained. If in situ stabilisation is at all likely the additional materials for testing must be discussed with the designer.

If the materials encountered on any two or more sections of pavement are similar, then only sufficient material will be sampled to carry out the envisaged laboratory testing programme. This may mean combining samples from various pits or not retaining samples from some of the pits. However, where differing basecourse and subgrade materials are encountered, then at least one bag of each of type of material should be sampled. This is because if all materials are sampled or retained, the volume of this material becomes a major logistical effort, not only to transport but also to store satisfactorily.

Photographs of the face of the excavated test pit (usually facing perpendicular to the centreline of the road) and surrounding pavement area shall be taken. For the pavement pit photographs, a whiteboard should be placed alongside the pit, stating project name, location, route position, date, pit number and comments (any relevant information). For comparing layer thickness a graduated white measuring rule with large highly visible numbers should be placed inside the pavement pit. To obtain a satisfactory photo place the whole of the face of the pavement pit and adjacent surrounds into complete shadow via the use of a large cardboard sheet or the like, and then use the flash mode of the camera for all photographs. Note, however, where all the materials contained within the pavement layer are of a similar dark brown or grey colour (i.e. no contrasting lighter shades), the photographs may not show all the details.

4.4 DEFLECTION TESTING

Deflection testing can use either a Falling Weight Deflectometer (FWD) or a Benkelman Beam (BB). The preference is for a FWD to be used unless the BB deflection bowl can be measured as described in Appendix E. The designer should be aware of the differences between the two types of measurements.

The default test frequency would normally be a test every 20 metres (or less) alternating between outer wheel paths. Tests shall be performed on all lanes. The testing frequency should ensure that a minimum of 20 test points are obtained although, where justified by the risks and the size of the site, a statistically significant number of test points should be obtained.

Where the proposed rehabilitation treatment is a hot mix overlay, then at a minimum the central deflection (D0) and the deflection at a distance of 200 mm (D200) will be obtained. This allows the curvature, defined as the difference between D0 and D200, to be calculated.

Back analysis of the FWD deflection bowl can provide a useful indication of the pavement stiffness and in particular the subgrade stiffness. These back calculated results should not be used in isolation. Care should be taken to ensure anisotropic or isotropic values are used where appropriate.
At each test point the surface condition shall be noted, for example if it is cracked, rutted, or patched. If the visual inspection indicated there were areas with higher levels of distress then in these areas deflection tests should be performed at a higher frequency.

One purpose of the initial deflection survey is to be able to divide the site into homogeneous sections. This can be performed using statistical techniques such as cumulative sum or a moving average.

The Traffic Speed Deflectometer (TSD) had surveyed some of the New Zealand State Highway network as well as Local Authority roads. While the TSD produces a bowl that is theoretically similar to the FWD deflection bowl this relationship is not perfect. NZ Transport Agency Technical Advice Note 17-04 gives some advice on converting between the two bowls. However, while the use of data generated by the TSD is encouraged, the results, particularly the conversion, should be used with caution and an awareness of the potential risks of relying on these data.

4.5 SUBGRADE INVESTIGATIONS

4.5.1 Field investigations

The subgrade CBR should be assessed using a dynamic cone (Scala) Penetrometer test. The inferred CBR values should be checked against a remoulded laboratory CBR test. There can be significant variation in results depending on the subgrade moisture conditions at the time of testing. This is why the subgrade moisture content is required for Level 3 testing.

Access to the subgrade should be sufficient to provide clean access to ensure an uninfluenced test. Dynamic cone (Scala 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. The Scala shall be used to a minimum depth of 1.3 metres below the top of the subgrade or to rejection, whichever occurs first.

It is important for Scala penetrometers to be taken to a significant depth to ensure a representative test result can be obtained for analysis and to determine the subgrade level. In Figure 3, is an example of results from Scala penetrometer testing on a subgrade. The results reveal a change in material at approximately 300 mm deep. If an in situ CBR test had been performed at the subgrade surface and no Scala tests had been performed then the soft material at a depth of 300 mm would not have been found and the test would have predicted a stronger subgrade than that which existed.
4.5.2 Design California Bearing Ratio

Taking the average value of the inferred results from a Scala test is not appropriate. This can significantly overestimate the strength of the subgrade.

For the majority of soil types, best correlation with CBR is achieved when the Scala DCP mm/blow is calculated from a weighted average of blows/50 mm for the first three 50 mm intervals using weightings of 0.7, 0.2 and 0.1 for each interval. A full description on the origin of these calculations can be found in Smits (1990). Smits’ weightings correspond very closely to those determined from a vertical stress distribution beneath a CBR plunger.

Therefore, the weighted Scala DCP results from the first 150 mm below the CBR test depth shall be used to determine the design CBR. Below this depth soil properties do not significantly affect results, however, this methodology should be tempered with the Austroads’ (2012) advice that evaluation of the actual support provided to the pavement structure by the subgrade can be complicated by the strength variations that often occur with depth. It is essential that the potential effects of any weak layers below the design subgrade level are considered in the pavement design process, particularly for low-strength materials occurring to depths of about one metre. For example, the design CBR example in Figure 3 would have to be adjusted for the lower strength underlying material. Where strength decreases with depth, the subgrade may be sublayered for the purposes of the mechanistic pavement design of flexible pavements and when calculating the effective subgrade strength for rigid pavement designs. For subgrade strengths that are constant or improve with depth, the support at the design subgrade level governs the pavement design.

The design CBR is then the tenth percentile of the results obtained over the treatment length.
4.5.3 **In situ California Bearing Ratio**

*In situ* CBR tests can also be performed according to the procedures specified in NZS 4402:1988 (Test 6.1.3). The number of CBR tests required will depend on the uniformity of the subgrade soil. Considerable input from the pavement engineer is required as both testing and interpreting the results can be difficult.

The *in situ* CBR test is a direct representation of *in situ* compacted subgrade CBR strength. However, this test is both time-consuming and expensive and is generally only undertaken if there is a granular subgrade, uncertainty over Scala penetrometer data, or if there is a dispute.

If using laboratory CBR testing then particular care should be taken in briefing the laboratory on:

- Conditions of the test (soaked or unsoaked),
- Compactive effort (NZ Standard or Heavy or Vibrating Hammer (NZS 4407:1991) or target density possibly to match *in situ* density testing),
- Moisture content (as per sampled, measured Optimum Water Content (OWC), or assumed OWC),
- Whether to use remoulded or “undisturbed” samples. The designer needs to talk to the laboratory personnel to determine the best way to obtain undisturbed samples, and
- The use of the appropriate surcharge.

Guidance on the design subgrade moisture conditions may be obtained from the report by Arampamoorthy and Patrick (2010) which recommends characterisation of the subgrade at saturation. If subgrade soil has been identified as fine grained (i.e. more than 35% passing the 75 micron sieve), Atterberg tests (Liquid and Plastic limits) shall be run in addition to the normal sieve analysis and CBR tests, so that assessment of the potential need for subgrade stabilisation or undercut can be determined.

Sufficient samples of subgrade must be collected in airtight bags for the required testing. If stabilisation is being considered, the sample size will be greater than if only classification tests are to be performed.

4.5.4 **Subgrade sensitivity**

Some subgrade soils are sensitive to being disturbed or remoulded. The subgrade sensitivity is a measure of the loss of strength that occurs when the soil is disturbed or remoulded. Sensitivity is defined as the ratio of the undisturbed strength to the remoulded strength as defined in equation 4.1 below.

\[
\text{Shear Strength Ratio} = \frac{\text{Undisturbed shear strength}}{\text{Remoulded shear strength}}
\]

4.1

As detailed in the *Field Description of Soil and Rock* (NZ Geotechnical Society Inc. 2005) the subgrade is defined as insensitive or normal when the shear strength ratio is less than two, moderately sensitive when it is between two and four and sensitive or worse when it is greater than four. Where a subgrade is identified as being sensitive or worse, the recommended pavement design will manage the risk of the sensitive subgrade particularly during construction.

4.6 **REINSTATEMENT**

For pavement pit reinstatement, all backfill material will be placed back into the pit in 100 – 200 mm thick layers and compacted using suitable compaction equipment. In most cases, where the materials are of poor quality or
large samples have been taken, “top up” material will be required and should be a high quality crushed rock basecourse.

When resealing, consideration shall be given to the existing surface and the pavement structure. The patch should give at least 24 months of maintenance free performance. For example for sealed roads, pavement pits might be compacted to a level 30 – 50 mm below the road surface and high volume roads capped off with a hot mix, while cold mix (plant mix) could be used on low volume roads. An emulsion tack coat should be used between the compacted basecourse material and the cold mix.

Finally, to assist the client and for reference purposes, after the pavement pit has been reinstated, a “site photo” shall be taken together with reference location and if available GPS coordinates. It is normal practice for this photograph to be taken facing towards the direction of the increasing route distances.

4.7 FIELD AND LABORATORY INVESTIGATION SUMMARY

Investigation scope is to be determined by an experienced pavement design engineer who will also be the person who will be doing the design. They shall also ensure:

- The location of the test pits and trenches are decided, note whether these are in cut or fill and the location of the test pit from the centre line or lane edge
- The pavement condition and the drainage are noted, e.g. cracked, shoved, rutted, stripped, patching
- The layer thicknesses are accurately measured, particularly noting the number of seal layers and individual thicknesses
- The physical characteristics of each layer are described, e.g. compactness, maximum sizes present, weathering, moisture
- The geological origin of aggregates and the nature and plasticity of fines and any contamination and the presence of stabiliser are identified
- Non-destructive testing is carried out first to help determine the test pit locations
- Test pits are obtained, recording location including offset from centre line or lane edge
- Test pit logs record depths to individual layers referenced to datum
- *In situ* tests of pavement layers are carried out, e.g. Scala, light weight deflectometer, shear vane, FWD, and/or BB,
- *In situ* strength of the subgrade is determined, e.g. CBR and shear strength.
- Bulk samples for later laboratory testing are taken, e.g. laboratory-soaked CBR or testing to determine suitability for stabilisation.

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

A root cause analysis integrates and interprets the various failure mechanisms, the existing pavement structure, and the traffic history to determine the root cause or causes of the pavement failure. The following sections detail some common faults and their causes but are by no means exhaustive additional information can be obtained from Austroads (2011).

In all cases, damage to the seal will allow the ingress of moisture and will accelerate the deterioration of the pavement. Furthermore, it should be remembered that seals, particularly first coat seals, are not fully waterproof and under the stresses induced by traffic allow some moisture to enter the pavement.

5.1 POTHOLES

5.1.1 Description

Figure 4 illustrates a pothole which can be described as a bowl shaped cavity extending into layers below the wearing course.

Figure 4 Example of a pothole.

5.1.2 Causes of failure

Potholes may be caused by a number of factors:

- lack of surface waterproofness;
- lack of cross fall causing water to pond on the surface, increasing moisture ingress;
- high fines/dirty unbound basecourse aggregate;
- water upflow, and
If the potholing is occurring early in the seal life, consider whether the surface matrix was inadequate prior to sealing.

5.1.3 Testing and investigations

If pavement renewal is being instigated to allow for future traffic growth and potholes are the only defects present, testing will probably yield little information. No rutting or shoving implies the structural strength does not need to be improved. However, if in combination with potholes there are other pavement distress modes, testing will be needed. The test requirements are detailed in the following sections. The particle size distribution should be tested to ensure the basecourse material is suitable for hoeing or to identify what stabilisation treatment might be appropriate, for example granular stabilisation. The plasticity index should be obtained to check for good potential bitumen adhesion.

5.1.4 Potential treatments

Treatment type and design method should target surface waterproofness along with improving crossfall and drainage. If investigations have shown that the basecourse is of poor quality then this deficiency should be addressed in the proposed treatment.

If basecourse stabilisation is being considered then Indirect Tensile Strength (ITS), Unconfined Tensile Strength (UCS) and/or flexural beam strength tests may be needed to confirm the reactivity of the materials with binder.

Minimal treatment is to only apply a pothole patch. Structural asphalt patches should be considered in areas that are flat and where the crossfall cannot be improved. However, this treatment should be developed in the context of the whole site and the interface between the different pavements will need to be considered.

When constructing a patch, an area greater than the area where surface damage is visible should be treated. The patch should also be constructed to allow adequate compaction and to ensure the longitudinal patch edge is not in a wheelpath.

5.2 DEFORMATIONS, HEAVES AND SHOVES

Actual and schematic examples of shoving are illustrated in Figure 5 below.

Figure 5 Example of shoving in the pavement
Heaving and shoving is most commonly a result of basecourse shear failure caused by:

- poor quality basecourse and/or water ingress due to lack of surface waterproofness and/or poor drainage,
- multiple unstable seal coats, as detailed in section 5.6,
- inadequate asphalt layer properties, and
- inadequate compaction, particularly in poorly reinstated trenches.

5.2.1 Testing and investigations

The aggregate from the upper pavement should be assessed in terms of rut resistance when dry and wet. Laboratory tests on aggregate to assess rut resistance are fines quality tests and particle size distribution. Compliance with NZTA M4 specification and/or conduct RLT tests are also checked as per the NZTA T15 (2014) specification.

Poor shear strength when wet can be inferred when:

- poor quality fines are observed,
- the broken faces are low,
- the surface is cracked, and/or
- it is obvious failure is due to water ingress.

Poor rut resistance when dry can likely be inferred when:

- the particle size distribution is poor,
- the broken faces are low,
- RLT tests results are poor, and/or
- the surface is waterproof but there is still significant rutting and shoving.

5.2.2 Potential treatments

The chosen treatment and design method should target basecourse poor quality, surface waterproofness and drainage improvement.

Higher risk treatment options include in situ stabilisation of the existing material or digging out a patch, both options should be combined with drainage improves and resealing of the whole site.

If the basecourse aggregate has insufficient engineering properties for the induced strains, the treatment should require a good quality overlay. It may also be necessary to stabilise the existing materials to correct the grading. Site constraints may also require the existing material to be removed and replaced, potentially with a stiffer more robust pavement.

5.3 RUTTING – SUBGRADE

Rutting may occur because of inadequate subgrade strength for the traffic loading, resulting in the build-up of plastic strain in the subgrade. Figure 6 below provides some examples of rutting. Merely observing the surface rut is insufficient to decide if the rutting is entirely contained in the subgrade or has also accumulated in the aggregate layers. However, ruts that have resulted primarily from subgrade deformation generally have wide ruts and are not accompanied by shoving or heaving.
If rutting is primarily in the subgrade, a pavement trench will typically reveal a shallow pavement depth and rutting in the subgrade soil only.

**Figure 6 Rutting in pavement subgrade**

### 5.3.1 Testing and investigations

If overlay treatment is the chosen rehabilitation treatment, laboratory testing of *in situ* aggregate may not be necessary. However, the suitability of the existing basecourse to act as a subbase material should also be assessed. Also, depending on the traffic levels, the suitability of the overlay material and depth may need to be confirmed. Test pits should still be excavated to determine the existing pavement configuration.

If stabilisation treatment is identified then tests are required with binder and *in situ* aggregate to check reactivity.

### 5.3.2 Potential treatments

Generally the only suitable treatment is an overlay to reduce stresses on the subgrade. Analysis using past precedent performance, detailed in 8.2, would frequently suggest a 70 mm overlay over the high spots is all that is required, provided all NZTA B2 (2005a) requirements are met. Any overlay should be combined with drainage improvements to dry the subgrade soil and thereby improve the effective strength of the subgrade.

*In situ* stabilisation of the existing pavement material with an overlay could be considered to reduce the risk of early failure. Simply stabilising the existing pavement is a potential treatment, however, it has high risk of premature failure unless the modular ratio is kept low.

### 5.4 Rutting – within pavement base layer

Rutting may occur primarily due to consolidation in the aggregate layers. Rutting/shoving and heaving at the surface can result from shear movement in the aggregate layers only. The source of the rutting can be confirmed by trenching. **Figure 7** shows some examples of rutting within the upper pavement.
Trenching will also enable other contributing factors such as quality and moisture content of the aggregate to be examined. The particle size distribution of the aggregate and the plasticity index will give an indication of the quality of the material and RLT testing will further support the suitability of the material.

5.4.1 Testing and investigations

The pavement depth and subgrade characteristics should be determined as detailed in Section 4.

The quality of the base layer can be assessed through looking at the particle size distribution and the Atterberg limits. Base Layer Quality/Rut Resistance can be more accurately determined as detailed in Section 7.4.

The test pit samples should be taken from the full depth of the various aggregate layers. The quality of each layer, in terms of rut resistance, should be assessed when the material is both dry and wet. Laboratory tests to do this can include fines quality tests and particle size distribution and checking compliance with NZ Transport Agency M4 specification.

If considered necessary conduct RLT tests as per the NZ Transport Agency T15 specification. T15 will indicate:

- Poor rut resistance when wet – Poor quality fines; failure likely due to surface being cracked and obvious failure are due to water ingress.
- Poor rut resistance when dry – Particle size distribution is poor; RLT when tested dry gives poor results; surface can still be waterproof but there is still significant risk of rutting and shoving.

5.4.2 Potential Treatment

Potential treatments for rutting could include granular and or chemical stabilisation. Granular stabilisation involves adding selected size aggregates to correct the grading deficiencies. An alternative is to apply a good quality overlay to reduce the stresses in the existing aggregate.
The treatment and design method should target basecourse poor quality, surface waterproofness and drainage improvement.

If rutting is highly localised in small areas then the potential exists to fix these small areas and then resurface the entire site. However, such an approach has a high risk of early failure unless proper design and testing is conducted. Validation that rutting within the basecourse is in localised areas and is unlikely to be a treatment length issue can be assisted with FWD testing of the site.

5.5 FLUSHING ONLY

This section applies where only flushing is present and no rutting or heaving is observed. An example is illustrated in Figure 8 which shows a heavily flushed pavement. Flushing can typically be attributed to chip breakdown, moisture induced binder rise, unstable chip seals, inappropriately designed reseals or a combination of all of these factors.

Flushing on its own should not require pavement rehabilitation, however, it may be a symptom of unstable seals or poor drainage.

![Figure 8 Pavement heavily flushed](image)

It should be noted; if flushing as well as other types of pavement distress are present then the treatments for the other distress modes should come first. Minimal tests will be required if the treatment simply involves water cutting and/or applying a sandwich seal to correct the flushing. Flushing may be a result of chipseal layer instability. This failure mechanism is discussed further in Section 5.6. If water cutting is identified as an appropriate maintenance treatment, the remaining bitumen should be estimated. This will ensure the surface retains its waterproof properties, otherwise rehabilitation will probably be soon required.

There is also an indication that the effects of water cutting are temporary. Figure 9 demonstrates flushing reduces in the year water cutting treatment was used (defined as year zero in the figure). However, the flushing extent recovers to pre-cutting levels and continues to deteriorate at the same rate observed prior to cutting. Water cutting
tends to only provide temporary relief from flushing. Figure 9 has been generated using flushing and water cutting data obtained from the Long Term Pavement Performance Project.

![Figure 9 Impacts of watercutting](image)

**Figure 9 Impacts of watercutting**

### 5.5.1 Testing and investigation plan

If a pavement renewal is designed, standard tests are needed. These include using test pits or RAMM records to determine pavement layer material properties and depths. FWD would also assist in determining if pavement strengthening is required for the future traffic.

Testing *in situ* aggregate not essential if an overlay treatment is proposed. If a stabilisation treatment is proposed then strength tests will probably be necessary using the proposed binder and *in situ* aggregate to check their reactivity.

### 5.5.2 Potential treatments

Flushing is not a pavement failure mechanism. It typically requires surfacing renewal rather than pavement renewal. The exceptions are when flushing is being driven by poor drainage or unstable seals.

*In situ* stabilisation and a reseal combined with drainage improvements is the most common fix for flushing where pavement renewal is required. However, there is a danger the underlying aggregate that is well compacted and waterproofed by the multiple seal layers is disturbed and the stabilised pavement will rut. RLT testing of the existing *in situ* aggregate blended with seal layers that will be incorporated into the stabilisation can check this risk.

An alternative treatment is to undertake drainage improvements, water-cut the surface, and apply a high void polymer modified binder sandwich seal. Layer instability should be eliminated as a failure mechanism before applying this treatment.
5.6 CHIPSEAL LAYER INSTABILITY

If a pavement could be failing due to chipseal layer instability, the following questions for a candidate length should be considered:

- Is surface shoving present?
- Is flushing a failure mode?
- Is loss of texture rapid and/or premature?
- Was flushing evident before the previous resurfacing?
- Was the life of previously applied reseals getting shorter and reducing after each successive treatment (referred to as a ‘pinching reseal cycle’)?

If most of the answers to these questions are affirmative consider taking a core sample of the layer for technical analysis. This core is analysed against the layer depth and binder stone ratio criteria. In this context binder is defined to be the bitumen and all material passing the 2.36 mm sieve contained within the seal layers. A core will also allow the underlying layer material types and thicknesses to be established.

A summary of the risks associated with various levels of binder stone ratios is provided in Table 9 below. The factors and triggers may be different in different climatic conditions. Various treatments are available: in some circumstances the existing surface layer should be removed or appropriately stabilised before an overlay is constructed; alternatively, an overlay of sufficient thickness to compensate for the unstable layers of chipseal may be appropriate. Water cutting is likely to provide only a temporary solution as indicated in Figure 9, in Section 5.5 above.

Table 9 Unstable chip seal layers

<table>
<thead>
<tr>
<th>Binder / Stone ratio (Binder = Bitumen plus material passing 2.36mm sieve (by mass))</th>
<th>Risk</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 12%</td>
<td>Low risk of seal instability.</td>
</tr>
<tr>
<td>12% to 15% (and cumulative depth of the seal layers exceeds 40mm)</td>
<td>Possible risk of instability. Water cutting suggested as a feasible treatment prior to reseals in conjunction with careful consideration of seal design.</td>
</tr>
<tr>
<td>15% to 20% (and cumulative depth of the seal layers exceeds 40mm)</td>
<td>High risk of seal instability. Water cutting suggested may be a feasible treatment prior to reseals in conjunction with careful consideration of seal design (for example use of sandwich seals).</td>
</tr>
<tr>
<td>&gt; 20% (and cumulative depth of the seal layers exceeds 40mm)</td>
<td>Very high risk / almost certain that instability will occur. Entire layer needs treatment (i.e. recycling, rehabilitation or another treatment).</td>
</tr>
</tbody>
</table>

5.7 CRACKING

Cracking is typically caused by fatigue failure of the wearing course, an example is shown in Figure 10 where the surface has failed in the wheelpaths through fatigue due to the pavement deformation under traffic. In addition rutting, shoving, or heaving of the underlying materials can generate high strains in the wearing course and exacerbate fatigue of the wearing course.
Cracking of the wearing course is often the result of the underlying pavement having low stiffness. Treatment can include overlay and/or stabilisation. However, in the cracking wearing course is thin asphaltic concrete, the suitability of the underlying pavement needs to be verified. It may be most cost effective to replace the thin asphalt with a chipseal which can tolerate higher levels of strain.

Figure 10 Cracking in both wheelpaths

Visual inspection showing extensive alligator skin type cracking frequently indicates fatigue failure of an underlying bound pavement layer. The failure of a cement stabilised layer will often manifest as block cracking. Oxidation of bitumen can result in a brittle binder which will also crack when subjected to excessive strains. Cracking can also be related to thermal shrinkage, environmental effects or reflective joint cracking.

The aim is to determine the cause of cracking so the rehabilitation treatment will prevent such failures from reoccurring. Austroads (2011) provides a good discussion on the various types of cracking observed on road pavements and their causes and potential treatments.

5.7.1 Testing and investigation plan

The testing and investigation plan should aim to determine the extent, type, depth and width of cracking and identify the pavement material that has cracked. Use of the FWD to determine central deflections and curvature is recommended. Curvature can be used as a proxy for surfacing strain. Guidance is provided in Section 7.10.1.

5.7.2 Potential treatments

Consider milling and removing the cracked pavement material and replacing it with good quality crushed rock. The pavement depth may also need to be increased. Alternatively, an asphaltic overlay can be designed following Austroads guidance for minimum cover to prevent reflective cracking. However, in New Zealand an overlay of 100 mm of unbound granular material is considered sufficient to eliminate reflective cracking.

In situ stabilisation (ideally with an overlay of crushed rock) could be considered if laboratory testing shows the source material is of sufficient quality. However, restabilising a once-stabilised road that is now cracked will probably result in the cracking reoccurring again in a similar timeframe. This is why in situ stabilisation should be treated cautiously.
For hairline cracks, a reseal with a polymer modified binder should be sufficient. This will be particularly effective when combined with sealing individual cracks greater than 2 mm and less than 5 mm.
6 Preliminary treatment design options

6.1 GENERAL

After assessing the faults identified in the treatment length and the factors leading to their development, a root cause analysis should be conducted. The root cause analysis results will lead the designer to select a number of treatment options to minimise the risk of the original causes driving premature failure again.

Any solution should be developed for design traffic in terms of Equivalent Standard Axles (ESAs) over a suitable design period. Typically this is 25 years. Traffic calculations shall follow the methodologies in Austroads (2017) and the NZ Guide to Pavement Structural Design (NZ Transport Agency 2018).

Typically, options available to a designer will involve:

- Granular rehabilitation treatments,
- Asphaltic concrete treatments, or
- Full depth reconstruction.

These treatment options are discussed in more detail in the following sections. Where possible, pavement treatments that incorporate the use of recycled materials are preferred.

6.1.1 Granular rehabilitation treatments

Implicit in the Austroads (2017) design methodology is the assumption that the pavement materials have sufficient strength to perform over the pavement design life without accumulating significant plastic strain. However, some in situ materials may deteriorate over time or may never have been suitable for the applied traffic levels. Therefore, an assessment of the existing aggregate’s rut resistance will help determine the appropriate treatment. However, the other Austroads’ design requirements, such as subgrades strain, still need to be met.

The initial assessment of whether the in situ existing aggregate has poor, medium or good rut resistance leads to the treatments and proposed pavement design assumptions in Table 10.
### Table 10 Pavement Rehabilitation Treatments

<table>
<thead>
<tr>
<th>Pavement rehabilitation treatment type</th>
<th>When this treatment would be considered</th>
<th>Design method and assumptions</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Granular Overlay - to reduce the wheel load stresses in the existing pavement material and subgrade.</td>
<td>Optional for all pavement distress types but is recommended where subgrade has rutted due to lack of pavement depth. (Where there is a concern the granular overlay may fail, an overlay following <em>in situ</em> stabilisation is required). <em>The design assumption will depend on the reason for treatment (either correcting for a poor quality existing aggregate or requiring more cover to the subgrade soil).</em></td>
<td>Mechanistic design method using CIRCLY where minimum overlay depth is either governed by total depth to the subgrade (rutting in subgrade) or cover needed to protect the existing pavement materials (rutting within aggregate layer). (See Section 7.6 Granular Overlay Design)</td>
</tr>
<tr>
<td>2. Bound Treatment Subbase - Stabilisation of the “new” subbase. <em>(Note there is more than one option for this. It can be <em>in situ</em> stabilisation on top of old pavement but overlaid with new basecourse aggregate</em>³ <em>Alternatively, the old basecourse may be removed and the <em>in situ</em> old subbase stabilised. In both cases the new aggregate is then put on top). It is designed specifically to ensure the stabilisation bonds remain intact over the design life. The approach is similar to a bound pavement design, to ensure the bound subbase can bridge the weak material below.</em></td>
<td>This treatment takes advantage of a bound subbase layer that bridges weak material underneath. This reduces the total pavement depth needed. Due to the risk of block and shrinkage cracking NZ Transport Agency does not allow bound basecourse. Thus, this bound treatment has been presented in terms of a bound subbase where it will always be overlaid with unbound or modified aggregate. The one exception is the construction of a HiLab basecourse, which will need to be approved by the Agency. This treatment is also needed where the existing aggregate is poor quality in terms of shear strength. Thus strength is reliant on the cement bonds in addition to stone on stone contact. For <em>in situ</em> stabilisation of the base hoeing-in top-up metal for granular (mechanical) stabilisation is recommended, along with a modified treatment design as per Treatment Type 3 below.</td>
<td><em>See Section 7.8 bound treatment (Limiting Tensile Stress Method)</em></td>
</tr>
</tbody>
</table>

³ The new basecourse thickness shall be a minimum of 100 mm of unbound material.
<table>
<thead>
<tr>
<th>Pavement rehabilitation treatment type</th>
<th>When this treatment would be considered</th>
<th>Design method and assumptions</th>
</tr>
</thead>
</table>
| **3. Modified Treatment - In situ** stabilisation designed specifically to result in a modified material where the binder is used to tidy up the fines to ensure adequate rut resistance should the pavement become wet. An overlay of aggregate can also be added. The overlay is usually large good quality crushed rock hoed in during the stabilisation process to improve the grading of the finished base layer for better rut resistance. | This treatment is needed where the existing aggregate is good quality in terms of good rut resistance when dry (i.e. good grading and properties similar to NZTA M4 Basecourse Specification). The rut resistance is reliant on the stone-on-stone contact rather than the cement bonds. The binder (e.g. cement, lime or bitumen) purpose is to tidy up the fines to prevent failure when wet.  
*Note: the key for this treatment’s success is the in situ aggregate must have a good grading and rut resistance that can be estimated using the RLT apparatus. If needed the source aggregate quality can be improved by top-up aggregate. The ITS of the stabilised mix is then used to multiply the life of the source aggregate found by RLT testing (NZTA T/15).* | Mechanistic design method using CIRCLY with unbound pavement design assumptions to check the total pavement depth. In addition, the stabilised base mix rutting life is checked. To check, use the rutting life of the unbound source aggregate multiplied by a factor dependent on the ITS value of the stabilised mix.  
An example is in situ stabilisation using foamed bitumen or low cement content. (See Sections 7.7 and 7.9 respectively) |
| **4. Reseal, resurface or rip and remake and improve drainage** | This treatment can be considered when the main and generally the only form of pavement distress is flushing. Section 5.6 gives more detail about unstable seal layers. | If required surface waterproofness is restored, plus drainage and shape is improved and design life is the same as past life (i.e. rut resistance of in situ materials remains the same). |

Note: 1. Chip seal layer instability can be treated as an existing base material that has “poor” rut resistance, although it is discussed further in the sections below:

Note: 2. Proposed in situ stabilised mixes tested in the laboratory should include a proportion of the existing chip seal layers and, if added, an overlay material as this could represent up to half of the mix (Note: a maximum of 30% chipseal is recommended in the in situ stabilised mix).

### 6.1.2 Asphaltic concrete overlay or inlay

Where an asphaltic cement overlay or inlay is to be designed, the methodology from *Austroads Guide to Pavement Technology Part 2* (Austroads 2017) and *Part 5* (Austroads 2011) should be used. Appendix A contains information on asphalt to supplement the information from *Austroads* (2017).

### 6.1.3 Full depth reconstruction

Where site constraints do not allow the existing pavement to be utilised in any manner, the only remaining option is a full depth reconstruction. The potential exists to recycle the existing materials as a subgrade improvement layer.
Furthermore, when considering full depth reconstruction, stabilising the subgrade should be considered as part of the treatment design process.
7  Treatment design

7.1  GENERAL

This section details the various pavement design methods and assumptions for treatment options identified in Section 6. Two design approaches are available. The first, detailed below, utilises traditional methods supplemented by laboratory testing. The second utilises testing from pavements with known performance history where in situ parameters have been assessed from back analyses of deflection tests (FWD). The second approach has a long history with unbound granular pavements, but for stabilised basecourse layers, suitably detailed FWD data have been collected for only a few years. For that reason, designers of stabilised pavements should focus on the traditional methods below, and use deflection results with caution, until good precedent performance is demonstrated in each region.

7.2  DESIGN CONSIDERATIONS

7.2.1  Pavement surfacing

An important component of pavement design is pavement surfacing. This should be designed to minimise the ingress of water into the pavement. Therefore, the use of a prime coat prior to chipsealing is encouraged. Thin asphalt surfacing layers are permeable, so primes and membrane seals should be placed under thin asphalt; in particular, care should be used where open graded porous asphalt is placed to ensure the pavement is relatively impermeable from the surface. Where chipseals are applied, the design should stipulate a second coat seal the following construction season. The seal design should be performed by an appropriately skilled surfacing engineer.

7.2.2  Chemically modified, bound and foamed bitumen treated materials

It is important to remember when designing a treatment that the stabilisation treatment performance depends on the underlying support. This is related to material quality, pavement construction, moisture conditions etc. Furthermore, under the same load conditions, cement- and foamed bitumen-treated materials do not lose stiffness as quickly as materials treated with cement only (Alabaster, et al. 2013).

There are a number of factors common to stabilisation failures. These are discussed below:

- Lack of support of the stabilised base caused by weak subgrades (high deflecting) and inadequate pavement depth. Therefore, optimum construction densities, and consequently full material strength, may not have been achieved. In addition, the stabilised state of the material under traffic deteriorates under the weight of vehicle loading, which results in the material returning to an unbound state;
- Inadequate thickness of the stabilised base, which results in the material deteriorating under the vehicle loading and rapidly returning the material to an unbound state;
- The material being stabilised has a poor grading (lack of a well graded crushed rock to give shear strength). This is a result of an old degraded pavement and/or the in situ hoe breaking down the material into “dust”. The lack of well-graded crushed rock results in material with insufficient shear strength to prevent rutting, relying on the strength provided by the foamed bitumen and cement bonds to achieve this instead;
• Pavement materials that differ from the materials tested in the laboratory;
• Poor quality control during construction; and
• Over stabilisation, where the chemical agent, particularly cement, causes block and/or shrinkage cracking.

All these factors need to be considered during the design and construction of chemically stabilised pavements.

7.2.3 Stress dependency

Strain softening materials lose stiffness with increasing strain. Conversely, strain-hardening materials increase stiffness with increasing strain. The stress dependency of the in situ subgrade will have a significant influence on the pavement performance. The mechanical behaviour of the subgrade should be considered during the rehabilitation design process; one possible approach is to use subgrade response to the shear vane.

7.2.4 Mechanistic pavement modelling

Designers are reminded the material properties typically provided in FWD reports are generally calculated assuming isotropic properties, while the Austroads methodology in pavement design in New Zealand assumes the materials are anisotropic. Care should be exercised when converting between the anisotropic and isotropic modulus values.

Similarly, where the Austroads’ methodology sublayers the granular layers and limits achievable modulus values, whereas the back calculation methods do not. Therefore, care is needed to understand if the modulus values used are averages or maximum values.

7.2.5 Geosynthetic reinforcement

Where geosynthetic reinforcement is proposed as part of the treatment design, the design shall follow the recommendations of the NZ Guide to Pavement Structural Design (NZ Transport Agency 2018). It is expected where geosynthetic reinforcement is placed, it will be at a depth that will allow for future pavement stabilisation.

7.2.6 Mix design

Where any proposed stabilisation treatment will incorporate the existing seal into the stabilised layer then laboratory mix design should include the appropriate proportion of the seal layer into the laboratory sample. As a general rule of thumb, the stabilised layer should contain no more than a third, by depth, of the seal layer. The design should also consider changes in mix proportions that might arise from geometric improvements that might be occur concurrent with any rehabilitation.

7.2.7 Customary practice moduli

Customary practice modulus values can be inferred from New Zealand in situ deflection testing.

Basecourse moduli back-calculated from a large database of historic FWD tests on roads throughout New Zealand were collated and analysed. The aim was to determine the typical values achieved for various types of basecourse: unbound, cement modified/ bound and also foamed bitumen stabilisation. The top layer of the pavement is represented in Figure 11 below. The results depend on the underlying support and the curve should not be used for design purposes without a full understanding of the pavement system.

Traditionally, cement bound and foamed bitumen stabilised basecourse layers have been assumed to have intrinsic moduli that were less affected by the stiffness of the underlying layer than is recognised for unbound granular materials. It is now clear from New Zealand in situ testing, that there is a substantial effect for most commonly used
materials. FWD central deflection (or BB deflection) provides a convenient guide for construction requirements. The lower bound isotropic moduli for basecourses (represented by the lines in the figures below) should be readily achieved by local contractors. They should be taken as default values unless the use of higher values can be substantiated. These values are the best current estimates of effective long term stiffnesses applicable to unbound granular and stabilised basecourses formed on an unbound granular subbase (not directly on a cohesive subgrade).

Checks for basecourse and subbase rutting, shoving (shallow shear), and flexural cracking of chip seal layers can be considered by examining results from back-analyses of deflection bowls. Appropriate means of characterising unbound or stabilised pavement layers is the subject of on-going research. New Zealand calibrated precedent strain with FWD is showing a lot of potential, however, the process is currently being validated and will not be included in this edition.

![Figure 11 Characteristic in situ moduli (Isotropic) for New Zealand Basecourse Layers (Unbound, Cement Stabilised and FBS)](image)

**7.2.8 Particle size distribution**

For design traffic levels greater than $5 \times 10^6$ ESA over a design period of 25 years, the particle size distribution for basecourse materials shall not travel from the fine side to the coarse side of the grading envelope. This will be demonstrated through the material having a Sand Grading Exponent not less than 0.4 as detailed in Stevens and Salt (2011). Failure to meet the Sand Grading Exponent will require testing of the proposed grading with the repeated load triaxial in accordance with NZTA T15 to establish the suitability of the material. For state highways, NZ Transport Agency’s Pavement Group must approve use of a basecourse that meets the NZTA M4 requirements but has a Sand Grading Exponent less than 0.4. The RLT and historic performance results for the proposed aggregate should be used to support the approval process.
7.2.9 Modelling of thin asphaltic surfacing

Mechanistic pavement design modelling shall not rely on the stiffness of the Thin Asphaltic Surfacing (TAS) being any greater than the stiffness of the underlying materials. If the underlying materials are stiffer than the TAS, the TAS stiffness is to be used in the modelling.

7.2.10 Notes regarding test methods and interpretation of results

Typically overseas researchers use ‘standard’ drop hammers for compaction, while the New Zealand industry uses vibrating hammers. Vibratory compaction provides more energy to the sample, so the resultant stiffness and strength values are greater. Therefore, the maximum dry density can be variable. Where possible, the target field density for proposed materials should be compared with achieved densities in previous construction projects that have performed satisfactorily using the same material. Alternatively, at least two maximum dry density tests should be performed.

7.2.11 Repeated load triaxial

The RLT test can be used as diagnostic tool to help clarify why a particular pavement has failed. It can also be used to rank the expected performance of various material sources. The RLT results can display some variability and are dependent on the level of compaction of the aggregate sample. Therefore, care needs to be taken in sample preparation and in interpretation of the results.

If a basecourse material that does not meet the NZ Transport Agency M4 Specification is proposed for a low volume road where the DESA is less than $5 \times 10^5$, then the suitability of the material shall be confirmed using the NZTA T19 RLT test procedure. The 25-year design traffic, in terms of ESA, shall be less than the calculated traffic required to generate a 10 mm rut.

For high volume traffic, where the 25-year design ESA is greater than $5 \times 10^6$, RLT testing is required to supplement the M4 material testing. In this case, the 25-year design traffic, in terms of ESA, shall be less than the traffic calculated to generate a 10 mm rut.

When relying on the RLT test there are a number of factors that should be considered:

- Adequacy of a sample population from a potentially variable environment,
- The repeatability of the RLT test which is associated with compaction repeatability, and
- The ability of the test to represent field compaction densities adequately in the laboratory.

These factors and their potential influence on the pavement design need to be considered when developing a design treatment. The RLT results should not be used in isolation, but should be considered with other information obtained on the materials.

7.2.12 Ripping the existing pavement seal

When constructing an overlay, there are two schools of thought in regard to the existing pavement seal.

One viewpoint is that the existing seal has intrinsic strength and removing the seal weakens the pavement. Various networks successfully apply granular overlays without the existing seal being treated. It is anticipated this approach would be used only where these treatments have provided good performance historically and the aggregate used in the overlay is demonstrated to have low moisture susceptibility.
The other point of view is that an overlay constructed with the existing seal intact has a high risk of premature failure due to any ingress of moisture being trapped in the layer of overlay material. This is exacerbated if the existing surface has ruts sufficiently deep to retain moisture. Furthermore, if the existing seal is unstable, it will continue to move if the depth of overlay is inadequate. It will also move if it continues to be subjected to stresses sufficient to induce plastic strain within the layer. If an unstable seal is removed the thickness of that seal should be replaced with equivalent granular material and an overlay thickness designed using the original pavement thickness.

7.3 PAVEMENT DEPTH AND CHARACTERISATION OF SUBGRADE

The field and laboratory investigations aim to determine the pavement structure and material properties of the pavement materials as well as the characteristics of the subgrade materials.

Define the existing pavement cross-section for modelling in a layered elastic model such as CIRCLY Pavement Design Software. The following information is needed:

- Existing pavement layer thicknesses - seal, asphalt (if applicable), basecourse, subbase and subgrade improvement layer;
- Estimated modulus of each pavement material layer either from the 10th percentile back calculated moduli from deflection testing and/or in situ Scala penetrometer tests (Vertical Modulus = 10×CBR and Horizontal Modulus = 5×CBR)⁴;
- If necessary adjust the modulus of the subgrade soil from 4 day soaked CBR test results (Vertical Modulus = 10×CBR and Horizontal Modulus = 5×CBR);
- If appropriate determine the modulus of the subgrade and/or basecourse and subbase aggregates by undertaking RLT tests at the expected field compaction density.

Bailey and Patrick (2001) have documented how volcanic soils have various relationships between the CBR and the resilient modulus. This is why using the relationship from AUSTROADS where Modulus = 10 CBR is not always appropriate. Instead of using a factor of 10, Sutherland et al. (1997) recommend using the equation Modulus = 3 CBR to estimate the subgrade modulus when calculating pavement deflections and strains for materials sourced from Whanganui. For volcanic soils, Bailey and Patrick (2001) suggested the following relationships between isotropic modulus and CBR were appropriate:

- \( M_r = \text{CBR for a typical pumice/sandy soil}, \)
- \( M_r = 3 \text{ CBR for a mixture of silty soils and brown ash, and} \)
- \( M_r = 10 \text{ CBR for clayey ash soils.} \)

Where \( M_r \) is the resilient modulus in MPa.

---

⁴ The relationship of the vertical modulus being equal to 10 times the CBR is a default value and should be confirmed prior to use. In practice the relationship between elastic modulus and CBR varies throughout the country.
7.4 BASE LAYER QUALITY/RUT RESISTANCE

The field and laboratory investigation aims to determine the rut resistance of the unmodified source pavement base material if it is kept dry as poor, medium, or good. Would this rut resistance be affected if seal layers are hoed in through in situ stabilisation/modification? Methods used to assess these questions may include:

a. Visual distress showing significant shoving and rutting that is not necessarily a result of water ingress - future rut resistance is estimated to be poor (note further tests below may change this view point);

b. From aggregate sampled from test pits, undertake appropriate NZTA M4 tests which may include: crushing resistance; weathering quality index; CBR; quality of fines including sand equivalent, clay index, plasticity index; broken face content; particle size distribution; and moisture content. If in situ aggregate is tested to be close to a compliant NZTA M4 - rut resistance is estimated to be medium or good (note further tests below may change this view point);

c. Undertake a dry performance based NZTA T15 RLT Permanent Strain Test on the existing in situ aggregate; in addition, where appropriate, a mixture of broken up seal layers (to simulate the base layer mixture if in situ stabilisation is used).

7.5 BASE LAYER DESIGN ESTIMATES

There are two methods of design for stabilisation treatments. These are either the unbound/modified granular method described in 7.5.1 and 7.5.2 below, or, for bound subbases, the limiting tensile stress method described in 7.8.

Both methods rely on conducting laboratory ITS tests and/or flexural beam tests of stabilised existing in situ aggregate mixes and/or new overlay aggregate to determine the most appropriate binder content.

7.5.1 Unbound granular design

Unbound granular design relies on aggregate interlock and the quality of the aggregate material to provide the rut resistance. Testing with the RLT can provide an estimate for the predicted rutting life of the material, this Unbound layer life is then used to estimate the Modified layer life.

7.5.2 Modified design

If the material is improved through modification then the life improvement is described in the following equation:

\[
\text{Modified layer life} = \text{Unbound layer life} \times \text{ITS improvement factor}
\]

where

\[
\text{ITS improvement factor} = 0.017 \times \text{ITS} + 1.7
\]

and the ITS is obtained from the NZTA T19 soaked ITS test.

The ITS improvement factor shall be limited to a maximum value of 3. Furthermore, to accommodate the deterioration of the bonds, the pavement design shall use the unbound modulus of the granular material.
7.6 GRANULAR OVERLAY DESIGN

7.6.1 Mechanistic pavement design or pavement thickness design chart

The overlay design thickness can simply be determined by the difference between the pavement thickness for the new design traffic, following the Austroads methodology, and the existing pavement thickness.

Unbound pavement designs using mechanistic pavement methods (CIRCLY) or using the pavement thickness design chart, figure 8.4 from Austroads (2017). The Austroads approach limits subgrade strain and implicitly assumes that the pavement materials have sufficient strength and durability. These assumptions need to be checked. Furthermore, if the pavement failure has been driven by drainage faults these also need to be addressed if the overlay is to have a reasonable chance of success.

The thickness of a granular overlay is restricted to a depth greater than 70 mm but less than 200 mm. It is determined by the following steps.

The minimum overlay thickness is 100 mm of M4 AP20 over the high spots. This is allowed if:

- The overlay thickness calculated is 100 mm or less; and
- The existing in situ aggregate has good rut resistance and any rutting is due to deformation in the underlying subgrade.

The minimum overlay thickness shall be increased to the minimum base thickness as required by figure 8.4 (Austroads 2017) over the high spots if:

- the rut resistance of the existing in situ aggregate is poor for the future traffic or
- the existing seal is unstable.

Overlay thickness is limited to a maximum of 200 mm over the high spots unless there has or will be a significant change in road use which justifies a greater depth. This maximum overlay thickness is given to limit the design thickness to a practical and economic limit as, if the existing road has survived for at least 10 years at its current pavement depth, it is unlikely to need more than 200 mm overlay.

7.7 FOAMED BITUMEN STABILISATION

This section details foamed bitumen treated basecourse mix and associated pavement design requirements that are considered suitable for acceptance in a pavement treatment design.

7.7.1 Background

Foamed bitumen stabilisation is a pavement rehabilitation method. It involves hoeing the existing pavement materials and mixing in small quantities of bitumen, water, cement and other modifying agents to bind the materials to give increased strength.

Although, considered a low risk option by the roading industry some early failures have occurred. Some possible reasons for these failures are discussed in Section 7.2.2. It may also be that the foaming process has not achieved the classic properties of a foamed bitumen stabilised material. Ideally very small spots of bitumen are dispersed throughout the sample. However, on occasion “stringers” occur which indicates poor dispersion.
After investigating failures and consultation with the New Zealand Industry the NZ Transport Agency has decided foamed bitumen stabilised pavements shall be considered as lightly modified pavements. Therefore, the Portland cement content will be ideally limited to 1% although, where justified, a maximum of 1.25% will be permitted. More emphasis is required for testing the quality and rut resistance of the source aggregate.

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

### 7.7.2 Adopted guideline

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

### 7.7.3 General

The mix design for the bitumen stabilised base or subbase aggregate will not be acceptable unless the criteria of this part of the Guide are met. In these cases, the pavement design and/or the stabilised mix design should be changed so the Guide’s requirements are met.

### 7.7.4 Design values

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

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

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

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

Poisson’s Ratio shall be equal to 0.3.

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

The foamed bitumen layer shall be equal to or thicker than the basecourse thickness required by Austroads figure 8.4 (Austroads 2017).
7.7.5 Mix acceptance for unbound granular design assumptions

7.7.5.1 Minimum source aggregate quality for all design loads

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

For construction where virgin aggregate is to be bitumen stabilised, the source aggregate’s particle size distribution must meet the TG2 (2009) requirements in the “ideal” category and all other NZTA M4 (2005) requirements. While it would be preferable to have an ideal grading for pavement materials that are to be recycled a “less suitable” grading could be considered. Table 11 details the grading requirements with these values being obtained from TG2 (2009). NZTA B5 (2008) provides guidance on the requirements for imported aggregates used to blend with recycled aggregates to improve their performance.

Table 11 Particle Size Distribution Requirements

<table>
<thead>
<tr>
<th>Sieve size (mm)</th>
<th>BSM Emulsion</th>
<th>BSM Foam</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ideal</td>
<td>Less suitable</td>
</tr>
<tr>
<td>50</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>37.5</td>
<td>87-100</td>
<td>87-100</td>
</tr>
<tr>
<td>26.5</td>
<td>77-100</td>
<td>100</td>
</tr>
<tr>
<td>19.5</td>
<td>66-99</td>
<td>99-100</td>
</tr>
<tr>
<td>13.2</td>
<td>67-87</td>
<td>87-100</td>
</tr>
<tr>
<td>9.6</td>
<td>49-74</td>
<td>74-100</td>
</tr>
<tr>
<td>6.7</td>
<td>40-62</td>
<td>62-100</td>
</tr>
<tr>
<td>4.75</td>
<td>35-56</td>
<td>56-95</td>
</tr>
<tr>
<td>2.36</td>
<td>25-42</td>
<td>42-78</td>
</tr>
<tr>
<td>1.18</td>
<td>18-33</td>
<td>33-65</td>
</tr>
<tr>
<td>0.6</td>
<td>12-27</td>
<td>27-54</td>
</tr>
<tr>
<td>0.425</td>
<td>10-24</td>
<td>24-50</td>
</tr>
<tr>
<td>0.3</td>
<td>8-21</td>
<td>21-43</td>
</tr>
<tr>
<td>0.15</td>
<td>3-16</td>
<td>16-30</td>
</tr>
<tr>
<td>0.075</td>
<td>2-9</td>
<td>9-20</td>
</tr>
</tbody>
</table>

The reactivity of the proposed foamed bitumen and cement additives with the aggregate shall be tested according to the TG2 (2009) using a two phase design life but with the modifications from Wertgen (2004) reproduced as equations 7.3 and 7.4 below.

\[
MR_{\text{Phase 1}} = \left( \log \frac{\text{ITS}_{\text{equ}} \times 3950 - 7000}{7000} \right) \times TSR \times F_{\text{drainage}} \quad 7.3
\]

\[
MR_{\text{Phase 2}} = \frac{MR_{\text{Phase 1}} \times TSR}{(0.5 \times \text{ITS}_{\text{equ}} + 0.7)} \quad 7.4
\]

Where

- \( MR_{\text{Phase 1}} \): Resilient Modulus during Phase 1, MPa
- \( \text{ITS}_{\text{equ}} \): ITS at equilibrium moisture content, kPa
- \( TSR \): Tensile strength retained (ratio of soaked to unsoaked ITS values)
- \( F_{\text{drainage}} \): Drainage factor from Table 12 below
New Zealand guide to pavement evaluation and treatment design

\[ MR_{\text{Phase 2}} = \text{Steady state resilient modulus} \quad \text{MPa} \]

\[ UCS_{\text{eq}} = \text{UCS at equilibrium moisture content} \quad \text{MPa} \]

**Table 12** Drainage factors, \( F_{\text{drainage}} \), for estimating field stiffness values of bitumen stabilised material

<table>
<thead>
<tr>
<th>Drainage quality</th>
<th>Mean annual rainfall (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>&lt; 200</td>
</tr>
<tr>
<td>Very good</td>
<td>1.4</td>
</tr>
<tr>
<td>Good</td>
<td>1.3</td>
</tr>
<tr>
<td>Fair</td>
<td>1.2</td>
</tr>
<tr>
<td>Poor</td>
<td>1.1</td>
</tr>
<tr>
<td>Very Poor</td>
<td>1.0</td>
</tr>
</tbody>
</table>

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

7.8 BOUND TREATMENT - LIMITING TENSILE STRESS METHOD

7.8.1 Tensile strength design

Tensile strength design of a bound subbase relies on the modifying or stabilising additives’ chemical bonds to provide resistance to plastic deformation, particularly rutting. Such a design may be necessary when the source material has poor rut resistance. Bound behaviour will only be retained if the stress applied is less than the elastic limit of the chemical bond. It is commonly accepted that if the stress applied to the bound layer is 50% less than the flexural strength of the cemented material, the stiffness will be retained. This yield strength can be estimated from the flexural strength as determined from the four point beam tensile strength test. The ITS test gives a strength value that is approximately 50% of the flexural strength. Therefore, the applied stress should not exceed the ITS value. Both the beam test and the ITS should be performed on a sample prepared at the maximum dry density. However, the achieved field density may be less than the maximum dry density and the strength results should be adjusted accordingly.

The limiting tensile stress method of design is suited for cases where the bound subbase layer is a minimum of 100 mm below a cover of modified or unbound aggregate. Turning the subbase into a bound material is a good solution to span weak subgrades, reduce pavement deflections, reduce pavement depth and give a good anvil to compact the upper base layer on.

The \textit{in situ} stabilisation depth and the overlay depth can be determined by checking the horizontal tensile stress at the base of the stabilised bound subbase layer is less than 50% of the stabilised flexural beam tensile strength or equal to the ITS obtained at the maximum dry density. Field compaction will typically not achieve maximum dry density, so the ITS may need to be adjusted to account for the lower achieved density. The stress can be determined by CIRCLY with an assumed high modulus of between 3,000 MPa to 10,000 MPa. The higher the modulus the greater the stress and thus assuming a high modulus evaluates the worst case, if the material becomes brittle.

Documented in Austroads (2017) Section 8.2.4 is an alternative design approach which can be used if desired.
7.8.2 Overlay requirements for bound subbase

The NZ Transport Agency requires an overlay of aggregate to prevent cracks reflecting through to the surface from the bound subbase layer.

7.8.3 HiLab

HiLab is a cement-bound, gap-graded pavement treatment that can be used either as a subbase or as a basecourse. Its use in rehabilitation sites requires consultation with the NZ Transport Agency Pavements Team. The requirement for an unbound overlay to be constructed above a bound subbase does not apply when a HiLab basecourse is constructed over a HiLab subbase.

7.8.4 Laboratory testing to determine the correct mixture of materials

Indirect tensile strength and flexural beam tests with a binder shall ensure the source material used in laboratory testing is a mixture to the correct proportions of existing seal layers, in situ aggregate and, if used, the overlay aggregate.

The in situ stabilisation depth is determined by mechanistic pavement design. Based on the existing pavement cross-section characteristics design a new pavement using Austroads procedures with CIRCLY. The stabilisation depth and overlay thicknesses are equal to the depth needed to ensure the calculated tensile stress at the base of the cemented layer is less than the limiting design tensile stress as calculated below.

7.8.5 Flexural beam flexural strength

Design tensile stress should be limited to less than 50% of the Flexural Beam Tensile Strength. This assumes compaction at 95% Maximum Dry Density (MDD) which is the maximum density usually achieved in sample preparation, due to difficulties in compaction in a rectangular mould.

7.8.6 Indirect tensile strength

Design tensile stress = ITS with the samples compacted to MDD or the field density achieved in a laydown trial. The ITS should be calculated according to NZTA T19. Alternatively:

\[
\text{Design tensile stress} = 0.6 \times \text{ITS}
\]

This reduction applies if compaction has not been controlled as samples are likely to be compacted to 100% of MDD. The forty percent reduction accounts for the loss of strength if the sample density is reduced to 95% of MDD.

Design modulus (bound, isotropic and non-sublayered) for the stabilised layer (excluding foamed bitumen or bitumen emulsion):

The maximum of the following methods to estimate the modulus of the stabilised layer (note – the worst case is when the modulus is the highest value possible):

\[
\text{Design modulus} = \text{Young’s Modulus}
\]

Where the modulus has been determined from ITS or flexural beam testing or
Design modulus = 10,000 × Design Tensile Strength

In design the layer shall be assumed to be isotropic and have a Poisson’s ratio of 0.2.

7.8.7 Adding an overlay aggregate

NZ Transport Agency requirements for a bound subbase design require an unbound or modified granular overlay on top. Mechanical stabilisation should also be considered. This involves hoeing the overlay of good quality aggregate into the existing aggregate to improve rutting resistance.

7.9 MODIFIED TREATMENT - STABILISATION WITH OR WITHOUT OVERLAY (UNBOUND METHOD)

The unbound method of design is suited for cases where the final hoed base mix when unmodified and dry has good (or medium for lower traffic volumes) rut resistance. In addition to the aggregate quality the future rut resistance relies on the stone-on-stone contact while the fines are “tidied up” by the binder. This ensures rut resistance is not substantially reduced when wet to prevent early rutting failures. The expected increase in rut resistance can be estimated by calculating a pavement life multiplier based on the ITS value.

The in situ stabilisation depth and whether an overlay is needed is determined by mechanistic pavement design.

Based on the existing pavement cross-section characteristics design a new pavement (Austroads procedures with a program like CIRCLY) and the in situ stabilisation depth, overlay thickness is equal to the depth needed. This will ensure the vertical strain at the top of the subgrade is less than the limiting design value given in Austroads (2017) for new pavement design.

The modulus of stabilised layer adopted for modelling should not exceed 3 x modulus of the underlying layer, or the limits in modulus gain for unbound granular materials as defined in Austroads (2017).

If an overlay and in situ stabilisation are used, the design requirements of a granular overlay in Section 8.2 shall be met.

Austroads criteria for new pavements shall be used to check the life of the subgrade material. The expected life of the unbound and modified granular materials can be checked using the strain criterion derived in Section 8.3.

Optional Laboratory Testing

Two RLT tests are recommended to support the expected performance of the stabilised layer:

1. Dry RLT (NZTA T/15) of the unmodified source in situ material plus correct proportions of seal layers and overlay aggregate. This is to prove the stone-on-stone contact has sufficient rut resistance;

2. Soaked RLT (NZTA T/15) of the stabilised in situ material plus correct proportions of seal layers and overlay aggregate. This is to prove the fines have been “tidied up” and there is sufficient rut resistance when wet (consideration should be given to breaking up the bound sample after 7 days cure and recompacting to replicated expected unbound behaviour in the field).

---

5 Derived from Austroads (2017) criteria of E = 1000×UCS and ITS = 0.1×UCS; the applicability of this equation for the materials being tested should be checked.
7.10 DESIGN OF ASPHALT OVERLAYS

The design of asphalt overlays is documented in the AUSTROADS Part 5: Pavement Evaluation and Treatment Design (2011). Section 7.10 is included as supplementary information for the design of asphalt overlays.

In the New Zealand context asphalt overlays will be designed for three different rehabilitation situations, namely:

1. The rehabilitation involves the application of an asphalt layer to rehabilitate a pavement that contains only granular materials,
2. The pavement requiring rehabilitation already contains asphalt materials and an asphalt overlay is used to overcome the deficiencies of the pavement, and
3. The pavement rehabilitation design uses the precedent strain method described in Section 8.2.

For situations where Case 1 is applicable then Section 6.2.4.2 in Austroads (2011) has a design method for this which involves design deflections and the characteristic curvature of the pavement before an overlay is applied.

Case 2 is addressed in Section 6.2.6 of Austroads (2011). The design method allows for changes in curvature associated with milling off some or all of the existing asphalt.

The precedent strain method, Case 3, provides an allowable subgrade strain for the future traffic. The rehabilitated pavement will still be designed using the materials modelled in CIRCLY (Mincad 2004). In doing so, any asphalt overlays will still be modelled and their fatigue life assessed according to the normal methods of Austroads (2017).

7.10.1 Thin asphaltic surfacing

The majority of the NZ Transport Agency network is constructed with flexible pavements surfaced with chipseals that are in general capable of withstanding the inherent traffic induced deflections. In recent years the stress on some high demand sites has become too much for chipseals and there is a need for these sites to be surfaced with a thin surfacing mix. If there is doubt over the pavement strength and deflections then the pavement should be tested and compared to the constraints outlined in Table 13. The central deflection, $d_0$, values quoted are from Sheppard (1989) and the curvature, $d_0 - d_{200}$, values have been estimated based on Sheppard’s bowl ratios. Opportunities should be taken to sample and test the subgrade if any pavement repairs are required and the pavement is opened up.
Table 13 Recommended deflection constraints for thin asphaltic surfacing mixes

<table>
<thead>
<tr>
<th>Traffic volume</th>
<th>Heavy trafficked pavements (ADT &gt; 5000)</th>
<th>Medium Trafficked pavements (ADT: 500 – 5000)</th>
<th>Lightly Trafficked pavements (ADT &lt; 500)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deflection criteria</td>
<td>Maximum 95 Percentile Beam Reading ( (d_0) )</td>
<td>Maximum 95 Percentile Beam Reading ( (d_0) )</td>
<td>Maximum 95 Percentile Beam Reading ( (d_0 - d_{200}) )</td>
</tr>
<tr>
<td>Surfacing mix type</td>
<td>( \text{Asphaltic Concrete} )</td>
<td>( \text{Open Graded Porous Asphalt} )</td>
<td>( \text{Slurry Seal} )</td>
</tr>
<tr>
<td>Asphaltic Concrete</td>
<td>0.70 mm</td>
<td>1.00 mm</td>
<td>0.70 mm</td>
</tr>
<tr>
<td>Open Graded Porous Asphalt</td>
<td>1.10 mm</td>
<td>1.60 mm</td>
<td>0.70 mm</td>
</tr>
<tr>
<td>Slurry Seal</td>
<td>0.70 mm</td>
<td>0.15 mm</td>
<td>0.70 mm</td>
</tr>
</tbody>
</table>

Table 13 has been developed for mixes using traditional bitumen binders, the fatigue behaviour will change when modifiers such as polymer are used. Also, while Table 13 has been generated using traditional New Zealand mixes, now denoted Dense Graded in M10, however, recent work has shown that the values adopted are applicable to AC mix designs as well.

Sheppard’s work was published in average daily traffic. Chappell (2017) has converted the ADT bands of 5000 to 500 vehicles per day to ESA numbers using the following assumptions:

- The heavy vehicles form 5% of the traffic spectrum,
- A factor of 1.44 ESA per heavy vehicle is average for the heavy vehicles carried, and that
- Surfacing will achieve a 10 year life and traffic growth is negligible over that period.

This results in the 5000 and 500 vehicles per day being equivalent to 1.31 and 0.13 million ESA respectively.

### 7.10.2 Asphaltic overlays

Two important design criteria are that the asphalt overlay thickness shall not be less than the thickness required to:

- Reduce the subgrade strain to the design level, and
- Limit the horizontal tensile strain at the bottom of the asphalt overlay, \( \varepsilon_{t, AC} \), to a value not exceeding the future traffic design strain as determined by the performance (fatigue) relationship given in Austroads (2017), Section 6.5.10.

Another and acceptable method of granular overlay design is to follow the method in the Austroads (2011) Sections 6.2.11 and 6.2.13. These sections provide design charts for determining the thickness of the asphaltic overlays given the observed FWD central deflections and calculated curvatures.
7.10.3 Structural asphalt

8 Strain criteria for mechanistic pavement design

8.1 MECHANISTIC DESIGN FOR REHABILITATION

In general mechanistic procedures, the reactions of different pavement rehabilitation designs under a standard wheel load (defined in Austroads 2017) are analysed using a computer program such as CIRCLY. Strains at various critical layers as detailed in Figure 12 (reproduction of Figure 8.2 in Austroads 2017) are computed for each design being considered. Figure 12 denotes the critical strain under the left hand inner wheel, however, depending on the axle configuration and pavement structure the critical strains may be between the wheels. Designs that incorporate bound materials that are acceptable are those which meet or exceed the performance criteria detailed in Austroads (20012): Section 6.5.6 for shpaltic cement and Section 6.4.5 for Portland cement.

Figure 12 Location of critical strains in a pavement in response to a standard wheel load. 
Where $\varepsilon_{AC}$ represents the tensile strain at the base of the Asphalt layer, $\varepsilon_{CM}$ represents the tensile strain at the base of the Cemented Material, and $\varepsilon_{SG}$ represents the vertical strain at the top of the subgrade.

The performance criterion for the subgrade is given by the Precedent Strain method described in Section 8.2, here in this document. Equation 6.3 has been rearranged from the subgrade strain criterion found in of Austroads (2017). 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), 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 and rutting of the
subgrade. This design process is similar to the design of new pavements as detailed in Section 8.2 of Austroads (2017).

### 8.1.1 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; and if applicable,
- The thickness determined necessary to prevent shear failure of the pavement base layers.

The material used for thin unbound granular overlays shall comply with TNZ M/4:2006 Specification for Basecourse Aggregate (NZTA 2006).

Mechanistic analysis of an unbound granular overlay involves modelling a trial overlay thickness over the existing pavement structure. The material characterisations should follow the Austroads (2017) methodologies.

### 8.2 PRECEDENT METHOD

This section details the establishment of suitable strain criteria for specific pavement layers, including subgrades in regions where the fatigue characteristics may differ from those for which the Austroads (2017) strain criterion was derived.

Section 8.2.1 gives the same precedent method established previously and known as the Precedent Method, this approach is based on historic subgrade strain performance.

Section 8.3 is an extension to the former method, using the same precedent method for subgrade performance, but in addition, now allows the designer to use the same precedent principle for other pavement layers, if the designer can reasonably establish that a particular layer is governing the specific distress mode affecting an individual section of pavement.

The precedent method can be used to determine a subgrade vertical compressive Strain Criterion based on the historic loading of the pavement.

The Austroads Subgrade Vertical Compressive Strain Criterion for mechanistic design of new pavements using CIRCLY is defined in Equation 6.1.

\[ N = \left[ \frac{0.00933}{\varepsilon} \right]^7 \]  \hspace{1cm} \text{8.1}

This equation shall be used to check the strain on top of the subgrade unless it can be proven that the source of pavement deformation is deep seated in the subgrade and there are good records to determine the past traffic. In this case the precedent method can be used to determine the appropriate design subgrade strain criterion as detailed below:

### 8.2.1 New Zealand Precedent Strain design

The New Zealand Precedent Strain method is a process to calibrate the past performance of a pavement when determining the potential rehabilitation treatments. The precedent strain method assumes that the structural integrity of the pavement materials is unchanged and that pavement distress is purely the result of deformation in the subgrade. Subgrade deformation is the dominant factor in producing a failed pavement with excessive rutting.
and roughness. The assumption that the materials used in the pavement have not lost their structural integrity during the life of the pavement needs to be confirmed. The material tests indicated in Chapter 4 will provide assurance that the materials are still fit for purpose. Should the existing pavement materials be suitable then the assumption is that rehabilitation of the pavement with shape correction will result in a pavement that can again sustain its historic traffic loading, that is, the pavement will be returned to as new condition.

The NZ Transport Agency allows the use of the ‘Precedent Strain’ method to forecast future allowable traffic. In effect, this method allows the number of cycles of the historic vertical subgrade strain as experienced by the pavement to determine a new subgrade strain criterion.

Thus, for rehabilitation treatments, the vertical compressive strain, computed at the top of the subgrade, shall not exceed the design strain ($\varepsilon_{des}$) as defined by the following equation

$$\varepsilon_{des} = \varepsilon_{cvs} \left( \frac{N_F}{N_P} \right)^{-0.23}$$

where:

- $N_F =$ Future design traffic (ESA),
- $N_P =$ Historic traffic loading of pavement (ESA), and
- $\varepsilon_{cvs} =$ Existing vertical compressive strain at the top of the subgrade (microstrain).

The future design traffic ($N_F$) is calculated according to the methods described in Austroads (2017). The historic traffic loading ($N_P$) is the estimated traffic carried by the pavement since it was last strengthened or smoothed (or since it was constructed if it has not been strengthened or smoothed), and until the level of serviceability for that class of road justifies rehabilitation. The existing vertical compressive strain at the top of the subgrade is calculated according to the methods described in Austroads (2017).

The new subgrade strain relationship is then used to determine the additional overlay depth required to ensure the subgrade design strain ($\varepsilon_{des}$) is not exceeded. The existing basecourse may then only be necessary to act as a subbase.

For rehabilitation treatments, the vertical compressive strain computed at the top of the subgrade, $\varepsilon_{v SG}$, shall not exceed the design strain, $\varepsilon_{des}$, as defined by Equation 8.2.

Alternatively, for use in Equation 8.3, Equation 8.4 shall be used to calculate $\varepsilon_{cvs}$ 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:

$$\varepsilon_{cvs} = \alpha - fs$$

where:

- $\alpha =$ 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; and
the pavement confidence factor is selected by the designer to provide a suitable level of confidence that the characteristic value will not be exceeded.

Table 14 Value of pavement confidence factor \((f)\) for Equation 8.3

<table>
<thead>
<tr>
<th>(f)</th>
<th>% of all deflection measurements which will be represented by the characteristic deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>97.5</td>
</tr>
<tr>
<td>1.65</td>
<td>95</td>
</tr>
<tr>
<td>1.3</td>
<td>90</td>
</tr>
</tbody>
</table>

Please note: the methodology discussed here has changed from that presented in previous versions; the \(f\) values in Table 14 are different from previous values. For confidence in the values used in Table 14 it is necessary to obtain 30 or more measurements.

The use of Equation 8.3 for calculating the design strain (Equation 8.3) will calculate one design vertical compressive strain \((\varepsilon_{des})\) 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.

This subgrade strain criterion (Equation 8.3) considers the past performance of the pavement. It is based on the design assumptions used in the State Highway Pavement Design Rehabilitation Manual (NRB 1989). The exponent \((-0.23)\) used in Equation 8.3 is derived from the vertical compressive strain criterion used to develop the pavement design charts in NRB (1989). Use of this exponent is considered appropriate. This is because slightly thicker overlays are calculated compared with those computed when using the exponent \((-0.143)\) adopted for the Austroads subgrade strain criterion (Equation 5.1, Austroads 2017). This conservative approach may also compensate for any degradation in basecourse strength.

8.2.2 Strain criterion for use in mechanistic pavement design

The strain criterion to use in a layered elastic design package such as CIRCLY is detailed in Equation 6.4:

\[
N = \left(\frac{k}{\varepsilon}\right)^{4.35} \tag{8.4}
\]

Where:

\(N\) = Calculated allowable traffic limit in ESAs for a subgrade vertical strain, \(\varepsilon\), and

\[
k = N_{Design}^{0.23} \times \varepsilon_{des} \tag{8.5}
\]

Where

\(N_{Design}\) = The future design traffic loading in ESAs

\(\varepsilon_{des}\) = The limiting design strain as calculated from Equation 8.4.
8.3 BASECOURSE AND SUBBASE GRANULAR STRAIN CRITERION

The life of the basecourse aggregate in terms of rutting and when a smoothing treatment is required must be calculated either through a laboratory approach (RLT) or in situ testing (FWD) as appropriate. Both should be considered for marginal designs. The methodology described in Section 8.3.1 shows promise in predicting performance. However, at this stage it should only be used to support an experienced Pavement Engineer’s estimate of material performance, rather than as a pass/fail tool. It should also be noted that the forecast performance will only be achieved if the laboratory test conditions are achieved in the field.

8.3.1 Laboratory approach

Basecourse rutting can be estimated using Equation 8.6. The maximum shear strain of the basecourse is usually at a depth approximately 80 mm below the surface. However, calculating the strain at other depths is recommended, to ensure the maximum resilient strain in the basecourse \( (BC) \) is used in Equation 8.6.

\[
N_{BC} = \left( \frac{\left(2 \times ITS_{\text{factor}}\right)^{1/\exp_{BC}} \times k_{BC}}{\text{Maximum\_resilient\_strain\_in\_BC}} \right)^{\exp_{BC}}
\]

Where:

\( N_{BC} = \) life of basecourse in ESAs (equivalent standard axles)

\( k_{BC} = \) constant found from RLT testing (NZTA T15, presumptive values shown in Table 15)

\( \exp_{BC} = \) constant found from RLT testing (NZTA T15, presumptive values shown in Table 15)

\( ITS_{\text{factor}} = 0.01672 \times (ITS) + 1.5691 (ITS \text{ Improvement Factor}) \) — maximum value = 5.0

\( ITS = T/19 \) soaked ITS value \( (ITS \text{ Improvement Factor} \) only valid if working horizontal tensile stress < ITS checked when modulus of stabilised layer = 10,000 ITS for cement or 4,000 ITS for foamed bitumen)

\( \text{Maximum\_resilient\_strain\_in\_BC} = \) maximum resilient strain found in the basecourse layer as calculated when using a program such as CIRCLY. (Hint: Split basecourse layer to add a top 80 mm layer from the surface and apply strain criterion to top of all basecourse layers in the model)

The life of the subbase \( (SB) \) aggregate layer is found from equation 8.7:

\[
N_{SB} = \left( \frac{\left(2 \times ITS_{\text{factor}}\right)^{1/\exp_{SB}} \times k_{SB}}{\text{Maximum\_resilient\_strain\_top\_of\_SB}} \right)^{\exp_{SB}}
\]

Where:

\( N_{SB} = \) life of subbase material in ESAs

\( k_{SB} = \) constant found from RLT testing (NZTA T15, presumptive values shown in Table 15)
expSB = constant found from RLT (NZTA T15, presumptive values shown in Table 15)

\[ \text{ITSfactor} = 0.01672 \times (\text{ITS}) + 1.5691 (\text{ITS Improvement Factor}) \] – maximum value = 3

ITS = T/19 soaked ITS value (ITS Improvement Factor only valid if working horizontal tensile stress < ITS checked when modulus of stabilised layer = 10,000 ITS for cement or 4,000 ITS for foamed bitumen)

Maximum Resilient Strain Top of SB = resilient strain at the top of the subbase layer

It is recommended designers conduct their own RLT tests (NZTA T15) to obtain the constants in the design strain criteria for subbase and basecourse aggregates. As a guide the range of presumptive values for unbound aggregates are shown in Table 15. The use of modified granular materials will require independent RLT tests (NZTA T15) under soaked conditions as the intent is to demonstrate the binder gives required strength when wet. This allows the determination the appropriate constants for the design stain criterion. Further, the constants in Table 6.2 were derived from dry tests in the RLT apparatus. A separate design is needed if the case for saturation of the granular layers is considered using equation constants derived from saturated/soaked RLT tests (NZTA T15).

Table 15 Presumptive constants and exponent values for CIRCLY design strain criteria

<table>
<thead>
<tr>
<th>Strain criterion</th>
<th>Subbase (from NZTA T/15 RLT tests using linear extrapolation to 3.3%)</th>
<th>Basecourse (from NZTA T/15 RLT tests using linear extrapolation to 3.3%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Strain criterion constants</td>
<td>Strain criterion constants</td>
</tr>
<tr>
<td></td>
<td>kSB</td>
<td>K (for CIRCLY)</td>
</tr>
<tr>
<td>Upper (best)</td>
<td>0.08</td>
<td>0.098</td>
</tr>
<tr>
<td>Middle</td>
<td>0.066</td>
<td>0.081</td>
</tr>
<tr>
<td>Lower (poor)</td>
<td>0.055</td>
<td>0.067</td>
</tr>
</tbody>
</table>
9 Compare design options

9.1 INTRODUCTION

There is generally more than one design option possible and in theory all can comply with the current Austroad’s Guides to Pavement Technology and the NZ Guide to Pavement Structural Design. Furthermore, the current Austroad’s Guides may lead the designer to believe that reducing the overlay and/or stabilisation thickness by 30 mm will result in a reduction in life from 25 years to 10 years. Actual life achieved for a particular pavement treatment is not governed just by the design thickness. Other factors are the quality of: the source material; the stabilised mix; drainage and surface waterproofness; and the compatibility of the treatment to the pavement distress that needs fixing. Traffic loading and past performance of neighbouring pavement treatments recorded in the RAMM database or local knowledge are huge factors in estimating the expected life.

9.2 TREATMENT TYPES AND LIFE ESTIMATE

Past performance of neighbouring pavement treatments recorded in the RAMM database or local knowledge are considered the most accurate predictors of pavement life. They should take precedence over the guidelines to estimate life set out in Table 16 below.

Life is estimated based on total traffic loading in Millions of Equivalent Standard Axles (MESAs). It assumes the design follows the current Austroads and New Zealand Guideline design procedures.

<table>
<thead>
<tr>
<th>Treatment</th>
<th>Pavement Distress</th>
<th>Life Expected and Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unbound granular overlay</td>
<td>Subgrade rutting or flushing</td>
<td>Should meet design life (unless unbound granular overlays perform poorly in the region); up to 5 MESA is an approximate limit for unbound granular material rutting;</td>
</tr>
<tr>
<td>Unbound granular overlay</td>
<td>Rutting within <em>in situ</em> existing aggregate layer and/or RLT on <em>in situ</em> aggregate shows poor rut resistance.</td>
<td>Unless overlay is the requisite basecourse depth for the traffic and the aggregate used has good rut resistance (evidenced by good past performance and/or good RLT result) to reduce shear stresses in underlying aggregate, then the expected life limit will 4 MESAs.</td>
</tr>
<tr>
<td>Treatment</td>
<td>Pavement Distress</td>
<td>Life Expected and Comments</td>
</tr>
<tr>
<td>-----------</td>
<td>------------------</td>
<td>----------------------------</td>
</tr>
<tr>
<td><strong>In situ stabilisation – modification &lt;2% cement</strong></td>
<td>Subgrade rutting</td>
<td>Modification tidies up the fines in the aggregate layer to maintain strength when wet but behaviour is still unbound granular; stresses on the subgrade will remain the same as pavement depth is the same. Life achieved is predicted to be the same as before, or may be less if existing pavement is down and waterproofed with many layers of seal. (This life can be checked with RLT testing.)</td>
</tr>
<tr>
<td><strong>In situ stabilisation – modification &lt;2% cement</strong></td>
<td>Rutting within <em>in situ</em> existing aggregate layer and/or RLT on <em>in situ</em> aggregate shows poor rut resistance.</td>
<td>Modification binds up the fines in the aggregate layer to maintain strength when wet, but behaviour is still unbound granular; rutting relies on the source aggregate stone-on-stone contact. Life can be estimated by the ITS improvement factor.</td>
</tr>
<tr>
<td><strong>In situ stabilisation – modification &lt;2% cement</strong></td>
<td>Rutting within <em>in situ</em> existing aggregate layer and/or RLT on <em>in situ</em> aggregate shows good rut resistance when dry.</td>
<td>Modification tidies up the fines in the aggregate layer to maintain strength when wet, as behaviour is still unbound granular and rutting relies on the source aggregate’s stone-on-stone contact; where the RLT has found good rut resistance when dry, the design life should be met up to 15 MESA.</td>
</tr>
<tr>
<td><strong>In situ stabilisation – bound subbase &gt;2% to 6% cement</strong></td>
<td>All distress types, but only necessary where the source material being stabilised is poor quality and has poor rut resistance when dry, as indicated by the RLT test.</td>
<td>Provided the construction meets the design expectations and the pavement design limits the tensile stress to 50% below the flexural beam strength, the design life should be met. There may be some shrinkage cracking in materials high in fines and PI greater than 12%. If not designed or constructed correctly, failure can be immediate by either cracking or returning to an unbound state.</td>
</tr>
<tr>
<td>Treatment</td>
<td>Pavement Distress</td>
<td>Life Expected and Comments</td>
</tr>
<tr>
<td>-----------------------------------</td>
<td>--------------------------------------------</td>
<td>-----------------------------------------------------------------</td>
</tr>
<tr>
<td>Foamed bitumen stabilisation</td>
<td>All distress types. Treatment can be performed with or without granular overlay.</td>
<td>Provided the construction meets the design expectations and the design limits excessive strains in the pavement, the expected design life can be up to 50 MESA</td>
</tr>
<tr>
<td>Structural asphalt</td>
<td>All distress types</td>
<td>Provided the construction meets the design expectations and the pavement design limits the tensile strains to appropriate levels, the design life should be met.</td>
</tr>
</tbody>
</table>

### 9.3 ASSESSMENT OF RECOMMENDED PAVEMENT TREATMENT

Table 2 and Table 4 from Chapters 2 and 3 respectively can be used to assess the risk of a proposed treatment or range of treatments. However, the following questions about the proposed pavement design need to be answered first through reviewing the design report and/or discussions with the pavement designer. If poor drainage and/or surface waterproofness are cited as the cause of the pavement distress, the recommended treatment should fix the drainage and restore surface waterproofness.

What is the design traffic loading for the design life being targeted?

- If greater than 10 MESA the design options should stabilise the base layer.

What is causing the predominant form of pavement distress?

- Subgrade rutting; base aggregate rutting; subbase aggregate rutting; poor drainage; lack of surface waterproofness; and/or unstable seal layers?

What is causing the predominant form of pavement distress?

- Subgrade rutting; base aggregate rutting; subbase aggregate rutting; poor drainage; and/or lack of surface waterproofness?

What is the dry rut resistance of the source unmodified/unbound top 200mm of the new rehabilitated pavement (note the RLT apparatus can be used to assess this)?

- Poor or good (adequate) dry rut resistance?

What is the dry rut resistance of the source unmodified/unbound subbase aggregate of the new rehabilitated pavement (note the RLT apparatus can be used to assess this)?

- Poor or good (adequate) dry rut resistance of the subbase?

How was the stabilised pavement layer designed?
• Either low binder content and assumed behaviour was unbound granular, or high binder content and assumed behaviour was bound designed, using the flexural beam test (or ITS).

When asking the above questions it is always a good idea to ask the designer how they derived their answers, to check whether adequate testing and investigation was undertaken.

Using Table 16 from Section 9.2 the life of various pavement treatments can be estimated. When costs are available, an economic analysis along with the desired life and available budget can be undertaken to decide the optimum treatment option.

9.4 DETERMINING THE LOWEST COST TREATMENT

As funds are constrained the lowest cost treatment determined by analysis over a suitable period is required. This treatment could simply involve treating the pavement distress areas (similar to pre-seal repairs) and then resealing the whole site, while accepting on-going maintenance costs will likely be high. To determine the risk of this proposed treatment’s early failure the history of similar sites and the guidelines given in Table 16, in Section 9.2 should be used.

The sites being renewed are generally those sites in the worst 1 to 2% of the whole roading network. The mere fact that these sites have failed while neighbouring road sections of similar age have not signals that these treatment lengths are the weakest components of the network. Investigations will often show the aggregate quality is very poor with excessive plastic fines and/or the subgrade is very weak. Correct investigation and design of pavement treatments to obtain a 25 year design life often results in extensive and expensive treatments. Changing the design life to 10 years simply reduces the design pavement depths by 20 to 50 mm but still requires most of the costs of the more robust treatment.

There is a temptation to adopt a lower cost initial treatment. However, any treatment that adopts a lowest initial cost treatment as an approach should recognise the potential consequences, particularly regarding premature failure and the associated treatment costs, and the probabilities of those failures.
10 Detailed pavement design report

The following sections detail the NZ Transport Agency’s expectations regarding the content of a pavement design report. The report should contain sufficient investigation information and a clearly documented decision making process supported by modelling for a reader to be confident the correct risk-based design process has been adopted.

10.1 BACKGROUND

The pavement design report background should detail:

- Site location, including a map and representative site photographs;
- The history of the site, construction date, rehabilitation date, surfacing history;
- Reasons for the rehabilitation;
- Future changes to the loading, particularly as it is related to changing traffic configurations as a result of high performance motor vehicles; and
- Any other details gleaned during the desktop investigation.

10.2 SITE INVESTIGATIONS

10.2.1 Ground investigations

The ground investigation record should include:

- Summary of defects observed on site;
- Site photos (more contained in appendix if considered necessary);
- Summary of test pits and Scala investigations and description of pavement type;
- Summary of FWD or BB test results; and
- Interpretation of test results and how the results relate to the faults observed.

Full Falling Weight Deflectometer or Benkelman Beam results, test pit records and the Scala investigation results should be reproduced in an appendix.

10.2.2 Laboratory test results

Summary and plots of laboratory test results should be included, with the full results presented in an appendix.

10.3 DESIGN TRAFFIC

The assumptions contained in the design traffic calculations should be clearly documented. As described in the Economic Evaluation Manual (NZTA 2016), traffic growth should be estimated from an analysis of the traffic count
data for at least the last five years and preferably for the last ten years. Full calculations should be presented in an appendix. A design life of 25 years should be used unless a greater period is required.

10.4 DESIGN

The design should start from a summary of the key investigation results and the root cause of faults, to inform a design decision.

The design process should detail how the treatments will address the site-specific risks. The construction techniques and materials should be appropriate for the site risk.

Assumed material properties for the subgrade, unbound granular layers and asphaltic layers should be clearly documented. For example, describe how the subgrade CBR been determined and how has the CBR to modulus relationship been established. The basis for adopting these values should also be clearly documented – for example, the assumed modulus, fatigue behaviour, operating temperatures and design speeds for the asphalt layers should be explicitly stated.

The constructability and the interface between the old and new pavement are often the causes of premature pavement failure, so these factors should be considered carefully and documented. Consideration should be shown to how the recommended treatment may impact on the surrounding pavement. For example, the designer needs to consider the potential that the treatment will create an impermeable layer that will not allow other sections of pavement drain effectively.

Safety in design requires the designer to consider risks associated with the construction, operation, maintenance and rehabilitation of the pavement and to eliminate, isolate, minimise, inform or control any risks. As a consequence, some treatments might be rejected. For example, high levels of maintenance activities on high volume traffic roads may not be appropriate.

Summary of pavement model details, where relevant, with the full models contained in an appendix.

10.5 COST ESTIMATE

A Net Present Value analysis, including economic indicators and traffic delays where appropriate should be summarised with the full calculations presented in an appendix. The calculations should include realistic forecasts of future maintenance costs. The cost estimates should have some allowance for unforeseen conditions, although the ground investigations should be sufficient to minimise this risk.

10.6 DISCUSSIONS AND RECOMMENDATIONS

Conclusions should provide a brief summary of the faults that have led to the rehabilitation decision and the identified causes of failure.

The recommended treatment design documents how the design will address the existing pavement defects and causes of failure. The design should clearly identify the design subgrade, design CBR, design traffic, and the recommended pavement structure. Assumed material properties and design temperatures should also be provided.

The design report should also include site plans and typical cross-section drawings and detail of the interface between the existing and new pavement.
Alternative treatments should be provided, in case the design assumptions are not met during construction.
11 Construction quality assurance

11.1 BACKGROUND

Suitable levels of pavement construction supervision can have a beneficial effect on the achieved life of a pavement. The purpose of pavement construction quality assurance is to identify those areas of the construction process for which monitoring will minimise the risk of pavement failure and improve the pavement performance. The following sections are not exhaustive and should only serve as an initial platform upon which to build a robust quality assurance program. However, they highlight the areas where particular focus is appropriate.

When undertaking quality assurance the site engineer should be suitably experienced and able to make decisions regarding the acceptability of the pavement construction. The site engineer shall consult with the pavement engineer if they have any concerns regarding the pavement construction.

The use of NZ Transport Agency specifications and standards are required for monitoring the quality of the pavement construction process. The following sections, while not exhaustive, detail some key factors from a number of specifications that should be monitored throughout the design and construction process.

A methodology by which the construction quality can be assured should be developed for each specific treatment. This will ensure that the constructed treatment meets the design assumptions detailed in the Detailed Design Report discussed in Chapter 10.

11.2 DESIGN OUTCOMES

It is imperative the design includes the desired outcomes and any assumptions or requirements the designer needs to be fulfilled through the construction process. The Design Outcomes would cover the areas suggested in Table 17 below and would:

- Describe the expected outcome from the pavement or surfacing design in construction-related terms,
- Identify what the purpose of the design is, and
- Detail the specific requirements to be met and how these can be ensured through the proposed Inspection and Testing Plan.

These details should be explicitly stated rather than assumed by the designer.
Table 17 Examples of Design Outcomes

<table>
<thead>
<tr>
<th>Pavement renewals</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pavement configurations</td>
</tr>
<tr>
<td>Design assumptions including construction thicknesses and material requirements</td>
</tr>
<tr>
<td>Applicable specifications</td>
</tr>
<tr>
<td>Construction techniques: what plant will be used and how?</td>
</tr>
<tr>
<td>Specific material parameters (e.g. maximum dry density for specific source material)</td>
</tr>
</tbody>
</table>

These Design Outcomes would then lead to the test requirements in the Inspection and Test Plan to demonstrate the design assumptions have been met.

### 11.3 CONSTRUCTION RISKS

A risk register will be generated to detail how construction risks are managed. These include:

- Construction timetable, particularly in relation to periods of high traffic levels and weather;
- Material supply and quality;
- Subbase material specification;
- Construction tolerances and quality;
- Construction plant;
- Material testing;
- Material handling (for example aggregate segregation);
- Asphalt compaction temperatures;
- Underground services; and
- Any other construction risks.

### 11.4 INSPECTION AND TEST PLAN

The Inspection and Test Plan should eliminate or mitigate the risks identified in the risk register. It should be designed to ensure the construction quality meets the design assumptions. For example a specific area often overlooked is establishing a clear link between the target density requirements and the material specific densities.

The inspection and test plan should also identify the actions required should any acceptance criterion not be met.

### 11.5 CONSTRUCTION PROCEDURES

Appropriate construction procedures should be detailed in the Quality Assurance documentation. An example for the construction of aggregate pavement layers is provided in Section 11.5.1 below. Such a process should be modified if chemical modification is to be included as part of the treatment.
11.5.1 Construction of aggregate pavement layers

Appropriate steps in the construction of aggregate pavement layers could include:

- Lay aggregate – from truck or bottom dumper
- Add water as required – to avoid segregation of the aggregates and maintain optimal moisture content
- Compact aggregate using vibrating roller – typically two passes
- Commence grading – monitoring need for watering
- Compact with vibrating roller
- Top-up aggregate to finish level
- Compact with vibrating roller – with water if required
- Do final trim with grader
- Do final rolling to get final surface
- Apply thin running course – if needed
- Prepare final surface
- Do long grade – to get final shape and ride
- Use drag broom and roller
- Continue dragging and rolling until surface is ready for surfacing.

In addition to the physical steps detailed above the quality assurance steps in the following Sections should also be considered and supplemented with additional testing where appropriate.

11.6 PRECONSTRUCTION APPROVALS

11.6.1 Subbase material specification

Subbase material specification requirements shall be developed considering the local source aggregate. The NZTA M3 (1986) notes may be used to assist in developing this specification.

The M3 document is not a specification; it does however provide guidance for specifying subbase materials. Some example clauses could be:

- The subbase material shall be graded with 100% by mass passing the 65 mm sieve and 45% by mass passing the 19 mm standard sieve.
- The laboratory soaked CBR tests shall all be greater than 35% as determined by NZS 4407:1991 Test 3.15.
- Materials must be free from all non-mineral matter.

11.6.2 Basecourse material specification

Basecourse aggregates shall satisfy the NZTA M/4 (Transit 2006) specification for the AP40 or the AP20 grading requirements. The requirements of Section 7.2.8 may also be applicable.

Testing certificates confirming the basecourse material satisfies the NZTA specifications shall be provided to the pavement engineer prior to construction.
11.6.3 Cement modification of basecourse

The suitability for the cement modification of the basecourse material in the existing carriageway and the proposed imported M/4 basecourse material should be confirmed prior to construction through laboratory testing. Cement modification of the basecourse shall be enough to ensure that the ITS shall not exceed 400 kPa. The level of modification shall be tested by the contractor prior to construction, to ensure strength in the required range can obtained, while excessive strength gains, with increased risk of block cracking, are not realised.

Representative samples of the materials to be included in the stabilised layer shall be tested according to NZS 4402: test 4.1.3 to determine the optimum water content and the likely maximum dry density target. In addition, the solid density of the representative stabilised material shall be determined according to NZS 4407: test 3.7.1.

Testing certificates shall be provided to the Engineer prior to construction confirming: the basecourse material satisfies the above strength characteristics; the optimum water content; the maximum dry density; and the solid density.

11.6.4 Chipseal design and asphalt mix design

Any chipseal design shall satisfy the requirements set out in the book Chipsealing in New Zealand (Transit et al. 2005).

Asphalt designs shall satisfy the NZ Transport Agency (NZTA) M/10:2014 specification with all required tests being performed on the aggregates, bitumen and proposed asphalt mix design.

The chipseal design and the asphalt mix design shall be submitted to the pavement engineer for approval prior to construction.

11.7 CONFIRMATION INVESTIGATION

11.7.1 Subgrade investigation

During the site preparation the subgrade shall be tested by Scala penetrometer to confirm the design CBR assumptions. At least three Scala tests shall be performed each separated by a minimum of two metres. The subgrade moisture conditions at the time of the testing shall be communicated to the pavement engineer.

A correlation between the Scala and the laboratory soaked CBR should be established, although it should be recognised that the sensitivity of the material may make this correlation difficult to establish.

The test results shall be communicated to the pavement designer. If the observed subgrade CBR values are less than the design CBR, an immediate response from the pavement designer shall be sought.

11.8 CONSTRUCTION VERIFICATION

For a particular construction project the accepted Contractor’s Quality Assurance Testing Plan should be followed. Records should be kept and regularly reviewed to allow appropriate corrective actions to be taken in a timely manner when necessary.

A Construction report should be generated detailing quality assurance test results and construction detail.
The Quality Assurance Testing Plan should include tests required by NZ Transport Agency specifications as well as any project specific specifications. Where appropriate, hold points need to be set to allow the Engineer to check the quality control test results before construction proceeds.

11.8.1 Subgrade preparation

Subgrade preparation shall be according to the NZTA specification TNZ F/1:1997. The existing soils shall be excavated to the required subgrade level. After this the subgrade material shall be prepared by proof rolling unless subgrade sensitivity has been identified as a risk.

11.8.2 Placement of geogrid and geotextile

The geogrid and geotextile materials shall be placed according to the manufacturer’s instructions and the NZTA specification TNZ F/7:2003.

11.8.3 Construction of unbound granular pavements

Unbound granular pavement layers shall be constructed according to NZTA B2: (NZTA 2005a) and the supporting notes. A particular focus shall be the construction tolerances and the compaction of the granular material as specified in the B2 document.

A minimum of five density tests shall be conducted with the test locations being determined by the site engineer. The test results shall satisfy the requirements of B2 which are reproduced in Table 18 below.

### Table 18 Mean and minimum value of pavement layer compaction as a percentage of maximum dry density

<table>
<thead>
<tr>
<th>Values</th>
<th>Sub-basecourse layer</th>
<th>Basecourse pavement layer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean value</td>
<td>≥ 95</td>
<td>≥ 98</td>
</tr>
<tr>
<td>Minimum value</td>
<td>≥ 92</td>
<td>≥ 95</td>
</tr>
</tbody>
</table>

11.8.4 Construction of foamed bitumen stabilisation

The best practice for construction and quality control shall follow the requirements stated in Appendix D, TG2 2009. In particular, construction should stop and the problem remedied if stringy bitumen pieces are found in substantial quantities through the mix. A simple test can check for stringy bitumen pieces as detailed below from TG2 (2009) (Appendix D: Construction Controls for Bitumen Treatment).

The “test” consists of holding the ball between the thumb and index finger and gently applying pressure on opposite sides of the ball to gauge the cohesiveness of the material. The ball should deform before falling apart. Inspect the face of the broken ball to see how well the bitumen has dispersed. If no bitumen can be seen, the mix is perfect. The more bitumen blobs or stringers that can be observed, the worse the quality of the mix.

Two factors that can influence the success of the foaming process are moisture content of the aggregate and the temperature of the pavement being foamed.
11.8.5 Construction of bound granular pavements

Bound granular pavement layers shall be constructed according to NZTA B/5:2008 or NZTA B7:2012 and the supporting notes.

The stabilisation process shall be monitored according to the NZ Transport Agency’s Draft Guidelines for the Sampling and Testing of Stabilised Materials during Construction (2000).

The pavement layers shall be compacted to a uniform, dense and stable condition. A suitable method of ensuring compaction consistency shall be proposed by the contractor. The compaction results shall be submitted to the site engineer for consideration. Construction of the next layer shall not proceed without the confirmation of the Engineer. The Risk Register will have identified the risks of premature trafficking of a bound layer, so appropriate risk mitigation will have been initiated. If premature trafficking of the bound layer cannot be avoided, the design process needs to recognise the reduction in stiffness and potentially the layer life.

11.9 PAVEMENT SURFACING

11.9.1 Surface preparation

Surface preparation shall satisfy the NZTA B2 (2005a) specification. One requirement is for a tightly consolidated surface when swept. In this: the large aggregate is exposed to the surface and is held in pace with a matrix of smaller aggregates; the smaller aggregate is held firmly in place by fine material; and the matrix does not displace under normal trafficking or sweeping. A surface that meets these requirements is illustrated in Figure 13. A poorly prepared surface is shown in Figure 14 below. The standard of sweeping shall be sufficient to remove all loose aggregate dirt, dust, silt and other damaging matter.

Figure 13 Stone mosaic ready for sealing
11.9.2 Chipseal

The placed seal shall at a minimum satisfy NZTA T10 (NZTA 2014) in terms of skid resistance and texture. Chipsealing aggregate shall satisfy NZTA M6 (NZTA 2011). The bitumen application rates and construction shall satisfy the approved seal design requirements.

The chipseal should be checked for bitumen application rates, chip application rates, chip adhesion, and chip rollover.

11.9.3 Asphalt

The following paragraphs are reproduced from the NZTA M10 (NZTA 2014) specification and should be used.

Manufacturing compliance may be assessed at two levels:

(a) Verification that the job-mix formula has been replicated, i.e. use of conforming components and NZTA M/10:2014 combination in the design proportions to achieve the job-mix formula grading and binder content.

(b) Verification that the design targets have been met, for example, testing of compacted samples for volumetric properties and other specified properties.

For many applications, compliance with the job-mix formula grading and binder content is adequate. It is considered best practice in New Zealand to monitor production consistency by also using the Maximum Specific Gravity.

Asphalt construction thicknesses are minimum values. The layer thicknesses shall not be less than the design thickness.

The asphalt construction shall satisfy the NZTA M10 specification. The minimum asphalt delivery temperature to the site shall be determined during the development of the Project Quality Plan and monitored during construction.

Open Graded Porous Asphalt and Epoxy Modified Open Graded Porous Asphalt shall be constructed according to the NZTA P11 and P11E specifications respectively.

Figure 14 Fatty surface, not ready for sealing
11.10 POST CONSTRUCTION TESTING AND MONITORING

It is important that the performances of rehabilitations from each year’s pavement renewal programme are monitored annually. The review’s purpose is to determine whether the treatments used are lasting their design/expected life. If there are indications of early failure, the causes need to be investigated and treatment designs and construction methodologies adjusted to best manage the risk. Investment in additional construction monitoring is expected to reduce the risks of premature failure. A combination of visual inspections, high speed condition data and FWD data should be sufficient for monitoring purposes. Data from the TSD should also be useful although experience with interpreting this data is still being developed. Therefore any conclusions derived should be treated with caution.

The main aim of post construction testing is to check the pavement design assumptions have been achieved in the constructed pavement. As an example for stabilised pavements, cores can be taken from the constructed pavement to check the stabilised depth and the indirect tensile strength meets the design expectations.

If there are any concerns regarding the consistency of the construction, deflection testing shall be used to determine construction uniformity. The exact testing methodology shall be determined after consultation with the site engineer and the pavement engineer.

Macrotextural requirements shall be as detailed in the Transit New Zealand Network Operations Division Memorandum No. NetO 1/05 (Transit 2005b). It should be noted that higher texture levels, compared to existing surfaces, are required for the construction of new surfaces. The document T/10:2013 (NZTA 2013) outlines the levels of skid resistance required for treatment selection.

The book Chipsealing in New Zealand (Transit et al. 2005) gives a good description of the physical effect of road surface on traffic noise and the impact the noise has on the adjacent community. NZ Transport Agency has published a Guide to State Highway Road Surface Noise (NZTA 2014). The surfacing shall follow the best practice detailed in this document.

For effective monitoring the pavement design assumptions, post construction tests and locations for future monitoring purposes need to be documented. The NZ Transport Agency Network Outcome Contracts provide guidance on the requirements for pavement rehabilitation post construction design assessment.

11.11 PAVEMENT CONSTRUCTION HOLD POINTS

Detailed in Table 19 are some recommended hold points for monitoring the quality of the construction of the pavement and identifying the timing at which the construction details need to be reviewed. Approval from a designated person will be required before moving on from a hold point onto the next stage.

Table 19 is provided as an example only. Depending on the actual pavement design, additional hold points may be needed or some of the listed hold points may be unnecessary. These hold points should be established before construction starts.
### Table 19 Hold points to be observed during the pavement construction process

<table>
<thead>
<tr>
<th>Hold point</th>
<th>Timing</th>
<th>Approval from</th>
<th>Test requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Preconstruction</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Confirmation of material compliance</td>
<td>Before establishment of site</td>
<td>Pavement engineer</td>
<td>Laboratory test results supplied</td>
</tr>
<tr>
<td>Supply of cement modification testing results to pavement engineer</td>
<td>Before establishment of site</td>
<td>Pavement engineer</td>
<td>Laboratory test results supplied</td>
</tr>
<tr>
<td>Supply of chipseal design and confirmation that material meets design expectation</td>
<td>Before establishment of site</td>
<td>Pavement engineer</td>
<td>Chipseal selection and design supplied. Design assumptions detailed</td>
</tr>
<tr>
<td>Supply of job mix formula and confirmation that material meets design expectation</td>
<td>Before establishment of site</td>
<td>Pavement engineer</td>
<td>M10 requirements supplied</td>
</tr>
<tr>
<td><strong>Granular Pavement Construction</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Confirmation of subgrade CBR</td>
<td>Before construction of pavement</td>
<td>Site engineer unless observed CBR values are less than the design CBR.</td>
<td>Design CBR confirmed using methodology identified in the Quality Plan.</td>
</tr>
<tr>
<td>Placement of the geogrid and geotextile</td>
<td>Before construction of the subbase layer.</td>
<td>Site engineer</td>
<td>Manufacturer’s instructions have been followed</td>
</tr>
<tr>
<td>Confirmation of compaction density</td>
<td>At the completion of the cement stabilisation of <em>in situ</em> and granular overlay materials</td>
<td>Site engineer</td>
<td>Nuclear densometer results taken in random locations in appropriate numbers</td>
</tr>
<tr>
<td>Confirmation of subbase construction thicknesses</td>
<td>At the completion of the cement stabilisation of <em>in situ</em> and granular overlay materials</td>
<td>Site engineer</td>
<td>Observed</td>
</tr>
<tr>
<td>Confirmation of basecourse construction thicknesses</td>
<td>At the completion of the cement stabilisation of <em>in situ</em> and granular overlay materials</td>
<td>Site engineer</td>
<td>Verification documentation supplied</td>
</tr>
<tr>
<td>Confirmation of correct levels of cement modification and compaction</td>
<td>At the completion of the cement stabilisation of <em>in situ</em> and granular overlay materials</td>
<td>Site engineer</td>
<td>Quality assurance results supplied</td>
</tr>
<tr>
<td>Hold point</td>
<td>Timing</td>
<td>Approval from</td>
<td>Test requirements</td>
</tr>
<tr>
<td>------------------------------------</td>
<td>------------------------------------------------</td>
<td>-------------------</td>
<td>----------------------------------------------------------------</td>
</tr>
<tr>
<td>Basecourse surface preparation</td>
<td>After confirmation of basecourse compaction levels</td>
<td>Site engineer</td>
<td>Verification documentation supplied</td>
</tr>
<tr>
<td>Chipsealing of pavement</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design assumptions met.</td>
<td>At the completion of the chipsealing.</td>
<td>Site engineer</td>
<td>Verification documentation supplied</td>
</tr>
<tr>
<td>Construction of asphalt surfacing</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Post construction deflection testing</td>
<td>At the completion of the construction</td>
<td>Pavement engineer</td>
<td></td>
</tr>
<tr>
<td>Post construction texture testing</td>
<td>At the completion of the construction</td>
<td>Site engineer</td>
<td></td>
</tr>
<tr>
<td>Construction of structural asphalt</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Asphalt properties</td>
<td>At the completion of the asphalting</td>
<td>Site engineer</td>
<td>Constructed asphalt lifts meet M10 requirements</td>
</tr>
<tr>
<td>Compaction temperatures</td>
<td>During asphalt placement and rolling</td>
<td>Site engineer</td>
<td>Compaction temperatures are within suitable range to obtain optimal compaction</td>
</tr>
</tbody>
</table>
12 References


13 Appendix A: Pavement life multipliers (Take Account of DAY and NIGHT Traffic)

These values are supplied to supplement Appendix B in Austroads (2017). 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.

PLMs provide a means whereby the performance of an asphalt-surfaced granular pavement in a given location, and with a specific day/night traffic split, can be readily assessed from the performance of the same pavement when the design asphalt temperature is 25°C.

During the general mechanistic procedure, the designer has to adjust the value of the design number of Standard Axles for asphalt (NSA x 365 x GF in Section 7.5) by dividing it by the value for PLM. This value can be determined from Appendix B of Austroads (2017). Having made this adjustment, the designer must adopt 25°C as the design asphalt temperature and proceed in the standard manner. For a conventional mix subjected to highway-speed traffic, the Guide has adopted 2800 MPa as a representative design asphalt modulus when the design asphalt temperature is 25°C. In essence, using the PLM adds to the established procedure by adding the fatigue life of the asphalt applicable to the specific location and day/night split of traffic. The PLMs are not applicable to pavements containing cemented material. This is because the dominant distress mode for such pavements is, for the vast majority of situations, fatigue cracking of the cemented material.
## Table 20 Pavement life multipliers for urban centres throughout New Zealand

<table>
<thead>
<tr>
<th>Centre</th>
<th>Asphalt Thickness</th>
<th>50 mm PLM&lt;sub&gt;D&lt;/sub&gt;</th>
<th>50 mm PLM&lt;sub&gt;N&lt;/sub&gt;</th>
<th>75 mm PLM&lt;sub&gt;D&lt;/sub&gt;</th>
<th>75 mm PLM&lt;sub&gt;N&lt;/sub&gt;</th>
<th>&gt;100 mm PLM&lt;sub&gt;D&lt;/sub&gt;</th>
<th>&gt;100 mm PLM&lt;sub&gt;N&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Whangarei</td>
<td>0.83</td>
<td>0.11</td>
<td>1.13</td>
<td>0.49</td>
<td>1.44</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Auckland</td>
<td>0.73</td>
<td>0.14</td>
<td>1.05</td>
<td>0.53</td>
<td>1.38</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Hamilton</td>
<td>0.58</td>
<td>0.07</td>
<td>0.99</td>
<td>0.45</td>
<td>1.36</td>
<td>1.00</td>
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<tr>
<td>Tauranga</td>
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<td>0.98</td>
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<td>Rotorua</td>
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<td>0.83</td>
<td>0.46</td>
<td>1.23</td>
<td>1.00</td>
<td></td>
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<tr>
<td>Taupo</td>
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<td>0.07</td>
<td>0.82</td>
<td>0.45</td>
<td>1.27</td>
<td>1.00</td>
<td></td>
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<tr>
<td>Gisborne</td>
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<td>0.08</td>
<td>1.07</td>
<td>0.47</td>
<td>1.43</td>
<td>1.00</td>
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<tr>
<td>Napier</td>
<td>0.60</td>
<td>0.09</td>
<td>1.02</td>
<td>0.48</td>
<td>1.40</td>
<td>1.00</td>
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<tr>
<td>New Plymouth</td>
<td>0.47</td>
<td>0.10</td>
<td>0.82</td>
<td>0.48</td>
<td>1.17</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Wanganui</td>
<td>0.47</td>
<td>0.09</td>
<td>0.86</td>
<td>0.48</td>
<td>1.22</td>
<td>1.00</td>
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<td>Palmerston North</td>
<td>0.41</td>
<td>0.07</td>
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<td>1.21</td>
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<td>Masterton</td>
<td>0.40</td>
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<td>0.45</td>
<td>1.29</td>
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<tr>
<td>Wellington</td>
<td>0.29</td>
<td>0.09</td>
<td>0.68</td>
<td>0.47</td>
<td>1.05</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Nelson</td>
<td>0.43</td>
<td>0.09</td>
<td>0.85</td>
<td>0.48</td>
<td>1.23</td>
<td>1.00</td>
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<tr>
<td>Westport</td>
<td>0.35</td>
<td>0.08</td>
<td>0.71</td>
<td>0.46</td>
<td>1.05</td>
<td>1.00</td>
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<td>Greymouth</td>
<td>0.30</td>
<td>0.08</td>
<td>0.69</td>
<td>0.46</td>
<td>1.05</td>
<td>1.00</td>
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<td>Kaikoura</td>
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<td>0.68</td>
<td>0.46</td>
<td>1.05</td>
<td>1.00</td>
<td></td>
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<tr>
<td>Christchurch</td>
<td>0.31</td>
<td>0.07</td>
<td>0.79</td>
<td>0.46</td>
<td>1.19</td>
<td>1.00</td>
<td></td>
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<tr>
<td>Timaru</td>
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### Table 21 Stiffness values for cracked asphalt as a function of temperature and loading speed

<table>
<thead>
<tr>
<th>Asphalt Temperature (°C)</th>
<th>Asphalt stiffness (MPa)</th>
<th>Vehicle speed 5 km/h</th>
<th>Vehicle speed 80 km/h</th>
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<tbody>
<tr>
<td></td>
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<tr>
<td>12.5</td>
<td>1850</td>
<td>2200</td>
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<td>1850</td>
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<td>1500</td>
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<td>1075</td>
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<td>1150</td>
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<td>670</td>
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Table 22: Relative stiffness values for sound asphalt as a function of temperature and loading speed

<table>
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<th>Asphalt Temperature (°C)</th>
<th>Relative asphalt stiffness</th>
<th>5 km/h (Benkelman Beam)</th>
<th>80 km/h (FWD, highway traffic)</th>
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<tr>
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