

Best practice guide for pavement stabilisation August 2017

W Gray
Opus International Consultants Limited

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NZ Transport Agency

Private Bag 6995, Wellington 6141, New Zealand

Telephone 64 4 894 5400; facsimile 64 4 894 6100

research@nzta.govt.nz

www.nzta.govt.nz

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Executive summary

The key objectives for this research project were to enable the New Zealand Transport Agency to publish a comprehensive, user relevant and best practice guide for pavement stabilisation in New Zealand, promote stabilisation best practice, maximise the opportunities presented by pavement stabilisation and provide a basis to support ongoing review, implementation and innovation.

Today stabilisation is a versatile and powerful technique used in constructing, rehabilitating and maintaining highways, public and private roads, ports, airports, domestic, commercial and industrial pavements. Stabilised ground improvement contributes to land development initiatives throughout New Zealand, notably rebuilding works in Canterbury following the 2011 earthquakes.

This project was not about active investigation and testing. It focused on examining current best practice and research findings, and resolving which of these best suited the New Zealand context.

The research team's comprehensive literature review considered a wealth of published research and technical guidance emanating largely from New Zealand, Australia and South Africa. In particular we considered the approach to technical guidelines on stabilisation published by AustStab that provided an excellent framework from which to launch this research project.

The research team members also drew on the evidence provided by stabilisation specialists worldwide, based on personal contacts and ongoing collaborative research and live project initiatives.

Then, following initial meetings with the project Steering Group, the key topic areas for the research report were selected in the following order: pavement stabilisation principles; treatment selection for stabilised pavements; laboratory and field tests for stabilised pavements; structural design for stabilised pavements; construction of stabilised pavements; quality management; ongoing research.

Pavement stabilisation is considered from first principles, including the history and background of the techniques and materials commonly used in New Zealand.

Treatment selection for stabilised pavements examines how asset managers and pavement designers shall approach the questions of problem identification and resultant treatment selection in practice, and how stabilisation can be considered along with other pavement maintenance or 'greenfield' treatment options.

The report then discusses laboratory and field tests for stabilised pavements, and provides background to current best practice recommendations, notably the use of the Indirect Tensile test in mix selection.

The structural design for stabilised pavements then discusses how to approach the design of stabilised pavements incorporating modified, lightly bound and bound materials. Worked examples are provided to demonstrate current design best practice in New Zealand and use of relevant software design tools.

Modern day construction and quality management processes are then examined to show the now diverse range of materials and project scenarios where stabilisation is used in New Zealand.

Finally, as research never sleeps, the research team discuss ideas for ongoing research.

We sincerely hope this research report provides a user relevant and practical best practice guide that enables asset managers, designers and contractors to utilise the tangible and sustainable benefits of pavement stabilisation.

Abstract

New Zealand pavement engineers, in collaboration with colleagues in South Africa and Australia, are recognised internationally as leaders in the use of stabilisation in highway, road, airport, port and industrial hardstand pavement applications. Stabilisation is used to rectify a deficiency in a soil, aggregate or surfacing material. Stabilised materials contribute to the strength and performance of pavements at all levels: subgrade; subbase; base and surfacing.

Applied research into and development of leading edge testing and design knowledge, coupled with significant improvements in the capacity and effectiveness of stabilisation construction plant and work site processes now offer the wider transport industry in New Zealand relatively safe, efficient and sustainable pavement construction, rehabilitation and maintenance options incorporating stabilisation.

This research was undertaken to bring together informed, current technical advice from a variety of sources to enable road controlling authorities, consultants and contractors in New Zealand to successfully investigate, design, construct, maintain and operate pavements with stabilised components.

1 Introduction

1.1 Purpose of this best practice guide

This guide's purpose is to bring together informed technical advice so road controlling authorities, consultants and contractors in New Zealand can investigate, design, construct, maintain and operate pavements with stabilised components.

Historic records and leading edge research in New Zealand, Australia and South Africa continue to show that pavement stabilisation is safe, efficient, affordable and sustainable.

This guide presents current best practice technical advice.

1.2 How to use the guide

The guide provides users with practical, relevant technical advice covering:

- pavement stabilisation principles
- treatment selection for stabilised pavements
- laboratory and field tests for stabilised pavements
- structural design for stabilised pavements
- construction of stabilised pavements
- quality management
- ongoing research.

Users are assumed to have a background in engineering, and a sound technical understanding of mechanical stress and strain, engineered soil and aggregate response.

First time users are encouraged to read the guide from the beginning.

The guide shows users how:

- the science behind material stabilisation can be incorporated in pavement treatment selection
- the investigation, design and construction processes employing pavement stabilisation are best delivered in the New Zealand context.

Extensive references to previously published information are provided in the guide's reference list. This includes related NZ Transport Agency (the Transport Agency) and Austroads design guides, material and construction specifications.

Worked structural design examples in chapter 5 of the guide include references to frequently used design charts, catalogue solutions, and design methods using proprietary pavement design software. The CIRCLY¹ pavement design software is mandated by Austroads and the Transport Agency.

¹ CIRCLY: Mincad Systems, Australia: New Zealand agent is Bartley Consultants

2 Pavement stabilisation principles

2.1 Introduction

Pavement stabilisation involves changing the existing material properties of the soil and aggregate layers used to build pavements by mechanically mixing them with one or more binders and water.

Pavement stabilisation methods have traditionally been defined by binder type: lime, cement, bitumen, granular stabilisation. Such binders are used individually and in combination.

Historic record and leading edge research in New Zealand, Australia and South Africa continues to show that pavement stabilisation is safe, efficient, affordable and sustainable. The reasons pavement stabilisation would be used in a particular project(s) will vary. The guide describes best practice methods.

2.2 History and background

Stabilisation was first used in New Zealand in 1943. Cement stabilisation for roads gained popularity in the mid-1960s and lime stabilisation followed in the 1970s, notably in the greater Auckland region (Hudson 1996).

Today stabilisation is a versatile and powerful technique used in constructing, rehabilitating and maintaining highways, public and private roads, ports, airports, domestic, commercial and industrial pavements. Stabilised ground improvement contributes to land development initiatives throughout New Zealand, notably rebuilding works in Canterbury following the 2011 earthquakes.

Stabilisation is used to rectify a deficiency in a soil, aggregate or surfacing material and to strengthen it. Stabilised materials contribute to the strength and performance of pavements at all levels: subgrade; subbase; base and surfacing.

The four types of stabilisation commonly used in pavements can be broadly categorised as: cementitious; bituminous; chemical or mechanical.

A survey of stabilisation practices (National Roads Board Road Research Unit 1985) showed in 1985 nearly half of New Zealand's road controlling authorities had used some form of stabilisation over a five-year period, notably on the northern and eastern areas of the North Island. Lime was the most commonly used stabilisation agent, followed by cement. Both sealed and unsealed roads were being stabilised. In-situ construction methods were the most common form of pavement stabilisation.

For unsealed roads pavement stabilisation is used to develop strength in the subgrade and pavement layers, and to control water sensitivity, rutting, dust and aggregate loss.

For sealed roads, pavement layer strength (stiffness) improvements and reducing moisture sensitivity at all pavement system levels following stabilisation enable pavements to carry more traffic and to be operated and maintained effectively.

Construction practices in the 1960s and 70s included: spreading cement or lime by hand or light machine; scarifying the pavement layer surface by grader or dozer tynes; mixing to shallow depths with low powered stabilisers (sometimes just heavy duty agricultural tractors) and compaction by self-propelled static and tow behind vibrating rollers. Dusty work trains with a grader, spreader, water cart, rotary mixer and roller would snake their way slowly down local roads and highways.

The situation today is different largely due to the improvements in the construction plant. In-situ pavement layer stabilisation now routinely involves directly injecting water and binder from bulk tankers to powerful, manoeuvrable and largely self-contained stabilising machines. Stabilising agents including cement, lime, refined kiln slag derived from steel production, foamed or emulsified bitumen and polymer chemicals are used individually and in combination. The depth of mixing and the mixing process effectiveness has improved. Compaction can now be achieved using highly efficient plant with combined static and vibrating capability, including variable amplitude/frequency control.

Pavement stabilisation is a highly effective design and construction option across New Zealand. This is even the case in those areas where abundant gravel supply has historically meant the economics of pavement stabilisation struggled when compared with the more traditional unbound pavement construction methods. The pavement layer strength and performance gains derived from stabilisation options today are recognised by designers, contractors and asset managers across the country, especially those working on heavily trafficked road networks. The improvements in construction plant, processes and technical understanding of stabilisation offer the wider transport industry in New Zealand relatively safe, efficient and environmentally sustainable pavement construction, rehabilitation and maintenance options.

2.3 Pavement stabilisation – applications

Pavement stabilisation supports improved whole of life performance and traffic load carrying capacity. It can be used effectively on both new pavement construction as well as existing pavement rehabilitation and maintenance projects in New Zealand because:

- Pavement materials (subgrade, subbase and basecourse) can be dried out, made stronger, more resilient and stable (less sensitive to changes in moisture content) under varying traffic, geology, climate, surface and groundwater conditions.
- Existing pavement materials (including surfacing materials) can be recycled, reducing demands on existing and/or new quarry resources and conversely on landfills. Stabilising existing aged materials can rejuvenate and strengthen layer properties in pavement reconstruction.
- Construction, rehabilitation and maintenance costs can be controlled with pavement stabilisation. This can be achieved by sustainable material input costs, raw material transport and construction costs and improved whole-of-life performance with fewer reactive maintenance interventions.
- The adverse social impacts of pavement construction, rehabilitation and maintenance on stakeholders (eg traffic delays, lane closures and disruption from the transportation of raw materials over longer distances) can be mitigated by using stabilised materials.
- Pavement stabilisation can be used on both sealed and unsealed roads, and in an unlimited variety of residential, commercial and industrial pavement settings.

To realise pavement stabilisation potential the pavement asset manager, pavement designer and contractor (builder) need to appreciate what ‘best practice’ means.

2.4 Unbound pavement stabilisation

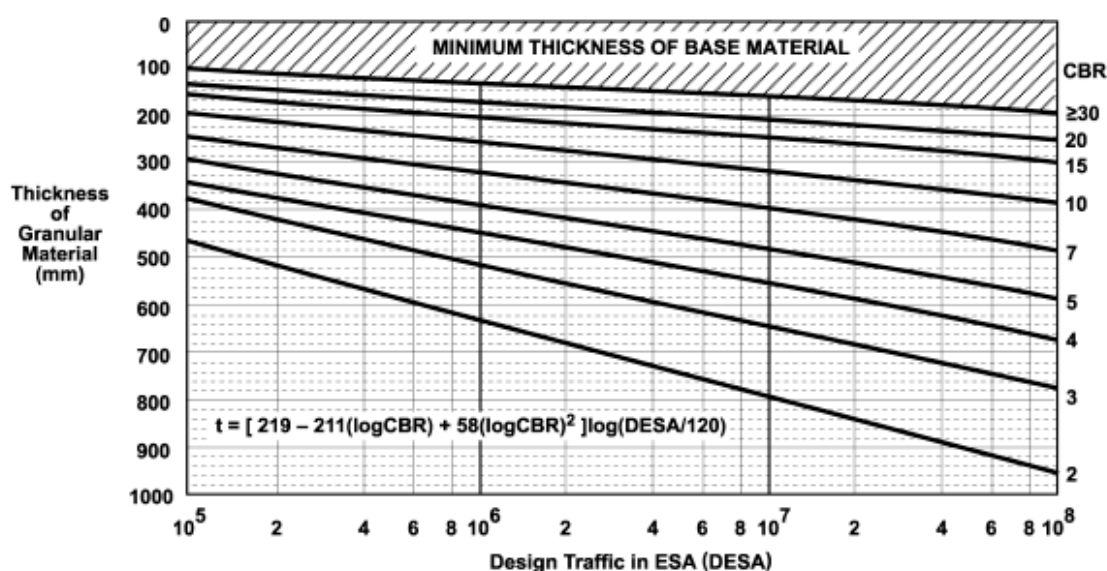
Unbound pavement systems typically use processed granular materials. These are sourced from a variety of natural quarry locations (eg stone quarry, river beds and coastal gravel bars) and are applied in subbase and basecourse production for use over existing or made ground (subgrade).

Published pavement design charts for unbound pavements in normal and light traffic conditions are included in relevant Austroads design guides for example (Austroads 2004; Austroads 2012).

A pavement design chart (see figure 2.1) enables the pavement designer to determine:

- the overall depth of unbound pavement needed for a given traffic load: design traffic in equivalent standard axles (ESA); subgrade (foundation) strength as represented by the California bearing ratio (CBR)
- the minimum thickness of base (basecourse) needed as the top unbound layer, to deliver the necessary upper pavement shear strength
- the balance of the total pavement depth would be subbase (and in some instances granular subgrade improvement material).

Figure 2.1 Unbound pavement and pavement design chart



Source: Austroads (2012)

Transport Agency material specifications prescribe the material properties of the basecourse.

The NZTA M/4 specification (NZ Transport Agency 2006) gives the source and production test properties to be met for compliance as an M/4 basecourse. These include: source rock crushing strength and weathering resistance, production particle size (grading), fines fraction sand equivalent, plasticity index and clay index.

Aggregate stabilisation typically involves blending new aggregate makeup into existing basecourse materials to change the properties of these materials. It also enables suppliers to process conforming M/4.

For example, if an existing basecourse material lacks the all-important sand fraction (Salt and Stevens 2011), then blending in gap graded crushed rock/sand fraction can help. The worked example in section 5.4.2 shows how this can be achieved.

The material specifications to be met for a subbase are typically source property tests (similar to M/4), production maximum stone size (usually 65mm) and particle size distribution. Best practice dictates that the permeability of the unbound subbase is higher than for the basecourse by limiting the fine fraction

passing the 75µm sieve. Aggregated stabilisation by blending new sand and finer gravel makeup can be used to improve permeability in subbase construction of new pavements.

The stabilisation of unbound aggregate materials can also utilise small quantities (typically <2% by dry mass) of reactive lime (Ca(OH)_2 or CaO , almost always oxide fines, 2–3mm top size for ease of slaking) as a modifying agent to reduce plasticity in the finer aggregate fraction. The worked example in section 5.4.3 shows how this can be achieved.

2.5 Lightly bound pavement stabilisation

Lightly bound pavement stabilisation of subgrade, subbase and basecourse layers is typically achieved using cementitious binders (cement, fly ash). Bitumen (foamed bitumen or bitumen emulsion) binders are more often confined to basecourse layer stabilisation.

Cementitious binders (typically <2% by dry mass) have been shown to readily deliver lightly bound materials (Gray et al 2011).

Lightly bound layers can help improve the pavement system load transfer.

Lightly bound basecourse layers are typically favoured over bound layers, which are at risk of thermal shrinkage and fatigue cracking.

A lightly bound subgrade improvement layer (SIL) can be constructed either in situ (eg stabilising existing subgrade) or using stabilised made ground (eg cement modified sand). The worked examples in section 5.5 show how this can be achieved.

2.6 Bound pavement stabilisation

Bound pavement stabilisation of the subbase is typically achieved using cement binder with cement contents of $\geq 4\%$ by dry mass reported from recent subbase stabilisation projects. Bound basecourse layers are not currently a generally accepted practice in New Zealand because of the risk of thermal shrinkage cracking (block cracking) and fatigue cracking.

Bound pavement layers deliver vastly improved load transfer in the pavement system. They can also help bridge softer foundation conditions. The bound pavement layer can be designed to either remain bound over the life of the project, or crack (in a controlled manner) and thereby migrate back to a pseudo unbound behaviour. The worked examples in section 5.6 show how to do this.

2.7 Binder types

2.7.1 Cementitious binders

The cement used for pavement stabilisation in New Zealand is described in the B series specifications, for example B/5 (NZ Transport Agency 2008) as follows:

- general purpose Portland cement – type GP
- general purpose blended cement – type GB
- special purpose low heat cement – type LH.

Type GP cement is typically used. Fly ash, pulverised blast furnace slag or other pozzolan rich materials can be combined with lime to form supplementary cementitious blended materials for use on pavement

stabilisation. Laboratory-based binder reactivity tests (NZ Transport Agency 2017a) should be used during the investigation stage of a project to see if a single cementitious material or a combination of these materials can be used. The usefulness of the binder/aggregate combination can be checked by taking in-situ cores from field trial sections and/or samples taken from behind the stabiliser and compacted into test moulds on site.

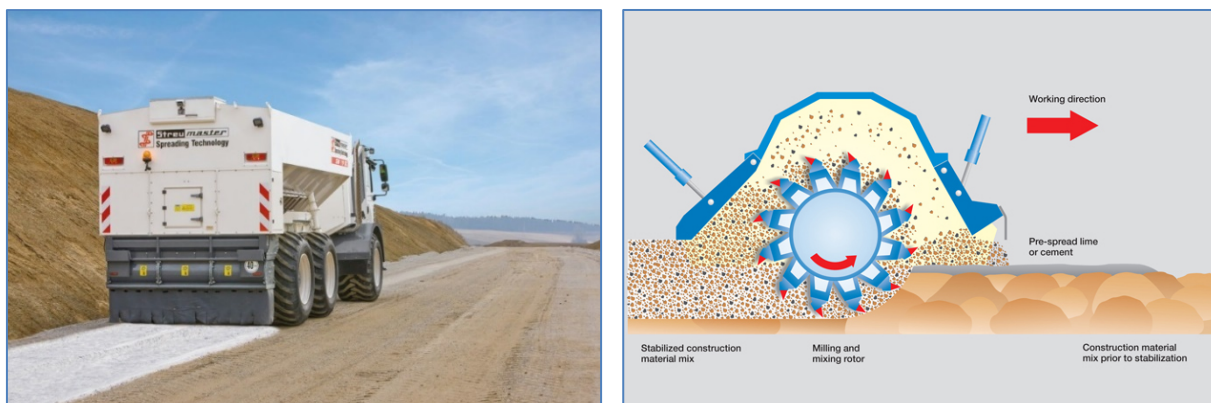
2.7.2 Lime binders

The lime materials used for pavement stabilisation in New Zealand are described in the M/15 specification. They are typically either hydrated lime ($\text{Ca}(\text{OH})_2$, calcium hydroxide) or burnt lime (CaO , calcium oxide) products (NZ Transport Agency 2012b). Burnt lime is typically used in New Zealand.

Hydrated lime is usually supplied either in bulk or in bags. It comes in fine powered form and is relatively safe to use, although the usefulness has been modified by prior hydration.

Burnt lime is usually supplied in pellet form (typically a 3mm top size) so can be spread with a chip spreader and easily slaked. Burnt lime reacts very quickly with any moisture in the soil or aggregate being stabilised, often with dramatic effect. Once lime (and cement) is applied to the surface it can be mixed in place (see figure 2.2).

Figure 2.2 Spreading and mixing lime (and/or cement)



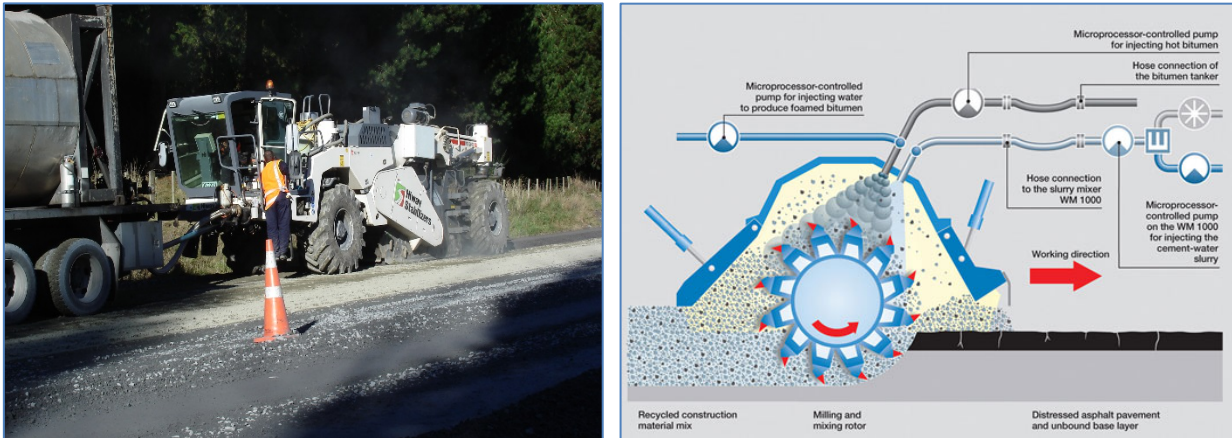
Source: William Gray and Wirtgen (2010)

2.7.3 Bitumen stabilised material binders

Bitumen stabilisation is typically carried out with either foamed bitumen or bitumen emulsion.

Foamed bitumen is a mixture of air, water and hot straight run bitumen. Injecting a small quantity of cold water into hot bitumen in a controlled manner makes the bitumen expand spontaneously. This in turn increases the surface area and reduces the viscosity of the bitumen, enabling mixing with damp and cold aggregates. The fine particles within the aggregate material are coated by bitumen, helping to create a bitumen-rich mortar that binds the stabilised material in a non-continuous manner akin to a multitude of small 'spot welds'. The term 'aggregate on steroids' has been used to describe foamed bitumen stabilised aggregate. Usually a small quantity of cement is also added to enhance short-term stability and tensile strength. The quantity of cement is expected to be less than 1.25% by dry mass (NZ Transport Agency 2017a) to mitigate against shrinkage cracking.

The process of foaming and mixing the bitumen binder into the stabilised layer happens in one operation, as shown in figure 2.3.

Figure 2.3 Foamed bitumen stabilisation

Source: William Gray and Wirtgen (2010)

The bitumen emulsion is cationic or anionic emulsion depending on the geology and chemistry of stone being treated. The process involved with supplying and mixing bitumen emulsion (essentially bitumen floating in water) is typically similar to more conventional lime or cement stabilisation.

2.7.4 Polymer binders

Polymer materials (typically short or long chain styrene-based chemical polymers) are supplied either in dry granular or liquid form, the latter being in solution with water. While the science behind selecting and using polymer additive (refer section 4.5) differs from other binder options, the application and mixing processes are typically the same.

2.8 Literature review

The technical and literary references used in the guide's preparation are listed in chapter 10. The leading-edge research on pavement stabilisation as it applies in the New Zealand context comes largely from New Zealand, Australia and South Africa.

The following documents are most relevant to best practice in New Zealand now.

Asphalt Academy (2009) *Technical guideline: bitumen stabilised materials. A guideline for the design and construction of bitumen emulsion and foamed bitumen stabilised materials.*

AustStab (2015) *Pavement recycling and stabilisation guide.* 2nd ed.

Austrroads (1988) *Guide to pavement technology part 4D: stabilised materials.*

Austrroads (2012) *Guide to pavement technology part 2: pavement structural design.*

NZ Transport Agency (2017a) *New Zealand guide to pavement evaluation and treatment design.*

NZ Transport Agency (2008; 2012a; 2012c; 2012d; 2016b) B series specifications.

NZ Transport Agency (2016c) *T19 Specification for the mix design testing of modified and bound pavement layers.* Draft.

NZ Transport Agency (2012b) *M/15 Specification for lime for use in soil stabilisation.*

Wirtgen GmbH (2010) *Cold recycling technology.*

2.9 Pavement stabilisation: key differences in New Zealand, Australian and South African best practice

There are differences in the ways pavement stabilisation best practice is delivered in New Zealand, Australia and South Africa. Guide users should be aware of the following, and defer to current best practice in New Zealand (as described herein) in the first instance.

2.9.1 BSM foam design

Current Australian practice (AustStab 2015) is to check a foamed bitumen layer (bitumen stabilised material ((BSM)) foam) for fatigue as if it were a structural asphalt concrete material. This results in more conservative outcomes.

In South Africa (Asphalt Academy 2009) the structural design of BSMs does not differentiate between BSM-foam and BSM-emulsion. Categories of BSM (BSM1, BSM2 and BSM3) are defined according to design traffic loading and reliability. Pavement designs for >1MESA uses a pavement number (PN) system (similar to AASHTO structural number; when it is below 1MESA a design catalogue is used. The PN and catalogue approaches both set limits on BSM layer modulus, layer thickness and modular ratio to underlying layer support.

Current best practice in New Zealand limits the BSM layer modulus to five times the underlying layer modulus to a maximum of 800MPa (NZ Transport Agency 2017a; 2017b). BSM mix design is support by the draft T/19 specification (NZ Transport Agency 2016c).

2.9.2 Post-effective long-term stiffness phase design in lightly cement bound pavements

Austrroads (2012) limits the cemented effective long-term stiffness (ELTS) layer modulus to a maximum of 500MPa, anisotropic, Poisson's ratio 0.35, no sub-layering. Anecdotal evidence from recent projects and ongoing research evidence suggest it is highly unlikely the cemented layer will retain these material characteristics over the full ELTS pavement life (eg 25 years).

Research at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) (Alabaster et al 2013) suggests when the lightly cement bound material cracks, it behaves as an unbound material, and therefore should be sub-layered. Anecdotal evidence from project work around New Zealand (notably in the central North Island) suggests while the unbound sub-layered rule is sensible, the effective stiffness of the micro-cracked layer in the ELTS would potentially be higher (between 650MPa and 800MPa) than the actual unbound material.

A lightly bound pavement layer between 200mm and 250mm thick (the optimum construction depth range) can conservatively be expected to deliver up to 1MESA (assuming the stress ratio (SR) is <0.5, see section 4.2.1) before flexural cracking occurs. The performance in the ELTS phase remains important.

The review completed for this guide suggests New Zealand best practice limits the vertical layer modulus E_v in the ELTS phase in a sub-layered material to a maximum 700MPa. The actual E_v value attained is dependent upon the E_v value of the layer below and the depth of the layer, controlled by sub-layering. The Austrroads (2012) granular material sub-layering rule should be used.

To mitigate the risk of 'eggshell behaviour' (stiff layer on thin pavement cracking) the ELTS material's residual unconfined compressive strength (UCS), using standard compaction and a 2:1 height: diameter ratio for the mould) is best controlled (< 1MPa) (see section 4.2.1).

3 Treatment selection for stabilised pavements

3.1 Introduction

Treatment selection for new pavements, pavement rehabilitation and pavement maintenance projects should be project and site specific. They should also take into account local conditions and materials.

Supporting information on pavement treatment selection can be found in AustStab (2015, sections 2 and 3), Austroads (2012, sections 5 and 6) and (NZ Transport Agency 2017a).

The stabilised pavement options available in New Zealand work well under the right conditions. Robust treatment selection is an essential ingredient of stabilised pavement performance, and should balance consideration of local, technical and economic factors.

3.2 Local factors

3.2.1 Introduction

Local factors can affect stabilised pavement performance in a number of ways, as discussed below.

3.2.2 Climate and geology

Stabilised pavements can be adversely affected by freeze and thaw conditions, high temperature climates, excess salt or salt water inundation, flooding, high ground temperature and material chemistry in geothermal areas, local aggregate and soil material variations. For example, even though clay rich volcanic ash materials can benefit from the drying effects of lime stabilisation, the greater strength loss following remoulding in some clay rich brown ash in the Bay of Plenty region can make such material impractical to use, although treatment with a blend of lime and cement can assist drying and improve strength in some materials. Reactivity testing (Standards New Zealand 1986; 2015) can be used to measure the effectiveness of stabilisation in this context.

Understanding the local climate and geology will also help the pavement designer to consider which binders are more likely to be effective (see section 4.3).

Laboratory-based material testing (see section 4.7) can be used to study the effects of location and climate which can affect stabilisation outcomes.

3.2.3 Pavement treatment history

Based on an area's local history, the question as to whether stabilisation has been used successfully before should be asked. Depending on the answer, the next question should be why, or conversely why not?

For example, in Canterbury and on the West Coast of the South Island, the local aggregate materials would probably respond well to stabilisation with cement. However, the abundance of accessible local aggregate materials and lower traffic volumes in many locations means stabilisation is unlikely to have the lowest whole-of-life cost (economic considerations) compared with more traditional, deeper unbound pavement design and construction methods.

3.2.4 Practical consideration

Practical considerations that may limit the use of pavement stabilisation include: shallow existing utility service depth, location and condition; proximity to sensitive residential dwellings, commercial or industrial

premises; construction and/or machinery access constraints; establishment/disestablishment and construction cost.

3.2.5 Iwi and social considerations

The project site's cultural sensitivity needs to be checked.

For example, some iwi (especially in more remote rural areas) may not support the use of imported materials, including bitumen and cement, and larger scale, expatriate construction operations.

3.3 Technical factors

3.3.1 Introduction

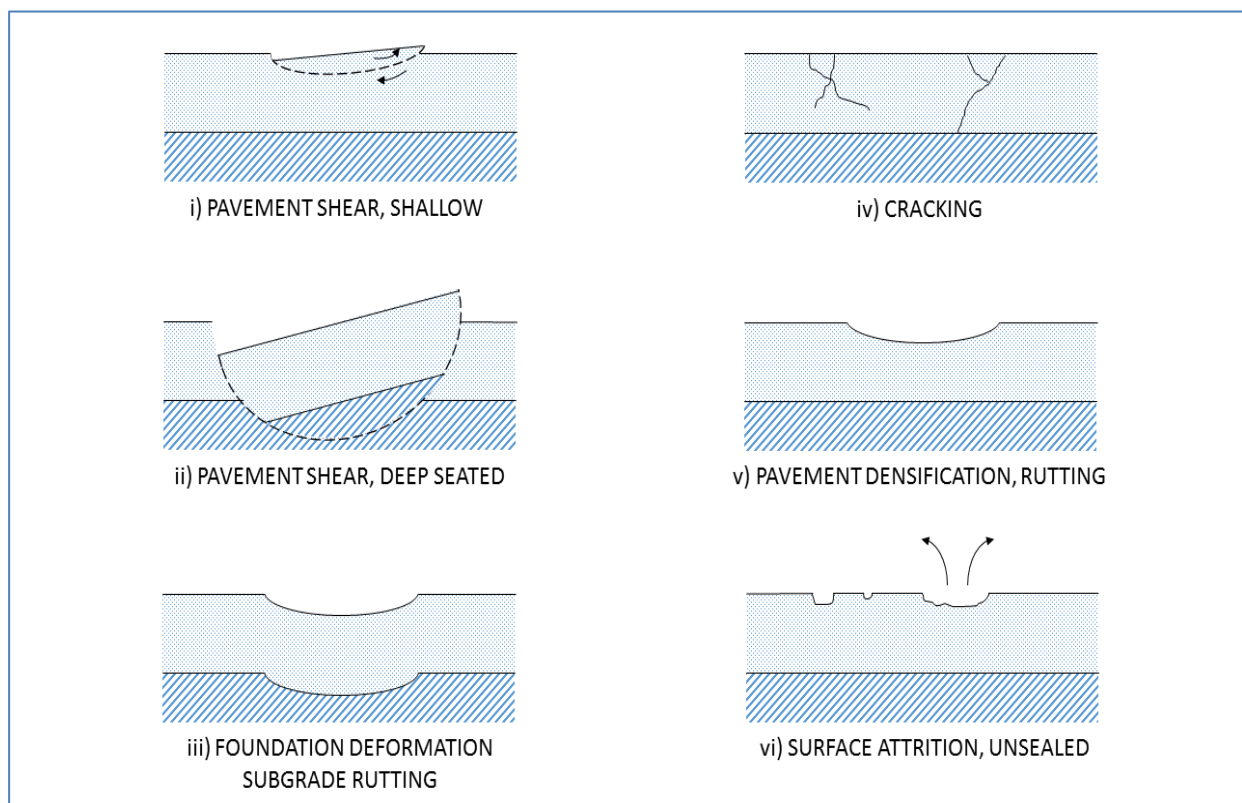
There are several technical factors influencing pavement stabilisation effectiveness on pavement construction, rehabilitation and maintenance works. These relate to how stabilisation addresses the probable cause(s) of the existing pavement defects and the selection of available binder types.

3.3.2 Pavement defects

Pavement rehabilitation and maintenance sites on existing roads and highways near new road 'greenfield' sites should be carefully assessed by the designer to ascertain: what the existing pavement defect(s) are; what this tells the designer about the existing pavement performance and the likely causes of the defect(s); and if/how pavement stabilisation could help to prevent similar defect(s) in the new pavement.

Common pavement defects are shown in figure 3.1.






Figure 3.1 Pavement defects




Source: National Roads Board (1987)

The common pavement defects are described further in table 3.1.

Table 3.1 Description and appearance of pavement defects (Source: William Gray and Google open source photographs)

Pavement defect	Typical appearance
<p>Pavement shear, shallow</p> <p>Deformation within the pavement layers resulting in movement sideways beyond the wheel track (usually combined with rutting). Caused by lack of shear strength (combined inter-particle friction and stable cohesion) within the unstable chip seal or asphaltic concrete (AC) layers, basecourse, basecourse/subbase.</p>	
<p>Pavement shear, deep seated</p> <p>Deformation within the pavement layers resulting in movement sideways beyond the wheel track (usually combined with rutting). Caused by lack of shear strength (combined inter-particle friction and stable cohesion) within the basecourse, subbase and subgrade. Often associated with outer wheel path shear into adjoining water tables where pavement shoulder support is lacking.</p>	
<p>Foundation deformation, subgrade rutting</p> <p>Deformation on the surface within the wheel track, through the pavement layers into the subgrade. If this occurs quickly, it can indicate the pavement is too thin. Controlled rut development of up to 20mm within the wheel path is expected over the design life of a typical unbound granular pavement.</p>	
<p>Cracking, within pavement</p> <p>Bound or lightly bound pavement layers that are too thin, and/or lack underlying support. These can crack when the tensile stresses imposed by the applied load exceed the tensile strength of the material, and display closely spaced alligator cracking.</p> <p>Widely spaced, often uniform transverse and/or longitudinal cracking in lightly bound and bound layers can also be caused by shrinkage following hydration.</p>	
<p>Pavement densification, rutting</p> <p>Densification (consolidation) within the pavement under traffic load can appear as rutting within the wheel path. Can occur quite quickly as a result of inadequate compaction and/or moisture control during construction, or more slowly during breakdown in the structure of the near surface pavement aggregate. Ruts tend to be sharper than rutting driven by subgrade deformation.</p>	

Pavement defect	Typical appearance
<p>Surface attrition, sealed or unsealed pavement</p> <p>In a sealed pavement, surface attrition of the sealed surface will manifest itself initially as seal scabbing (loss of chip) and shallow potholes. If left untreated can lead to deep potholes, water intrusion and basecourse failure.</p> <p>In unsealed pavements surface attrition of the wearing course is caused by dust loss in dry weather and the metal moving to the side of the road.</p>	

3.3.3 Probable cause of common pavement defects

Experience across New Zealand shows the probable cause(s) for the common pavement defects shown in figure 3.1 and table 3.1 are as described in table 3.2; refer also to NZ Transport Agency (2017a).

Table 3.2 Probable causes for common pavement defects

Pavement defect	Probable causes
Pavement shear, shallow	<p>Lower-quality basecourse includes little or no crushed faces, poorly graded or open graded material, poor aggregate durability, and lacks granular interlock and stable cohesion</p> <p>Surface or in-pavement water ingress and moisture sensitive aggregates</p> <p>Unstable seal layers includes higher bitumen content in thicker (>50mm deep) aged, multiple seal layers</p> <p>Overloading, notably high shear loads caused by turning movements or adverse road surface camber</p>
Pavement shear, deep	<p>Lower quality pavement layers (subbase and subgrade)</p> <p>Weak foundation support (subgrade support)</p> <p>Thinner pavement than needed to carry traffic load</p> <p>In-pavement or groundwater ingress over time</p> <p>Overweight traffic loads</p>
Foundation deformation, subgrade rutting	<p>Low strength foundation (subgrade)</p> <p>Thinner pavement than needed to carry traffic load</p> <p>Water infiltration, high ground water table/fluctuation</p> <p>Overweight traffic loads</p> <p>Pavement design life has been reached</p>
Cracking within pavement	<p>Lower strength foundation (subgrade) causing flexure</p> <p>Thinner pavement causing flexure</p> <p>Water infiltration into sensitive pavement and/or subgrade materials causing reduced foundation support</p> <p>Overweight traffic loads</p>
Pavement densification, rutting	<p>Inadequate pavement layer compaction (eg low density, high air voids, saturation effects)</p> <p>Water infiltration into sensitive pavement materials resulting in reduced strength and 'lubrication' of the fines matrix in the presence of plastic fines</p> <p>Pavement aggregate in-service breakdown</p> <p>Overweight traffic loads</p>

Pavement defect	Probable causes
Surface attrition	<p>Unsealed surface lacking cohesion and wet/dry stability</p> <p>Poor surface water drainage</p> <p>Aged seal with brittle binder, causing chip loss and cracking</p> <p>Poor adhesion of first coat seal to basecourse surface, absence of dense stone mosaic surface in basecourse, wet, poor 'slurry cake' surface prior to sealing</p> <p>High temperature seal instability (bleeding) then chip loss</p> <p>Sliding of thin AC surface layers</p>

3.3.4 Treatment selection – response outcomes

Table 3.3 Pavement treatment response to common pavement defects

Pavement defect	Pavement treatment response
Pavement shear, shallow	<p>Waterproof surface (reseal)</p> <p>Modify existing multiple seal layers (stabilisation/recycling)</p> <p>Strengthen basecourse or overlay</p> <p>Reduce moisture sensitivity in upper pavement layers</p> <p>Pavement drainage</p>
Pavement shear, deep	<p>Increase new pavement depth or overlay existing pavement</p> <p>Strengthen subgrade and/or use subgrade improvement layer (more practical for 'greenfield' pavements)</p> <p>Increase upper pavement shear strength to increase load capacity and reduce strains in lower pavement</p> <p>Reduce moisture sensitivity in the pavement and subgrade</p> <p>Improve pavement shoulder support</p> <p>Pavement drainage</p>
Foundation deformation, subgrade rutting	<p>Infill ruts and reseal (re-set pavement design life) if the overall pavement sound</p> <p>Increase pavement depth, overlay existing pavement</p> <p>Stabilise/recycle basecourse and reshape</p> <p>Strengthen subgrade and/or use subgrade improvement layer (more practical for 'greenfield' pavements)</p> <p>Reduce moisture sensitivity in the pavement and subgrade</p> <p>Increase upper pavement shear strength to increase load capacity and reduce strains in subgrade</p> <p>Pavement drainage</p>
Cracking, within pavement	<p>Waterproof surface (reseal) and cracked seal</p> <p>Strengthen lightly bound and bound layers</p> <p>Overlay with unbound or lightly modified aggregate</p> <p>Increase subgrade strength</p> <p>Reduce moisture sensitivity in the pavement and subgrade</p> <p>Pavement drainage</p>
Pavement densification, rutting	<p>Infill ruts and reseal</p> <p>Waterproof surface (reseal) only if rutting not too bad</p> <p>Strengthen basecourse or overlay</p> <p>Rip and remake with adequate compaction and moisture control</p> <p>Reduce moisture sensitivity in the pavement</p> <p>Pavement drainage</p>

Pavement defect	Pavement treatment response
Surface attrition	Make up lost wearing course in unsealed road with clay rich, low silt material Waterproof surface (reseal) and fill potholes Strengthen basecourse or overlay, allowing a clean stone mosaic surface to be achieved which will bond well with a new seal Reduce moisture sensitivity in the pavement Pavement drainage

3.3.5 Stabilisation – binder types and benefits




The typical pavement treatment responses described in table 3.3 can all make use of pavement stabilisation.

A guide to selecting the best binder for use in pavement stabilisation is shown in table 3.4.

Table 3.4 Guide to selecting common stabilisation binder types in New Zealand

Characteristic pavement material particle size		Fine grained pavement material > 25% passing 0.425 mm sieve			Coarse grained pavement material < 25% passing 0.425mm sieve		
Plasticity index (PI)		PI ≤ 10	10 < PI < 20	PI ≥ 20	PI ≤ 6	PI ≤ 10	PI > 10
Binder type	Cement and cementitious blends*		Lime pre-treatment desirable	Lime pre-treatment essential			Lime pre-treatment desirable
	Lime as hydrated or burnt lime (CaO)	Additional drying action with CaO	Additional drying action with CaO	Additional drying action with CaO	Additional drying action with CaO	Additional drying action with CaO	Additional drying action with CaO
	Hot bitumen						
	Bitumen emulsion**						
	Foamed bitumen**		Lime pre-treatment desirable	Lime pre-treatment essential			Lime pre-treatment
	Granular		Lime pre-treatment desirable	Lime pre-treatment essential		Lime pre-treatment desirable	Lime pre-treatment desirable
	Polymer***						

KEY

	Usually suitable
	Doubtful or supplementary binder required
	Usually not suitable

Notes: * Includes fly ash ** Bitumen emulsion and foamed bitumen can be used with other binders (typically small quantities of cement) *** Includes proprietary polymer materials used as dust suppression and finer soil particle modifier

3.3.6 Expected benefits from pavement stabilisation

The expected benefits from using pavement stabilisation in new pavement construction and existing pavement rehabilitation and maintenance are discussed in table 3.5. This information is framed within the context of commonly used binders and expected benefits for a number of pavement layer and expected layer stabilisation options. These start with the subgrade and progress through to the basecourse. The SIL is delivered either by in-situ stabilisation of the existing subgrade's upper levels, or by new earthworks using cut to fill or borrow to fill materials (NZ Transport Agency 2016a). The framework for table 3.5 is drawn from AustStab (2015, table 1).

Table 3.5 Expected benefits from pavement stabilisation in New Zealand

Pavement layer	Stabilisation option	Commonly used binders	Expected benefits
Subgrade	Unbound, modified subgrade	Lime (hydrated or burnt) Lime/cement blends	Dry out material during construction Reduce long-term moisture sensitivity Strength improvement: CBR around 10% to 15%, underlying layer stiffness usually drives dependable long-term strength – including consideration for modular ratio of ~3 maximum.
	Lightly bound subgrade	Lime/cement blends Cement or cementitious blends	As above: Pure sand subgrades would benefit from cement alone; however, in cases where cohesive soil is present lime/cement blends would be needed.
Subgrade improvement layer (SIL)	Unbound, modified SIL	Lime (hydrated or burnt) Lime/cement blends	Dry material during construction Reduce long-term moisture sensitivity Strength improvement: CBR up to 15% depending on quality of SIL material, and sub-layering rules.
	Lightly bound SIL	Lime/cement blends Cement or cementitious blends	As above Strength improvement: CBR up to 15%, depending on quality of SIL material, and sub-layering rules.
Subbase	Unbound, modified subbase	Lime/cement blends Cement or cementitious blends	Dry material during construction Reduce long-term moisture sensitivity CBR 30% to 80% UCS < 1.0MPa ITS < 200kPa
	Lightly bound subbase	Lime/cement blends Cement or cementitious blends Bitumen/bitumen emulsion Foamed bitumen/cement	CBR 60% to >100% 1.0MPa < UCS < 2.0MPa 200kPa < ITS < 600kPa $E_{resilient} < 2000\text{MPa}$
	Bound subbase	Lime/cement blends Cement or cementitious blends Bitumen/bitumen emulsion	ITS > 600kPa UCS > 3.0MPa $E_{resilient} > 2,000\text{MPa}$ which reduces over time if cracking occurs or reduces immediately if layer pre-cracked. Consider potential impacts of fatigue or thermal cracking

Pavement layer	Stabilisation option	Commonly used binders	Expected benefits
Basecourse (top surface layer can include previous surfacing)	Unbound, modified basecourse	Lime/cement blends Cement or cementitious blends	Dry material during construction Reduce long-term moisture sensitivity CBR 80% to 100% UCS < 1.0MPa ITS < 200kPa, when indexed to the T/19 test method
	Lightly bound basecourse	Lime/cement blends Cement or cementitious blends Bitumen/bitumen emulsion Foamed bitumen/cement	CBR > 100% 1.0MPa < UCS < 2.0MPa 200kPa < ITS < 600kPa $E_{resilient} < 2,000\text{MPa}$ which reduces over time if cracking occurs or reduces immediately if layer pre-cracked
	Bound basecourse	Lime/cement blends Cement or cementitious blends Bitumen/bitumen emulsion	ITS > 600kPa $E_{resilient} > 2,000\text{MPa}$ which reduces over time if cracking occurs or reduces immediately if layer pre-cracked Depth and strength of layer needed to mitigate unplanned fatigue cracking

Note: Abbreviations: CBR = California bearing ratio; SIL = subgrade improvement layer; UCS = unconfined compressive strength; ITS = indirect tensile strength

4 Material characterisation and testing

4.1 Introduction

New pavement and pavement rehabilitation projects can make use of material stabilisation in a variety of ways: foundation (subgrade) improvement; pavement layer (subbase and basecourse) modification; surfacing (AC and stone mastic asphalt (SMA)).

The material characterisation and testing to support pavement stabilisation in New Zealand is normally categorised according to the binder type. The categories are cement stabilisation, lime stabilisation, bitumen stabilisation, polymer stabilisation and granular stabilisation.

Supporting information on material characterisation and testing is published in AustStab (2015, chapters 4, 7, 8, 9, 10 and 11).

The following section discusses stabilised material characterisation and testing in New Zealand.

Specific reference is made to the worked design examples in chapter 5.

4.2 Cementitious stabilisation

Cementitious stabilisation uses either cement or supplementary cementitious materials. These include fly ash or pulverised blast furnace slag (typically from the Glenbrook steel plant near Auckland) with or without lime. Lime may be considered as a cementitious material when mixed with other pozzolans. Anecdotal and published research evidence from older lime stabilised pavements in New Zealand (eg the Paremoremo Road north of Auckland) shows lime stabilised pavement layer strength at least remains constant and can even increase over time.

Supporting information on material characterisation and testing to support cementitious stabilisation is published in chapter 7 of AustStab (AustStab 2015).

Cement stabilisation typically delivers lightly bound or bound pavement outcomes (Gray et al 2011) depending on the cement binder content and often on construction conditions.

The required binder content will vary depending on desired outcome and the characteristics of the aggregate (or soil) being stabilised. Typical cement stabilised material properties reported from New Zealand project work, are shown in table 4.1.

Table 4.1 Typical properties using cement stabilisation

Material	Design thickness (mm)	Cement content (% of dry mass)	Strength ² (MPa)	Resilient modulus (MPa)
Unbound (modified)	Varies	<1%, depending on plasticity in treated material	UCS < 1MPa ITS < 200kPa	<1,000MPa
Lightly bound	200mm to 300mm	1% to 2%	1MPa < UCS <2MPa ITS <600kPa	≤2,000MPa
Bound	>300mm, built in two layers	>2%	UCS >2MPa ITS >600kPa	3,500 to 15,000MPa

² Sample compaction to NZ Standard NZS 4402 (test 4.1.1)

The sample compaction level and sample aspect ratio will affect the UCS material strength measurements for both lightly bound and bound materials. The UCS strength ranges shown in table 4.1 are typical of standard compaction and 2:1 sample aspect ratio. Anecdotal evidence has UCS >3MPa for bound materials under heavy compaction (Standards NZ 2015).

For indirect tensile strength (ITS) testing, the T/19 specification (NZ Transport Agency 2016c) uses the vibrating hammer compaction (Standards NZ 2015, test 4.1.3). The ITS strength ranges shown in table 4.1 are for vibrating hammer compaction.

The practicalities of constructing an unbound (modified) stabilised material using cement are often limited because of the very low cement content (refer table 4.1) needed to truly deliver an unbound material. Any variability of cement content application upwards, fines in the mixed material and construction water content can quickly lead to lightly bound or even bound behaviour (at least initially).

The design of pavements including unbound cement modified materials is demonstrated in the worked examples in chapter 5.

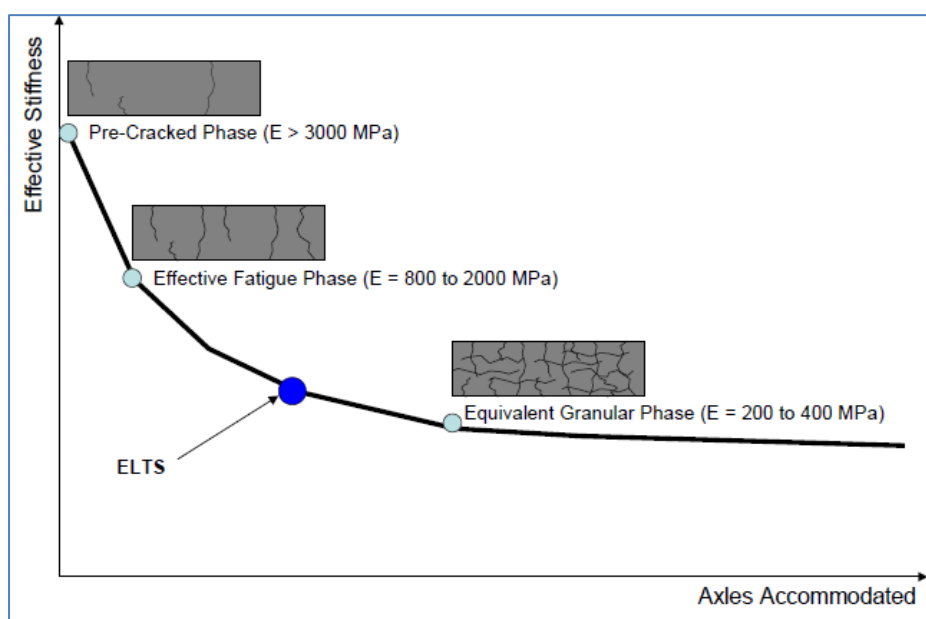
For lightly bound and more particularly bound pavement layers, the necessary design thickness (refer to table 4.1) is affected by tensile capacity and fatigue performance in the stabilised material and by construction variables including available plant mixing capacity, fines in the mixed material and construction water content.

4.2.1 Lightly bound pavement layers

A lightly bound stabilised material with $1\text{MPa} < \text{UCS} < 2\text{MPa}$ and $\text{ITS} < 600\text{kPa}$ can be made. The process involves adding a quantity of cement (1% to 2% by dry mass) to an existing well-graded aggregate via either proprietary pug mill or in-situ stabilisation processes. Lightly bound cement stabilised aggregates are modelled mechanistically as isotropic, a resilient modulus < 2,000MPa, a Poisson's ratio of 0.2 and no sub-layering.

New Zealand and international research shows that when controlled microcracking occurs the lightly bound material can migrate back to a near equivalent unbound condition over time. South African guidelines (SARA 2014) describe this as the ELTS (see figure 4.2).

Figure 4.2 Effective long- term stiffness



Source: SARA (2014)

Current Austroads best practice limits the cemented ELTS layer modulus to: three times the underlying layer modulus; or a maximum of 500MPa, anisotropic, Poisson's ratio 0.35, with no sub-layering (Austroads 2012).

Current best practice in New Zealand means the pre-cracked phase is not considered and sub-layering is required. Unbound, maximum $E_v < 700\text{MPa}$, anisotropic, Poisson's ratio 0.35, sub-layered. The actual E_v value attained is dependent upon the E_v value of the layer below and the depth of the layer, controlled by sub-layering. To mitigate the risk of 'eggshell behaviour' (stiff layer on thin pavement cracking) the residual UCS of the ELTS material is $< 1\text{ MPa}$ using standard compaction and a 2:1 height: diameter ratio for the mould.

The recommended thickness of the lightly bound layer should not be $< 200\text{mm}$ compacted depth.

Pavement layers in the ELTS phase continue to provide some enhanced load-bearing support, so long as the sealed surfacing's integrity is maintained and the underlying pavement support is consistent and sustainable.

In cases where the underlying pavement support is weak, and/or not consistent and sustainable, the ELTS phase material should be treated as unbound, maximum $E_v < 500\text{MPa}$, anisotropic, Poisson's ratio 0.35, sub-layered.

The design of pavements including lightly cement bound materials is demonstrated in the worked examples in section 5.5.

4.2.2 Bound pavement layers

In bound pavement layers, cracking is controlled by a combination of traffic loading, stabilised layer strength, layer depth and consistent, sustainable underlying pavement support.

A bound stabilised material with UCS $> 2\text{MPa}$ under standard compaction and ITS $> 600\text{kPa}$ could be achieved. It would likely involve adding 4% or more by dry mass of cement to an existing well-graded aggregate via either proprietary pug mill or in-situ stabilisation process.

Bound cement stabilised aggregates are modelled mechanistically as isotropic, a resilient modulus typically $> 3,500^3\text{MPa}$, a Poisson's ratio of 0.2, without sub-layering.

The common fatigue relationship for bound pavement layers is shown in equation 4.1⁴.

$$N = \left(\frac{k}{\mu\varepsilon} \right)^n \quad \text{(Equation 4.1)}$$

The fatigue constant k in equation 4.1 is related to the bound layer's resilient modulus, and exponent n is typically 12 for cemented materials.

Austroads research on fatigue constants (Austroads 2014) concludes cement bound materials can be designed based on strain and stress control. Strain control fatigue constants can be determined in the laboratory using the flexural beam test (Austroads 2014; NZ Transport Agency 2017a).

Austroads report presumptive fatigue constants for bound basecourse and subbase quality aggregates. These are shown in table 4.2, reproduced from Austroads (2014).

³ Austroads uses 2,000MPa

⁴ Published fatigue relationships are currently under review both in New Zealand and Australia based on in-service and accelerated pavement testing research.

Table 4.2 Presumptive fatigue constants for bound pavement layers

Property	Base quality granular material 4–5% cement	Subbase quality crushed rock 3–4% cement	Subbase quality natural gravel 4–5% cement
Typical modulus (MPa)	5000	4000	3000
Typical flexural strength (MPa)	1.4	1.2	1.0
In-service fatigue constant K	272	270	304

Bound pavement layers would be configured to be strong and deep enough, with adequate support to carry the design traffic load N without cracking.

In some cases the bound pavement layer may be pre-cracked. If cracking were to occur, the bound pavement layer could be designed to perform in the post-cracked ELTS phase. The pavement designer needs to carefully consider whether controlled pre-cracked materials in basecourse materials can be achieved consistently. Great care is still needed to mitigate against the risk of post-construction shrinkage cracking. This would include careful control of construction cement and water contents.

Current Austroads best practice limits the cemented ELTS layer modulus to a maximum of 500MPa, anisotropic, Poisson's ratio 0.35, with no sub-layering. Mix design is supported by the T/19 specification. Additional support can be provided by limiting the layer modulus to three times the underlying layer modulus. The difficulty the designer faces here is estimating, measuring or back calculating the underlying mid-pavement layer modulus from deflection measurements.

In New Zealand, controlled micro-cracking is now more common when used in conjunction with a bound subbase layer.

The cemented pavement layer resilient moduli are reported as hugely variable both in the laboratory and even more in the field. They can range from 2,000MPa to 20,000MPa. The tensile strength at the break point is more reliably measured in the laboratory, either as flexural strength or twice (2x) the ITS.

Until now the most commonly used test for characterising the strength of cementitious materials was the UCS test. The results for this test can be used to give a preliminary estimate of the resilient modulus:

$$E=1,000*UCS \text{ MPa} \quad \text{(Equation 4.2)}$$

Experience in New Zealand shows the UCS result variability can be high. A fine grading in an aggregate material when mixed with low cement contents (1%) can still give high UCS results. If the UCS test were used to help the designer estimate the design modulus, it should always be used in conjunction with the ITS.

Specification T/19 (NZ Transport Agency 2016c) now promotes the use of the ITS test (with wet/dry ITS comparison) for strength characterisation in cementitious materials. This is because research suggests the compression testing is more sensitive to variations in the properties of materials and sample size than tensile testing where results are more closely grouped.

The modulus E for the cement bound layer can be used in mechanistic design to estimate the governing tensile stresses in the material. By varying the layers' strength and depth, and the underlying support conditions, the tensile stresses can be kept <50% of the tensile strength.

Current New Zealand-based research shows if the stress ratio (SR) is kept below 0.5, the design traffic can be conservatively estimated as up to 1MESA without risk of fatigue cracking.

4.2.3 Roller compacted concrete

Unreinforced concrete pavements are known to be used in some commercial applications.

According to sources from the United States (Harrington et al 2010) rollers used to compact it into its final form. RCC is similar to, and has similar strength properties as, conventional concrete: it is composed of well-graded aggregates, cementitious materials and water. RCC has different mixture proportions and a higher percentage of fine aggregates. These allow for tight packing and consolidation, but can increase the risk of shrinkage cracking if the moisture content is wrong.

Fresh RCC is stiffer than typical zero-slump conventional concrete. The consistency is stiff enough to remain stable under vibratory rollers, yet wet enough to permit adequate mixing and paste distribution without segregation.

RCC is typically paver laid with a standard or high-density screed, followed by a combination of passes with rollers for compaction. Final compaction is generally achieved within one hour of mixing. Unlike conventional concrete pavements, RCC pavements are constructed without forms, dowels or reinforcing steel. Joint sawing should be specified to mitigate the risk of shrinkage cracking. Transverse joints are spaced farther apart than with conventional concrete pavements.

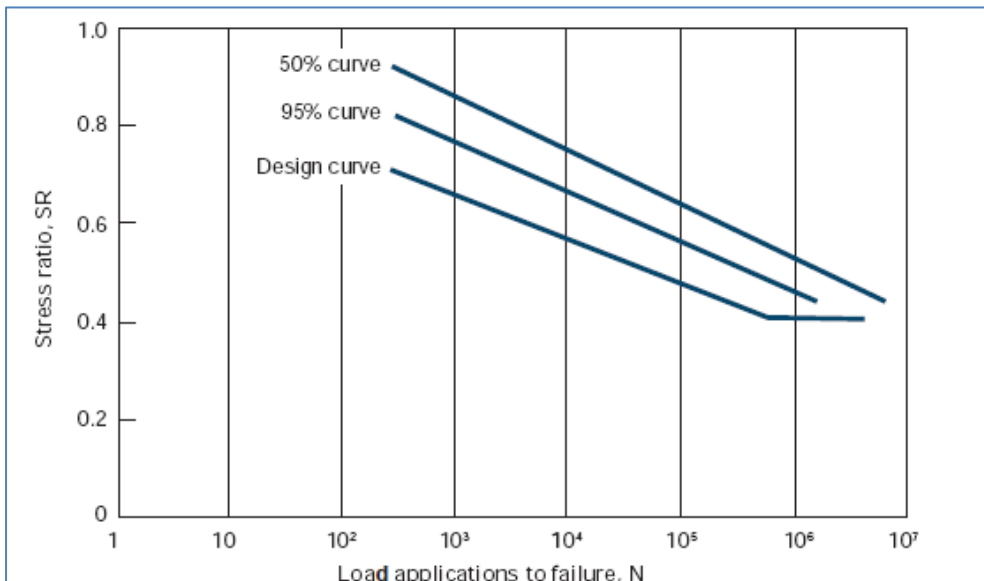
A typical relationship for converting 28-day compressive strength to 28-day flexural strength for concrete with crushed aggregate is shown in equation 4.3:

$$f_{cf} = 0.75 \sqrt{f_c} \tag{Equation 4.3}$$

Where: f_{cf} = 28-day concrete flexural strength (MPa)
 f_c = 28-day concrete compressive strength (MPa)

The RCC pavement designs control the tensile stress within the bound layer to less than 40% of the tensile strength of the material. This is shown in figure 4.3. Pavements designed using cement bound materials are demonstrated in the worked examples in section 5.6.6.

Figure 4.3 Fatigue relationship for RCC based on stress ratio



Source: Harrington et al (2010)

4.3 Lime stabilisation

Lime is an effective binder for plastic soils $PI > 10$. It improves both workability and strength, and reduces moisture content. This enables the stabilised material to be re-classified accordingly. In granular soils, lime reduces plasticity in the fine particle fraction and overall moisture content, while providing some cementation, as a result of pozzolan reaction.

Supporting information on material characterisation and testing to support lime stabilisation is published in chapter 8 of AustStab (2015).

Typical material properties of lime stabilisation for use in mechanistic design are given in table 4.3.

Table 4.3 Typical properties of lime stabilisation

Material	Design thickness (mm)	Lime content (% of dry mass)	Strength attained ⁵ (MPa)	Resilient modulus (MPa)
Unbound soil	Varies	3% to 5%	CBR 10% to 30%	$E = 10 \times \text{CBR}$, max CBR 15%, selected subgrade sub-layering applies (refer section 5.4.1)
Unbound aggregate	150mm to 200mm	3% to 5%	CBR 30% to 100%	Austrroads unbound granular <500MPa sub-layered

Pavements designed using lime stabilised materials, including a lime stabilised subgrade improvement layer and modified basecourse, are demonstrated in worked examples in chapter 5.

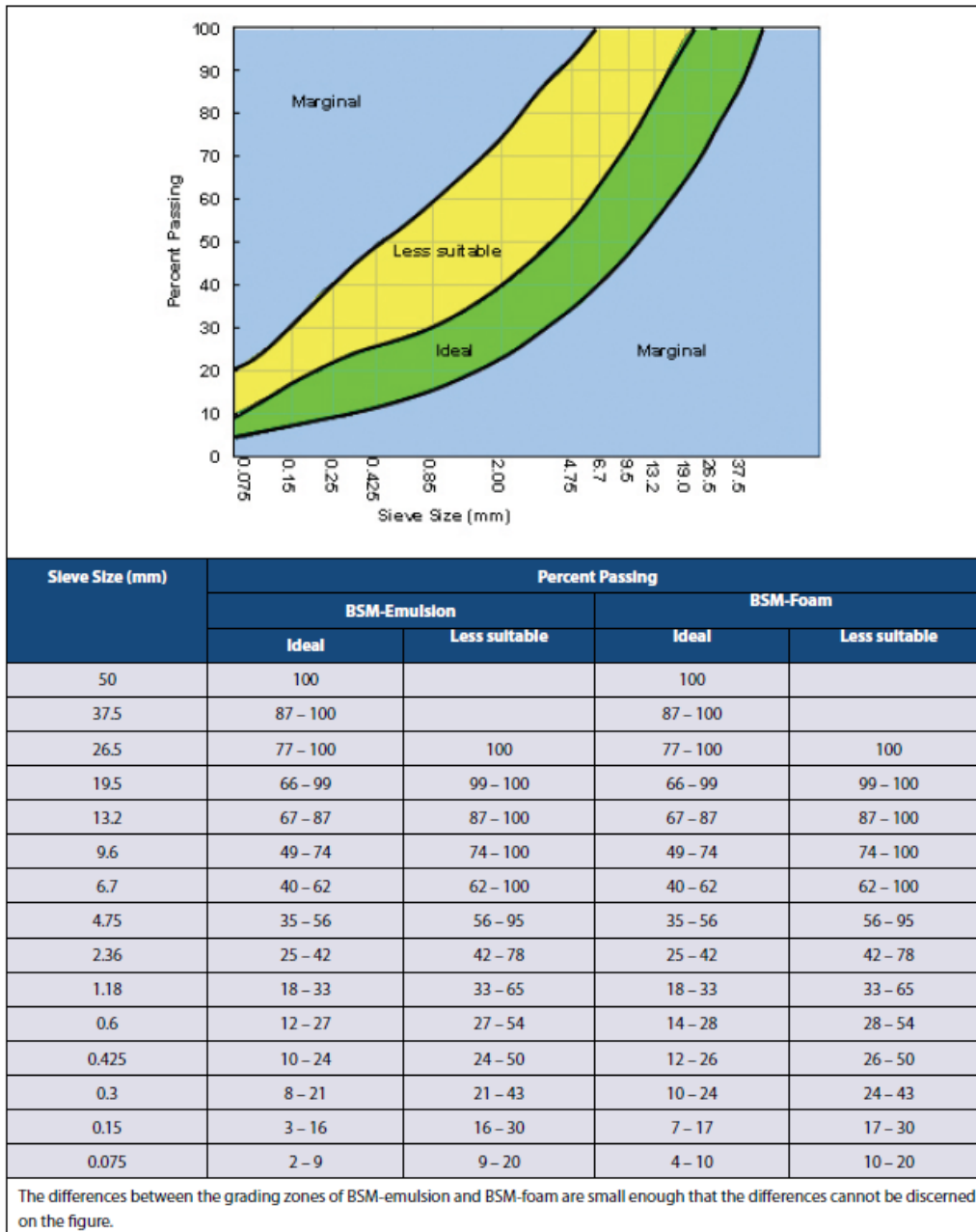
4.4 Bitumen stabilisation

Bitumen stabilised materials (BSMs) are pavement materials treated with either foamed bitumen (BSM-foam or FBS) or bitumen emulsion (BSM-emulsion), with or without added fillers including lime and cement.

BSM suit mixing with fine graded granular materials and recycled pavement materials with increased fines (over M/4), including those with recycled aged seals or asphalt, as shown in figure 4.4.

⁵ Sample compaction to New Zealand heavy (test 4.1.2)

Figure 4.4 Aggregate grading for BSMs



Source: Asphalt Academy (2009)

BSM are used to introduce strength and cohesion into granular materials, and to reduce moisture sensitivity in stabilised materials.

Supporting information on material characterisation, mix design and testing to support BSM is published in chapter 9 of AustStab (2015).

Around New Zealand, anecdotal evidence shows foamed bitumen stabilised (FBS) material can form a ‘super granular’ pavement layer (typically as a FBS basecourse).

Mix design utilises the foaming characteristics of the bitumen binder and the dry/wet ITS to allow the producer to formulate the optimal job mix formula. This is in line with the Asphalt Academy's (2009) recommendations, and the test methods given in the T/19 specification (NZ Transport Agency 2016c).

Mix design with bitumen emulsion binder would be similar, but excludes consideration of bitumen foaming characteristics.

Current best practice in New Zealand limits the FBS layer modulus to five times the underlying layer modulus to a maximum of 800MPa (NZ Transport Agency 2017a) and (NZ Transport Agency 2017b) BSM mix design is supported by the T/19 specification (NZ Transport Agency 2016c).

Pavements designed using BSM as lightly bound materials are demonstrated in the worked examples in section 5.5.

4.5 Chemical polymer stabilisation

Chemical polymers are used successfully as dust suppressants and as aids to fine grained material strength improvement and reduced water sensitivity. When used in combination with quantities of lime and/or (more typically) cement, the chemical polymer stabilisation of fine graded pavement aggregates has demonstrated improved performance (resistance to cracking) over cement-only pavement maintenance repairs in regions of New Zealand including central and east Waikato.

Chemical polymer additives can be added in dry powder, pellet or liquid forms.

Supporting information on material characterisation, mix design and testing to support polymer stabilisation is published in chapters 11 and 13 of AustStab (2015).

Pavements designed using chemical polymer stabilisation would be undertaken in a similar manner to either unbound or lightly bound cementitious stabilisation. They would be based on laboratory ITS or flexural beam response and stress ratio control, as demonstrated in the worked examples in section 5.5.

4.6 Granular stabilisation

Improving an existing granular material with one or more imported granular make up materials is referred to as granular stabilisation. This approach mechanically alters the particle size distribution. This in turn affects other material properties including permeability, sand equivalent and plasticity. Most importantly, it has the proven potential to improve the stabilised aggregate's density/granular interlock/shear resistance.

Supporting information on material characterisation, mix design and testing to support granular stabilisation is published in chapter 10 of AustStab (2015).

Pavements designed using granular stabilisation are demonstrated in the worked examples in chapter 6.

4.7 Laboratory testing

Laboratory testing is used to support the informed investigation, design, construction, maintenance and operation of stabilised pavements.

The range of tests can be broadly categorised as:

- classification tests – including particle size distribution (grading), plasticity index, sand equivalent, clay index, permeability determination

- reactivity tests – to consider which binder (or mix of binders) reacts with the pavement materials, the CBR test can be used to test strength gain (soaked and unsoaked) when mixed with different binder combinations
- compaction tests – including maximum dry density (MDD), optimum moisture content (OMC), total and air voids, saturation index determination in the laboratory under either standard, heavy or vibrating hammer compaction, and in the field typically using either the nuclear densometer or sand replacement tests
- resistance to load tests – including CBR determination
- strength tests – including unconfined compression strength, indirect tensile strength, shear strength, flexural strength and fatigue strength characterisation.

Supporting information is published in chapter 4 of AustStab (2015).

4.7.1 Classification, reactivity and compaction tests

In relation to the classification and compaction tests described above, reference should be made to the following guidelines and standards (and any subsequent updates):

- *Field description of soil and rock* (NZ Geotechnical Society 2005)
- *NZS 4407:2015 Methods of sampling and testing road aggregates* (Standards NZ 2015)
- *NZS 4402:1986 Methods of testing soils for civil engineering purposes* (Standards NZ 1986a)

Reactivity tests typically make use of the CBR test (refer section 4.7.2)

4.7.2 Resistance to load test – CBR

The CBR test is completed by an International Accreditation New Zealand (IANZ) accredited laboratory using one of two test methods from NZS 4402 standard specification (Standards New Zealand 1986a), as shown below:

- *NZS 4402. 6.1.1: 1986 Methods of testing soils for civil engineering purposes –soil strength tests – determination of the California bearing ratio (CBR) – test 6.1.1 Standard laboratory method for remoulded specimens*
- *NZS 4402. 6.1.2: 1986 Methods of testing soils for civil engineering purposes – soil strength tests – determination of the California bearing ratio (CBR) – test 6.1.2 Standard laboratory method for undisturbed specimens.*

The CBR remains a quintessential test method in New Zealand. In this the pavement subgrade's load bearing strength is characterised for pavement design. The subgrade material's reactivity to stabilising agents can also be assessed by comparing the CBR outcomes for treated and untreated samples.

Key aspects the pavement designer needs to consider when briefing the laboratory to complete CBR tests include:

- The sample moisture content and target density outcomes should be representative of the poorer in-situ conditions
- Both soaked and unsoaked CBR tests should be completed when the designer needs to assess the potential adverse effects of changes in groundwater levels (if these are not being otherwise controlled by subsoil drains, embankment heights etc as part of the wider geometric design considerations).

The CBR test is also a routine production property test for pavement aggregates. A premium M/4 basecourse is expected to have a minimum CBR >80%, typically 100%. CBR values higher than 100% are often reported when testing cement binder reactivity with aggregate materials. Such high values are indicative of a good cement reaction. They should not be used to derive modulus values for subsequent mechanistic design.

4.7.3 Strength tests

The strength tests currently in variable use in New Zealand include the unconfined compression strength, shear strength, flexural beam strength and fatigue characterisation, and indirect tensile strength.

4.7.3.1 Unconfined compression test (UCS): NZS 4402, test 6.3.2

The UCS test is used to characterise stabilised materials as either lightly bound or bound (refer table 4.1) and then to support the design of bound pavements, using the UCS/modulus conversation ($E=1000*UCS$ MPa). Care needs to be taken with the aspect ratio of the sample when completing the UCS test. The level of compaction used on the sample (standard, heavy or vibrating hammer compaction) will also influence the UCS test outcome. Presumptive UCS test results reported herein generally come from using standard compaction samples.

4.7.3.2 Indirect tensile strength test

For stabilised materials the ITS test is completed in accordance with the T/19 specification (NZ Transport Agency 2016c). The test is less dependent on the sample aspect ratio than it is for the UCS test. The tensile strength of the material is twice the ITS (tensile strength = 2*ITS). Therefore the ITS test is useful when designing lightly bound and bound pavements (refer section 4.2).

4.7.3.3 Shear strength test: repeat load triaxial (RLT)

The shear strength measurement in a compacted, stabilised soil or aggregate sample can be undertaken using the RLT.

The RLT test for use in New Zealand for pavement subgrade and aggregate materials is provided in the T/15 specification (NZ Transport Agency 2014). In this test method, the sample is subjected to repeat loading under six different axial/confining stress combinations. The subsequent stress strain measurement and reporting support the: resilient modulus determination; sample vertical compressive strain stability and rut potential under the ESA loading.

Supporting information on the RLT test is published in the *New Zealand guide to pavement evaluation and treatment design* (NZ Transport Agency 2017a).

The direct shear and shear vane tests are only applicable to unbound fine grained soils.

4.7.3.4 Flexural beam test

The flexural beam test measures the tensile strength, flexural modulus and fatigue life of stabilised material.

The tensile strength can then be used in the design of bound pavements, where the tensile stresses are kept <40% of the tensile strength ($SR < 0.4$), refer section 4.2.

The flexural modulus and fatigue history under repeated loads can be used to determine the constant 'k' in the fatigue relationship below, when examined under the RLT test to a pre-set tensile strain level. This is determined by the designer based on the expected in-service pavement response, refer section 4.2.2.

Supporting information on the RLT test is published in NZ Transport Agency (2017a).

4.8 Field testing

Field testing is used to support the investigation, design, construction, operation and maintenance of stabilised pavements.

The range of tests can be broadly categorised as:

- classification tests – including in-situ field water content, soil and rock descriptions (NZ Geotechnical Society 2005)
- compaction tests – nuclear densometer, sand replacement
- strength tests – including shear strength testing using the hand-held shear vane (NZ Geotechnical Society 2001)
- resistance to load tests – including in-situ CBR determination, dynamic cone penetrometer (DCP: Scala penetrometer), Loadman, Clegg hammer test and layer surface deflection testing (falling weight deflectometer ((FWD)) and Benkelman beam tests)

Supporting information on quality assurance and field testing is published in chapter 14 of AustStab (2015), and in *Minimum standard Z/8* (NZ Transport Agency 2017d).

Technical information supporting the classification, compaction and strength tests listed above is also available from a number of other sources, including NZS 4402 (Standards NZ 1986a) and NZS 4407 (Standards NZ 2015). See these sources for further information.

4.8.1 Resistance to load tests

4.8.1.1 In situ CBR test: NZS 4402: test 6.1.3

The in-situ CBR test should be completed on soil or aggregate layers compacted to optimum moisture content and density outcomes, and are present at representative long term in-situ moisture contents. Because the in-situ CBR test is slow and time consuming, parallel testing with a DCP: Scala test, hand-held shear vane, Loadman or Clegg hammer can be used. These tests will provide local soil calibrations between in-situ CBR, in-situ moisture content and other test results, enabling the latter to be used more frequently and cost effectively for routine quality control.

4.8.1.2 Dynamic cone penetrometer (DCP: Scala): NZS 4402: test 6.5.2

The Scala penetrometer is probably the most common field test used in New Zealand for CBR and bearing capacity estimation. The Scala is best suited for use in sand and finer-grained granular soils. Estimating CBR and bearing capacity is made using the 'DCN' number or 'e' value (mm/blow).

While much reliance is made of the DCP/CBR estimation, great care is needed. Published research (Wesley 1998) described the CPT test's unreliability in pumice sands, particularly when comparing relative density. Like its older CPT cousin, the DCP Scala will probably be unreliable when used to test light-weight, high-friction pumice sands. At the other end of the material spectrum, care is needed with heavy plastic clay to prevent the soil 'grabbing' the DCP shaft. (This can be mitigated in part by combined hand auger/DCP test.) On-site calibration between DCP and in-situ CBR tests for example (tested at a lower bound representative moisture content) is used to improve the reliability of the DCP outcome at a project level.

4.8.1.3 Loadman and Clegg hammer

These tests correlate the surface impact resistance of the pavement layer by decelerating the dropped weight on the surface to a single index (eg Clegg impact value) and interpolated CBR or modulus. On-site

calibration between test and in-situ CBR tests for example (tested at a lower bound representative moisture content) should be used to improve the reliability of the test outcome at a project level.

4.8.1.4 Surface deflection testing

The FWD, and to a lesser degree the Benkelman beam test, enable designers to measure the deflection bowl beneath the pavement design axle. Back analysis of the deflection bowl using software tools, such as ELMOD, EFROMD2 and CIRCLY, allow designers to estimate pavement layer moduli. The more information available to support the back analysis (eg test pit information measurement material type and layer depth) the more reliable the pavement back analysis.

In New Zealand, the significant nationwide FWD historic data records enable FWD analysts to compare pavement performance. These comparisons include stabilised pavements. Detailed analysis now enables pavement designers to consider and use regional subgrade precedent strain and fatigue relationships in lightly bound and bound stabilised pavement layers.

Chapter 8 of this guide has further information on these research advances.

5 Pavement structural design

5.1 Introduction

Pavement stabilisation design opportunities extend from within the foundation (subgrade) through subgrade improvement layer(s) to the pavement layers (subbase and basecourse). They also extend into the surfacing (AC and SMA) where end-of-life treatments for aged surfacing can include recycling options with stabilisation. Structural design includes determining the stabilised material's initial and long-term dependable properties, and deciding on the contribution stabilisation can make to individual pavement layer and overall pavement system performance.

The structural design of stabilised pavements is described in chapter 5 of the AustStab (2015) guide.

The following section discusses how pavement structural design methods are to be implemented in New Zealand using current best practice.

5.2 Design methods

Empirical and mechanistic design methods are used in New Zealand to design stabilised pavements.

Empirical design methods typically use design charts, (eg Austroads 2012, figure 8.4) in combination with national and local material specifications for unbound and stabilised materials. Layered pavement solutions from published design charts are derived using the following design inputs: the subgrade strength (CBR); known or indexed pavement material properties; future design traffic, based on the passage over the design life of an expected number of design axles, ESA.

Mechanistic design makes use of design charts and/or proprietary software design tools (eg CIRCLY) and informed characterisation of the pavement materials properties (eg resilient modulus, Poisson's ratio) to predict stress and strain in all pavement layers under the ESA load. The number of allowable ESA repetitions is then determined using Austroads performance criteria or specific fatigue relationships of the materials based largely on historic in-service precedent performance. The pavement layer design depth is determined iteratively. This task is made more efficient by using pavement analysis software.

Unbound (modified), lightly bound and bound pavement solutions incorporating stabilised materials can be delivered using both the empirical design and mechanistic design methods.

In New Zealand, the ongoing performance of unbound, lightly bound and bound pavements continues to be reviewed using in-service performance information and data.

The worked design examples of unbound, lightly bound and bound pavements presented in sections 5.4, 5.5 and 5.6 demonstrate how empirical and mechanistic methods are used alongside our understanding of stabilised pavement performance in New Zealand. A number of design examples are given which are typical of local pavements. Specific project requirements may vary, however.

The best practice worked examples do not always deliver the least-depth pavement. The varying structural forms (unbound, lightly bound and bound layers) within the overall pavement system require designers to consider a number of factors. Besides the theory described in this guide, they also need to consider constructability, material and traffic variability and initial versus end performance operation and maintenance expectations.

For example, practical depths for construction (mixing and compaction) of a single layer of in-situ stabilised basecourse layers are between 175mm and 250mm. Mix designs proposing additive contents

(eg cement) above 1% are reasonably common in such circumstances. If the underlying pavement structure does not provide the necessary foundation support for the stabilised basecourse, or if the construction process controls do not deliver a consistent end product (eg with unplanned variations in cement and water contents) then long-term performance of the renewed pavement is unlikely.

5.3 Baseline project

For consistency across the worked examples below, the following baseline project data has been used in all cases. The worked examples' presentation assumes the reader is familiar with empirical and mechanistic pavement design principles and with using CIRCLY:

- location – greenfields rural two-lane highway, adequate side drainage options
- surfacing – chipseal, with potential for future overlay with thin lift (<40mm) AC or SMA
- design traffic: two scenarios – normal, 2 million ESA (MESA) with unbound and lightly bound pavement options, and high, 20 MESA with lightly bound and bound pavement options
- subgrade – fine-grained moderately plastic slightly clayey SILT, in-situ CBR 2% (10%ile value) under-representative long-term winter groundwater and surface water conditions

The following stabilised pavement worked examples are presented under three headings: unbound pavement (section 5.4); lightly bound pavement (section 5.5); bound pavement (section 5.6).

5.4 Unbound pavements incorporating stabilised layers

An unbound pavement design in New Zealand can typically incorporate stabilised materials in two ways:

- 1 Unbound pavement on stabilised subgrade (to form an upper level subgrade improvement layer or SIL)
- 2 Unbound pavement that uses aggregate materials modified by stabilisation either mechanically (eg granular stabilisation to correct grading deficiency in the basecourse) or chemically (eg lime and/or cement stabilisation to reduce plasticity and improve shear strength in the basecourse and strength/stability in the subbase).

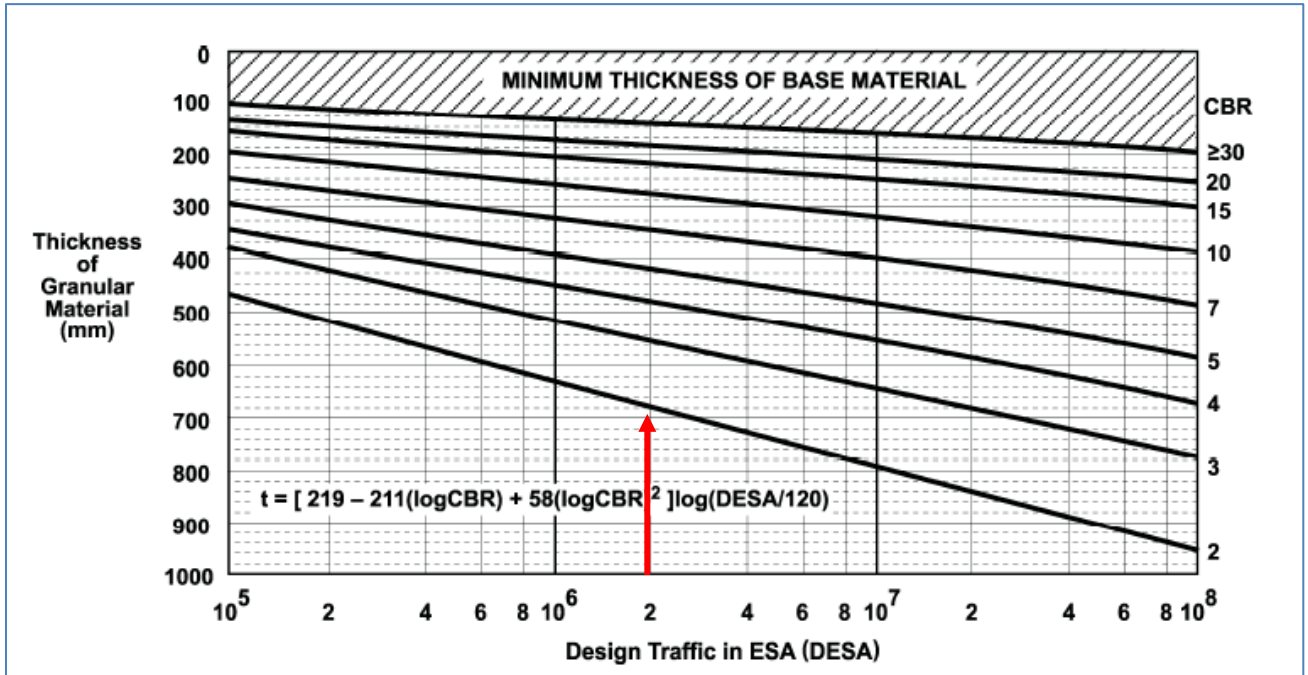
5.4.1 Worked example: unbound pavement with stabilised subgrade

In this worked example the designer wants to make use of a conventional unbound pavement. This is because good quality aggregate materials are available locally and are known to be cost effective. The low strength foundation conditions warrant using an SIL.

The natural subgrade has a low strength of CBR 2%. Laboratory testing of the in-situ subgrade material has shown the addition of cement/lime can potentially improve the CBR of the soaked subgrade material to 15% maximum (based on laboratory CBR reactivity testing). The designer knows the extent to which this can be realised in the field will depend on the foundation support conditions during and post-construction.

The designer's assessment of the trial depth of unbound pavement (680mm) and minimum thickness of base material (150mm basecourse) needed to meet the design traffic (2MESA) and low strength subgrade conditions (design CBR 2%) are derived from empirical design charts (eg figure 5.1).

Figure 5.1 Unbound pavement design chart (from Austroads, figure 8.4)



Source: Austroads (2012, figure 8.4)

The minimum thickness of base (basecourse) material is 150mm (see hatched area at the top of figure 5.1). The balance of the pavement below the basecourse (680mm - 150mm = 530mm) could be made up of unbound subbase aggregate.

In this case the designer wants to make use of the SIL to mitigate the risk of unbound subbase construction difficulties on a low-strength subgrade, and to reduce the reliance on deep excavations and imported subbase material.

Mechanistic principles are then used to refine the initial empirical trial design depth of 680mm.

The designer selects a compacted depth of the SIL as 250mm (a practical depth from an in-situ stabilisation construction perspective).

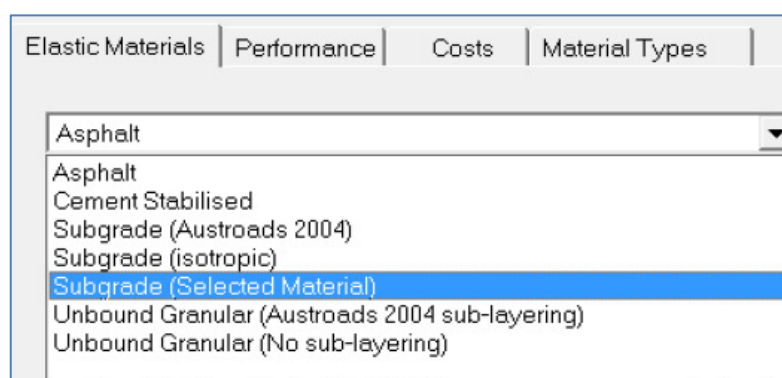
Austroads (2004) equations 8.1 and 8.2 are used to sublayer the SIL. Note: these equations are named as equations 19 and 20 in Austroads (2012).

- existing subgrade CBR 2%
- $E_{v \text{ in situ subgrade}} = 20\text{MPa}$ ($E = 10 * \text{CBR}$, appropriate as the subgrade is not volcanic in origin (Bailey and Patrick 2001))
- $E_{v \text{ selected subgrade top sublayer}} = 63\text{MPa}$
- $R = 1.26$ (equation 8.2), giving sub-layering (layer moduli and layer thickness) in the selected stabilised SIL as shown in table 5.1.

Table 5.1 Sub-layering in the selected stabilised subgrade

Layer	Selected subgrade	Layer modulus (MPa)	Sub-layer thickness (mm)
Layer 1		25	50
Layer 2		32	50
Layer 3		40	50
Layer 4		50	50
Layer 5		63	50

The sub-layering described above is normally completed automatically in CIRCLY by categorising the SIL as a 'selected subgrade' material (see figure 5.2).

Figure 5.2 Selected subgrade material tab in CIRCLY

The designer then uses mechanistic modelling (in this case with CIRCLY, which is currently the only Transport Agency designated software) and the following design information to optimise the design:

- design traffic loading 2MESA (project reliability 95%)
- subgrade performance traffic multiplier 1.2, as per Transit NZ (2007)
- unbound granular pavement ($E_{max} = 500\text{MPa}$) sub-layered

SIL modelled in CIRCLY as the 'selected subgrade' (refer table 3.5):

- subgrade ($E_{max} = 20\text{MPa}$)
- subgrade cumulative damage factor (CDF) < 1.

The recommended unbound pavement solution is given in table 5.2.

Table 5.2 Unbound aggregate on stabilised subgrade layer

Pavement layer	Material type	Minimum compacted layer thickness*
Basecourse	M/4 crushed basecourse	150mm
Subbase	Crushed, low fines AP 65	250mm
Stabilised SIL	Lime/cement stabilised in-situ subgrade	250mm
Subgrade	Natural, in-situ, underlying in-situ stabilised subgrade layer	
Total depth	650mm	

Note*: Layer thickness rounded up to nearest practical depth for construction

The as-built surface deflections on an unbound pavement system like the one described in table 5.2 can be estimated using CIRCLY and in this case are expected to be in the range of 1.5mm to 2mm.

Well-maintained chipseal surfacing can perform well under such movement but considerable care is required when including other surfacing types (eg thin-lift AC) as the resulting curvature ($D_0 - D_{200}$) may be too high (>0.2mm) and result in premature AC fatigue cracking.

5.4.2 Worked example: unbound pavement with granular stabilisation of the basecourse

In this worked example the designer wants to incorporate a local basecourse aggregate modified using granular stabilisation, rather than importing fully complying M/4 basecourse.

The local basecourse is poorly graded, and lacks coarse sand or finer gravel material, with 9% by weight passing a 75 μ m sieve. The designer knows this material is water sensitive, potentially reducing effective shear strength (a key basecourse attribute).

The local basecourse will be changed by blending with a more gap-graded finer gravel and coarse sand make-up material, in the proportions of 80% original material and 20% make-up material by dry weight.

A spreadsheet solution (refer table 5.3) is used to design the granular stabilisation.

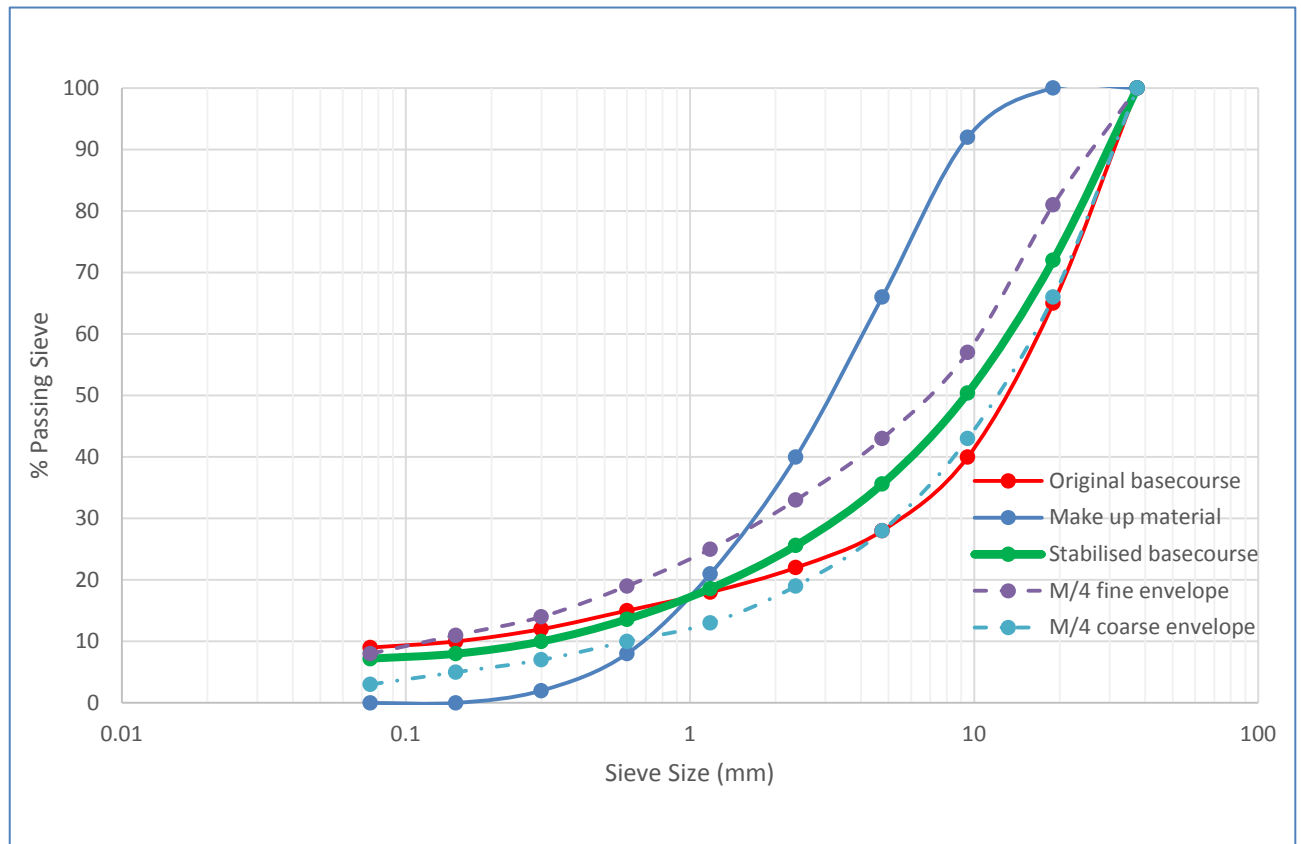
Supporting information on material characterisation and testing to support granular stabilisation is published in AustStab (2015, chapter 10).

Table 5.3 Granular stabilisation spreadsheet example

Sieve size (mm)	0.075	0.15	0.3	0.6	1.18	2.36	4.75	9.5	19	37.5
Original basecourse	9	10	12	15	18	22	28	40	65	100
Make up material	0	0	2	8	21	40	66	92	100	100
Stabilised basecourse	7	8	10	14	19	26	36	50	72	100
M/4 fine envelope	8	11	14	19	25	33	43	57	81	100
M/4 coarse envelope	3	5	7	10	13	19	28	43	66	100

When the results from table 5.3 are plotted (refer figure 5.3) the benefits of granular stabilisation on the local basecourse grading become apparent, resulting in a well-graded although still finer basecourse.

If the designer is still concerned about strength and rut resistance in the granular stabilised local basecourse this can be checked using the CBR and the RLT tests (refer section 4.7).

Figure 5.3 Aggregate granular stabilisation grading curve outcome

Source: William Gray

5.4.3 Worked example: unbound pavement using blended unbound basecourse with lime stabilisation

In this worked example, the designer notes (in section 5.4.2): the local basecourse material has a finer grading than the specified M/4 basecourse and a high PI > 10.

A quantity of lime (as burnt or hydrated lime during production or by in-situ stabilisation) combined with the granular stabilisation will help to deliver a basecourse within the M/4 specification. The amount of lime can be calculated using reactivity (CBR) and PI testing. It will probably be in the order of 2% to 3% by dry mass.

In this worked example, the laboratory RLT tests on the blended basecourse material show it is dense and rut resistant and has a lower resilient modulus of up to 350MPa based on the RLT test. Care is needed when using the resilient modulus from the RLT test on stabilised materials in subsequent mechanistic analysis because of the relatively low strain test conditions used to derive the E_v outcomes.

In this case the designer chooses to test the effect reduced stiffness in the modified basecourse layer would have on overall pavement depth. This test must note that a premium M/4 basecourse could be expected to achieve $E_{v \text{ resilient}}$ of between 450MPa and 500MPa under the right underlying support conditions.

The designer can use mechanistic modelling (in this case they used CIRCLY) to assess the effect of this change. This is shown below.

- design traffic loading 2MESA (project reliability 95%)
- subgrade performance traffic multiplier 1.2
- unbound granular pavement (assumed $E_{\max} = 350\text{MPa}$, taken as lower bound from the RLT test) sub-layered
- SIL modelled in CIRCLY as the 'selected subgrade'
- subgrade ($E_{\max} = 20\text{MPa}$)
- subgrade CDF < 1.

The recommended modified unbound pavement solution is given in table 5.4.

In the worked example, the overall pavement depth has increased by only 20mm in response to the RLT test outcome on the basecourse. The designer chose to increase the basecourse layer's depth because its strength and integrity are very important: a subbase layer depth of 270mm could introduce construction (compaction) variability concerns if placed in one layer.

Table 5.4 Modified unbound aggregate on stabilised subgrade layer

Pavement layer	Material type	Compacted layer thickness
Basecourse	Modified local crushed basecourse	170mm
Subbase	Crushed, low fines AP 65	250mm
Stabilised SIL	Lime/cement stabilised in-situ subgrade	250mm
Subgrade	Natural, in-situ, underlying in-situ stabilised subgrade layer	
Total depth	670mm	

5.5 Lightly bound stabilised pavement

Lightly bound pavements have a layer (or layers) with some tensile capacity. These can be used to enhance the pavement's repeat load carrying performance, provided the designer ensures unplanned cracking does not govern future performance. Lightly bound pavement designs can typically incorporate stabilised materials as:

- lightly bound cement stabilised basecourse
- lightly bound foamed bitumen/cement (BSM) stabilised basecourse.

5.5.1 Worked example: lightly bound cement stabilised basecourse on stabilised subgrade

For this worked example, the designer wants to use a cement stabilised basecourse material. In this way they can make more effective use of a locally-available lower grade basecourse material and help control rut development and shallow shear in the basecourse. The low strength foundation conditions (refer to section 5.3) justify using stabilised materials in a SIL.

IANZ-accredited laboratory testing following the T/19 specification (NZ Transport Agency 2016c) has shown the existing basecourse aggregate is reactive to cement. Laboratory testing with cement added at 2% by dry weight gave an ITS of 290kPa. The design (target) tensile stress is selected in this case as 40% of the flexural strength, which is estimated from the ITS test result as follows:

- Flexural strength is twice the ITS (580kPa).

- Design tensile stress is 40% of 580kPa or 232kPa.

The stabilised subgrade's compacted depth is selected as 250mm (a practical depth from an in-situ stabilisation construction perspective).

The design is undertaken in two parts.

5.5.1.1 Part 1: Lightly bound basecourse in pre- crack phase

In the first step, check how well a pavement made up of 200mm of cement stabilised basecourse on 200mm of subbase would protect the pavement from unplanned cracking. This is the equivalent pavement depth to the unbound pavement in table 5.2. The designer would use the following inputs:

- design traffic loading 2MESA (project reliability 95%)
- subgrade performance traffic multiplier 1.2
- lightly bound cement stabilised basecourse (pre-crack phase), $E=2,000\text{MPa}$, isotropic, Poison's ratio 0.2, not sub-layered
- unbound granular subbase ($E_{\text{max}} = 150\text{MPa}$ maximum) sub-layered
- SIL modelled in CIRCLY as the 'selected subgrade'
- subgrade ($E_{\text{max}} = 20\text{MPa}$)
- subgrade CDF < 1.

A CIRCLY check of the horizontal tensile stresses at the base of the proposed cement stabilised basecourse layer 200mm deep delivers predicted tensile stress >600kPa. This significantly exceeds the design tensile stress of 232kPa. This would lead to cracking early in the pavement life that would result in a significant reduction in basecourse stiffness and could adversely affect pavement performance.

The practical compacted depth of the cement stabilised basecourse is increased to 250mm (maximum depth for a single layer).

A further CIRCLY check shows tensile stresses at the base of the 250mm basecourse are around 500kPa. This is still too high. The designer therefore expects the pavement will micro-crack and move quickly to the ELTS phase.

The designer could increase the depth of the stabilised layer and iterate back through the part 1 analysis. A layer depth >250mm would probably mean multi-layer construction. In this worked example the designer chooses to move onto the ELTS phase.

5.5.1.2 Part 2: Lightly bound on post- crack ELTS phase

The designer checks the pavement for the ELTS phase, using the following inputs:

- unbound granular layer with $E=700\text{MPa}$ max, sub-layered
- SIL modelled in CIRCLY as the 'selected subgrade'
- subgrade ($E_{\text{max}} = 20\text{MPa}$)
- subgrade CDF < 1, actually 0.6.

The recommended pavement treatment in this worked example would be as shown in table 5.5.

The pavement layers shown in table 5.5 could be 'tweaked' so they are a bit smaller, to bring the CDF in closer to 1. However, the designer believes the minimum compacted layer thicknesses of 250mm for the

basecourse and 150mm for the subbase are sensible from a construction perspective. They provide additional support for the pavement given the low-strength subgrade conditions.

A robust first coat seal is needed to mitigate the risk of surface water intrusion into micro cracks in the stabilised basecourse.

Table 5.5 Lightly bound cement stabilised basecourse on stabilised subgrade layer

Pavement layer	Material type	Minimum compacted layer thickness
Basecourse	Cement stabilised crushed local aggregate	250mm*
Subbase	Crushed, low fines AP 65	150mm
Stabilised SIL (working platform)	Lime/cement stabilised	250mm
Subgrade	Natural, in-situ	
Total pavement depth	650mm	

Note * the total pavement depth above the SIL is 400mm. The maximum modulus achieved under the sub-layering rule and the underlying conditions is 584MPa in this instance.

5.5.2 Worked example: lightly bound BSM basecourse (BSM-foam) on stabilised subgrade

For this worked example, the designer wants to use a foamed bitumen/cement stabilised basecourse material. The aim is to make more effective use of a locally available lower grade basecourse material that will help control rut development and shallow shear in the basecourse. The low-strength foundation conditions justify using a SIL.

IANZ-accredited laboratory testing, including wet/dry ITS testing, from the T/19 specification (NZ Transport Agency 2016c) shows the existing basecourse aggregate is suitably reactive to foamed bitumen and a small quantity of cement.

The designer has selected the stabilised subgrade's compacted depth as 250mm. This is a practical depth from an in-situ stabilisation construction perspective.

The designer uses the following inputs:

- design traffic loading 2MESA (project reliability 95%)
- subgrade performance traffic multiplier 1.2
- unbound granular subbase 200mm deep ($E_{\max} = 193\text{MPa}$) sub-layered. This means the overlaying BSM layer modulus is limited to 800MPa
- SIL modelled in CIRCLY as the 'selected subgrade'
- subgrade ($E_{\max} = 20\text{MPa}$)
- subgrade CDF < 1.

Current best practice in New Zealand limits the BSM layer modulus to five times the underlying layer modulus or a maximum of 800MPa, anisotropic, Poisson's ratio 0.3 (NZ Transport Agency 2017a)

From a practical construction and cost perspective, the BSM basecourse depth is first trialled as 180mm. CDF for the subgrade is 0.6 and the modular ratio is ~4.

The designer considers the relative costs of the conforming M/4 basecourse and BSM-foam material, and trialled the BSM layer at 175mm. This delivered a maximum CDF on the subgrade of just over 0.7. Even though the BSM depth could theoretically be reduced further, a compacted depth of 175mm is a practical depth from an in-situ stabilisation construction perspective.

The recommended pavement treatment in this worked example would be as shown in table 5.6.

Table 5.6 Lightly bound BSM basecourse on stabilised subgrade layer

Pavement layer	Material type	Minimum compacted layer thickness
Basecourse	Foamed bitumen/cement stabilised crushed aggregate	175mm
Subbase	Crushed, low fines AP 65	200mm
Stabilised SIL (working platform)	Lime/cement stabilised	250mm
Subgrade	Natural, in-situ	
Total pavement depth	625mm	

A robust first coat seal is needed to mitigate the risk of surface water intrusion into the stabilised basecourse.

5.6 Bound stabilised pavement

Bound pavement designs can typically incorporate stabilised materials in the following ways:

- bound subbase layers supporting upper pavement layers and surfacing.
- bound upper pavement layers/surfacing, incorporating:
 - reinforced concrete
 - unreinforced concrete
 - structural asphaltic concrete
 - paving systems incorporating concrete or clay pavers
 - cement treated base
 - cement or bitumen bound Macadam pavement layers.

This guide does not include information on the design of reinforced concrete, structural AC or block pavement systems.

The following worked examples explore the use of bound subbase and basecourse options.

5.6.1 Worked example: bound subbase with unbound granular overlay

In this worked example, the designer wanted to make use of a cement bound granular subbase with an M/4 basecourse overlay. It would be used for the heavily trafficked new road pavement (20MESA) over the soft subgrade (CBR 2). The designer also wants to make more effective use of a locally available subbase material. The low strength foundation conditions (refer to section 5.3) justify using a modified subgrade improvement layer (SIL).

IANZ-accredited laboratory testing to the T/19 specification (NZ Transport Agency 2016c) has shown the existing AP65 subbase aggregate is reactive to cement. With cement additive contents of >5% by dry mass it can deliver a bound layer, with $E=3,500\text{MPa}$ and $\text{ITS} >600\text{kPa}$.

In this worked example, the designer has two options.

Option 1 considers the subbase as bound at all times. This has the expected advantage of lower pavement surface deflections under traffic load over the life of the pavement. The designer expects this to require a thicker pavement.

Option 2 allows the bound subbase layer to micro-crack sufficiently so the layer migrates to the ELTS state (refer to section 4.2.1). Light rolling during construction (three to four passes of a light vibrating roller) can be specified to help manage the micro-cracking. Care will also need to be taken to mitigate the risk of post-construction shrinkage cracking.

5.6.1.1 Option 1: Bound subbase, fatigue control

The designer used mechanistic principles to develop the design. Due to the higher traffic loading, in-situ stabilisation of the subgrade was replaced with a purpose-built 300mm-deep SIL using locally available sand.

The design inputs to the mechanistic analysis are as follows:

- design traffic loading 20MESA (project reliability 97.5%)
- subgrade performance traffic multiplier 1.2
- cement bound performance traffic multiplier 3.6
- unbound granular pavement ($E_{v \text{ max}}=500\text{MPa}$) sub-layered, anisotropic
- bound granular subbase, pre-cracked (ELTS) phase, $E=3,500\text{MPa}$, isotropic, not sub-layered
- fatigue constant for cement bound layer $k=350$ (refer section 4.2.2)
- subgrade improvement layer, sand, modelled in CIRCLY as the 'selected subgrade'
- subgrade ($E_{v \text{ max}}=20\text{MPa}$)
- $\text{CDF} < 1$.

The compacted depth of the sand SIL is selected as 300mm; this is a practical maximum depth from an earthwork construction perspective.

Austrroads (2004) equations 8.1 and 8.2 are used to sublayer the SIL for analysis.

- existing subgrade CBR 2%
- $E_{v \text{ in situ subgrade}}=20\text{MPa}$ ($E=10*\text{CBR}$, appropriate as the subgrade is not volcanic in origin)
- layer thickness 300mm
- $E_{v \text{ selected subgrade top sublayer}}=80\text{MPa}$ (equation 8.1)
- $R=1.32$ (equation 8.2), giving sub-layering (layer moduli and layer thickness) in the selected stabilised subgrade (SIL) as shown in table 5.4. The sub-layering described above and shown in table 5.7 is usually completed automatically in CIRCLY by categorising the SIL as a 'selected subgrade' material.

Table 5.7 Sub-layering in the selected sand SIL from the bottom layer 1

Layer	Selected subgrade	Layer modulus (MPa)	Layer thickness (mm)
Layer 1		26	60
Layer 2		35	60
Layer 3		46	60
Layer 4		61	60
Layer 5		80	60

The recommended pavement treatment in this worked example is shown in table 5.8.

Table 5.8 Bound subbase with unbound granular overlay (fatigue control)

Pavement layer	Material type	Minimum compacted layer t
Basecourse	M/4 crushed basecourse	180mm*
Bound subbase	Bound subbase AP 65, fatigue control	365mm**
Stabilised SIL	Sand	300mm
Subgrade	Natural, in-situ	
Overall pavement system	845 mm	

Note* Practical depth from a construction perspective, ** Constructed as two layers.

The recommended pavement treatment shown in table 5.8 has a relatively thickly bound subbase layer based on the presumptive fatigue relationship. The designer then trials how controlling stress levels in the base of the bound subbase to in this instance <40% of flexural strength would influence the depth of the bound subbase, to deliver at least 1MESA traffic carrying capacity before microcracking occurs.

IANZ-accredited laboratory testing to the T/19 specification (NZ Transport Agency 2016c) shows the existing AP65 subbase aggregate is reactive to cement. With cement additive contents of >5% by dry mass it can deliver a bound layer, with E=3,500MPa and ITS >600kPa.

The flexural strength is estimated as twice the ITS or 1,200kPa. CIRCLY is used to optimise the depth of the bound subbase to keep the tensile stress in the bound layer to <40% of the flexural strength, or <480kPa. All other layers are kept the same as shown in table 5.8.

The recommended pavement treatment to control stress levels in the bound subbase is shown in table 5.9. The bound subbase layer and overall pavement depth contribute to reduced surface deflections (estimated in CIRCLY as <1.2mm)

Table 5.9 Bound subbase with unbound granular overlay (stress level control)

Pavement layer	Material type	Minimum compacted layer thickness
Basecourse	M/4 crushed basecourse	180mm
Bound subbase	Bound subbase AP 65, stress level control (<40% flexural strength)	260mm
Stabilised SIL	Sand	300mm
Subgrade	Natural, in-situ	
Overall pavement system	740 mm	

5.6.1.2 Option 2: Post ELTS phase

The designer then considers how the pavement shown in table 5.9 will behave once the subbase layer has migrated to the post-cracked (ELTS) phase: all other layers would stay the same.

- design traffic loading 20MESA (project reliability 97.5%)
- subgrade performance traffic multiplier 1.2
- unbound granular pavement ($E_{\max} = 500\text{MPa}$) sub-layered, anisotropic
- current best practice in New Zealand either limits the cemented ELTS layer modulus to three times the underlying layer modulus or a maximum of 500MPa, anisotropic, Poisson's ratio 0.35, no sub-layering, or models the layer as unbound completely. In this worked example the layer only achieves $E_{v\max} = 240\text{MPa}$, anisotropic, Poisson's ratio 0.35, no sub layering
- subgrade improvement layer, sand, modelled in CIRCLY as the 'selected subgrade'
- subgrade ($E_{\max} = 20\text{MPa}$)
- subgrade CDF <1, actually 0.96.

The pavement shown in table 5.9 would be acceptable, but the migration to the unbound condition results in an increase in surface deflection (estimated in CIRCLY as >1.5mm). This could affect future surfacing decisions.

The first coat seal needs to be robust and well maintained. This is because fine micro-cracks will inevitably migrate through to the surface over time, even in the fatigue and stress-controlled bound subbase options.

5.6.2 Worked example: bound basecourse with unbound subbase

The designer in this example wanted to make use of a cement bound granular basecourse to make effective use of a locally available material. They also wanted to have lower pavement deflections (<1mm) to allow for future overlay with thin-lift, low-noise asphalt-bound surfacing. The low-strength foundation conditions (subgrade CBR <2%) warrant use of a modified SIL.

IANZ-accredited laboratory testing to the T/19 specification (NZ Transport Agency 2016c) has shown the existing AP40 basecourse materials are reactive to cement. They also show with cement additive contents of >5% by dry mass they can deliver a UCS of 5MPa, giving an estimated resilient modulus of 5,000MPa.

The ITS is 0.65MPa, giving a flexural strength 1.3MPa.

The designer used mechanistic principles to develop the design. Due to the higher traffic loading, in-situ subgrade stabilisation was replaced with a purpose-built 300mm-deep subgrade improvement using locally available sand.

The design would be completed in two parts.

5.6.2.1 Part 1: Bound pavement, control stress ratio <0.4 for minimum 1MESA traffic

- bound granular basecourse: pre-cracked phase $E = 5,000\text{MPa}$, isotropic, Poisson's ratio 0.2
- unbound granular subbase ($E_{\max} = 320\text{MPa}$) sub layered, anisotropic
- subgrade improvement layer, sand, modelled in CIRCLY as the 'selected subgrade'
- subgrade ($E_{\max} = 20\text{MPa}$)
- stress ratio <0.4, hence maximum allowable tensile stress in bound layer is 0.52MPa (520kPa).

The sand SIL's compacted depth was selected as 300mm. This is a practical maximum for single layer construction depth from an earthworks' construction perspective. The sub-layering is completed automatically in CIRCLY by categorising the SIL as a 'selected subgrade' material (refer to section 5.4.1).

A bound base layer depth of 250mm and subbase layer of 200mm delivers maximum tensile stresses at the base of the layer of about 650kPa in the pre-crack phase. This is expected to support approximately 1MESA, with controlled micro cracking leading to the ELTS phase.

5.6.2.2 Part 2: ELTS phase

- design traffic loading 19MESA (project reliability 97.5%)
- subgrade performance traffic multiplier 1.2
- unbound granular layer with $E=700\text{MPa}$ max, sub-layered (refer section 4.2)
- unbound granular subbase, $E=250\text{MPa}$, sub-layered
- subgrade improvement layer, sand, modelled in CIRCLY as the 'selected subgrade'
- subgrade ($E_{\text{max}}=20\text{MPa}$)
- subgrade CDF <1 , actually 0.85.

The recommended pavement treatment in this worked example, to enable the pavement to work effectively across the ELTS transition is shown in table 5.10.

Table 5.10 Bound basecourse in ELTS phase

Pavement layer	Material type	Minimum compacted layer thickness
Basecourse	Cement bound AP40 in ELTS phase	235mm
Subbase	Crushed, low fines AP 65	200mm
Stabilised SIL	Sand	300mm
Subgrade	Natural, in-situ	
Overall pavement system	735mm	

Even though the bound basecourse is designed to migrate to the ELTS phase without 'fatal flaws' occurring (eg block cracking) the surface seal needs to be robust and well maintained. This is because fine micro-cracks will inevitably migrate through to the surface over time. Careful control of the water content of the construction will also help mitigate the risk of transverse and longitudinal shrinkage cracking.

At the present time, the Transport Agency is unlikely to endorse the use of bound basecourse as described above (NZ Transport Agency 2017a).

5.6.3 Worked example: roller compacted concrete base

In this worked example, the designer wanted to use a strong, well-bound basecourse material as a lean mix concrete. This approach is not common in New Zealand, but could become more useful in future commercial applications. It could enable the use of local basecourse materials and support long-lasting smooth-ride, low-noise and low-spray bitumen bound surfacing layers, while mitigating the risk of unplanned transverse and longitudinal shrinkage cracking.

Mechanistic principles are used by the designer to develop the design, based on the principles of RCC (refer section 4.2.3). Due to the higher traffic loading, in-situ stabilising of the subgrade will be replaced by a purpose-built subgrade improvement using 300mm-deep locally available sand.

The design inputs to the mechanistic analysis are follows:

- design traffic loading 20MESA (project reliability 97.5%)
- subgrade performance traffic multiplier 1.2
- RCC base layer ($E=15\text{GPa}$, as $1000 \times \text{UCS}$), isotropic, not sub-layered, with tensile stress control ($\text{SR} < 40\%$)
- unbound granular subbase, $E=250\text{MPa}$, anisotropic, sub-layered
- subgrade improvement layer, SAND, modelled in CIRCLY as the 'selected subgrade'
- subgrade ($E_{\text{max}}=20\text{MPa}$)
- subgrade CDF < 1 .

The compacted depth of the modified sand SIL is selected as 300mm (a practical depth from an earthwork construction perspective).

Austrroads (2004) equations 8.1 and 8.2 are used to sub-layer the SIL for analysis (refer section 5.4.1).

The reported compression strength f_c for the RCC is 15MPa, giving $f_{cf}=2.9\text{MPa}$. The maximum tensile stress in the RCC should be $< 0.4f_{cf}$ or 1.2MPa.

Results from the iterative CIRCLY analysis for the RCC pavement system, allowing for a depth of RCC of 225mm (practical depth from a construction perspective given the intention to lay the material post pug mill production with a purpose specified mechanical paver) show the maximum tensile stress in the base of the RCC is 1.06MPa. This is $< 0.4 f_{cf}$.

The recommended pavement treatment in this worked example is shown in table 5.11. Estimated post-construction surface deflections are $< 1.0\text{mm}$, in a 'perpetual pavement system'.

Table 5.11 Roller compacted concrete basecourse

Pavement layer	Material type	Minimum compacted layer thickness
Basecourse	Roller compacted concrete	225mm
Subbase	Unbound, crushed stone aggregate	200mm
Stabilised SIL	Lime/cement stabilised sand	300mm
Subgrade	Natural, in-situ	
Overall pavement system	725mm	

The surface seal needs to be robust and well maintained, as fine micro-cracks will inevitably migrate through to the surface over time. Transverse saw-cut joints prior to first coat sealing are required to control shrinkage cracking.

At the present time, the Transport Agency is unlikely to endorse the use of bound basecourse as described above (NZ Transport Agency 2017a). However RCC has been used successfully overseas, and could under the right conditions, including informed investigation and analysis, be given more detailed consideration in specific cases.

6 Construction

6.1 Introduction

Stabilised pavement construction processes continue to advance. This is a result of improvements in technology (construction plant and equipment) and research and development (the engineering science behind investigation, testing, design and construction quality assurance).

Supporting information on construction can be found in the following references:

- *Pavement recycling and stabilisation guide* (AustStab 2015, chapter 12)
- *New Zealand guide to pavement evaluation and treatment design* (NZ Transport Agency 2017a)

The following discussion on stabilised pavement construction looks at:

- construction setting: in-situ stabilisation; plant-based stabilisation (pug mill)
- construction materials: mixed in-situ materials; plant manufactured materials.

6.2 Construction setting

6.2.1 In situ stabilisation

In-situ stabilisation involves mixing pavement materials with stabilisation agents and water in the place they will be used. The water is used primarily to support compaction and in some cases supports stabilising agents' slaking (eg burnt lime, CaO).

Improvements in construction plant and technology means it is now routine to expect stabilising agents (eg cement, foamed bitumen, water) to be directly injected into or immediately in front of the stabiliser rotor chamber. The stabiliser (a bespoke hoeing machine, refer to figure 6.1) now routinely includes computer controls for binder application rates, mixing depth, ground speed, speed of mixing, drum rotation etc. All these enable the contractor to deliver well-engineered, process-controlled construction outcomes.

Industry preference for stabilisers is to have the rotor inside the axles for superior depth control. This is essential for aggregate stabilisation and desirable for subgrade stabilisation.

Size does matter. A larger stabilising plant can routinely mix and place deeper lifts, without 'bouncing'. It can pulverise and mix a greater variety of materials including previously recycled aggregates, existing bitumen bound seal layers, layers of lower strength unreinforced concrete and previously stabilised materials.

Figure 6.1 Bespoke pavement and soil stabilisers



Source: William Gray

Pavement stabilisation is often synonymous with recycling. From a plant perspective, a pavement stabiliser as shown in figure 6.1 is different from a pavement recycler, which is more likely to consist of a milling and replace operation for use on deep-lift asphalt roads.

The arrangement and shape of the 'teeth' on the mixing drum in the modern stabiliser can also affect the stabilisation outcomes. Varying the teeth and drum arrangement (refer figure 6.2) can make a particular item of plant better suited to soil stabilisation for example, rather than aggregate stabilisation. On-site trials supported by enquiry to equipment manufacturers are recommended for providing optimised stabilisation outcomes.

Figure 6.2 Different teeth and drum arrangements on modern pavement stabilisers



Source: William Gray

The in-situ pavement material stabilisation must be successful for all concerned (asset owner, designer and contractor). Therefore, the pavement designer's assumptions about the expected strength outcomes and behaviour of the stabilised material must be based on sound investigation (refer to chapter 4). This must also meet current design best practice (refer to chapter 5).

The in-situ stabilisation of mixed existing pavement materials will probably encounter a range of materials. These include combinations of unbound fine soil and stone aggregate, previously stabilised materials (as lightly bound or bound layers), old and more recent seal (with and without polymer modified bitumen materials etc), and occasionally even old geosynthetic materials.

While the construction processes (adding stabilising agents and water, hoeing, shaping, compaction) are now routine, project site-specific compaction targets (maximum dry density, optimum water content, target air void content) vary. Skilled operators need to respond to these on-site variables during construction.

In-situ pavement stabilisation, like many other construction processes, needs time to develop strength and stability. Cement and lime stabilised materials can display early strength gain. This is due primarily to the moisture reduction in the material and early cementation.

Typically, stabilised pavement construction success requires:

- 1 Site investigation to confirm the type and variability of the existing pavement materials, mix design using representative pavement materials to assist the designer in setting realistic performance targets and for the contractor to confirm the optimum binder type and application rates for the specified mix design 'recipe'
- 2 Clearly defined contract specifications for the pavement area to be worked, technical requirements for the stabilised pavement (materials to be stabilised, binder type, application rate, depth of mixing and any project specific requirements such as pre-cracking for bound materials). This will enable the contractor to provide suitable plant
- 3 Well-planned and executed consecutive construction operations – site preparation and set out, makeup material applied (if specified), surface scarification (if needed), binder and water application and mixing, initial compaction immediately after mixing, shaping (usually by grader) and final compaction
- 4 Carefully managing the stabilised pavement surface after construction. Once a pavement layer has been stabilised, the surface needs to be protected from either drying out or becoming over wet; at the same time, the stabilised layer needs to 'dry back' to the appropriate degree of saturation prior to surfacing. The contractor must protect the stabilised layer surface from uncontrolled loading from construction and general traffic. They must do their best to cover it as soon as possible with an additional layer, and with near surface layers provide proactive traffic management for an extended period.

Further information on the construction processes, as these relate to stabilising the subgrade, SIL and subbase and basecourse layers, can be found in the specifications listed in figure 6.1.

Table 6.1 Specifications applicable to in- situ pavement stabilisation

Pavement construction (or part thereof)	Relevant technical standards
Subgrade preparation	NZTA F/1
SIL construction	NZTA F/1 NZTA B/9
Stabilising agents' source and properties	NZTA M/1, M/15, M/4
Pavement aggregate source and production properties	NZTA T/15, M/4
Modified, lightly bound or bound pavement layer geometry, consistency, strength and/or stiffness	NZTA B/2, B/5, B/6, B/9

6.2.2 Plant-based stabilisation

Plant-based stabilisation by fixed batch plant or mobile pug mill offers potential for enhanced process control over supplying and mixing the material components in situ.

Improvements in construction plant and technology mean it is now routine to expect plant-based pug mill and mixing systems to be able to mix different aggregate materials with a variety of stabilising agents.

These include cement, lime, bitumen emulsion and foamed bitumen. Batch weighing pre-selected, dry and clean aggregate components and having a computer-controlled supply of agents and water provides excellent process control at the plant.

For in-situ stabilisation, the primary compaction has to be completed within a specified period of time (as per the B series specifications). While this is usually easily managed for in-situ stabilisation, ex-situ stabilisation requires effective time management and proximity to the construction site. This is because the management process needs to incorporate batching, transportation, placement and primary compaction within the permitted timeframe.

On-site construction is best achieved using a suitable paver, supported by well organised material supply to the paver and then compaction operations. Great care is needed when the pre-mixed materials are being transported and placed to prevent segregation. Best practice on site would also seek to avoid adverse changes in temperature, wind speed and moisture content.

The specifications listed in table 6.2 support plant-based pavement stabilisation in New Zealand.

Table 6.2 Specifications applicable to plant-based pavement stabilisation

Pavement construction (or part construction)	Relevant technical standards
Subgrade and SIL construction	NZTA F/1
Plant mixed modified, lightly bound or bound pavement layer geometry, consistency, strength and/or stiffness	NZTA B/2, B/7, B/8

6.3 Construction materials

6.3.1 Binder materials

The typical binder materials used in pavement stabilisation (cement, lime, bitumen and polymer) are discussed in detail in chapter 4.

The material specifications listed in table 6.3 support pavement stabilisation in New Zealand.

Table 6.3 Specifications applicable to binder materials

Binder agent(s)	Relevant technical standards
Lime (burnt lime CaO or hydrated lime Ca(OH) ²)	NZTA M/15
Cement (general purpose Portland Cement type GP, general purpose blended cement type GB)	NZS 3112
Bitumen (80/100 or 180/200 penetration grade). Current practice in the wider Pacific uses multigrade materials, but not yet in New Zealand	NZTA M/1
Water – free from impurities that may negatively affect the setting, hardening or strength of the stabilised materials	

6.3.2 Mixed in-situ materials

Stabilising existing pavement materials (with or without make up), and old seal layers in situ invariably encounters differences. These occur in the 'as-received' material type, mix composition, density and water content. The designer needs to undertake informed investigation and design work of the expected

in-situ material 'cocktail' to enable suitable design and construction performance measures to be specified. The contractor then in turn needs to operate with effective on-site process controls under the watchful eye of experienced operators.

The pavement designer should consider how the expected variability could affect design strength outcomes, compaction achievement standards and preparation for surfacing. They should reflect these expectations in the contractor specification and quality assurance requirements.

NZTA B/5 (NZ Transport Agency 2008, section 7.7) notes in particular that fluctuations in density and moisture content during construction result from in-situ material variability, which to some extent is unavoidable. This will often require some compromise around the target water contents and compaction density outcomes, plus informed, proactive decision making on site.

In contrast, avoidable factors such as poor workmanship, lack of good planning and preparation and machinery breakdown etc, have little to do with in-situ material variability. They should not be used as an excuse by the contractor for not achieving well-conceived specified targets.

NZTA B/2 (NZ Transport Agency 2005) in clause 7.8 requires the basecourse pre-seal finish to present a tightly consolidated surface when swept:

- The large aggregate is exposed to the surface and held in place 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 traffic or well-managed sweeping.

This sort of unbound granular basecourse surface prior to seal is called a 'clean, stone, mosaic surface'. It can be less than straightforward to deliver with stabilised in-situ materials, because of the in-situ materials' variability, level of fines or contamination in aged pavements and material breakdown during mixing.

For example, in figure 6.3, the in-situ stabilisation of the mixed fine gravels on the left resulted in a less than desirable pre-seal surface. While it did present some larger stone mosaic, it was 'choked' with finer, lightly bound silt. In this case, the contractor worked the surface with a drag broom and coarse-graded running course for several days. At the same time they proactively controlled live traffic speeds and placement to prepare the dense stone mosaic surface for chipseal.

Figure 6.3 In-situ cement stabilisation and finished surface appearance



Source: William Gray

Both the designer and contractor need to take these risks into account and manage encountered materials to optimise the outcome. From the design perspective, if the in-situ material is shown by investigations to

be either fine graded to start with or likely to breakdown during mixing, the option of introducing hard stone, coarse graded make up metal should be considered. This would provide supporting 'aggregate stabilisation' (refer to section 4.6).

The specifications listed in table 6.4 support pavement stabilisation with mixed in-situ materials in New Zealand.

Table 6.4 Specifications applicable to mixed in- situ materials

Binder agent(s)	Relevant technical standards
Testing	NZTA T/19
Construction with mixed in-situ materials	NZTA B/2, B/5, B/6 and B/9

6.3.3 Stabilisation with processed materials

Processing prepared aggregate and binder materials through a fixed or mobile processing plant brings with it the opportunity of material consistency.

Points to note when considering this approach should include:

- Successfully blending consistent aggregate materials (coarse and fine grained) requires a good knowledge of the grading properties of the source materials, and confidence the required grading will not change. This is particularly important for fine-grained components. While silt and sand-sized particles might all look the same in the storage bins, subtle changes in source grading (for example a move to finer sand) can alter the effective surface area of the materials; this in turn influences binder and water content. Similar effects can be observed if the source material changes from round sand or silt particles (eg river run materials) to crushed, angular grains (eg crushed sand by-products from concrete production).
- Successfully processing aggregate and binder materials needs good control of weights, water contents, and as noted above, particle size distribution.
- Once blended, all the good work can be undone if transporting the processed material to site results in segregation, changes in water content and temperature or delays compromising binder reactivity and time to primary compaction.
- Blended, stabilised aggregate materials should be placed through a purpose-built 'fit for purpose' paver, with suitable controls on screed height and width, supported by well-organised compaction.

The specifications listed in table 6.5 support pavement stabilisation with mixed in-situ materials in New Zealand.

Table 6.5 Specifications applicable to processed material

Binder agent(s)	Relevant technical standards
Testing	NZTA T/19
Construction with processed materials	NZTA B/2, B/7 and B/8

7 Quality assurance

7.1 Introduction

The following discussion on quality assurance assumes construction works will be carried out in accordance with NZS 3910 (Standards NZ 2013). Subject to the approval of the engineer (refer clause 6 NZS 3910:2013), or specific requirements of the contract, operational and/or principal's requirements and relevant technical standards, tables 7.1 and 7.2 describe the minimum requirements to inspect, record, measure, test, verify and certify pavement construction by either the contractor or engineer (principal's agent). The wording used below is similar to that expected in a contract document.

These minimum requirements would be delivered under an integrated programme by the engineer and contractor using the respective quality management plans, refer *Minimum standard Z/1* (NZ Transport Agency 2017c) that shall provide the principal, engineer (principal's agent) and contractor with current, explicit information about the pavement to support informed decision making, leading to consistent, high-quality outcomes.

Testing in support of pavement verification and certification can and should include a sensible combination of laboratory and field tests, including cores taken from the stabilised layers for post-construction laboratory strength testing (eg ITS) and calibration with the design intent, refer *Minimum Standard Z/8* (NZ Transport Agency 2017d)

7.2 Relevant technical standards

The minimum requirements to inspect, record, measure, test, verify and certify pavement construction shall use the test methods prescribed by the engineer in the contract specification, or an approved alternative, as discussed previously in the guide.

7.3 Verification

Verification means the contractor's review of the adequacy of the documentation or stage of work being verified in accordance with the design, specification and relevant technical standards. This will be based on evidence (both supplied and sought), inspection and testing. Statements of verification shall be prepared and signed by a suitably qualified professional, and include IANZ accredited laboratory results.

7.4 Certification

Certification means the engineer's confirmation of the adequacy of the completed works (or parts thereof) being certified in accordance with the design, specification and relevant technical standards. This will be based on evidence (both supplied and sought), independent monitoring and random verification testing. Statements of certification shall be delivered using templates such as the PS1 to PS4 Producer Statement templates prepared by the Association of Consulting Engineers New Zealand (ACENZ).

7.5 Random verification testing (RVT)

The contractor shall enable the engineer to undertake RVT. This will be undertaken as specified in the contract, or as or when notified by the engineer by writing at least 24 hours in advance.

RVT testing shall be used by the engineer to verify compliance with the contract specification. In the event RVT testing identifies any non-conformance, the contractor shall be informed immediately, and undertake corrective actions in accordance with the quality management plan.

7.6 Minimum requirements: contractor

The minimum requirements for the contractor to inspect, record, measure, test and verify the successful delivery of pavement construction works or parts thereof are given in table 7.1.

Table 7.1 Minimum requirements: contractor to inspect, record, measure, test and verify

Pavement construction (or part thereof)	Relevant technical standards
Subgrade material consistency and strength	NZTA F/1
Top of subgrade surface geometry and condition	NZTA F/1
Subgrade improvement layer (SIL) consistency, strength and/or stiffness	NZTA F/1 NZTA B/9
Top of SIL surface geometry and condition	NZTA F/1
Stabilising agents source and properties	NZTA B/5 to B/9
Pavement aggregate source and production properties	NZTA M/4 NZTA T/15
Modified or bound pavement layer geometry, consistency, strength and/or stiffness	NZTA B/5 to B/9
Unbound pavement layer geometry, consistency and stiffness	NZTA B/2
Pavement surface prior to first coat sealing	NZTA B/2

7.7 Minimum requirements: engineer

As a minimum the engineer must monitor, test (RVT) and certify the successful delivery of pavement construction works or parts thereof in accordance with the contract specification, relevant technical standards and the design, using the ACENZ construction monitoring level CM4.

Unless otherwise specified, the relevant technical standards are prescribed in table 7.2 below.

Table 7.2 Minimum requirements: engineer to certify

Pavement construction (or part thereof)	Relevant technical standards
Subgrade	NZTA F/1
Subgrade improvement layer (SIL)	NZTA F/1 NZTA B/9
Modified or bound pavement layer	NZTA B/5 to B/9
Unbound pavement layer	NZTA M/4 NZTA B/2
Pavement surface prior to sealing	NZTA B/2

8 Ongoing research: in- situ testing and analysis

8.1 Introduction

Ongoing research and development in New Zealand (Alabaster et al 2013) supports a growing awareness by industry of the merits of back analysis of pavement layer properties and future pavement performance prediction. This uses in-situ performance testing (largely deflection testing using the FWD and recently the traffic speed deflectometer). It also uses test outcomes from accelerated load testing at the Transport Agency CAPTIF facility in Christchurch

Transport Agency-sponsored research (Gray et al 2011) gave specific attention to the characterisation and use of stabilised pavements in New Zealand. This is based largely on in-situ pavement performance, stemming in part from pavement layer recycling initiatives (Gray and Hart 2003).

The previous research investigated the ongoing performance of a number of pavement sites using the extensive data inventory for cement-stabilised sites. In some cases, evidence of the timing of 'premature failures including cracking and rutting' in the top surface stabilised granular layer can be compared with existing theory.

The previous researchers suspected the Austroads fatigue criterion for cement bound materials with layer moduli $< 10,000\text{MPa}$ was conservative with respect to design expectations when using cement-stabilised pavement layers with low cement contents ($< 3\%$ cement by dry mass). Their analysis of the inventory data, including pavement sites where the ongoing performance had either exceeded that predicted by Austroads or in contrast, had exhibited premature failure in the stabilised granular layer (eg cracking or rutting, or both), enabled them to prepare a conceptual pavement performance model for cement-stabilised pavements. They then used this model to predict the observed performance of pavements using stabilised granular materials as lightly bound layers.

At that time (2011) there was only limited performance data for foamed bitumen/cement-stabilised (FBS) project sites, a maximum of five years' service life, and a lack of identified failure modes. All these factors prevented the researchers from confirming a conceptual performance model for FBS.

Since then, developing the conceptual models has made use of the expanding data sets from performance testing and condition monitoring around New Zealand.

Case study projects referred to in the 2011 research report have continued to be used as 'baseline reality checks' for the conceptual models in ongoing work by the transport sector.

8.2 Ongoing research initiatives

8.2.1 Conceptual model: fatigue relationships

In-situ performance testing (Gribble and Stevens 2016) of cement stabilised and FBS sites around New Zealand was used to continue the background development of conceptual models for pavements incorporating unbound granular, cemented and FBS layers.

FBS performance on Auckland motorways has led to recommendations (Chappell pers comm) for consideration during design and construction of FBS layers. These include: assumptions for layer modulus; stress dependency; design guidance for phase 1 and 2 moduli; limits for tensile stress at the base of the

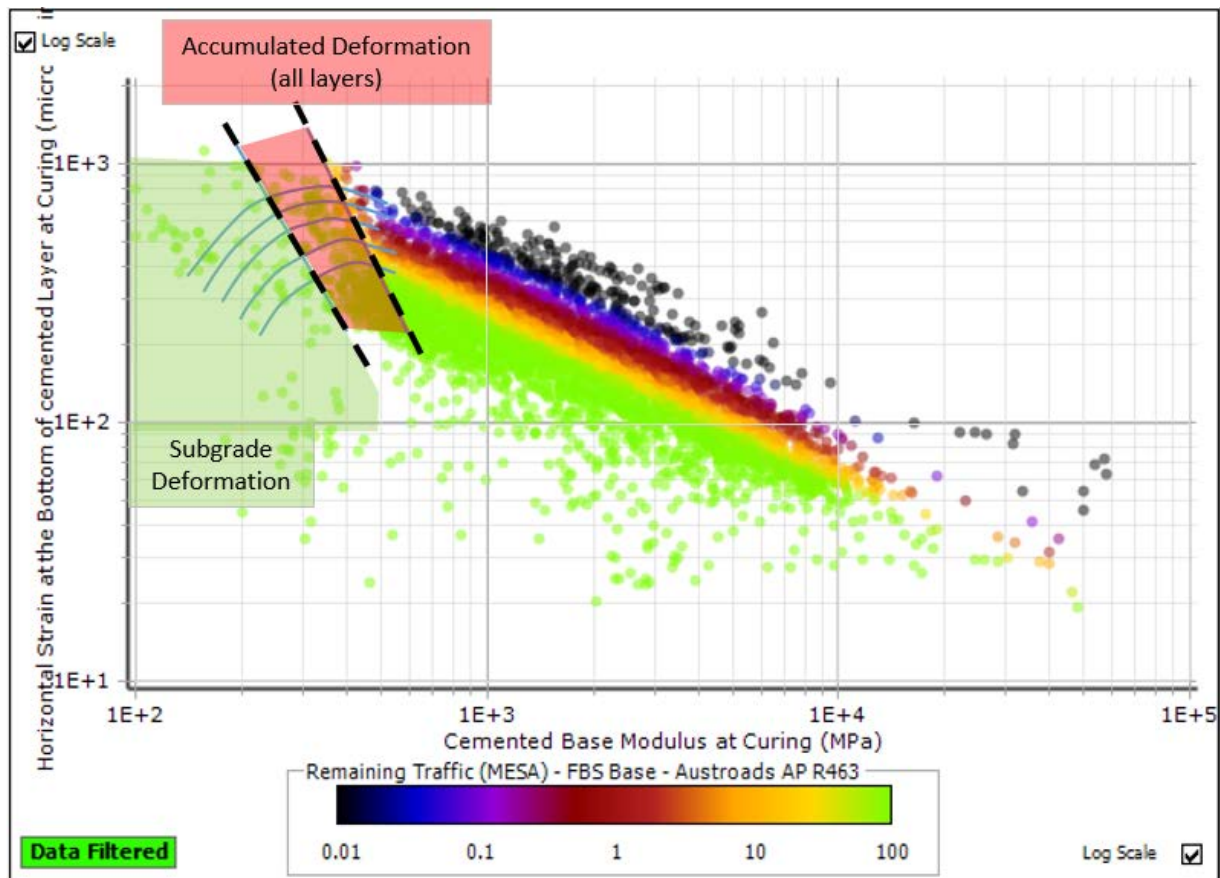
stabilised layer; model expected deflections using an initial higher modulus (phase 1) for immediate construction verification; verification of model assumptions during construction and phase 2 assumptions after one year.

Ongoing research continues to address the above, as far as practical given the data collated so far.

Initial modelling (Gray et al 2011) started by seeking limits to the horizontal strain at the base of the stabilised layer as a function of the modulus of the layer. The convenience of this form of presentation is that it lends itself to extending the model to cover the full spectrum of material properties from unbound, through to the lightly and heavily bound materials. Initially only a single phase was considered (ELTS concept, refer section 4.2.1).

Figure 8.1 demonstrates the possible form of a future pavement performance relationship with layer modulus and horizontal strain in a cemented top layer of the pavement (which will be influenced by binder content and underlying pavement and subgrade support conditions) potentially used to predict likely initial failure mechanism (varying between subgrade deformation at lower base modulus and cracking at higher base modulus) and remaining life (distinguished in figure 8.1 by the coloured data points). Are similar relationships possible in the future with FBS?

Figure 8.1 Conceptual pavement model incorporating cemented top layer



Source: Graham Salt, Geosolve

8.2.2 Pavement performance over time

The ongoing research effort has considered how a stabilised layer matures over time. A curing phase was considered and all stabilised layer moduli were standardised to a 'cured' state (12 months from date of construction). This assumed adequate support was present so only a small proportion of the pavement life (ideally less than 1/25th) would be consumed in the bedding-in interval. This was intended to facilitate quantifiable comparisons between sites which would often be tested at varying times after completion.

Extensive work with the expanding stabilisation database has been carried out. This sought to quantify the improvement of in-situ modulus that could be expected when an existing unbound granular pavement had the upper layer (200mm thick) subjected to stabilisation of a given type.

The term 'normalised effective stiffness' or NES was explored to characterise the ratio of the stabilised layer modulus to that for an unbound layer of the same thickness with the same underlying support structure. A value of 1.0 implies no improvement from unbound and values of 2 or more would be typical.

Techniques were developed to predict NES beginning with a simple two-parameter equation. As the database grew, by including CAPTIF data and extensive data from South African in-situ stabilisation trials, the number of relevant parameters had to be increased progressively to address inconsistencies encountered with the more simplistic models. To achieve a rational model that adequately explained all relevant characteristics of the database, the researchers found at least four variables to be included.

The basis of the model has been developed but the work is proving time consuming. Further work is needed, including a more rigorous treatment of modulus non-linearity (Ullidtz 1987) which has been over-simplified for expedience but is limiting⁶ the current accuracy.

A presentation of the combined dataset can potentially be advanced as a preferred form of model (incremental-recursive⁷). In this the stabilised moduli are updated in prescribed time steps so the revised stresses, strains and non-linear moduli can be recalculated for a more realistic time-related pavement model. The change of the stabilised layer modulus with trafficking can potentially be tracked to cover what might be termed progressive de-bonding of the stabilised material.

Once the de-bonding is complete the previously stabilised material becomes unbound. In this state there is a much slower change in modulus, as no further bonds are lost. The mechanism becomes limited to one of breakdown of the aggregate component particles or 'attrition', with increase in fines as a result of the dynamic shear strains from successive loads.

The key difference from the current approach is there is no 'intrinsic' or single unbound modulus in the ELTS phase: the relevant unbound modulus changes with the support condition (subgrade CBR) as evidenced by the differences in central deflection.

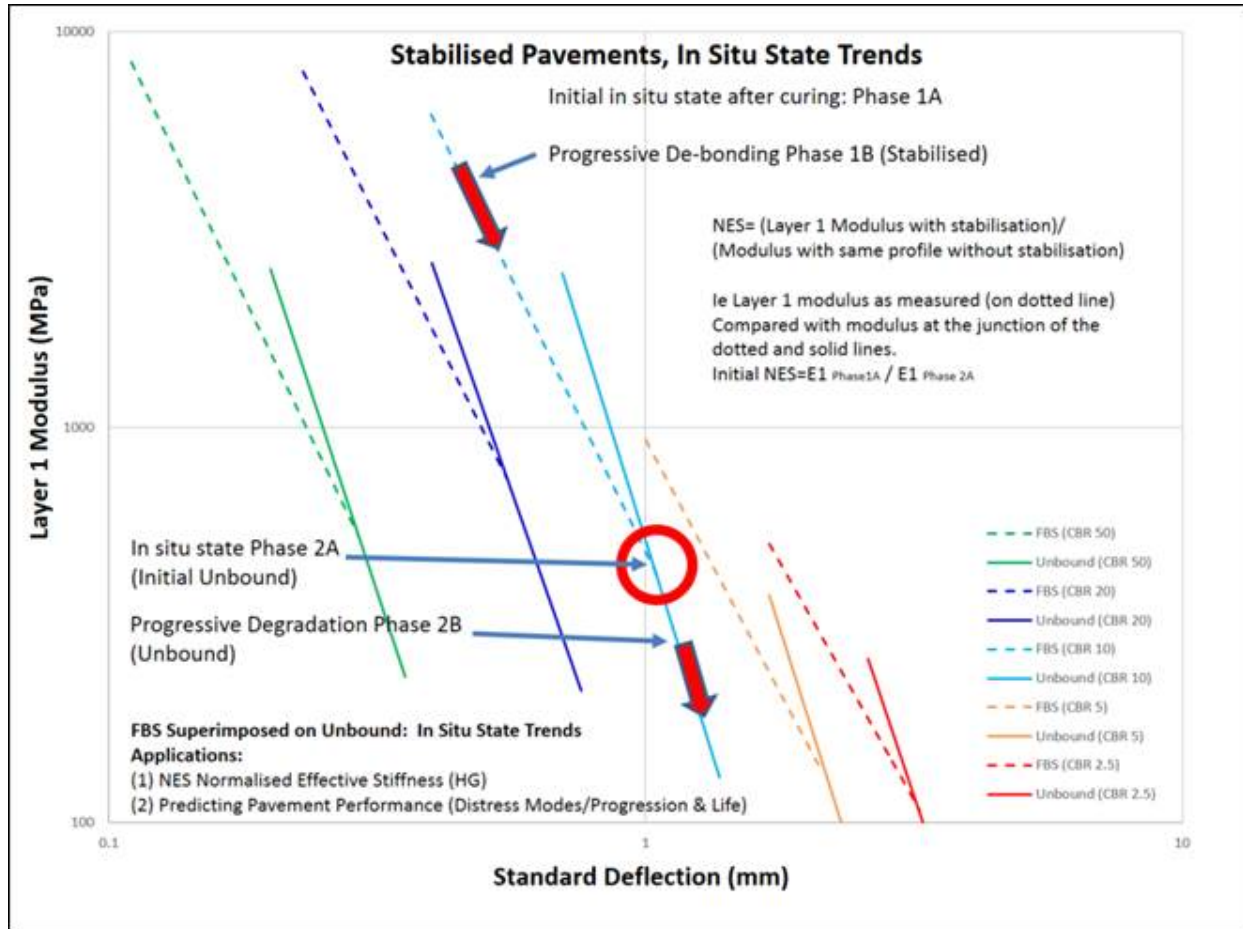
These concepts are described in figure 8.2. Potentially the progression from lightly bound to unbound and then beyond can be modelled and calibrated using in-service central deflection, based on a prior knowledge of the subgrade and underlying pavement support conditions?

The two previous stabilisation models, for cement alone, and for foamed bitumen, (Gray et al 2011) have been regularly revised and outputs generated to enable comparison of the model predictions with reality. Ongoing observations have been used for improvements of life prediction.

⁶ Per Ullidtz has frequently pointed out that ignoring or improperly characterising subgrade modulus non-linearity creates substantial errors in the other layer moduli especially that of the top layer, hence the approach adopted.

⁷ Literature, particularly that by Ullidtz promotes incremental-recursive models as the appropriate method for dealing with fatigue of bound layer models.

Figure 8.2 Conceptual model for stabilised pavement layer over time



Source: Graham Salt, Geosolve

9 Conclusions

The key objectives for this research project were to enable the New Zealand Transport Agency to publish a comprehensive, user relevant and practical best practice guide for stabilisation for use in New Zealand, promote stabilisation best practice, maximise the opportunities presented by pavement stabilisation and provide an proactive basis to support ongoing review, implementation and innovation.

Today stabilisation is a versatile and powerful technique used in constructing, rehabilitating and maintaining highways, public and private roads, ports, airports, domestic, commercial and industrial pavements. Stabilised ground improvement contributes to land development initiatives throughout New Zealand, notably rebuilding works in Canterbury following the 2011 earthquakes.

The research has utilised national and international research and development and evidence from current investigation, design and construction best practice to present herein practical guidance for stabilisation in New Zealand, including the explanation and development of relevant worked examples.

The research highlights the future research and development opportunities presented by conceptual pavement performance models and the ever expanding pavement in service performance dataset.

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Appendix A: Glossary

AASHTO	American Association of State Highway and Transportation Officials
BSM	bitumen stabilised material
CAPTIF	Canterbury Accelerated Pavement Testing Indoors Facility
CBR	California bearing ratio
CDF	cumulative damage factor
CIRCLY	pavement analysis software, MINCAD Systems
DCP	dynamic cone penetrometer (or Scala penetrometer)
ELTS	effective long-term stiffness
ESA	equivalent standard axles
FBS	BSM-foam
FWD	falling weight deflectometer
IANZ	International Accreditation New Zealand
ITS	indirect tensile strength
MESA	millions of equivalent standard axles
NES	normalised effective stiffness
PN	pavement number
RCC	roller compacted concrete
RLT	repeat load triaxial
RVT	random verification testing
SARA	South African Roads Agency
SIL	subgrade improvement layer
SMA	stone mastic asphalt
SR	stress ratio
Transport Agency	New Zealand Transport Agency