

Guide to Pavement Technology Part 4D

Stabilised Materials



Austroads



Guide to Pavement Technology Part 4D: Stabilised Materials



Austroads

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Guide to Pavement Technology Part 4D: Stabilised Materials

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Abstract

The use of stabilisation technology for stabilising and recycling materials for pavement construction and maintenance is widely accepted as a cost-effective method of improving long-term performance and reducing whole-of-life costs of modern, heavily-trafficked pavements.

Guide to Pavement Technology Part 4D: Stabilised Materials described in detail the:

- types of stabilisation undertaken in improving pavement materials and subgrades
- types of binders used in stabilisation
- types of materials suited to particular binders
- laboratory determination of the type and quantity of binder required to achieve a particular type of stabilised material (mix design).

This part of the Guide to Pavement Technology does not detail quality control aspects of manufacture or performance attributes of stabilised materials nor the safety aspects of using specific binders.

Keywords

Guideline, pavement technology, stabilisation, recycling, cement, lime, bitumen, pozzolan, pavement design, pavement materials, specification, construction, compaction, surfacing, performance.

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Edition 2.1 corrects Figures 5.4 and 5.5.

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Edition 2.0 is a complete revision of edition 1.0 published in 2006 and includes:

- restructuring the Guide based on stabilisation materials
- editorial changes and minor technical changes throughout
- major technical changes to the guidance relating to cementitious stabilised materials and foamed bitumen stabilised materials.

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Austrroads' purpose is to support our member organisations to deliver an improved Australasian road transport network. To succeed in this task, we undertake leading-edge road and transport research which underpins our input to policy development and published guidance on the design, construction and management of the road network and its associated infrastructure.

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Summary

Stabilisation may be defined as a process by which the intrinsic properties of a pavement material or earthworks materials are altered by the addition of a stabilisation binder or granular material to meet performance expectations in its operating, geological and climatic environment.

Part 4D Stabilised Materials of the Austroads Guide to Pavement Technology describes in detail:

- the types of stabilisation undertaken to improve pavement materials and earthworks materials
- the types of binders used in stabilisation
- the types of binders suitable for particular materials
- the laboratory determination of the type and quantity of binder required to achieve a particular type of stabilised material (mix design).

It does not detail quality control aspects of manufacture or all the performance attributes of stabilised materials nor the safety aspects of using specific binders.

This Guide is an update of the guide originally published in 2006.

Stabilisation of unsealed roads is described in *Guide to Pavement Technology Part 6: Unsealed Roads*.

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1. Introduction

Stabilisation may be defined as a process by which the intrinsic properties of pavement materials or earthworks materials are altered by the addition of a stabilisation binder or granular material to meet performance expectations in its operating, geological and climatic environment.

The use of stabilisation technology for stabilising and recycling materials for pavement construction and maintenance is widely accepted as a cost-effective method of improving long-term performance and reducing whole-of-life costs of modern, heavily-trafficked pavements.

This part of the *Guide to Pavement Technology* discusses:

- the types of stabilised pavement and earthworks materials
- the binders associated with various types of stabilised pavement materials
- methodologies for the determination of the appropriate mix proportions in the manufacture of stabilised pavement materials
- specification considerations for manufacture and supply of stabilised materials.

Additional issues related to stabilisation technology are discussed in other parts of the Austroads *Guide to Pavement Technology* are listed in Table 1.1.

Table 1.1: References to stabilisation technology in Austroads *Guide to Pavement Technology*

Technology	Guide Part
Structural design of new stabilised pavements	Part 2: Pavement Structural Design
Surfacings for stabilised pavements	Part 3: Pavement Surfacing
Unbound granular materials	Part 4A: Granular Bases and Subbase Materials
Recycled materials	Part 4E: Recycled Materials
Bituminous binders	Part 4F: Bituminous Binders
Stabilisation of subgrades	Part 4I: Earthworks Materials
Stabilising binders	Part 4L: Stabilising Binders
Structural design of in situ stabilisation treatments	Part 5: Pavement Evaluation and Treatment Design
Stabilisation of unsealed pavements	Part 6: Unsealed Pavements
Maintenance practices for stabilised pavements	Part 7: Pavement Maintenance
Construction of stabilised pavements	Part 8: Pavement Construction

Stabilisation of unsealed roads is described in *Guide to Pavement Technology Part 6: Unsealed Pavements (Austroads 2009a)*.

Table 1.2 lists websites from which pertinent publications on stabilisation technology such as specifications, technical notes, guidelines, work tips and safety data can be obtained.

Table 1.2: Key websites pertinent to stabilisation technology

Organisation	Website
Austroads	www.austroads.com.au
Australian Stabilisation Industry Association	www.auststab.com.au
World Road Association (PIARC)	www.piarc.org

2. Overview

2.1 Purpose of Stabilisation

Stabilisation in new road pavement construction, and the rehabilitation of existing pavements, is broadly undertaken for the following purposes:

- correct any mechanical deficiencies (particle size distribution (PSD) and/or plasticity) in unbound granular materials and earthworks materials
- increase the strength of a material, i.e. Unconfined Compressive Strength (UCS) and flexural strength
- increase the modulus or bearing capacity of a material, i.e. Californian Bearing Ratio (CBR), indirect tensile modulus, flexural modulus and resilient modulus
- reduce the permeability and/or moisture sensitivity, which can result in a loss of strength, of the material
- provide cost-effective pavement configurations through the provision of stabilised pavement and earthworks layers
- improve the wearing characteristics of unsealed pavements
- provide a means by which existing pavements can be recycled
- improve the bearing capacity of subgrades so they can accept construction traffic
- enhance the compaction of overlying pavement materials.

2.2 Manufacture of Stabilised Materials

Stabilised materials can be manufactured using the following procedures.

2.2.1 Plant-mixed

This involves stationary pugmill mixing of a stabilisation binder with an unbound granular material sourced from quarrying or reclaimed construction and demolition waste (generally concrete). The quality of unbound granular pavement materials used in plant mixing typically conforms to road agency specifications for PSD, plasticity and source rock hardness. The stabilised material is delivered to the site in trucks and then paved or spread, compacted, shaped and cured in preparation for the placement of the overlying layers, e.g. additional stabilised or unbound granular layers, asphalt, concrete or thin bituminous surfacings.

2.2.2 In situ-stabilised Materials

This is a mobile process, during which a stabilisation binder is added to an existing pavement or earthworks material and mixed with a purpose-designed road recycler.

The quality of in situ stabilised pavement materials is often variable and, in some cases, requires the addition of other unbound granular materials to correct deficiencies in PSD, particle shape and/or plasticity. In situ stabilised material is shaped then compacted and cured prior to the placement of overlying layers, e.g. additional stabilised or unbound granular layers or bituminous surfacing.

In situ stabilisation of a subgrade is undertaken as either a construction expedient or to improve the design CBR or modulus of the top of subgrade prior to the placement of an overlying pavement.

2.3 Stabilisation Binders

The binders most commonly used in road stabilisation include:

- lime
- Portland cement
- cementitious blends, including lime and/or cement combined with ground granulated blast furnace slag (GGBFS) and/or fly ash
- bituminous materials, including foamed bitumen and bitumen emulsions often including lime or cement as a secondary binder
- granular materials, including crushed rocks, aggregates and sands
- chemicals, including salts, organic and polymer compounds.

2.4 Categories of Stabilised Materials

Categories of stabilised materials described in this Part are:

- cementitiously-modified pavement materials (Section 3.7)
- cementitiously-bound pavement materials (Section 3.8, Section 3.9)
- cement-treated subgrades and earthworks materials (Section 3.10)
- lime-modified pavement materials (Section 4.5)
- lime-stabilised subgrades and earthworks materials (Section 4.6, Section 4.7, Section 4.8)
- foamed bitumen stabilised pavement materials (Section 5.3, Section 5.4)
- bitumen emulsion-stabilised pavement materials (Section 5.3, Section 5.5)
- granular stabilised materials (Section 6)
- stabilisation using other chemical binders (Section 7).

Table 2.1 summarises the types of stabilised materials, typical strengths achieved after stabilising and commonly achieved performance attributes. Table 2.1 excludes bituminous-stabilised materials which are discussed in Section 5.

Table 2.1: Stabilisation categories and characteristics

Category of stabilisation	Indicative laboratory strength after stabilisation	Binders adopted	Anticipated performance attributes ^(5, 4)
Subgrade and formation treatments			
Stabilised earthworks materials	1 < UCS ^(2,3) ≤ 2 MPa or CBR ⁽¹⁾	<ul style="list-style-type: none"> Lime and/or cementitious binder (high plasticity soils) Cement and/or cementitious binder (low plasticity soils) 	<ul style="list-style-type: none"> Improved constructability Improved subgrade CBR and modulus Improved shear strength Reduced heave and shrinkage
Pavement material treatments			
Granular stabilisation	CBR ⁽¹⁾ > 30%	<ul style="list-style-type: none"> Blending other granular materials which are classified as binders in the context of this Part 	<ul style="list-style-type: none"> Improved pavement modulus Improved shear strength Improved resistance to aggregate breakdown
Modified materials	UCS ⁽²⁾ < 1 MPa	<ul style="list-style-type: none"> Addition of small quantities of cement or cementitious binder Addition of lime Addition of chemical binder 	<ul style="list-style-type: none"> Improved long-term rut-resistance Improved pavement layer modulus after curing After curing, reduced sensitivity to loss of strength due to increasing moisture content Similar to unbound granular materials, moisture content prior to sealing needs to be limited to inhibit premature distress At low binder contents can be subject to erosion where cracking is present
Lightly-bound cemented materials	1 ≤ UCS ⁽³⁾ ≤ 2 MPa	<ul style="list-style-type: none"> Addition of small quantities of cementitious binder, commonly less than 3% binder 	<ul style="list-style-type: none"> Greater rut-resistance than modified materials May be susceptible to fatigue cracking but cracking finer than bound materials At low binder contents can be subject to erosion where cracking is present
Bound cemented materials	UCS ⁽⁴⁾ > 2 MPa and/or flexural modulus and flexural strength	<ul style="list-style-type: none"> Addition of greater quantities of cementitious binder, commonly binder content of 3% or more 	<ul style="list-style-type: none"> Increased pavement modulus Thickness design needs to consider susceptibility to fatigue cracking Some binders introduce transverse shrinkage cracking

1. Four or 10-day laboratory soaked CBR.
2. Values determined from test specimens stabilised with General Purpose (GP) cement and prepared using 100% standard Proctor compactive effort at 100% standard optimum moisture content, normal curing for a minimum 28 days in moist condition without soaking in water.
3. The Unconfined Compressive Strength (UCS) criteria defining lightly-bound materials listed are those currently used by Queensland Department of Transport and Main Roads (TMR 2018a). These limits are currently the subject of Austroads research.
4. Values determined from test specimens stabilised with GP cement and prepared using 100% standard Proctor compactive effort at 100% standard optimum moisture content (OMC), normal curing for a minimum 28 days. Some road agencies soak specimens in water prior to testing.
5. Not all of the performance attributes may be improved by all of the common binders.

2.4.1 Earthworks Materials Stabilisation

In the context of this Part, earthworks materials stabilisation implies the addition of a binder to a soil by in situ stabilisation. Other methods of improving subgrades, such as geofabrics or geogrids, are discussed in *Guide to Pavement Technology Part 4G: Geotextiles and Geogrids* (Austroads 2009b).

Stabilisation of earthworks materials can be undertaken to provide a short-term higher-bearing capacity, a layer more capable of carrying construction traffic (construction platform), and/or a layer which will assist in the compaction of the subbase and base layers. In addition, the stabilised soils may be designed and constructed to provide long-term bearing capacity that may be considered in the structural design of the overlying pavement.

Lime stabilisation of subgrades with high plasticity is the most-commonly used stabilisation treatment (Section 4.6 to Section 4.8).

Stabilisation of soils with cementitious binders is described in Section 3.10.

2.4.2 Modified Granular Pavement Materials

Modified granular pavement materials are those to which small quantities of cement, cementitious, lime or other chemical binders have been added to improve the performance attributes of the material but not to the extent that the stabilised material has significant tensile strength.

As discussed in Section 4.5, lime is effective in modifying excessive plastic properties of subbase and base materials. Such modification of unbound granular base materials is a widely accepted and successful practice as it decreases moisture susceptibility. At lime contents of less than about 3%, the risk of undesirable shrinkage cracking is low, and it would rarely be necessary to take special measures to combat reflective cracking.

Cementitious modification (Section 3.7) is adopted when it is desirable to increase rut-resistance, modulus and/or decrease moisture susceptibility while at the same time maintaining flexible pavement characteristics.

The distress mechanisms of modified pavement materials are vertical deformation and shear. The intention is that the amount of cementitious binder used is limited so that cracking is not a distress mechanism.

2.4.3 Cementitiously-bound Pavement Materials

Cementitiously-bound pavement materials are produced by the addition of stabilising binders to granular materials in sufficient quantities to produce a material which has tensile strength, this being significantly higher for bound materials compared to lightly-bound materials. Binders commonly used include cement and cementitious materials.

A bound material acts like a 'beam' in the pavement to resist traffic loading and has significantly increased structural capacity compared with unbound granular and modified materials. However, shrinkage and premature fatigue cracking in the pavement layers needs to be controlled (Figure 2.1).

Figure 2.1: Example of fatigue cracking of a cementitiously-bound base



Lightly-bound materials are commonly manufactured using lower quantities of cement or cementitious binders. These stabilised materials are also susceptible to fatigue cracking; however, the nature of the cracking differs from bound cemented materials (Figure 2.1). The material cracks into smaller size blocks, possibly due to the lower strength of the material and the use of thinner layers. As a result, crack widths tend to be initially smaller than for bound materials. If surface cracking of lightly-bound materials does occur, it may be followed by erosion caused by pumping of fines and subsequent deformation.

In relation to the use of lightly-bound bases, surface cracking has been observed in some projects and not others. Further research is being undertaken to improve the characterisation of these materials to reduce the risk of cracking.

2.4.4 Stabilisation using Bitumen

Bitumen-stabilised granular materials are produced using either foamed bitumen or bitumen emulsion either plant-mixed or in situ stabilised. The materials treated are normally granular pavement materials, previously cement stabilised materials or reclaimed asphalt pavement (RAP). Where an existing pavement is recycled, old seals or asphalt surfacings are usually mixed with the underlying layer and treated to form a new base or subbase layer.

Foamed bitumen is a mixture of air, water and bitumen. Injecting a small quantity of cold water into hot bitumen produces an instantaneous expansion of the bitumen to form a fine mist or foam. In this foamed state, the bitumen is then suitable for mixing with the host material. Foamed bitumen-stabilised pavement materials commonly have 2.5–3.5 % residual bitumen and 1–2% of lime or cement as the secondary binder. Lime is added for the following purposes (Ramanujam & Jones 2007):

- flocculate and agglomerate the clay fines
- stiffen the bitumen binder
- act as an anti-stripping agent to assist the dispersion of the foamed bitumen throughout the host material
- increase the modulus and improve the early-life rut resistance of the stabilised material.

The most common distress mode is cracking; it may be load-induced fatigue cracking or cracking due to moisture changes in underlying highly expansive subgrades.

There is currently limited use of bitumen emulsion-stabilised materials in Australia. Where they are used, the empirical mix design process is intended to produce modified pavement materials with enhanced modulus and rut-resistance but without being susceptible to fatigue cracking.

2.4.5 Granular Stabilisation

Granular stabilisation is the process of adding another granular material to correct a deficiency in the intrinsic material properties of the parent material. These stabilising materials include natural gravels, crushed rock and fine-grained soils.

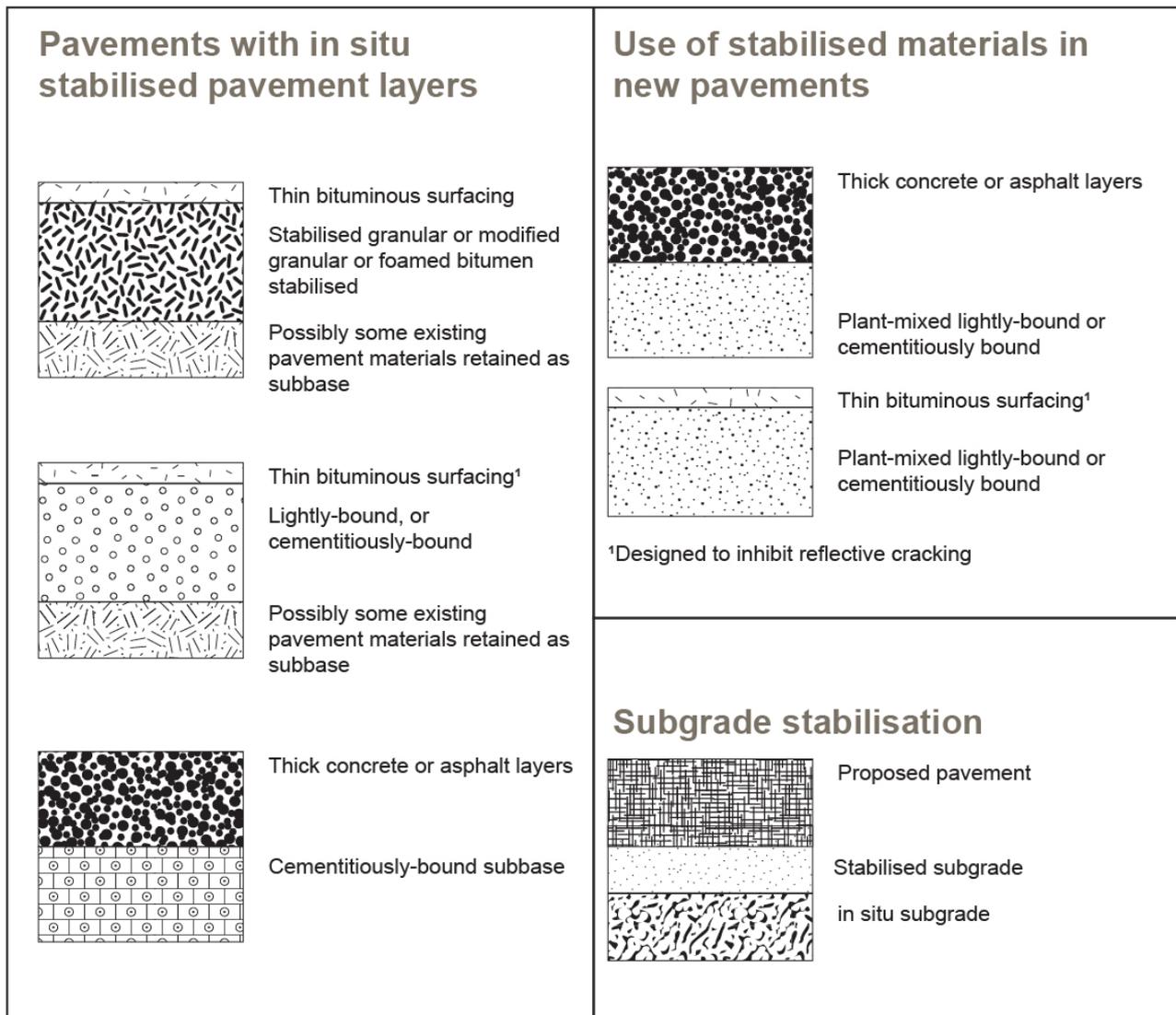
Granular stabilisation alters the intrinsic properties of the parent material, e.g. PSD, particle shape, plasticity and aggregate hardness. The load-bearing capacity performance attributes are improved through increased mechanical interlock, increased inter-particle friction, decreased aggregate breakdown and increased resistance to weakening by water ingress (permeability).

The common distress mechanisms for granular-stabilised pavement materials are rutting and shoving. Such stabilised materials remain unbound and are not susceptible to fatigue cracking.

2.5 Pavement Configurations Incorporating Stabilised Materials

Stabilisation treatments can be incorporated in a variety of pavement configurations including composites of different materials. Examples of pavement configurations with stabilised materials are shown in Figure 2.2.

Figure 2.2: Typical pavement configurations incorporating stabilised materials



2.6 Binders used in Stabilisation

2.6.1 Principal Binders and Combinations

Stabilisation binders used in road construction are manufactured to either Australian or New Zealand Standards or road agency specifications. There are several ways to categorise binder types; the format used in this Guide is in common usage in Australia and New Zealand, viz.:

- lime AS 1672.1-1997
- cement AS 3972-2010, NZS 3122:1995
- hot bitumen AS 2008-2013, NZTA M1:2011
- bitumen emulsion AS 1160-1996, NZTA M1:2011
- GGBFS AS 3582.2-2016
- fly ash AS 3582.1-2016
- chemicals commonly propriety products, therefore no recognised standards.

In addition, granular stabilisation through the addition of gravels, sands, etc. is a common practice.

Stabilisation binders are categorised in terms of their main constituent, viz.:

- Lime:
 - hydrated lime [Ca(OH)₂]
 - quicklime [CaO]
- Cement:
 - general purpose (GP) cement
 - general purpose blended (GB) cement
- Cementitious blends consisting of combinations of pozzolanic material and cement and/or lime such as:
 - lime and fly ash
 - lime and GGBFS
 - lime, GGBFS and fly ash
 - cement and GGBFS
 - cement and lime
 - cement, lime and fly ash
 - cement, GGBFS and fly ash

Note that all blends incorporating lime require hydrated lime for the cementing process. This may be derived from direct addition of hydrated lime or in situ slaking of quicklime. However, it is common practice to simply refer to it as lime.

- Bitumen Class 170 (C170) bitumen is typically used in stabilisation as:
 - foamed bitumen
 - bitumen emulsion.

Foamed bitumen stabilisation usually incorporates a supplementary binder, e.g. lime or cement:

- Other proprietary chemical products, including lignin derivatives, polymers and salts (see Section 7).

More detailed guidance on lime, cement, fly ash and GGBFS is provided in *Guide to Pavement Technology Part 4L: Stabilising Binders* (Austroads 2009c), whilst *Part 4F: Bituminous Binders* (Austroads 2017a) provides information relating to bituminous binders.

2.6.2 Characteristics of Stabilisation Additives

Table 2.2 presents an overview of the attributes associated with commonly-adopted binders/additives.

Table 2.2: Characteristics of various stabilising binders/additives

Stabilisation binder/additive	Stabilising action	Stabilisation effect	Applicable material types
Cement	<ul style="list-style-type: none"> Cementitious inter-particle bonds are developed 	<ul style="list-style-type: none"> Low binder content (< 2%): decreases susceptibility to moisture changes, resulting in modified materials High binder content: increases modulus and tensile strength significantly, resulting in bound materials 	<ul style="list-style-type: none"> Not limited apart from materials which contain deleterious components (organics, sulphates, etc.) which retard cement reactions Suitable for granular materials but inefficient in predominantly one-sized materials and heavy clays May be suitable for low-plasticity soils that are not reactive to lime
Cementitious blends	<ul style="list-style-type: none"> Cementitious inter-particle bonds are developed but rate of development is slow compared to cement 	<ul style="list-style-type: none"> Generally, like cement but rate of gain of strength similar to lime Generally, reduces shrinkage cracking compared to cement 	<ul style="list-style-type: none"> Similar to cement
Lime	<ul style="list-style-type: none"> Cementitious inter-particle bonds are developed but rate of development is slow compared to cement 	<ul style="list-style-type: none"> Improves handling properties of cohesive materials and initial strength Higher binder content: long-term increases in CBR, modulus and tensile strength 	<ul style="list-style-type: none"> Suitable to modify granular materials with high plasticity using lower binder contents Suitable to stabilise cohesive subgrade soils in the long-term if higher binder contents used Requires clay components in the soil/gravel that will react with lime Organic materials will retard reactions
Bitumen (emulsion)	<ul style="list-style-type: none"> Agglomeration of fine particles 	<ul style="list-style-type: none"> Decreases permeability and improves cohesive strength Decreases moisture sensitivity by coating fines 	<ul style="list-style-type: none"> Applicable to granular materials with low cohesion and low plasticity
Bitumen (foamed)	<ul style="list-style-type: none"> Inter-particle bonds are developed due to bitumen and secondary binders 	<ul style="list-style-type: none"> Increases modulus and tensile strength significantly, resulting in bound materials 	<ul style="list-style-type: none"> Applicable to granular materials with low cohesion and low plasticity
Granular material	<ul style="list-style-type: none"> Mixing two or more materials to achieve target PSD and plasticity 	<ul style="list-style-type: none"> Some changes to material strength, permeability, volume stability and compactability Materials remain granular 	<ul style="list-style-type: none"> Poorly-graded soils and natural gravels with a deficiency in the particle shape, PSD or plasticity
Other proprietary chemical products	<ul style="list-style-type: none"> Agglomeration of fine particles and/or chemical bonding (refer trade literature) 	<ul style="list-style-type: none"> Typically, increased dry strength, changes in permeability and volume stability 	<ul style="list-style-type: none"> Typically, poorly-graded soils and gravels

2.7 Selection of Stabilisation Type

2.7.1 Introduction

Table 2.3 summarises the various stabilisation treatments that may be used for earthworks and pavement materials assuming the binder type is selected in accordance with Table 2.4. A large number of factors influence whether a treatment is applicable for a project, including quality and thicknesses of existing pavement materials, subgrade strength, future traffic loading, climate, drainage, road geometry, availability of construction equipment and materials and whole-of-life costs. These issues are discussed in more detail in:

- *Guide to Pavement Technology Part 2: Pavement Structural Design* (Austroads 2017b)
- *Guide to Pavement Technology Part 5: Pavement Evaluation and Treatment Design* (Austroads 2011).

For the design of pavement rehabilitation treatments (Austroads 2011), it is important that the causes and modes of distress are identified to enable the most cost-effective use of stabilisation treatments.

Table 2.3: Types of stabilisation and typical binder applicable

Binder	Subgrade treatment	Pavement material treatments		
		Modified	Bound	
			Lightly-bound	Bound
Cement	Yes ⁽¹⁾	Yes	Yes	Yes
Cementitious blends	Yes ⁽²⁾	Yes	Yes	Yes
Lime	Yes	Yes	No ⁽³⁾	No
Bitumen	No	Yes	Yes	Yes
Other proprietary chemical products	Yes ⁽⁴⁾	Yes ⁽⁴⁾	No ⁽⁵⁾	No ⁽⁵⁾

1. *In some situations, cement has been successfully used after an initial lime treatment to improve the subgrade strength.*
2. *Need to use a blend with a high lime content, e.g. 70% lime and 30% GGBFS.*
3. *May be used as a pre-treatment*
4. *Not common. Refer trade literature.*
5. *Sometimes used as a secondary binder in association with a primary cement or cementitious binder.*

2.7.2 Sampling and Testing

It is essential with all stabilisation work that materials are thoroughly assessed and their reactions with the specific binder to be used in the stabilisation process be properly validated by laboratory testing before any field work commences.

Stabilised pavement materials should be tested to determine their quality and uniformity. Testing requirements will depend on the type of stabilisation contemplated. These requirements are described in the sections of the Guide dealing with the relevant methods of stabilisation.

Section 8.2 describes sampling of materials to be stabilised from existing pavements.

2.7.3 Preliminary Binder Selection

To gain a preliminary assessment of the type of stabilisation required for a particular material, PSD and plasticity are commonly used.

Table 2.4 provides initial guidance on the application of binder types before laboratory testing with particular materials and particular stabilising binders. Materials with less than 25% passing the 75 µm sieve are commonly used for base and upper subbase layers. In this Guide, materials with greater than 25% passing the 75 µm sieve are considered subgrade materials, although not all subgrade materials meet this definition (e.g. coarse sands).

Binders may be used in combinations or as part of staged construction. For example:

- bitumen and lime combined is commonly used in foamed bitumen stabilisation
- lime stabilisation may be adopted to dry out materials and reduce their plasticity prior to the application of other binders.

Table 2.4: Preliminary selection of binder/additive type

Particle size	More than 25% passing 75 µm sieve			Less than 25% passing 75 µm sieve		
	PI ≤ 10	10 < PI < 20	PI ≥ 20	PI ≤ 6 & PI x %passing 75 µm ≤ 60	PI ≤ 10	PI > 10
Plasticity index (PI)						
Binder type						
Cement and cementitious blends ^(1,3)	Usually suitable	Doubtful	Usually not suitable	Usually suitable	Usually suitable	Usually suitable
Lime	Doubtful	Usually suitable	Usually suitable	Usually not suitable	Doubtful	Usually suitable
Bitumen	Doubtful	Doubtful	Usually not suitable	Usually suitable	Usually suitable	Usually not suitable
Bitumen/lime blends	Usually suitable	Doubtful	Usually not suitable	Usually suitable	Usually suitable	Doubtful
Granular	Usually suitable	Usually not suitable	Usually not suitable	Usually suitable	Usually suitable	Doubtful
Dry powder polymers	Usually suitable	Usually suitable	Usually unsuitable	Usually suitable	Usually suitable	Usually not suitable
Other proprietary chemical products ⁽²⁾	Usually not suitable	Usually suitable	Usually suitable	Usually not suitable	Doubtful	Usually suitable

1. The use of some chemical binders as a supplementary addition can extend the effectiveness of cementitious binders in finer soils and soils with higher plasticity.
2. Should be taken as a broad guideline only. Refer to trade literature for further information.
3. TMR uses triple blend and have a method based on % passing 0.425 mm sieve and linear shrinkage (Volker & Hill 2016).

2.7.4 Final Selection of Binder Type and Content

Following the selection of candidate stabilisation treatments, binder contents are determined using the mix design processes described in Section 3 to Section 7.

There may be a number of feasible stabilisation methods. The decision on which to choose is generally a financial one, when the life cycle costs of all feasible alternatives are compared as described in Part 2 and Part 5 of the *Guide to Pavement Technology* (Austroads 2017b; Austroads 2011).

3. Cementitious Stabilisation

3.1 Introduction

Cementitious stabilisation refers to stabilisation using either cement or supplementary cementitious materials. Supplementary cementitious materials are mixtures of pozzolanic materials such as fly ash or pulverised GGBFS and lime. A pozzolan is a siliceous or alumino-siliceous material which possesses little or no cementitious value but which, in finely divided form, may be mixed with lime or Portland cement to form a cementitious material.

The primary reaction of cementitious binders is between the binder and the water in the host material, which leads to the formation of cementitious material. These reactions occur almost independently of the nature of the granular material. It is for this reason that cementitious stabilising binders can be used to stabilise a wide range of materials, from cohesionless sands and gravels to silts and low-plasticity cohesive materials.

Cementitious binders have two important effects on the behaviour of the material to be stabilised:

- They reduce the moisture susceptibility of some host materials, giving enhanced volume and strength stability under variable moisture conditions.
- For bound materials, cementitious binders can result in the development of interparticle bonds in host materials, endowing the stabilised material with tensile strength and high elastic modulus.

While cement stabilisation can offer many possibilities with respect to changing host material properties, other factors, including construction requirements, traffic loading, shrinkage characteristics, etc. can limit its application.

Lime may be considered as a cementitious material when mixed with pozzolans and this is discussed in this section. However, when lime is the primary stabilising binder it reacts with the natural pozzolans within the host material; as a result, the effect depends on the host material type more than when stabilising with cementitious materials. Section 4.5 discusses the use of lime without pozzolans to modify pavement materials.

Most of the discussion in this section relates to the stabilisation of granular pavement materials; specifically, Section 3.7 covers cementitiously-modified materials, Section 3.8 lightly-bound materials and Section 3.9 bound materials. Generally, soils with more than 25% passing the 75 μm sieve are not suitable to be cementitiously stabilised unless they have low plasticity (Table 2.4). The treatment of earthworks materials to expedite construction is discussed in Section 3.10.

3.2 Commonly used Cements

Cement used for stabilisation must conform to AS 3972-2010, *General Purpose and Blended Cements* or NZS 3122:1995 *Specification for Portland and Blended Cements (General and Special Purpose)* as appropriate. There are a variety of commercially-produced cement types and blends in Australia and each has different properties and characteristics. As discussed in Section 2.6.1, the principal cement types available are:

- Type GP – general purpose cement
- Type GB – general purpose blended cement.

GP cements are produced from a mixture of calcium carbonate, alumina, silica and iron oxide which, when calcined and sintered at high temperatures, results in the production of a new group of chemical compounds which are capable of reacting with water. The composition of individual cements can vary depending on the nature and composition of the raw materials being used. AS 3972-2010 defines type GP cement as: 'a hydraulic cement which is manufactured as a homogeneous product by grinding together cement clinker and calcium sulphate and which, at the discretion of the manufacturer, may contain up to 5% of mineral additions'.

Type GB cement is defined as hydraulic cement containing cement and a quantity comprised of one or both of the following:

- greater than 5% of fly ash or GGBFS, or both
- up to 10% silica fume.

Although types GP and GB cements are used in stabilisation, there is an increasing trend towards the use of supplementary cementitious materials (Section 3.3) because their increased working time allows more time to meet specified compaction and rideability requirements. In addition, early trafficking to induce closely-spaced microcracks can minimise the risk of larger, wider-spaced cracks appearing later.

Special-purpose cements are not commonly used for stabilisation in Australia.

3.3 Supplementary Cementitious Binders

3.3.1 General

Fly ash, GGBFS or other pozzolanic type materials may be combined with lime to form supplementary cementitious materials as outlined in Section 2.6. Supplementary cementitious materials provide an alternative to type GP cement, on the grounds of economy or for extended working time for compaction and finishing.

3.3.2 Lime

The properties and types of lime are described in Section 4.

For lime pozzolan stabilisation and other supplementary cementitious materials, the lime and the pozzolan (or other component additives) are dependent variables and this requires a comprehensive testing program to determine the optimum lime-to-pozzolan ratio (or ratio of other components).

In this regard the following should be considered:

- the costs involved for each of the additive components
- the need or otherwise for filler to correct a PSD deficiency.

3.3.3 Pozzolanic Materials

Cementitious binders may contain a pozzolanic additive which is a siliceous or alumino siliceous material. In finely divided form and in the presence of moisture, it chemically reacts at ordinary room temperatures with calcium hydroxide released by the hydration of cement or lime to form compounds possessing cementitious products. Pozzolanic additives include GGBFS and fly ash, which may be combined with lime or cement to form cementitious binders.

Ground granulated blast furnace slag (GGBFS)

GGBFS is used in road stabilisation. It is often manufactured pre-blended with lime as a proprietary product. It is commonly sourced from Wollongong in New South Wales (NSW) and must conform to AS 3582.2-2016.

GGBFS, which acts as a slow-setting hydraulic cement by itself, also reacts exceptionally well with lime and so it is an excellent pozzolanic material. Some GGBFS materials naturally contain small amounts of free lime, but the minimum lime content required for the reaction is one-part lime for each 10 parts GGBFS.

The most common combination of GGBFS/lime blends is 85:15. In addition to Accelerated Loading Facility (ALF) pavement trials (Jameson et al. 1995; Moffatt et al. 1998), which demonstrated good performance of GGBFS/lime blends in bound pavement stabilisation, it has been successfully used in NSW for at least the past 25 years.

Fly ash

Fly ash is a product of the coal power generation industry. The type of coal used, and the mode of operation of the plant, determines the chemical composition and PSD of the fly ash. Consequently, not all sources of fly ash are suitable for stabilisation. Generally, fly ash derived from the burning of black coal is high in silica and alumina and low in calcium and carbon and hence it is well suited for use in stabilisation. On the other hand, fly ash derived from the burning of brown coal contains large percentages of calcium and magnesium sulphate and chlorides and other soluble salts and hence it is unsuitable for use in stabilisation.

As unburned organic carbon breaks the continuity of contact in the cementitious reactions, its proportions should be limited to about 10%.

Fly ash must conform to AS 3582.1-2016 and be a 'fine grade type', i.e. solid material extracted from the flue gases of a pulverised coal fed boiler must have at least 75% passing the 45 µm sieve and a maximum 4% loss on ignition.

Other ash products, such as power station bottom ash, have also been successfully used in lime stabilisation (e.g. Ash Development Association of Australia 2005; Chapman & Youdale 1982; Francis 1994; Jameson et al. 1996a, 1996b; Symons & Poli 1996).

Bottom ash comprises about 10% of the ash produced at coal-fired power stations and, while it has a similar chemical composition to fly ash, it contains greater quantities of carbon and is relatively inert because it is coarser and more highly fused than fly ash.

3.4 Water

Water which would be considered as suitable for the making of concrete will also be suitable for use in cement stabilisation; for example, potable water is generally considered to be suitable. Water which contains organic matter or a high concentration of sulphates (e.g. above 0.05%) is unsuitable. Where other than normal potable water is to be used in construction, it is desirable to assess the effects of the water in the laboratory test program.

The quantity of water added to a cement-stabilised mixture is determined by the requirements of maximum density. Curing water may be needed to keep the pavement moist while hydration takes place. The provision of an adequate supply of water is frequently a major consideration. Saline waters are generally unsuitable for curing.

3.5 Deleterious Materials

Deleterious materials are usually not present in plant-mixed quarry products or in situ granular materials obtained from quarries. However, there may be projects where there may be concern about the quality of the host material. Organic matter, excess salt contents (especially sulphates and ferrous oxide) can retard or prevent proper hydration of cementitious binders (Ingles & Metcalf 1972; Queensland Department of Transport and Main Roads (TMR) 2018a, 2018b) for the following reasons:

- Organic matter interferes with the hydration process and also competes for available paste of cementitious binder and water. Uneconomical quantities of stabiliser are therefore usually required to achieve the required strengths.
- Sulphates can also interfere with pozzolanic reactions due to the formation of very expansive hydrates, which, if formed after compaction, can result in heave.
- Ferrous oxide can also interfere with pozzolanic reactions; however, there are no specific limits in the literature at this stage. Based on recent TMR experience (TMR 2018a, 2018b), contents greater than 10% have a deleterious effect and contents above 2% should be investigated further.

Table 3.1 lists TMR guidance on deleterious limits related to use of stabilisation with cementitious binders.

Table 3.1: TMR deleterious limits related to use of stabilisation with cementitious binders

Property	Test method	Limit
Organic content	TMR test method Q120B (TMR 2014a)	≤ 1.0%
Sulphate content (water soluble)	AS 1289.4.2.1-1997	≤ 1.9 g/L (SO ₄)
Ferrous oxide (FeO)	Not specified	≤ 2.0%

Note: The limits shown are a guide; advice should be obtained before stabilising materials with ferrous oxide contents > 2% or organic contents > 1%.

Source: TMR (2018a, 2018b).

3.6 Reaction of Materials with Cementitious Binders

The primary reaction during stabilisation is the hydration reaction of the cementitious binder with the water in the host material. This leads to the formation of cementitious material (calcium silicate and aluminium hydrates as in concrete). These reactions occur almost independently of the nature of the host material and it is for this reason that cement can be used to stabilise materials ranging from cohesionless sands and gravels, to silts and low-plastic clays (Table 2.4). The hydration reaction releases hydrated lime (about 30% by mass of the added cement in the case of GP cement stabilisation) and this can cause secondary reactions with any pozzolans within the host material. The secondary reaction produces cementitious products similar to those from the primary reaction.

The hydration reaction starts immediately on contact of the cementitious binder with water. It proceeds rapidly if cement is used and there are very significant strength gains in the first day. The secondary reactions with pozzolans are similar to those that occur in lime stabilisation and proceed slowly with time.

The reactions that occur with the use of supplementary cementitious binders are similar to the secondary reactions that occur with cement; they take place more slowly than the reaction that occurs with cement.

Pozzolanic reactions are usually slow but they continue over a long period provided that adequate moisture is present. Reactions are also temperature sensitive, the rate of reaction increasing with increasing temperature. Organic material and sulphates may cause retardation of the reaction.

The effectiveness of stabilisation using supplementary cementitious binders depend on the amounts of lime, pozzolan and the host material. For fly ash, a ratio of about one part of lime to two parts of fly ash by volume will produce maximum strength of the paste. Usually the amount of lime plus fly ash added to a host material should not exceed about 5% by mass. These proportions should be confirmed by testing.

3.7 Properties of Cementitiously-modified Pavement Materials

3.7.1 General

Cementitiously-modified pavement materials are those with small amounts of cementitious binders added to improve rut resistance and modulus or to correct other deficiencies (e.g. reducing plasticity) without causing a significant increase in tensile capacity (i.e. producing a bound material). Modified materials have a maximum 28-day UCS of 1.0 MPa tested after moist curing but without soaking at 100% standard Proctor maximum dry density (MDD) and optimum moisture content (OMC).

For the purposes of pavement design, modified materials are treated as unbound granular materials without consideration of their susceptibility to cracking.

If modification is only required to reduce the plasticity of subbase and base materials, then lime alone is usually a better binder option (Section 4.5).

3.7.2 Distress Types

The amount of stabilising binder is limited such that the cementitiously-modified pavement material exhibits limited or fine cracking, unlike the widely-spaced blocking cracking of some bound materials. Low cementitious contents are required such that the cementitious bonds are likely to be weak with respect to the applied traffic and thermal stresses; they may be discontinuous.

To address this risk of fine surface cracking of modified bases, consideration may need to be given to the surfacing type to inhibit moisture ingress and pumping of fines, particularly for heavily-trafficked roads in wet climates. Early-life trafficking can assist in the development of micro-cracking which may be beneficial in relation to the life of the surfacing.

After modification these materials commonly have greater rut-resistance and higher modulus than the host materials from which they are produced.

The moisture content of modified materials before sealing and trafficking needs to be limited to reduce the risk of premature distress.

3.7.3 Materials Suitable for Stabilisation

Like unbound granular materials, the performance of cementitiously-modified granular materials is highly dependent on the PSD and plasticity of the host material. Accordingly, for plant-mixed modified materials, the required host materials' properties are often the same as for unbound granular materials (e.g. Roads and Maritime Services (Roads and Maritime) 2014). In relation to the stabilisation of in situ materials, there may be some relaxation of requirements depending on the traffic loading and the costs of importing new granular materials to blend with the existing materials. However, materials with more than 25% passing the 75 µm sieve and with a PI in excess of 10 are commonly not considered suitable.

Finely-graded gravels, clayey gravels, silty sands (> 50% passing 425 µm sieve) and other materials without significant particle interlock are not suitable for use as modified basecourse materials. The life of such base materials will generally be short and rapid disintegration of the pavement may occur with the onset of cracking.

The deleterious effects of organic matter and salts in retarding or preventing the hydration of cementitious binders is discussed in Section 3.4 and Section 3.5.

3.7.4 Mix Design

For cementitious binders, including cement and GGBFS/lime and fly ash/lime blends, a maximum binder content of 12% is typically required to achieve the structural characteristics of modified materials. It is important, however, to recognise that there is an upper limit on strength which, if exceeded, may lead to a lightly-bound or bound pavement which has different performance characteristics (see Section 3.8 and Section 3.9).

Depending upon the type of binder spreader or plant-mix control used, binder contents less than 5 kg/m² may be difficult to apply uniformly. Consequently, construction plant limitation may result in a lower limit on the thickness that can be stabilised to produce a modified material.

Table 3.2 details the mix design process for cement-modified pavement materials.

Table 3.2: Mix design procedure for modified materials

Step	Activity	Reference
1	Obtain a representative sample of the untreated material to be stabilised, including any recycled surfacing materials.	
2	Measure PSD and assess whether it is necessary to improve the grading by adding other granular materials.	
3	Measure liquid limit (LL), plastic limit (PL) and PI, and assess whether the material needs to be pre-treated to reduce plasticity.	
4	Measure the organic matter, sulphates and ferric oxide contents to assess whether the material is suitable for chemical stabilisation.	Section 3.5
5	Select the binder or range of binders that may be suitable.	Table 2.4
6	Select a binder content for initial assessment and adjust in accordance with laboratory testing taking due consideration of practical spread rates possible.	
7	Combine a sample of the untreated material and the select binder content. Measure the standard or modified Proctor MDD and OMC.	
8	Mix a sample of untreated material with the selected binder content and at a moisture content determined from step 7. Compact UCS test specimens to a density determined from step 7 in accordance with road agency requirements.	
9	Demould the specimens and moist cure the test specimens for 28 days without soaking in water.	
10	Measure the 28-day UCS strength (Section 8.4.4) and check that it does not exceed the maximum allowable for a modified material. If the strength is excessive, either reduce the binder content and repeat steps 7 to 10 or select another binder and repeat steps 5 to 10. If a higher UCS is required, increase the binder content in increments of 0.5% and repeat steps 7 to 10.	Table 2.1
11	The following tests may also be undertaken at the design binder content (step 10): <ul style="list-style-type: none"> • shrinkage • capillary rise • working time • free swell • resilient modulus using the repeat load triaxial (RLT) testing method. 	

3.8 Properties of Lightly-bound Pavement Materials

Shown in Table 2.1, lightly-bound materials are treated with cementitious binders such that they have higher strength and modulus than cementitiously-modified materials, yet below the properties of bound materials. Consequently, lightly-bound materials have higher rut-resistance than modified materials; however, they are more likely to exhibit a network of finely-spaced cracks. Again, as for modified materials, consideration may need to be given to the surfacing type needed to inhibit moisture ingress and the pumping of fines, particularly for heavily-trafficked roads in wet climates.

In recent years, TMR has developed design methods (TMR 2018b, 2018c) and specifications for the use of lightly-bound materials. The TMR mix design method is based on a requirement that the UCS be 1–2 MPa, similar in approach to that described for modified materials. This method has been developed based on TMR experience with Queensland materials, environments, traffic loading, etc. It may not be directly applicable to other areas of Australasia.

In relation to the materials suitable for stabilisation, the advice listed in Section 3.7.3 also applies for lightly-bound materials.

Austrroads research is currently being undertaken to improve understanding of the distress types, mix design and structural design methods of lightly-bound materials.

3.9 Properties of Cementitiously-bound Pavement Materials

3.9.1 General

Cementitiously-bound pavement materials are described as a combination of a cementitious binder, water and granular material which is mixed together and compacted in the early stages of the hydration process to form a pavement layer which is subsequently cured. In the *Guide to Pavement Technology Part 2: Pavement Structural Design* (Austroads 2017b) such materials are called cemented materials.

The cementitious binder may consist of GP cement, GP blended cement, lime, or other chemical binder and may include one or more supplementary cementitious materials such as fly ash or GGBFS. The binder should be added in sufficient quantity to produce a bound layer with significant tensile strength.

3.9.2 Distress Types

Cracking is the primary and predominant distress type of cementitiously-bound materials. There are two principal forms of cracking:

- cracking from hydration and drying shrinkage
- fatigue cracking.

Cracking created from within the layer is mainly of concern in basecourses although reflective cracking from lower courses may occur.

Shrinkage cracking and erosion

A combination of subgrade restraint, high shrinkage and tensile strength in cementitiously-bound materials can cause widely-spaced (commonly 0.5–5.0 m) transverse and/or block cracking to occur. Though this may reduce the ride quality of the pavement it usually does not lead to serious structural problems provided the cracks are less than 2 mm wide and are sealed, or an appropriate modified bituminous surfacing is applied. Geotextile reinforced seals have also been used successfully to mitigate the effects of cracking.

If the cracks are not sealed, then moisture may enter the pavement, which may lead to pumping of fines from erosion and rapid deterioration of the pavement under the action of traffic.

The likelihood of pumping of fines from the base can be minimised by ensuring an adequate binder content for the stabilised layer(s) that is above the erodibility limits (Howard 1990) – see Figure 3.1.

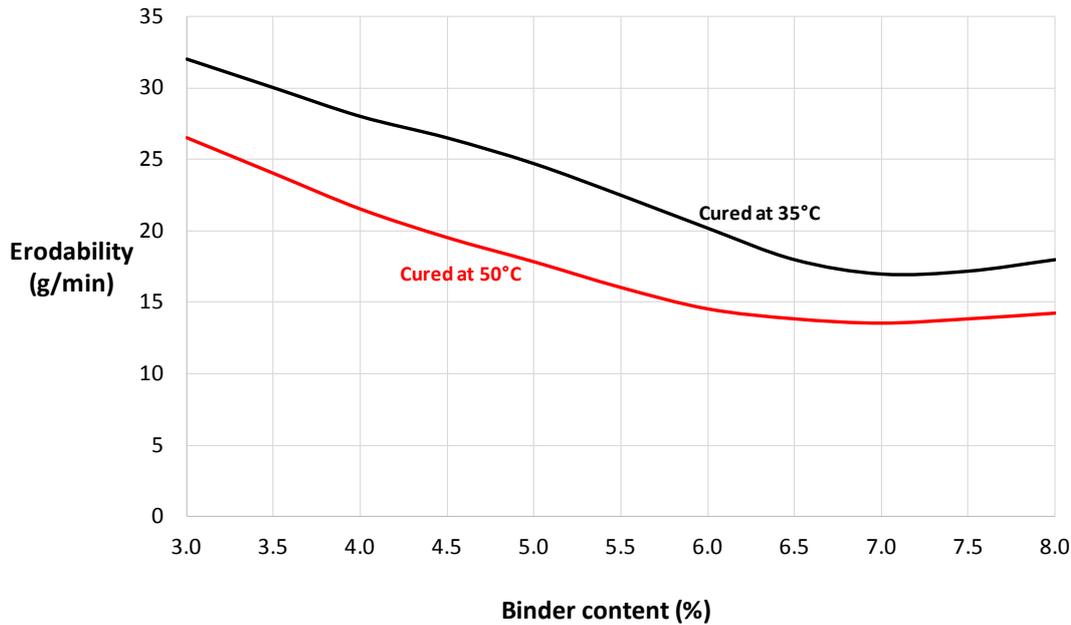
On heavily-trafficked roads the application of a polymer modified asphalt will lessen the impact loading of vehicles on the cracks reducing the fretting of the cracks at the surface.

Even if significant cracking does occur, provided moisture ingress into the pavement can be prevented, there may still be considerable life left in the pavement in the post-cracking phase, where the cementitiously-bound layer acts as an unbound granular or a modified granular material. This is particularly so if the layer is a subbase layer. This post-cracking phase life can be quantified using the design procedures in the *Guide to the Pavement Technology Part 2: Pavement Structural Design* (Austroads 2017b).

For pavements with thin bituminous surfacings, early trafficking of cementitiously-stabilised layers is considered beneficial as it facilitates the development of closely-spaced fine cracks which are easier to manage than widely-spaced large cracks which tend to occur without early trafficking (Yamanouchi 1975). The increased use of slow-setting cementitious binders such as Type GB cements and supplementary cementitious materials, together with the careful use of curing and early trafficking and the use of improved crack sealing and wearing surface technologies, can reduce the adverse effects of shrinkage cracking in cementitiously-bound bases.

The common practice now is to use pozzolans (Section 2.6) in cementitious binders. These are slow-setting and reduce cracking problems previously associated with in situ cement stabilisation of basecourses.

Figure 3.1: Example of erodibility decreases as the binder content increases



Source: Adapted from Howard (1990).

Fatigue cracking

Fatigue cracking occurs when the number of repetitions of tensile strain induced in the cementitiously-bound layer by the passage of traffic exceeds the capacity of that layer. Prevention of premature fatigue cracking is the principal criteria for the structural design of cementitiously-bound pavement courses (Austroads 2017b).

Recent research (Austroads 2014a, 2014b) has provided improved understanding of the material properties that influence the fatigue of cementitiously-bound materials. Test methods were developed for the manufacture of test beams and the measurement of flexural modulus, flexural strength and fatigue (Austroads test method AGPT/T600). The structural design guide (Austroads 2017b) provides a method of estimating in-service fatigue life either from laboratory-measured lives or indirectly from the measured flexural strength and flexural modulus. The modulus alone is insufficient to predict service fatigue life.

3.9.3 Materials Suitable for Stabilisation

Like unbound granular materials, the performance of cementitiously-bound materials is dependent on the PSD and plasticity of the host material. Accordingly, for plant-mixed materials, the required host materials properties are often the same as for plant-mixed unbound granular materials (e.g. Roads and Maritime 2014).

In relation to the stabilisation of in situ materials, there may be some relaxation of requirements depending on traffic loading and costs of importing new granular materials to blend with the existing materials. However, materials with more than 25% passing the 75 µm sieve and with a PI in excess of 10 are commonly not considered suitable. Note that the greater the deviation of PSD and plasticity from that required for plant-mixed materials, the greater the need for flexural strength testing for use in the pavement structural design.

In relation to deleterious materials, the guidance in Section 3.7.3 is also relevant for cementitiously-bound materials.

3.9.4 Mix Design

For cementitious binders, including cement and GGBFS/lime and fly ash/lime blends, typically 3% or more binder is required to achieve the structural characteristics of cementitiously-bound pavements.

The principal purpose of material mix design for bound pavements is to determine the quantity of binder required to reach the desired fatigue properties.

Although laboratory flexural modulus, strength and fatigue are the key parameters affecting in-service fatigue performance, the preparation and testing of flexural beams for mix design has yet to be implemented apart from test methods developed in research (Austroads 2018). Instead, the UCS test continues to be commonly used due to the availability of equipment, duration of testing and experience with this test.

Austroads (2017b) contains procedures to estimate design modulus from measured UCS values. Although some data relating UCS and flexural strength is available (Austroads 2014a), further research is required to finalise the relationship to predict flexural strength from UCS.

Table 3.3 details the mix design method for cementitious-bound materials, noting these procedures may vary across Australian and New Zealand.

In some cases, the selected binder may not meet the requirements of capillary rise and other optional tests but meet the UCS requirements. In these cases, engineering judgement may override the test results.

If there is no improvement in the stabilised material after increasing the binder content, it is suggested that granular stabilisation (Section 6) and/or another binder type be investigated.

Working time may need to be determined to set the time period in which mixing, compaction and trimming needs to be completed.

Table 3.3: Mix design procedure for cementitious-bound materials

Step	Activity	Reference
1	Obtain a representative sample of the untreated material to be stabilised, including any recycled surfacing materials.	
2	Measure PSD and assess whether it is necessary to improve the gradings by adding other granular materials.	
3	Measure LL, PL and PI, and assess whether the material needs to be pre-treated.	
4	Measure the organic matter, sulphates and ferric oxide contents to assess whether the material is suitable for chemical stabilisation.	Section 3.5
5	Select the binder or range of binders that may be suitable.	Table 2.4
6	Select a binder content for initial assessment and adjust in accordance with laboratory testing taking due consideration of practical spread rates possible.	
7	Combine a sample of the untreated material and the select binder content. Measure the standard or modified Proctor MDD and OMC.	
8	Mix a sample of untreated material with the select binder content and at a moisture content determined from step 7. Compact UCS test specimens to a density determined from step 7 in accordance with road agency requirements.	
9	Demould the specimens and moist cure the test specimens for 28 days without soaking in water.	
10	Measure the 28-day UCS (Section 8.4.4) and check that it exceeds the minimum allowable value. If a higher UCS is required, increase the binder content in increments of 0.5% and repeat steps 6 to 10. If the UCS is unnecessarily high, either reduce the binder content and repeat steps 6 to 10 or select another binder and repeat steps 5 to 10.	Table 2.1
11	The following tests may also be undertaken at the design binder content (step 10): <ul style="list-style-type: none"> • working time • shrinkage • free swell • flexural strength and flexural modulus • laboratory beam fatigue • erosion. 	

3.10 Treatment of Earthwork Materials

Silt and clay soils, particularly when they are wet, can be associated with construction problems. These soils can be soft, sticky, plastic, and difficult to compact to the extent that they are unsuitable for subgrade preparation. In such cases, soil stabilisation may provide a means of expediting construction. As seen from Table 2.4, cement can be used to treat silts and low plasticity ($PI \leq 10$) clays.

Commonly, the use of cement stabilisation on these subgrades improves stability by reducing excess moisture and plasticity and hence improving the ability to work and compact the subgrade. That is, it is often a short-term treatment aimed at expediting construction rather than necessarily resulting in long-term changes that can be allowed for in the thickness design of the overlying pavement.

In such cases the amount of cement used is typically selected based on engineering judgement and experience in relation to the compactability of the treated soil. However, PI and laboratory CBR testing may provide useful insight in the workability and strength of the subgrade after treatment. Reducing the PI to a value of 12 to 15 has been suggested as a target (Portland Cement Association (PCA) 2008) .

The cement application rates for such cement-modified soils usually vary from 2–5%, with the application rate being higher for finer-grained soils (e.g. types ML, CL) than coarser-grained sands.

The use of cement in cohesive soils can result in shrinkage cracks occurring on the stabilised layer. The use of cement-stabilised soil to create low-permeability layers is therefore discouraged.

4. Lime Stabilisation

4.1 Introduction

Lime stabilisation of subgrade soils and granular bases and subbases is usually undertaken to:

- increase bearing capacity
- reduce plasticity and seasonal swell and shrinkage
- reduce moisture sensitivity
- improve compactability
- reduce in situ moisture content (using quicklime) to provide trafficability for construction.

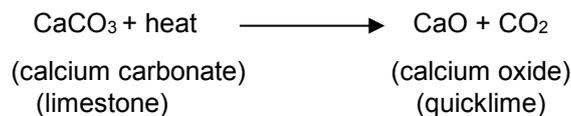
Table 2.4 shows the range of materials that can be treated with lime. The advantages of using lime instead of a cementitious binder increases with increasing plasticity and fines content of the soil. Soils with more than 25% passing the 75 µm sieve and PI > 10 are the most likely candidates. Generally, soils with a PI ≤ 10 will respond better to cementitious binders (Table 2.4). However, comparative testing is advisable. Soils with organics content above 1–2% by weight may not be suitable.

For materials stabilised with lime, the rate of strength gain (tensile strength or UCS) is considerably less than materials stabilised with cement or cementitious binders. Stabilised materials gain strength over time provided curing is sustained. If high strengths are required, then careful control needs to be exercised over field procedures, particularly moisture control, early compaction and effective curing. Shrinkage cracking of lime-stabilised materials is not usually a major problem.

If the aim of the stabilisation treatment is to modify the material to reduce its plasticity, without achieving high strengths, then lower binder contents are sufficient. As a consequence, the stabilised material can be reworked one or two days after initial compaction.

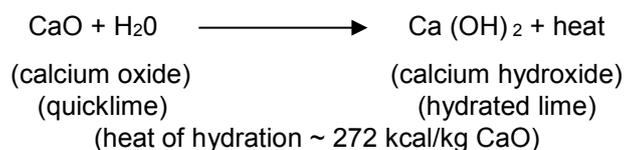
4.2 Lime Manufacture

The manufacture of lime involves the heating of excavated limestone in a kiln to temperatures above 900 °C, resulting in carbon dioxide being driven off and calcium oxide (quicklime) being produced. The chemical equation is as follows:



Limestone feedstocks used for calcination are not pure calcium carbonate and the kilning processes have inherent inefficiencies. This means that commercial quicklime will never be 100% calcium oxide and typically yields an 80–95% Available Lime Index (Section 4.4).

At temperatures below 350 °C, the calcium oxide component of quicklime reacts with water to produce hydrated lime (calcium hydroxide) as well as liberating heat. Approximately 320 litres of water are required to hydrate 1 tonne of calcium oxide. The chemical equation is as follows:



For lime stabilisation, the quantity of calcium hydroxide is the active component that reacts with the host material. In the laboratory, hydrated lime is used to determine the amount of lime to achieve the desired material attributes. However, in construction, quicklime (calcium oxide) is often used which, if added at the laboratory-determined application rate, will result in an increased amount of calcium hydroxide being available. Table 4.1 indicates that quicklime has approximately 30% more effective lime for stabilisation than hydrated lime (i.e. equivalent Ca(OH)_2 /unit mass when slaked). This needs to be considered in the mix design of stabilised materials.

Table 4.1: Properties of quicklime and hydrated lime (assuming pure lime)

Property	Hydrated lime	Quicklime	Slurry lime
Chemical composition	Ca(OH)_2	CaO^*	Ca(OH)_2
Form	Fine powder	Granular	Slurry
Equivalent Ca(OH)_2 per unit mass (available lime)	1.00	1.32	0.56 to 0.33**
Bulk density (t/m^3)	0.45 to 0.56	1.05	1.25

* $\text{CaO} + \text{H}_2\text{O} \rightarrow \text{Ca(OH)}_2 + \text{heat}$.

** Moisture contents of slurries could vary from 80% to over 200%.

Note that lime can react with moisture and carbon dioxide. Careful storage is required to maintain the integrity of the lime to produce reliable results.

Further details are given in *Guide to Pavement Technology Part 4L: Stabilising Binders* (Austroads 2009c).

4.3 Types of Lime

Lime comes in a number of forms:

- hydrated (or slaked) lime (calcium hydroxide)
- quicklime (calcium oxide)
- dolomite lime (calcium/magnesium oxide)
- agricultural lime (calcium carbonate)
- limestone (calcium carbonate).

Agricultural lime is not suitable for stabilisation and dolomitic lime is not usually as effective as hydrated lime or quicklime.

All commercial lime products have impurities such as carbonates, silica, alumina, etc. which dilute the active additive but are not harmful to the stabilisation reaction.

Hydrated lime comes as a dry, very fine, powder or as a slurry. The water contents of common lime slurries can range from 80% to over 200%.

Quicklime and dolomitic limes are commonly much more granular than the hydrated products and are available only as a dry product. These forms of lime react rapidly with available water, producing hydrated lime and releasing considerable amounts of heat.

Quicklime, and to a lesser extent hydrated lime, is particularly suitable for treating wet, plastic clay subgrades, providing effective working platforms from otherwise untraffickable situations. The process is based on the exothermic reaction of the lime as it hydrates, reducing the moisture content of the soil.

Table 4.1 summarises the properties of lime (assuming pure lime). Lime contents are expressed as the equivalent of 100% pure hydrated lime.

While the type of lime does not appear to be significant in terms of determining the long-term structural properties of the stabilised materials, it has considerable influence on the construction processes. In selecting the type of lime for a project, the following should be considered:

- **Cost** – the cost of quicklime per tonne is significantly less than hydrated lime. Approximately 30% less quicklime (by mass) compared to hydrated lime is used to achieve the same stabilisation outcome, therefore reducing the cost of the treatment.
- **Dust nuisance** – hydrated lime can cause a dust problem, even with very light winds. Its use in urban or windy areas should be minimised. Generally, dust is not a serious problem with quicklime, except for some very fine batches. Lime slurry is dust free.
- **Steam nuisance** – water is added to quicklime so that slaking occurs. The exothermic reaction generates some steam for a short period of time. There may be a possibility, particularly in colder climates, that traffic control is required if steam affects visibility.
- **Soil moisture** – quicklime and hydrated lime are effective in drying out wet soils, but slurry limes cannot be used for this purpose. Slurry limes are very suited to dry soil conditions where water may be required to achieve effective compaction. Quicklime dries out materials, as it uses the soil moisture as part of the reaction. The exothermic reaction can also cause evaporation of excess water.
- **Lime content** – quicklime is particularly effective if the content of the additive is to be kept low. In terms of available $\text{Ca}(\text{OH})_2$ per unit mass, it is equivalent to 1.3 units (2.5 units per unit volume). The total amount of lime slurry will usually be limited by the soil moisture content; generally lime slurry is limited to low additive contents (< 3 %) and dry construction conditions.
- **Water availability** – Less water is required using hydrated lime. If water availability is limited, hydrated lime may be preferred.
- **Lime storage** – site silos hold more (approx. double) quicklime by mass than hydrated lime. Similarly, a delivery tanker can carry more (by mass) quicklime, unless it is a high capacity vessel.
- **Lime availability** – quicklime is more readily available in large quantities than hydrated lime. There is often less production and storage capacity for hydrated lime.
- **Available equipment and expertise** – these factors are always important. Automated spreaders, adequate mixing and compaction equipment are essential to achieve good results.

While quicklime can be immediately mixed into wet subgrades without problems, the lime should be allowed to hydrate before it is mixed into the subbase and base materials as unhydrated particles of quicklime may cause expansion with possible heaving of compacted materials.

4.4 Available Lime Index

As discussed in Section 4.2, calcium hydroxide is the active component that reacts with the host material. A complicating factor in the design and specification of lime stabilisation treatments is that the proportion of pure calcium oxide or calcium hydroxide in quicklime or hydrated lime respectively varies depending on the source of the limestone.

To allow for this variation in the proportion of active lime components, use is made of the Available Lime Index (ALI) which is the content of pure calcium oxide or calcium hydroxide in quicklime or hydrated lime respectively. It is expressed as a percentage of the total mass of lime determined in accordance with AS 4489.6.1-1997.

In specifying the lime content to use in construction, allowance needs to be made for the difference in the ALI of the lime used in the laboratory mix design testing and that of the lime used in construction. One means of addressing this variation is to specify a lime content applicable to an ALI of 100% and to adjust for the actual ALI used at the time of construction. In addition, the specified lime content needs to allow for construction tolerance: an additional 0.5% may be added to the value obtained from the laboratory testing.

4.5 Lime Stabilisation of Pavement Materials

Lime is effective in modifying excessive plastic properties of subbase and base materials (Table 2.4). Such modification of plastic natural gravels is a widely accepted and successful practice. At lime contents less than about 3%, the risk of undesirable shrinkage cracking due to the lime is low and it would rarely be necessary to take special measures to combat reflective cracking.

If an increase in modulus and strength is required, a pozzolan may be used with the lime to produce a cementitiously-modified material as described in Section 3.4.

Lime without the addition of pozzolans is not effective in cohesionless or low cohesion materials.

The lime-pavement material reactions are slowed by lower temperatures (below 15 °C) and are impeded by high organic contents.

4.6 Properties of Earthworks Materials Stabilised with Lime

4.6.1 General

For clayey soils, the effect of lime on the volume and moisture stability, strength and elastic behaviour, are similar to the effects of cement. The following sections are concerned only with those aspects of materials stabilised with lime that are significantly different from materials stabilised with GP cement.

Long-term strengthening (pozzolanic reactions) occurs in a highly-alkaline environment (pH > 12.3). It promotes the dissolution of the clay, particularly at the edges of the clay plates and permits the formation of calcium silicates and aluminates in the material. These cementitious products are similar in composition to those of cement paste.

The addition of lime to clay results in the following:

- It has an immediate effect on clay, improving its grading and handling properties by promoting flocculation of the clay particles. The effect varies with the actual clay minerals present being large with montmorillonite group clays and low with kaolinite clay groups. That is, the effect is dependent on the degree of pozzolanic material in the soil.
- It has longer-term effects on strength, resulting in continuing strength gains with time. This process is relatively slow because the available lime has to diffuse through both the matrix of the material and the cementitious materials produced during the treatment.

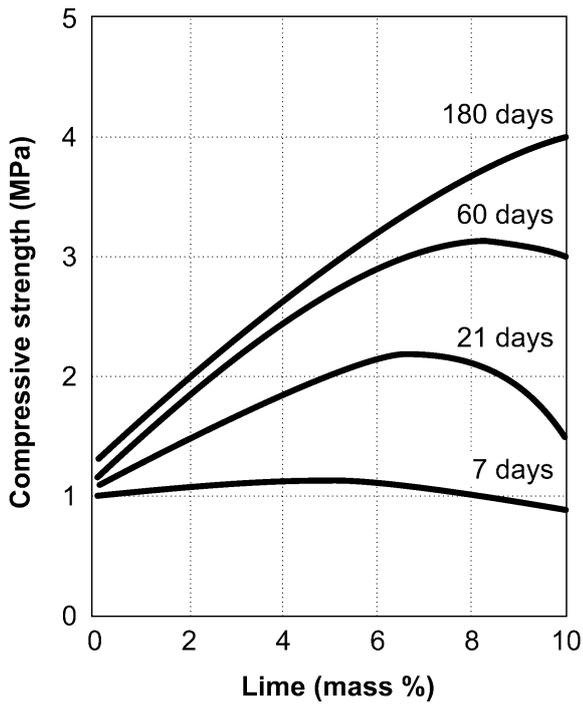
4.6.2 Rate of Strength Gain

For materials stabilised with lime, the rate of strength gain (tensile strength or UCS) is considerably less than with cementitiously-bound materials. Materials stabilised with lime and supplementary cementitious materials will continue to gain strength over time provided curing is sustained.

The rate of strength gain is temperature sensitive and also depends on the lime content. Caution should therefore be exercised in accepting results of high-temperature accelerated testing without validation at field temperatures. High temperatures can cause other types of bonds to form that would not normally occur in the field. Accelerated curing temperatures should not exceed 40 °C.

In laboratory UCS testing, lime-stabilised materials are usually evaluated after 7 and 28 days of curing. Figure 4.1 illustrates an example of the variations in strength with time and lime content. Note that high lime contents will not necessarily produce high early strengths. However, the ability of lime to dry out wet materials and reduce plasticity to make otherwise unsuitable materials useful, renders early strength gains a secondary consideration in some instances.

Figure 4.1: Variation in compressive strength as a function of lime content and time

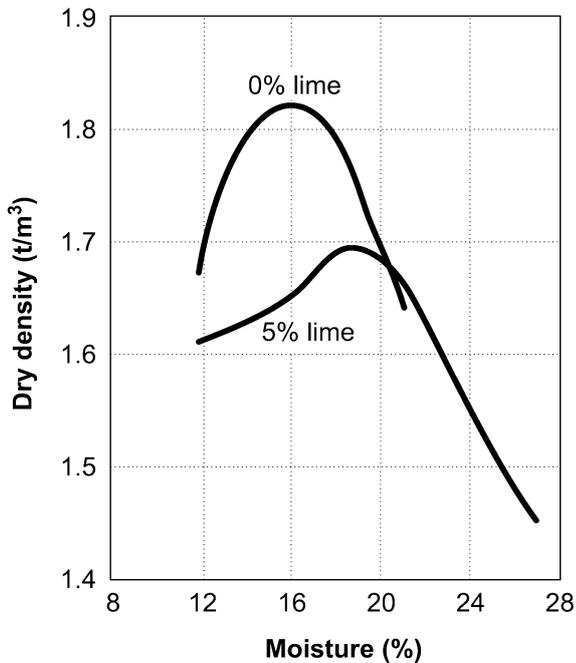


Source: Kezdi (1979).

4.6.3 Moisture-density Relationship

Increasing lime contents increases the OMC and reduces the MDD of the material being stabilised (Figure 4.2). This effect is further increased by delaying compaction once the lime is added.

Figure 4.2: Effect of lime content on maximum dry density and optimum moisture content



Source: Terrel et al. (1979).

4.7 Appropriate Conditions for Lime Stabilisation of Earthworks Materials

4.7.1 General

The following factors need to be considered in assessing the suitability of lime stabilisation for a project:

- material factors, dealing with the composition of the material to be stabilised and its response to lime
- production and construction factors.

4.7.2 Material Factors

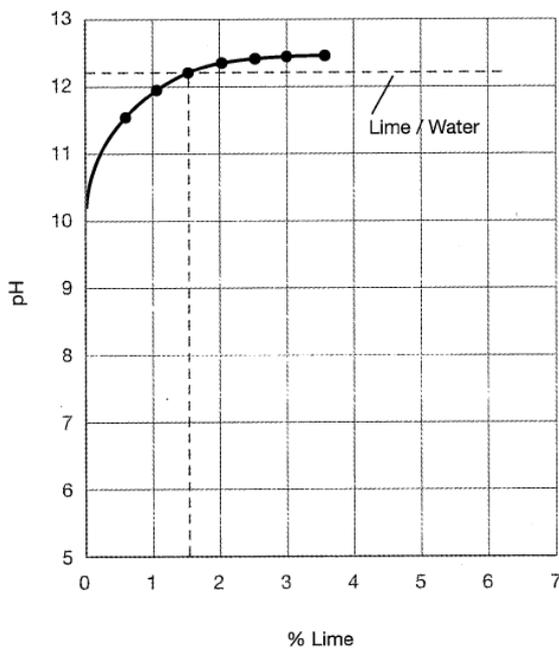
For lime to be effective, the material to be stabilised must contain clay particles or pozzolanic materials that are reactive with lime. In general, the more plastic the clay fines and the higher the clay content, the larger the lime content required to produce a specific strength gain or other effect. However, the amount of bonding achievable with lime is limited by the amount of reactive material. Generally, soils with a PI less than 10 will respond better to cementitious binders. However, comparative testing is advisable.

Sugars and reactive organic materials can retard the development of cementitious bonds with both cementitious binders and lime.

For PI reduction and workability improvement using lime modification, sufficient lime should be added so that additional quantities of lime do not result in further changes in PI.

Where lime stabilisation is used for long-term strength gain a Lime Demand test is required (Sherwood 1993). By testing the pH at different percentages of lime, a plot of lime content versus pH is obtained. The lime percentage where the pH of the soil-lime-water is the same as a lime-water is the required lime content for long-term strength gain (Figure 4.3). An additional 0.5% lime is usually used in construction to allow for losses and variations in host material properties, lime quality and mixing.

Figure 4.3: Example of Lime Demand test results



Source: AustStab (2015).

Apart from the Lime Demand test, it is important to verify the lime reactions increase strength/modulus. This is discussed further in Section 4.8.

4.7.3 Production and Construction Factors

The following factors significantly affect the quality of lime stabilised materials:

- quality of water
- quality of lime
- uniformity of mixing and curing
- compaction.

The addition of lime normally promotes granulation of the material being stabilised. For materials that are difficult to break down, the lime-material mix is sometimes moist cured – for time periods ranging from a few hours to a day – after light rolling to reduce contact with air, and then remixed. Lime addition may be added in two stages in a process called ‘mellowing’ with the second addition being undertaken between 4 and 72 hours after the first addition.

Lime will diffuse slowly throughout clays and stabilise the lumps. Unless high early strength is particularly important, it is unnecessary to seek fine granulation. It is suggested that 80 to 90% of the soil's clods should pass the 26.5 mm sieve. If temperatures are low at the time (< 15 °C), then more attention should be given to breakdown.

The use of quicklime to establish a working platform on a wet clay is a useful construction expedient and utilises the exothermic reaction of the lime as it hydrates to reduce the moisture content of the soil.

The initial rate of reaction with lime gives time to achieve adequate compaction. If high strengths are being sought, early compaction is necessary to achieve as high a density as possible. Delayed compaction lowers the density but the rate of reduction of maximum density is not as severe as with cementitious binders.

4.8 Selecting the Lime Content for Long-term Property Changes

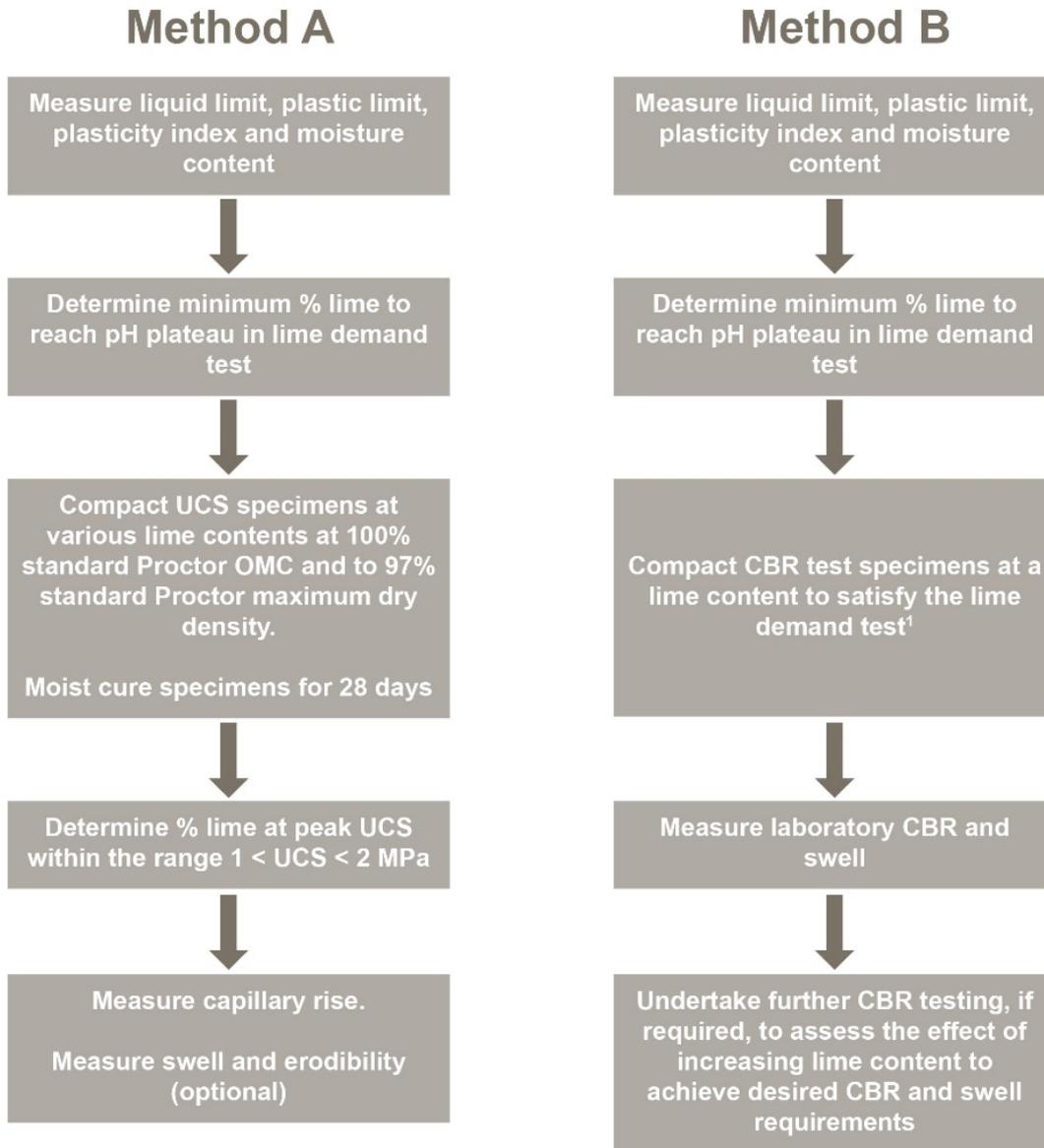
For lime to be effective, the material being treated must contain clay particles or pozzolanic materials that react with lime. Accordingly, an important first step in selecting the lime content is to assess whether the pavement or earthworks material is reactive to lime. This is undertaken using the Lime Demand test which estimates the lime content required for the pH of the material to be stabilised to plateau, as described in Section 4.7.2. This will ensure that the stabilised design properties are achieved in the long term (Little 1996; Sherwood 1993). The presence of organic materials in acid soils will increase the amount of lime required for stabilisation.

Where the objective of the lime stabilisation is a long-term strength gain, strength testing is undertaken after determination of the lime content to satisfy the Lime Demand test. The required lime content can be determined in two ways as indicated in Figure 4.4:

- In Method A the lime content is determined such that it satisfies the Lime Demand test; it is also based on the content to achieve the peak UCS within the range $1 < \text{UCS} \leq 2$ after 28 days curing (Figure 4.5). When no peak is observed and the UCS exceeds 1 MPa, the lime content selected is that which satisfies the Lime Demand test and has the slowest capillary rise. This method is described in TMR Technical Note 151 *Testing of Materials for Lime Stabilisation* (2018d).
- In Method B the minimum lime content is determined such that it satisfies the Lime Demand test. An additional 0.5% to 1% lime is usually specified for construction to allow for losses and variations in host material properties, lime quality and mixing. The laboratory soaked CBR and swell are measured on the lime-treated subgrade for use in the pavement structural design. Code of Practice RC 500.23 *Lime Stabilised Earthworks Materials – Available Lime, Assigned CBR and Swell* (VicRoads 2016) is an example of the use of this method. Consideration should also be given to measuring the CBR and swell of the untreated material to provide an indication of the effectiveness of the lime treatment. As specific requirements for material pre-treatment, preparation, compaction and CBR and swell assessment varies across road agencies, Figure 4.4 Method B provides a general outline of the determination of lime content of earthworks material using the CBR approach.

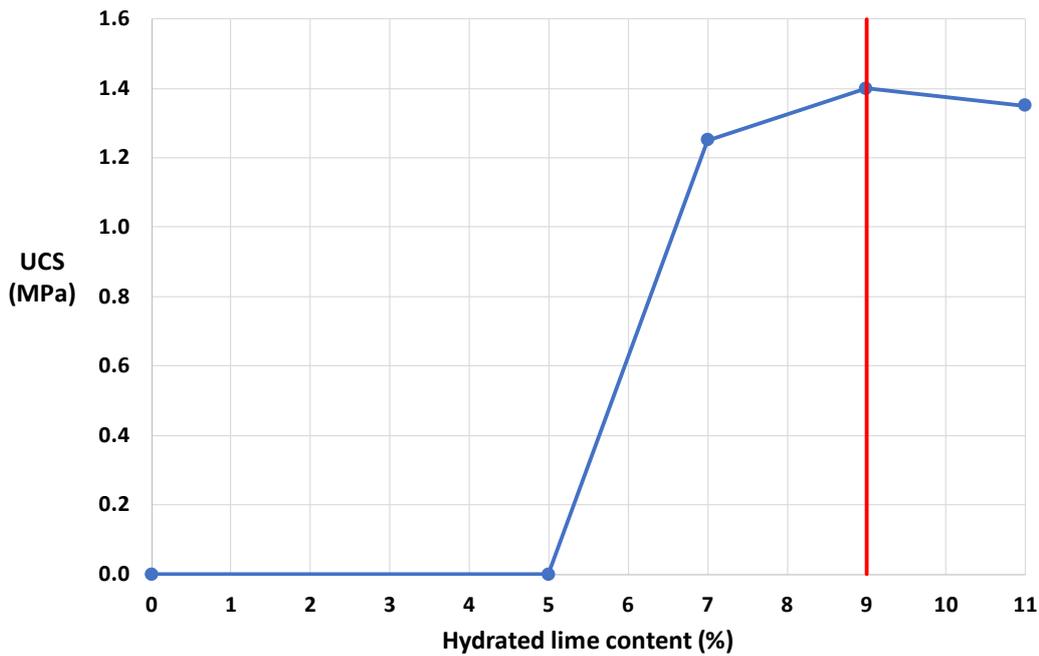
When assessing the CBR of lime stabilised material, consideration should also be given to measuring the CBR and swell of the untreated material to provide an indication of the effectiveness of the lime treatment. Note that it will be usually necessary to increase the lime content above that required to satisfy the Lime Demand test in order to comply with specified minimum CBR or maximum swell values.

Figure 4.4: Determination of lime content of earthworks materials



1. CBR test procedures vary between road agencies in relation to moisture content and density of CBR specimens and soaking prior to testing.

Figure 4.5: Example of lime content determined using Method A



Source: TMR (2012).

4.9 Selecting the Lime Content to Expedite Construction

Clay soils, particularly when they are wet, can be associated with construction problems. These soils can be soft, sticky, plastic, and difficult to compact to the extent that they are unsuitable for subgrade preparation. In such cases, soil stabilisation may provide a means of expediting construction. As seen from Table 2.4, lime can be used to treat soils with moderate to high plasticity ($PI > 10$).

For PI reduction and workability improvement using lime modification without the aim of providing long-term property changes, the lime content may be selected such that any additional amount does not result in a further decrease in PI.

For difficult-to-compact and overly-wet subgrade materials, consideration can be given to using a lime content that reduces the moisture content closest to the OMC of the lime-treated subgrade. A combined treatment of lime and granular stabilisation may also be suitable.

5. Bituminous Stabilisation

5.1 Introduction

Bitumen-stabilised granular materials are produced using either foamed bitumen or bitumen emulsion either plant-mixed or in situ stabilised. The materials treated are normally granular pavement materials, previously cement-treated materials or reclaimed asphalt pavement (RAP). Where an existing pavement is recycled, old seals or asphalt surfacings are usually mixed with the underlying layer and treated to form a new base or subbase layer.

Bituminous stabilisation of granular materials is usually intended either to introduce some cohesion into non-plastic materials or to make a cohesive material less sensitive to loss of stability with increased moisture. Generally, materials that are suitable for in situ stabilisation are (TMR 2018d):

- well-graded PSD (Section 5.3)
- good particle shape, not rounded
- a low plasticity $PI \leq 10$ or linear shrinkage ≤ 6
- may include up to 20% RAP
- previously stabilised materials.

Foamed bitumen stabilisation commonly includes supplementary cementitious binders (e.g. lime, cement).

5.2 Bitumen

5.2.1 General

Bituminous stabilisation may be carried out using any of the following materials:

- foamed bitumen
- bitumen emulsion, either as cationic or anionic emulsion.

In association with bitumen stabilisation, secondary binders, which are generally lime or cement, are usually added to increase the modulus and rut-resistance of the mix.

Approximate equivalents between Australian and New Zealand bitumen grades are shown in Table 5.1.

Table 5.1: Australian and New Zealand bitumen classifications

Australian classification	Minimum penetration	New Zealand classification
C170	62	80/100
C240	53	–
C320	40	60/70

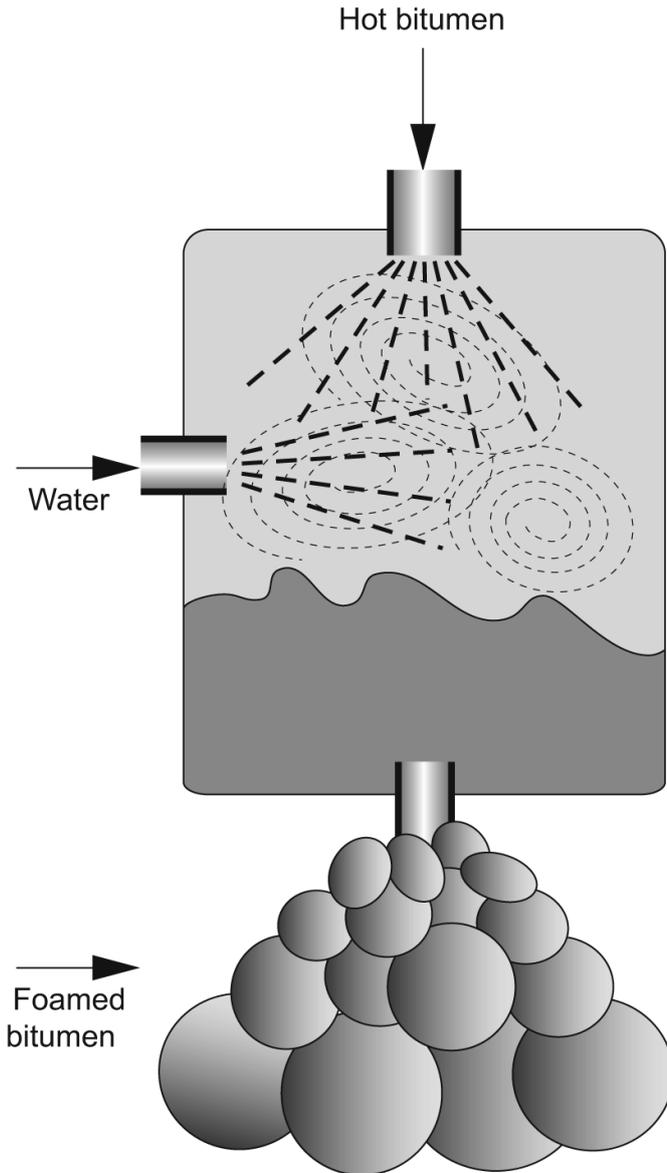
5.2.2 Foamed Bitumen

Foamed bitumen is a mixture of air, water and hot bitumen. Injecting a small quantity of cold water into hot bitumen produces an instantaneous expansion of the bitumen – up to 15 times its original volume forming foam. The concept of manufacturing foamed bitumen is illustrated in Figure 5.1.

When the bitumen is in a foamed state it is ideal for mixing with fine materials because its large surface area bonds to fine particles. As the foam collapses very quickly, rapid mixing is required to adequately disperse the bitumen throughout the material.

For most bitumens, a foaming agent needs to be added to the bitumen to provide the appropriate foaming characteristics. C170 bitumen is commonly used.

Figure 5.1: Manufacture of foamed bitumen



5.2.3 Bitumen Emulsion

Bitumen emulsions are dispersions of fine droplets of bitumen in water. Standard grades comprise approximately 60% bitumen and 40% water with a small proportion of emulsifier. Some types of emulsion include additives such as polymer or cutter and different proportions of bitumen. The setting and curing of emulsions involve the separation and removal of water ('breaking') leaving solid bitumen. The type and quantity of emulsifier determines the setting characteristics of the emulsion.

Bitumen emulsions are manufactured to comply with AS 1160-1996, which allows for the following two classes depending on the charge of the suspended particles:

- anionic bitumen emulsion, where the particles of bitumen are negatively charged
- cationic bitumen emulsion, where the particles of bitumen are positively charged.

Both classes of bitumen emulsion are prepared in two grades: rapid setting (RS) and slow setting (SS). Only the slow-setting grades are suitable for use in stabilisation.

Whilst most manufacturers produce all classes and grades, emulsions of the same class but produced by different manufacturers can react differently with the same soil. It is therefore important that the same emulsion to be used in the field is also used in any laboratory testing program. More than one product should be tested to allow a fair assessment to be made of the types of bitumen proposed.

Bitumen emulsion is normally manufactured with C170 bitumen, which is usually satisfactory for materials with lower fines content (0–10% passing the 75 µm sieve). For materials with higher fines content (15–25%) passing the 75 µm sieve, it may be necessary to use softer bitumen if difficulty in distribution is encountered. Under extreme conditions, emulsion containing oil may have to be used.

Bitumen emulsions may be readily mixed with damp host materials to produce a good dispersion of bitumen throughout the host material.

5.3 Materials Suitable for Bitumen Stabilisation

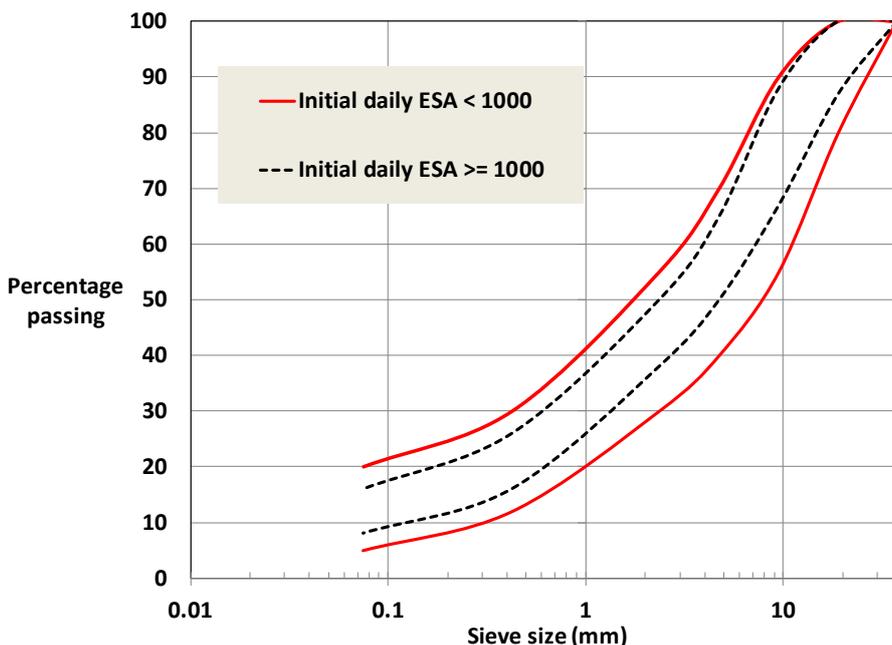
The application of bituminous binder (as a foam or emulsion) in stabilisation is highly dependent on the PSD of the host material. Recommended distribution limits are shown in Table 5.2 and Figure 5.2.

In the event that PSD of the host material needs to be adjusted, Section 6 describes the process of blending materials to achieve a target PSD.

Table 5.2: PSD envelopes for bituminous binders

% Passing sieve (mm)	Initial daily ESA < 1000	Initial daily ESA ≥ 1000
37.5	100	100
19.5	80–100	87–100
9.5	55–90	67–88
4.75	40–70	50–65
2.36	30–55	38–50
0.425	12–30	16–26
0.075	5–20	8–16

Figure 5.2: Suitable PSD envelopes for bituminous binders



5.4 Foamed Bitumen Stabilisation Mix Design

5.4.1 Introduction

The foamed bitumen mix design process includes consideration of the required properties of the material to be stabilised, requirement in the foaming characteristics of the bitumen proposed to be used and the bitumen and secondary binder contents required to meet minimum indirect tensile modulus requirements.

Table 5.3 lists the Austroads mix design test methods.

Table 5.3: Austroads test methods for FBS mix design

Number	Title
AGPT/T301	<i>Determining the Foaming Characteristics of Bitumen</i>
AGPT/T302	<i>Mixing of Foamed Bitumen Stabilised Materials</i>
AGPT/T303	<i>Compaction of Test Cylinders of Foamed Bitumen Stabilised Mixtures: Part 1 Dynamic Compaction using Marshall Drop Hammer</i>
AGPT/T305	<i>Resilient Modulus of Foamed Bitumen Stabilised Mixtures</i>

5.4.2 Bitumen Foaming Characteristics

Prior to the mix design being conducted it may be necessary to undertake a binder design to determine if the foaming characteristics of the bitumen are pertinent to the particular site conditions. When selecting a bitumen for foaming, the expansion ratio should not be less than ten times and the half-life at least 20 seconds.

This testing can only be undertaken using specially-designed laboratory apparatus (e.g. Figure 5.3) and experienced technicians. The Austroads test method is AGPT/T301 *Determining the Foaming Characteristics of Bitumen*.

Binder design is undertaken to determine the half-life of the foam, i.e. the time for the foam to settle to half its initial height. Generally, a foaming agent is added to the water to enhance the expansion ratio, i.e. the increase in volume due to foaming, with a typical value being between 12 and 15 times.

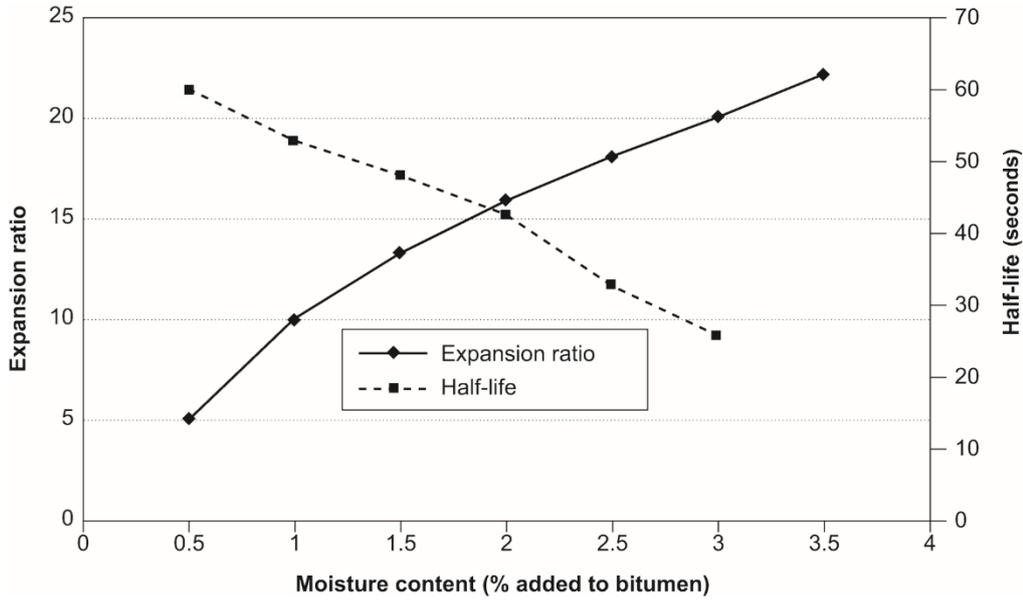
Figure 5.3: Laboratory bitumen foaming apparatus



Source: Wirtgen Group brochure WLB10S- WLM30, <https://media.wirtgen-group.com>.

The influence of water content on the foaming characteristics of bitumen affects both the expansion ratio (increase in volume) and the half-life (time for half the expanded volume to collapse) as shown in Figure 5.4.

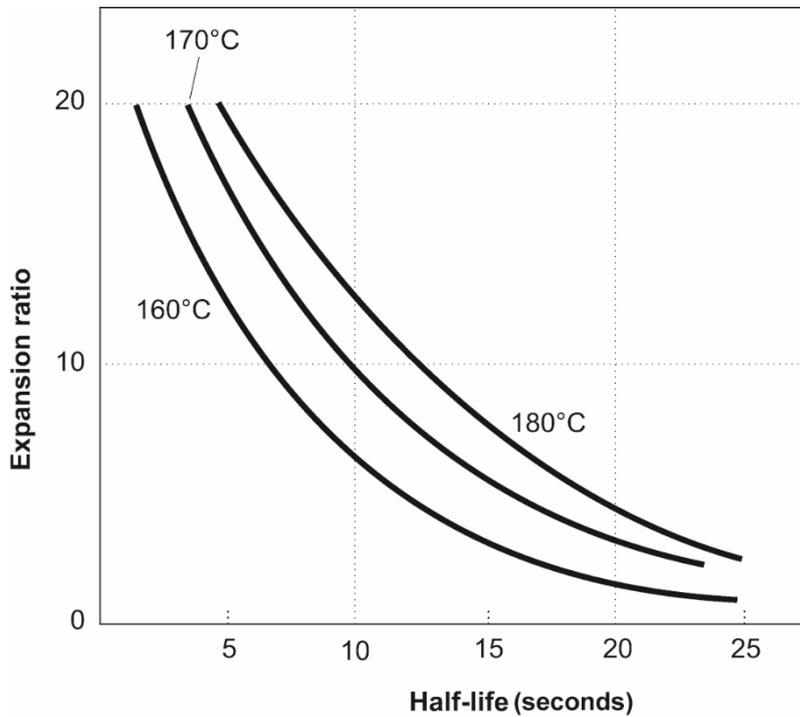
Figure 5.4: Effect of water on foaming characteristics



Source: Kendall et al. (2001).

In addition, as the bitumen temperature decreases slightly below 180 °C the half-life duration reduces, and this can have an effect on the foaming process (Figure 5.5).

Figure 5.5: Example of the effect of temperature on foam expansion ratio and half-life



5.4.3 Secondary Binders

A secondary binder, generally either lime or cement, is used to:

- stiffen the bitumen binder
- act as an anti-stripping agent (lime only)
- assist in dispersion of the bitumen throughout the material
- improve the initial modulus, and early life rut-resistance, of the stabilised pavement
- reduce moisture sensitivity of the stabilised material.

5.4.4 Mixing Moisture Content

The moisture content of the loose mixture of host granular material, foamed bitumen and lime/cement affects the distribution of the binders, the effect of the secondary cementitious binders and, importantly, the density to which the test cylinders can be compacted. Accordingly, the mixing moisture content can have a very significant effect on the mix moduli. Figure 5.6 shows a pugmill commonly used to mix the constituents in accordance with Austroads test method AGPT/T302.

It is common practice in Australia to derive the mixing moisture content by testing the laboratory compaction characteristics of the untreated material. The mixing moisture content is commonly either:

- 70% standard Proctor OMC of the untreated material, or
- 80–85% modified Proctor OMC of the untreated material.

Figure 5.6: Laboratory mixing using a pugmill



Source: Wirtgen Group brochure WLB10S- WLM30, <https://media.wirtgen-group.com>

5.4.5 Mix Design

The mix design method for foamed bitumen stabilised mixtures is shown in Table 5.4 and Table 5.5. In Australia, it is common for projects to use bitumen contents of 2.5% to 3.5% and hydrated lime contents of 1–2% for host materials with up to 20% RAP.

Table 5.3 lists the mix design test methods. Note that the indirect tensile modulus specimens are compacted into a 152 mm internal diameter Marshall mould using a 10.2 kg Marshall drop hammer (Figure 5.7).

Table 5.4: Selecting mix components

Step	Activity	Reference
1	Obtain a representative sample of the untreated material to be stabilised, including any recycled surfacing materials.	
2	Measure the PSD and assess whether it is necessary to improve the grading by adding other granular materials.	Table 5.2
3	Measure LL, PL and PI and assess whether the material needs to be pre-treated.	Table 2.4
4	Obtain a representative sample of C170 bitumen and determine the expansion ratio and half-life of samples with 2, 3 and 4% moisture content.	AGPT/T301
5	Compare measured expansion ratio and half-life against limits. If the limits cannot be achieved at any moisture, assess whether a foaming agent or another bitumen is needed and repeat step 4. Otherwise, select the moisture content for foaming and proceed to step 6.	
6	Select whether the secondary binder is lime or cement, obtain a representative sample, and check for compliance.	

Table 5.5: Determination of design bitumen content

Step	Activity	References
7	From the specification for construction, determine whether standard or modified Proctor MDD will be used in assessing field compaction and determine the OMC of the untreated material. Based on this testing, select a range of moisture contents for use in testing (e.g. 70% standard Proctor or 80–85% modified Proctor OMC).	
8	Select the trial bitumen content and a trial secondary binder content.	
9	Add water to the untreated granular material to bring it to the selected moisture content. It may be necessary to adjust the moisture content to allow for hydration of the secondary binder.	AGPT/T302
10	Mix the granular materials and secondary binders at a mixing moisture content and leave the mixture to cure for 45 minutes.	AGPT/T302
11	Thoroughly mix the foamed bitumen into the mixture.	AGPT/T302
12	Compact at least three test cylinders by placing the loose mixture into 152 mm diameter moulds and compact by applying 50 blows per face with a 10.2 kg Marshall hammer.	AGPT/T303
13	Cure the specimens for three hours at 25 °C and measure the initial indirect tensile modulus – M_i .	AGPT/T305
14	Dry the specimens for three days at 40 °C and measure the dry indirect tensile modulus – M_d .	AGPT/T305
15	Soak the specimens in a vacuum chamber for 10 minutes at a partial vacuum and measure the wet indirect tensile modulus – M_w .	AGPT/T305
16	Measure the specimen dry densities.	
17	Calculate the ratio of wet to dry indirect tensile modulus.	
18	Compare the measured results of steps 13–17 with the specified values (e.g. Table 5.6, Table 5.7 and Table 5.8). If the results are below the requirements, increase the bitumen content and/or secondary binder and repeat steps 8–17. If the results are above the requirements, decrease the bitumen content and repeat steps 8–17.	
19	Select the design bitumen and secondary binder contents.	

Figure 5.7: Large-scale Marshall mould and drop hammer



Source: Downer personal communication via email (1 March 2015).

In relation to modulus requirements, Table 5.6, Table 5.7 and Table 5.8 show examples of modulus requirements for in situ-stabilised materials (TMR 2018d). Table 5.9 shows an example of additional modulus requirements for plant-mixed FBS mixes during production.

Table 5.6: Example of initial modulus limits for in situ stabilised foamed bitumen materials

Average daily traffic in design year of opening (ESA)	Minimum initial modulus ^(1,2) (MPa)
< 100	500
≥ 100	700

1. Foamed bitumen mixes mixed and compacted at about 70% standard Proctor OMC of the untreated material.
2. Initial sample curing time of three hours at 25° C required prior to initial resilient modulus testing being completed.

Source: TMR (2018d, 2018e).

Table 5.7: Example of cured and soaked modulus limits for in situ stabilised foamed bitumen stabilised base

Average daily traffic in design year of opening (ESA)	Minimum three day cured modulus ⁽¹⁾ (MPa)	Minimum three day soaked modulus ⁽¹⁾ (MPa)	Minimum retained modulus ratio ⁽²⁾
< 100	2500	1500	0.40
100–1000	3000	1800	0.45
> 1000	4000	2000	0.50

1. Foamed bitumen mixes mixed and compacted at about 70% standard Proctor OMC of the untreated material.
2. Retained modulus ratio = three-day soaked modulus divided by the three-day cured modulus.

Note: Base is defined as a stabilised pavement layer with a bituminous seal surfacing or less than 100 mm of asphalt overlying it.

Source: TMR (2018d, 2018e).

Table 5.8: Example of cured and soaked modulus limits for in situ stabilised foamed bitumen stabilised subbase

Average daily traffic in design year of opening (ESA)	Minimum three day cured modulus ⁽¹⁾ (MPa)	Minimum three-day soaked modulus ⁽¹⁾ (MPa)	Minimum retained modulus ratio ⁽²⁾
< 100	2500	1500	0.40
100–1000	2500	1500	0.45
> 1000	2500	1500	0.50

1. Foamed bitumen mixes mixed and compacted at about 70% standard Proctor OMC of the untreated material.

2. Retained modulus ratio = three-day soaked modulus divided by the three-day cured modulus.

Note: Subbase is defined as a stabilised pavement layer with at least 100 mm of asphalt overlying it.

Source: TMR (2018d, 2018e).

Table 5.9: Example of modulus limits for plant-mixed foamed bitumen stabilised mixes during production

Average daily traffic in design year of opening (ESA)	Minimum three-day cured modulus ⁽¹⁾ (MPa)	Minimum seven-day soaked modulus ⁽¹⁾ (MPa)	Minimum 14-day soaked modulus ⁽¹⁾ (MPa)	Minimum retained modulus ratio ⁽²⁾
All	1000	1400	1800	0.45

1. Foamed bitumen mixes mixed and compacted at about 70% standard Proctor OMC of the untreated material.

2. Retained modulus ratio = three-day soaked modulus divided by the three-day cured modulus.

Source: TMR (2018e).

On large projects additional testing may be undertaken including flexural fatigue and wheel tracking testing.

5.5 Bitumen Emulsion Mix Design

There is currently limited use of bitumen emulsion stabilised materials in Australia. Where they are being used, the empirical mix design process is intended to produce modified pavement materials with enhanced modulus and rut-resistance but without being susceptible to fatigue cracking.

Emulsion application rates of 2–3% (by mass) of residual binder are commonly used. Lower rates of about 0.5–1% may be satisfactory for well-graded materials in dry climates (Roads & Traffic Authority (RTA) NSW 1995). Lower application rates, when added to granular base materials, can be useful as a construction expedient to reduce raveling and potholing under traffic. In any case, the application rate should be determined by laboratory testing.

In some applications cement or lime is added as a secondary binder. For example, bitumen emulsion stabilised-subbase limestone has been used in Western Australia, for many years. A typical application rate would be 2% residual binder, coupled with about 1% cement or lime for some projects.

6. Granular Stabilisation

6.1 Introduction

The improvement of one material by blending it with one or more granular materials is referred to as granular or mechanical stabilisation. This type of stabilisation provides a direct means of altering the PSD. Plasticity changes may also result.

Granular stabilisation may involve the following:

- mixing of materials from various parts of a deposit at the source of supply
- mixing of selected, imported material with in situ materials
- mixing two or more selected, imported natural gravels, soils and/or quarry products on site or in a mixing plant
- mixing recycled materials with existing pavements.

Materials produced by granular stabilisation have properties similar to conventional unbound materials and can be evaluated by conventional methods.

Some typical applications of granular stabilisation are:

- correction of PSD generally associated with gap-graded or high fines-content gravels
- correction of PSD and increasing plasticity of dune or river-deposited sands which are often single sized
- correction of PSD and/or plasticity of crushed products, quarry wastes and environmentally acceptable industrial by-products
- correction of particle shape by adding crushed products to round gravels
- decrease in particle breakdown of soft aggregate through the addition of harder aggregate.

6.2 Materials

Materials requiring granular stabilisation have properties which make them deficient to be used as base or subbase materials. Typically, such materials are:

- poorly-graded products
- poorly-shaped aggregates (e.g. uncrushed river gravels)
- dune or river deposited sands
- scoria, shales
- silty sands, sandy clays, silty clays
- crusher run products
- waste quarry products
- industrial by-products
- high plasticity granular materials.

6.3 Design Criteria for Granular Stabilisation

6.3.1 Introduction

The principal properties affecting the load bearing capacity of base and subbase materials are internal friction and cohesion.

Internal friction is generated primarily because of the characteristics of the coarser soil particles and the PSD.

Cohesion (and shrinkage, swelling and compressibility) results primarily from the quantity and nature of the clay fraction as indicated by the plastic properties, Sand Equivalent and Maximum Dry Compressive Strength (MDCS).

Preliminary mix design is usually based on that required to achieve a target PSD, particle shape and plastic properties. Strength testing is generally undertaken using the CBR test and/or triaxial shear test.

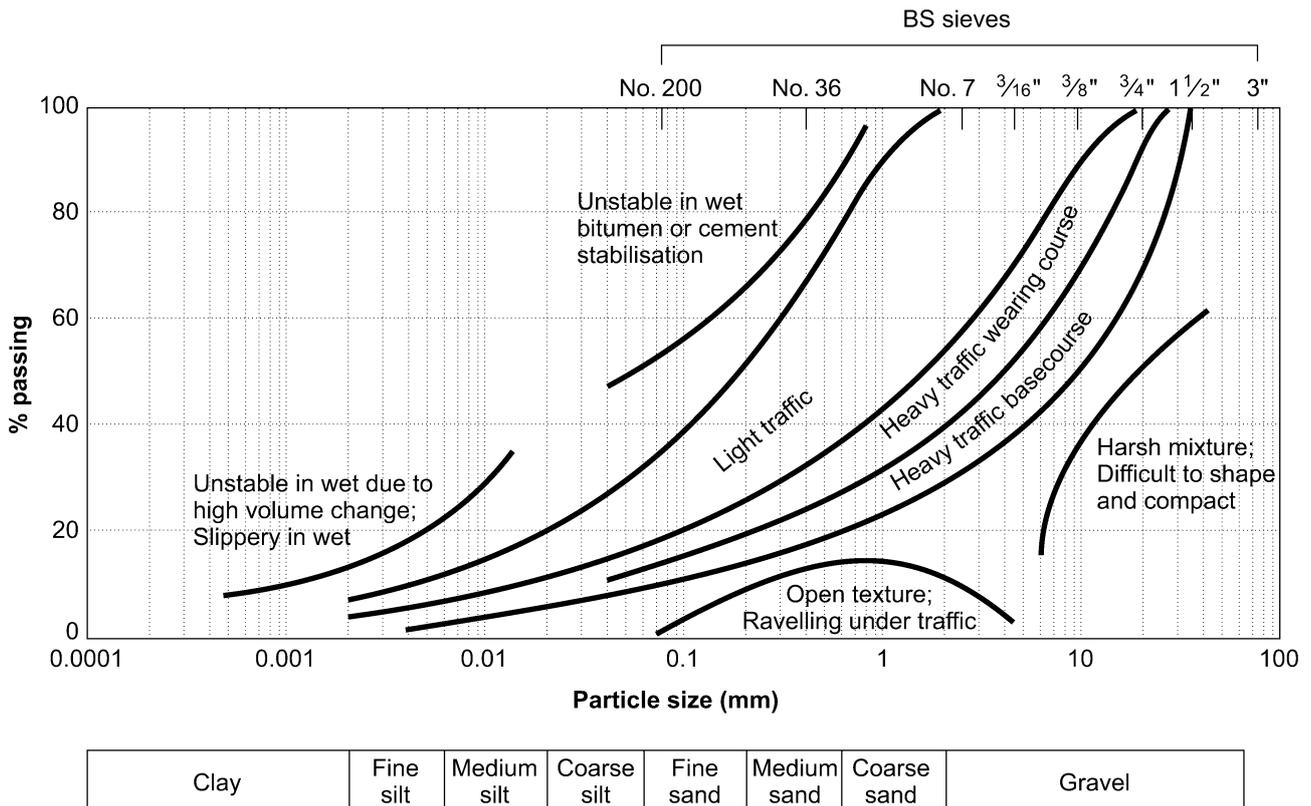
The resilient modulus of granular stabilised materials (used in mechanistic-empirical pavement design) can be determined using AS 1289.6.8.1-1995.

6.3.2 Required PSD

The strength of a granular material is derived solely from the mechanical interlock of particles throughout the particle size spectrum. While maximum frictional strength does not necessarily coincide with maximum density, achieving high density will generally provide high friction. This is often referred to as the 'maximum density PSD principle' in which successively smaller-sized particles fit into the remaining void space thereby reducing the void space to a minimum (maximising density).

Figure 6.1 provides broad guidance on appropriate and inappropriate uses of granular pavement materials with various PSDs.

Figure 6.1: Examples of PSDs of unbound granular materials



Source: Wooltorton (1954).

Road agency PSD specifications for base and subbase materials reflect the characteristic suited to local jurisdiction conditions. These specifications are useful in developing a target PSD for the stabilisation design of pavement materials.

6.3.3 Required of Plasticity Properties

The volume stability of subgrades and pavement materials is dependent upon the amount of clay present and its plasticity. These two parameters are linked to express potential volume stability in the Weighted Plasticity Index defined as:

$$\text{Weighted Plasticity Index (WPI)} = \% \text{ passing } 0.425 \text{ mm} \times \text{Plasticity Index (PI)}$$

Example values of WPI for granular pavement layers specified by road agencies are shown in Table 6.1.

Table 6.1: Examples of limits on plasticity

Material	Weighted plasticity index
High-quality basecourse	< 100
Standard-quality basecourse	< 200
Subbase	< 400
Selected subgrade material, fill	< 1500

In addition to stability, the plasticity of a basecourse can influence the performance of a bituminous seal in terms of adhesion of the bitumen to the surface of the base layer and resistance to raveling under traffic during construction.

6.3.4 Required Aggregate Hardness and Durability

The use of soft aggregates in granular pavements can result in particle breakdown and subsequent rutting of the surface. This degradation can also, in some rock types, result in the release of highly-expansive clay minerals, resulting in an increase in the plasticity of the crushed rock. Some soft and fissile sedimentary rock types can also exhibit excessive swell if broken from the rock mass and subjected to compaction in the roadbed.

As a typical example, particle breakdown from stabilisation mixing and heavy compaction of soft sandstones may result in poor compacted density. Blending in harder aggregates provides protection for the soft aggregate, resulting in higher bearing capacity.

6.3.5 Required Strength and Rut-resistance

For strength tests, such as the Modified Texas Triaxial test and the CBR test, the criteria normally specified for base and subbase materials should be used. Repeated load triaxial (RLT) testing may also be utilised to characterise the elastic and permanent deformation characteristics of granular stabilised materials.

An alternative, recently developed method of characterising permanent deformation properties is using a wheel-tracking device. This method is described in Austroads test method AGTP/T054 *Determination of Permanent Deformation Characteristics of Unbound Granular Materials by the Wheel-tracking Test*.

The selection of design criteria, particularly for lightly-trafficked roads, should take into account local experience. Some materials that do not meet 'normal' specifications perform well in lightly-trafficked, well-drained situations.

6.4 Mix Design

The mix design process is based on blending the existing material with one or more other materials to achieve the required PSD properties (Section 6.3.2), plasticity (Section 6.3.3) and hardness/durability (Section 6.3.4).

The first step in the mix design process is to identify suitable materials to stabilise the existing material. The PSD, plasticity and other relevant properties of these materials are then measured.

A trial blend of these materials is then selected, and an assessment made as to whether the required properties are attained. If the required properties are not achieved, additional trial blends are evaluated.

As illustrated in the worked example in Appendix A, the PSD resulting from a trial blend of materials is a simple proportion calculation of the percentage of the constituent materials passing respective sieves. For instance, when two materials are blended, the PSD of this blend can be determined using Equation 1.

$$[(A\% \times A_{\text{pass}})/100] + [(B\% \times B_{\text{pass}})/100] \quad 1$$

where

A% = percentage of material A being added by mass

A_{pass} = percentage of material A passing allocated sieve by mass

B% = percentage of material B being added by mass

B_{pass} = percentage of material B passing allocated sieve by mass

Note: as there are two materials A% + B% = 100.

Where blending is undertaken to correct plasticity or aggregate hardness, a similar proportionate calculation is undertaken. However, it is not as reliable as PSD corrections and so it may be necessary to test a trial blend for these properties to confirm the calculations. In addition, where two materials are blended to alter plasticity or aggregate hardness, the PSD should also be checked to ensure that it remains satisfactory.

After the PSD, plasticity and other relevant properties have been corrected, strength tests or repeat load triaxial testing to determine modulus and permanent deformation characteristics may also be required depending on the road agency specification.

7. Other Forms of Chemical Stabilisation

7.1 Introduction

The vast majority of pavement stabilisation carried out in Australasia is conducted using cement, lime or bituminous stabilisers or blends of these stabilising binders. However, there are other forms of stabilisation used and these are briefly described in this section. Many of these proprietary chemical binders have been used successfully as dust suppressants but may also have the ability to alter other properties such as strength and permeability. These binders should be assessed for their ability to improve the structural performance of pavements in a similar way to the cementitious, lime and bituminous binders.

One application where other methods of stabilisation are used is for unsealed pavements or pavements under construction to reduce dust nuisance and improve safety. This is an application that is continuously developing, and individual stabilising binders should be assessed on the merits of their performance for a particular project. Stabilisation of unsealed roads is described in the *Guide to Pavement Technology Part 6: Unsealed Pavements* (Austroads 2009a).

Broad guidance for the application of other stabilisation binders for sealed roads follows.

7.2 Structural Improvement

There are a number of proprietary products being promoted for stabilisation of base and subbase courses. The two most promising products of this type are:

- polymer in a dry powder form
- polymer additives used in conjunction with lime.

These act in significantly different ways.

7.2.1 Dry Powder Polymers

Dry powder polymers (DPP) consist of a high-grade polymer thermally bound to an 'inert fine carrier' such as fly ash. It is handled in the same way as other powdered products, being delivered by bulk tanker and placed through a conventional cement spreader.

The product distributed through the pavement material will improve volume stability and reduce capillary rise, pore pressure and moisture-induced shrinkage. This is achieved by altering the physical properties of the soil and not by cementitious action. Hence, the pavement is not subject to shrinkage.

Materials successfully modified will retain a high proportion of their dry strength and bearing capacity even after lengthy soaked conditions. After stabilisation, the material may continue to behave as an unbound material.

7.2.2 Additives to Lime and Cementitious Binders

In some applications DPP are used in conjunction with hydrated lime. The lime is added to flocculate and prepare clay particles for adhesion to the polymer. The lime is not coated with polymer.

7.3 Mix Design

The large variety of proprietary products available and classified as chemical or polymer binders, coupled with varying degrees of quality performance data, make them less definitive in their selection compared to cement, cementitious, lime or bituminous binders.

Information available from product literature together with field examples may provide insight about binder application rates. In addition, the following performance properties may also to be considered in selecting chemical binders:

- increased rut-resistance
- resistance to erosion
- capillary rise
- swell
- resistance to leaching
- long-term durability.

In deciding the relevant mix design for each product, elements of the design processes for cementitious (Section 3), lime (Section 4) and bituminous stabilisation (Section 5) may be adapted.

8. Sampling and Testing for Mix Design

8.1 Introduction

The determination of the most appropriate binder and quantity required to meet the selected type of stabilised pavement material (i.e. granular, modified, lightly-bound or bound) should be undertaken through a laboratory test program using specialist laboratories or, for small projects, on the basis of significant experience with binders and specific pavement materials or soils.

In mix design it is important to recognise the fact that the cost of the binder is often more than 40% of the total cost of the in situ stabilisation. Therefore, in addition to determining the desired engineering parameters, the selection of the most efficient binder type as well as quantity can significantly affect the viability of stabilisation as a pavement option.

Details of the sampling and laboratory testing associated with stabilisation mix design are described below.

8.2 Field Sampling for In Situ Stabilisation

When undertaking a mix design for in situ stabilisation, the samples of pavement material selected for testing must be representative of the material to be stabilised. There can be significant variability between sampling sites because of:

- intermittent maintenance applications (patching and section reseals)
- material variations, particularly where natural gravels are encountered
- PSD variations resulting from construction operations such as grid rolling and rock busting
- pavement thickness variations, both longitudinally and transversely
- natural subgrade variability due to topography and geomorphology
- seasonal variations in pavements and subgrades (wet or dry environments)
- the use of existing subbase/subgrade materials with base materials for lightly-trafficked roads
- sampling under sealed shoulders rather than in the traffic lane
- construction issues, e.g. shape control (the lifting of levels or changing the cross fall to correct drainage or level control to improve ride quality).

The selection of representative samples for testing therefore requires bulk samples to be classified according to a visual description and the results of standard laboratory classification tests (i.e. PSD and plasticity).

8.3 Laboratory Sample Preparation

8.3.1 Particle Breakdown During Construction

During in situ stabilisation, the mixing action typically takes place at a rate of 150–200 rpm using current rotors consisting of 'bullet' style engaging ends on long apertures. Some pavement materials with 'soft' coarse aggregates, such as tuff and sandstone, may break down under the mixing process, resulting in a finer PSD. The rotor speed can be reduced to minimise this breakdown. During site investigations these 'soft' gravels should be identified.

A pre-treatment of the material to be stabilised by repeated compaction in the laboratory can usually take into consideration this effect in the field (e.g. Roads and Maritime 2012a, 2012b). When sampling, some practitioners have sought to use profiler attachments on skid-steer loader as a way of simulating the PSD of the material after mixing.

8.3.2 Water Quality

In most cases water is obtained from town water supplies. However, during prolonged drought periods, the use of treated waste water effluent can be considered provided it does not impact on the strength or durability of the stabilised material.

It is essential that, where possible, the actual water source to be used in the field is also used for the laboratory testing program. If required, a Materials Safety Data Sheet should be provided.

An example standard for stabilisation of water supply (Roads and Maritime 2015):

Water must be free from amounts of materials which are injurious to the stabilisation process such as oils, salts, acids, alkalis and vegetable substances. Water taken from other than a town water supply system must not contain more than:

- (a) 600 parts per million of chloride ion in accordance with RMS T1004 (RMS 2012c)
- (b) 400 parts per million of sulphate ion in accordance with RMS T1014 (RMS 2012d)
- (c) 1% by mass of undissolved solids in accordance with AS 3550.4-1990.

8.3.3 Binders

It is essential that the binder/s proposed for construction is of the same brand and quality as that used in the laboratory mix design process, except for quicklime, which is not commonly used in laboratory due to occupational health and safety considerations. Fresh supplies of binder should also be obtained prior to undertaking laboratory testing and testing for compliance with road agency specifications.

Grouped samples may represent particular sections of the project and, when tested, may require different binder contents to match the various sections. However, it is not common practice to change binder types in these circumstances unless the test results show significant differences in the desired stabilisation strength.

It is also noted that, for the type of stabilised pavement selected, a number of binders and binder contents can achieve the same target strength, but each may have different performance characteristics (e.g. potential of cementitious binders to crack or aversion to capillary rise).

8.3.4 Laboratory Compaction of Lime-stabilised Subgrade Test Specimens

In Australia, field compaction of earthworks materials is commonly expressed relative to the standard Proctor MDD (AS 1289.5.1.1-2017). Accordingly, the laboratory test specimens for strength evaluation are compacted to a density related to the standard Proctor MDD of the lime-treated subgrade.

When Method A (Section 4.8) is used to determine the lime content, the UCS test specimens are compacted to 97% standard Proctor MDD according to TMR practice (TMR Technical Note 151 (TMR 2018d)).

Similarly, when Method B (Section 4.8) is used to determine the lime content, the CBR test specimens are compacted to 95% standard Proctor MDD according to VicRoads practice (VicRoads 2017).

8.3.5 Laboratory Compaction of Cementitiously-stabilised Pavement Test Specimens

The laboratory compaction method for sample preparation and compliance varies across Australia. Some road agencies express field compaction in terms standard Proctor MDD (AS 1289.5.1.1-2017), while others use modified Proctor (AS 1289.5.2.1-2017). Most of the documented experience and research work in Australia has been based on specimens compacted to 100% standard Proctor MDD and the associated OMC. There is no agreement regarding the conversion of test values for samples compacted using modified compaction to standard compaction.

Practitioners should always take note of the relevant laboratory compaction method specified by the road agency for mix design testing. The guiding principal is that the level of compaction in the laboratory testing should be similar to the specified compliance level in the works. The default compaction level is 100% standard Proctor MDD.

8.3.6 Laboratory Compaction of Bituminous-stabilised Pavement Test Specimens

In Australia, field compaction of bitumen-stabilised pavement materials varies across Australia. Some road agencies express compaction in terms standard Proctor (AS 1289.5.1.1-2017), while others use modified Proctor (AS 1289.5.2.1-2017).

In relation to the preparation of the design of foamed bitumen-stabilised mixes, indirect tensile modulus specimens, 152 mm in diameter, are compacted using a heavy (10.2 kg) Marshall hammer as described in Austroads test method AGPT/T305. It is suggested that the density of these modulus specimens is measured as part of the mix design testing to enable comparison to field compaction values.

8.4 Laboratory Tests Associated with Stabilisation Mix Design

8.4.1 General

The principal reasons why a laboratory evaluation of stabilised materials is conducted are to:

- determine the most appropriate binder type in terms of compatibility with the host material, i.e. does the binder work?
- determine the optimum binder content to achieve the type of stabilised pavement required, i.e. how much binder is required?
- provide parameters for the empirical or mechanistic-empirical design of the road pavement.

In undertaking laboratory testing for the determination of the mix design there are some tests specifically associated with the binder type and other tests (generally structural in nature) associated with all binders. Table 8.1 lists Austroads and national standard test procedures appropriate to the evaluation of stabilised pavement materials. Note, however, that local road agencies may also have their own version of these test procedures.

In relation to the properties of the materials to be stabilised, properties such as PSD, particle shape, durability and plasticity, are described in *Guide to Pavement Technology Part 4A: Granular Bases and Subbase Materials* (Austroads 2008). The standard and modified Proctor laboratory compaction methods are also described.

Table 8.1: Laboratory test methods for stabilised materials

Test	Test method reference
Commonly used classification tests	
PSD	AS 1289.3.6.1-2009, AS 1141.11-2009, NZS 4407:1991
Plasticity (LL, PL, PI, LS)	AS 1289.3.1.1-2015, .3.2.1-2009, .3.3.2-2009, .3.4.1-2008, .3.9.1-2015
Particle shape	AS 1141.14-2007, .15-1999, .16-2007, .18-1996
Los Angeles abrasion (LA), crushing value, wet/dry strength variation	AS 1141.21-1997, .22-2008, .23-2009 .24-2013, NZS 4407:1991
Compaction (standard Proctor and modified Proctor)	AS 1289.5.1.1-2017, .5.2.1-2017, NZS 4407:1991

Test	Test method reference
Modified granular mix design	
Californian bearing ratio (CBR)	AS 1289.6.1.1-2014, .6.1.2-1998, NZS 4407:1991
Unconfined compressive strength (UCS)	AS 5101.4-2008
Compressive resilient modulus	Austrroads AGPT/T053, AS 1289.6.8.1-1995
Permanent deformation	Austrroads AGPT/T053, AS 1289.6.8.1-1995, Austrroads AGPT/T054
Triaxial shear strength and Texas triaxial test	AS 1289.6.4.1-2014, Roads and Maritime 2012e, NZS 4402:1986
Lime and cement content	AS 5101.3.1-2008, AS 5101.3.2-2008, AS 5101.3.3-2008
Vertical saturation	Department of Planning, Transport and Infrastructure 2014
Binder leaching	AS 4439.3-1997
Erodibility	Roads and Maritime 2012f
Lightly-bound and bound mix design	
Unconfined compressive strength (UCS)	AS 5101.4-2008
Flexural beam test methods for cemented materials	Austrroads AGPT/T600
Indirect tensile modulus	AS/NZS 2891.13.1-2013
Drying shrinkage of stabilised pavement materials	AS 1289.3.4.1-2008
Cement content	AS 5101.3.1-2008, AS 5101.3.2-2008, AS 5101.3.3-2008
Lime demand	Roads and Maritime 2012g
Available lime	AS 4489.6.1-1997 Rec:2016
Working time	TMR 2017, Roads and Maritime 2012h
Lime stabilised mix design	
Californian bearing ratio (CBR)	AS 1289.6.1.1-2014, .6.1.2-1998, NZS 4407:1991
Unconfined compressive strength (UCS)	AS 5101.4-2008
Lime demand	Roads and Maritime 2012g, TMR 2014b, VicRoads 2013
Available lime	AS 4489.6.1-1997 Rec:2016
Capillary rise, absorption and swell	AS 5104.5-2008
Lime content	AS 5101.3.1-2008, .3.2-2008
Foamed bitumen stabilisation mix design	
Determining the foaming characteristics of bitumen	Austrroads AGPT/T301
Mixing of foamed bitumen stabilised materials	Austrroads AGPT/T302
Compaction of test cylinders of foamed bitumen stabilised materials Part 1: dynamic compaction using Marshall drop hammer	Austrroads AGPT/T303
Resilient modulus of foamed bitumen stabilised materials	Austrroads AGPT/T305
Working time	TMR 2017b

8.4.2 Lime Demand Test

The aim of the Lime Demand Test is to identify the quantity of lime needed to reach a pH plateau to produce long-term reactions (Figure 4.3). Some materials may not gain strength due to a dominant ion exchange process in the presence of organic substances in the material.

Typically, testing in the laboratory will involve hydrated lime whilst contractors may use quicklime in the field. Therefore, the application rate of quicklime needs to be converted from the rate of hydrated lime used in the testing.

8.4.3 California Bearing Ratio (CBR)

The California Bearing Ratio (CBR) test (AS 1289.6.1.1-2014) was originally devised to provide a method of comparing natural gravel and crushed rock bases, and subsequently developed as a means of assessing subgrades for pavement design purposes. In the context of stabilised materials, the test is most commonly associated with granular stabilisation of pavement and earthworks materials. It is also used as part of the Method B lime stabilisation mix design process (Section 4.8).

In the laboratory test a cylindrical plunger is penetrated at a standard rate into a compacted, confined sample (refer Figure 8.1). The CBR is calculated by expressing the load required to cause a specific penetration as a percentage of 13.344 kN, the load required to cause the same penetration in a standard material. The standard material, which was a crushed Californian limestone, is defined as having a CBR of 100%.

The CBR is an empirical value, which unlike modulus, does not accurately relate to any fundamental engineering property. Since the material in the test is predominantly subject to shear deformation, the test can be regarded as an indirect measure of the shear strength. The test is normally carried out on material passing the 19 mm sieve. However, the repeatability on medium- to coarse-grained soils is not as good as on fine-grained soils. The advantage of the test lies in the confidence that can be placed in its application as a result of its successful use in the field over a long period of time, and under a wide range of conditions.

Although the standard laboratory test (AS 1289.6.1.1-2014) is usually carried out on soaked specimens, the procedure allows samples to be tested at whatever moisture content and density is considered appropriate to field conditions (AS 1289.6.1.2-1998 (R2013)). The CBR value will vary according to the conditions of test.

Figure 8.1: Laboratory measurement of California Bearing Ratio (CBR)



Source: Austroads (2008).

Tests are undertaken under a condition of both moulded moisture content (generally OMC) and after four or 10 days of soaking. In addition to the actual CBR value, other useful information includes the increase in moisture content under soaking and any vertical swell that has occurred.

8.4.4 Unconfined Compressive Strength (UCS)

The UCS test (AS 5101.4-2008) provides a measure of the maximum vertical stress an unconfined cylindrical specimen of rock or bound material can withstand before crushing or collapsing. Test specimens, typically with a height to diameter ratio of 2:1 must be capable of standing unsupported in the test equipment (Figure 8.2).

Figure 8.2: UCS test



Source: AustStab (2015).

This is a simple test to assess the strength of material and for that reason it is commonly used in specifications for stabilised materials and also used to define the stabilisation category, viz. modification, lightly-bound or bound as detailed in Table 2.1.

The test can be used to indicate the gain in strength over time associated with cementitious binders, in particular slow-setting binders. In addition, the test can also be used to assess the strength development of different binders when the pavement temperature is cooler in winter (or warmer in summer).

Some road agency test methods require the test to be undertaken after soaking in water.

8.4.5 Repeated Load Triaxial Test

Engineering considerations of the load-carrying ability of granular stabilised and modified granular materials operating under dynamic loading relate to their permanent deformation and resilient modulus. The behaviour of a granular material under dynamic wheel stress is complex as it depends not only on the material type but also on the moisture/density condition and the way in which the stress is applied.

In AS 1289.6.8.1-1995 – repeated load triaxial (RLT) laboratory test method (Figure 8.3) – a compacted cylindrical specimen of the test material is placed in a triaxial cell in which both the lateral stress and vertical stress are applied dynamically. This method uses simple repeated load triaxial equipment which applies static confining pressure and uses dynamic vertical stress.

Figure 8.3: Repeated load triaxial test equipment



Source: Austroads (2008).

The same sample preparation and loading apparatus are utilised to determine both the permanent deformation and resilient compressive modulus properties from a single specimen prepared to a specified density and moisture condition. The permanent deformation measures the vertical permanent strain at three stress conditions using three levels of repeated vertical stress and a static lateral stress. Each stress condition consists of 10 000 repetitions of vertical stress application.

A cylindrical sample, 100 mm diameter and 200 mm high, is used, which allows for testing up to 20 mm maximum particle size. (Note: there is increasing interest in testing materials up to a maximum size of 37.5 mm, which requires samples 150 mm in diameter and 300 mm high.) The approximated stress conditions include static confining pressure and a range of stress levels for base, upper subbase and lower subbase materials. For simplicity, off-sample vertical strain measurement is adopted. This test has been found to produce acceptable accuracy of modulus determination up to a maximum of 700–1000 MPa. This Austroads test method is not accurate for lightly-bound and bound materials.

The resilient compressive modulus determination characterises the vertical resilient strain response over 66 stress conditions using combinations of applied repeated vertical and static lateral stresses. Based on the test results, stress-dependent characteristics of both permanent strain and resilient modulus for the specimen can be determined.

8.4.6 Wheel-tracking Test

For modified materials and granular stabilised materials, the newly-developed wheel-tracking test method (AGPT/T054) may be used to characterise resistance to permanent deformation under a rolling wheel load.

The test material is compacted into rectangular mould to an appropriate density level using segmental roller (Figure 8.4). For each test slab up to 40 000 cycles of loading are applied using a pneumatic tyre loaded to 8 kN (Figure 8.5). The surface shape is periodically measured.

Figure 8.4: Compaction of wheel-tracking test specimens



Source: Austroads (2017c).

Figure 8.5: Application of rolling wheel load



Source: Austroads (2017c).

8.4.7 Indirect Tensile Modulus

For lightly-bound materials and bound materials, the indirect tensile modulus may be used to determine the resilient modulus of cylinders in a similar manner as used for asphalt. Indirect tensile modulus is carried out in accordance with AS/NZS 2891.13.1-2013 (Figure 8.6, Figure 8.7). A repeated vertical compressive force is applied parallel to and along the vertical diametrical plane and the horizontal displacements are measured mid-height through the horizontal diameter.

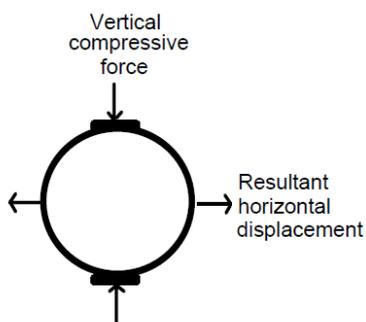
Austrroads test method AGPT/T305 details the measurement of the indirect tensile modulus of foamed bitumen stabilised materials.

Figure 8.6: Indirect tensile modulus test



Source: AustStab (2015).

Figure 8.7: Diagrammatic representation of indirect tensile test



Source: Austrroads (2014c).

8.4.8 Flexural Modulus, Strength and Fatigue

The in-service fatigue performance of cement bound materials can be estimated from their flexural modulus, flexural strength and flexural fatigue testing (Austrroads 2014a, 2014b, 2018).

Test slabs are compacted by placing the loose mixture into a rectangular mould 400 mm long x 320 mm wide x 145 mm high and compacting the material using a segmental roller, a similar manner to the process used to compact asphalt slabs for wheel-tracker testing (Figure 8.8).

Test beams, 100 mm square and 400 mm long, are cut from the slabs.

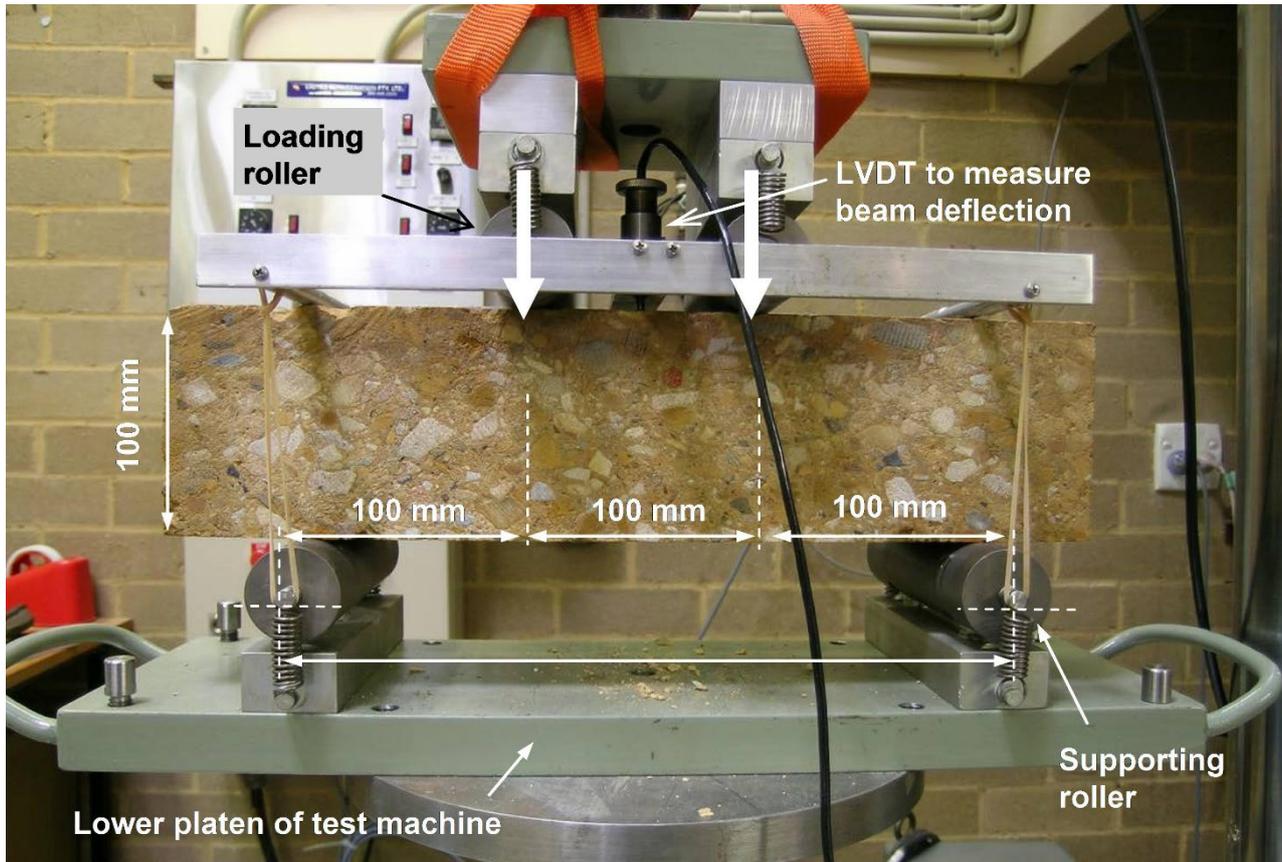
Figure 8.9 shows the apparatus used to measure the flexural modulus, flexural strength and fatigue life (Austroads Test method in press).

Figure 8.8: Compaction of test slabs from which test beams are sawn



Source; Austroads (2014a).

Figure 8.9: Flexural modulus, strength and fatigue test



Source: Austroads (2014a).

8.4.9 Capillary Rise

The Capillary Rise test (Figure 8.10) is associated with lime, cement, cementitious and chemical binders. It is defined as the ratio, in per cent, of the capillary rise in the specimen to the initial specimen height.

There is no absolute level of acceptance for capillary rise of stabilised materials. Capillary rise testing is currently being undertaken for comparative purposes on large projects for the selection of binder types or contents. In addition, current research is studying the rate of capillary rise as well as the absolute value.

The Australian Standard test (AS 5101.5-2008 (R2017)) for capillary rise also determines the absorptivity and the swell of the specimen, thus providing additional data.

Figure 8.10: Capillary rise test



Source: *Georgees and Hassan (2017)*.

Moisture infiltration effects are critically dependent on:

- the distribution of air trapped in voids, which varies with the initial moisture content and the time after initial wetting
- the initial moisture content of the soil, i.e. the drier the soil the greater the increase in the rate of wetting of the sample
- the infiltration process, which slows as the period after wetting has commenced becomes longer.

Remoulding pavement and earthworks material in the laboratory may remove structural discontinuities, which, in the field, would greatly accelerate infiltration.

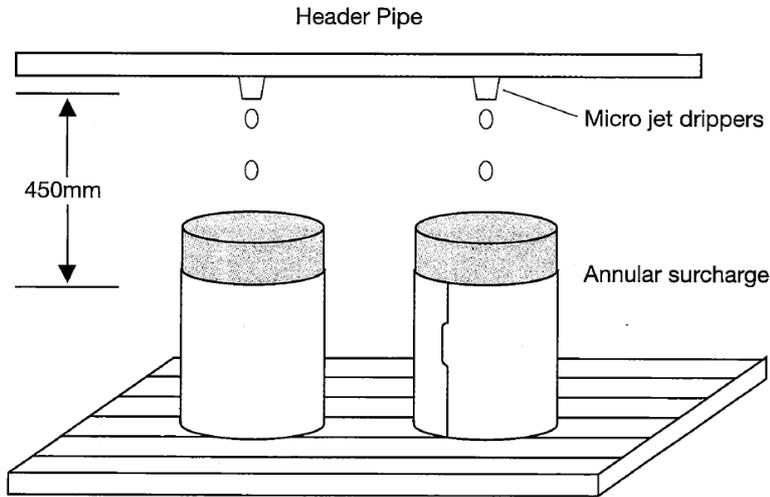
8.4.10 Vertical Saturation

The South Australian Department of Planning, Transport and Infrastructure (DPTI) has developed an empirical test to determine the effectiveness of a binder and the treated material's sensitivity to moisture (DPTI 2014).

The method measures the effect of dripping water onto a compacted specimen of stabilised material. The effectiveness of the binder is judged by monitoring the time for the specimen to collapse under an annular surcharge (Figure 8.11).

The method has been developed with particular reference to the evaluation of the suitability of various types of proprietary chemical binders (Section 7).

Figure 8.11: Vertical saturation test



Material passing 2.36 mm sieve compacted in five layers 105mm diam x 115mm high

Source: AustStab (2015) based on DPTI (2014).

8.4.11 Working Time

Materials stabilised with cement and other cementitious blends need to be compacted before the bonding of the particles reaches a stage which inhibits proper compaction. The time available to work the material is termed the 'working time'.

Whilst several definitions of 'working time' appear to be in use, one of the most common in mix design is:

The nominated working time for any proposed mix is the time measured from the commencement of the addition of the binder to the compaction of the stabilised material, which corresponds to 80% of the mean value of three determinations of UCS, for samples compacted one hour after incorporation of the binder.

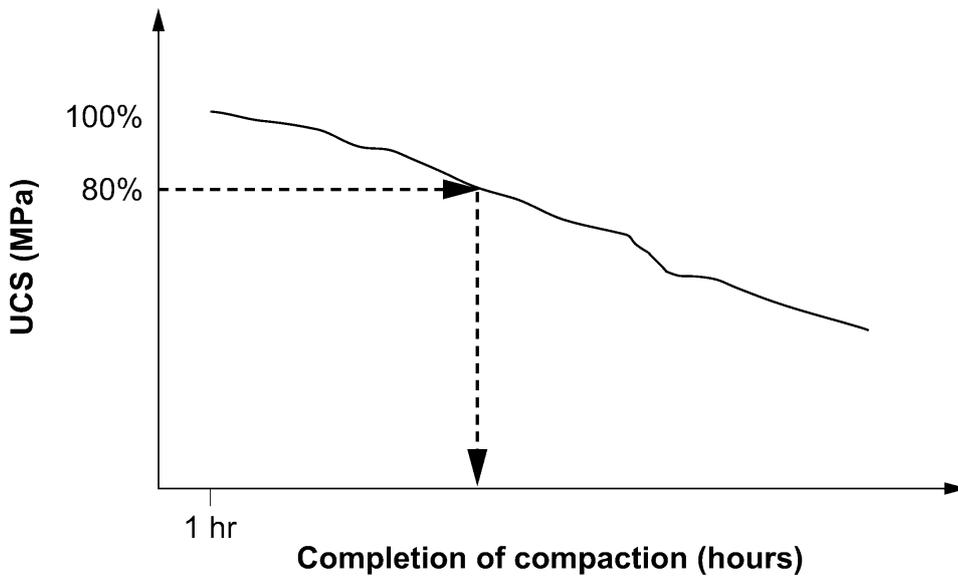
However, other definitions supplement strength with density reduction. For example, TMR test method Q136A (TMR 2017a) also defines working time as the time at which the density decreases to 97% of a reference density.

Working time is determined from preparing loose stabilised material sealed in an airtight container and cured for designated times at which the density and UCS is to be determined.

Generally, curing temperatures are specified as 23 ± 2 °C. However, in cool climates a winter-simulated curing may be considered, e.g. May to September: 10 °C to 15 °C.

The determination of working time is illustrated in Figure 8.12.

Figure 8.12: Diagrammatic representation for working time of cemented materials



Source: AustStab (2015).

Materials stabilised with foamed bitumen also need to be compacted within a certain time to ensure uniform distribution of the bitumen binder. TMR test method Q136B is an example of the testing process. In this method the working time is defined as the time when the three-day soaked indirect tensile modulus is 80% of the mean modulus determined on samples compacted one hour after incorporation of the binder.

The working time is a function of the material and also the ambient conditions and, as a result, allowance should be made for temperature and humidity.

8.4.12 Erodibility

Erodibility is generally described as a loss of material; it is therefore reported in units of grams/minute to the nearest whole number.

There are no specifications or guidelines published for erosion limits, except for the case where erosion of a material needs to be reduced as much as possible, in which case the requirement is for nil erosion. However, it may not be possible to achieve such a material state and increasing the amount of binder may cause other problems associated with the pavement field performance. An example is increased drying shrinkage, which may exacerbate the erosion potential of the pavement by permitting additional moisture into the pavement.

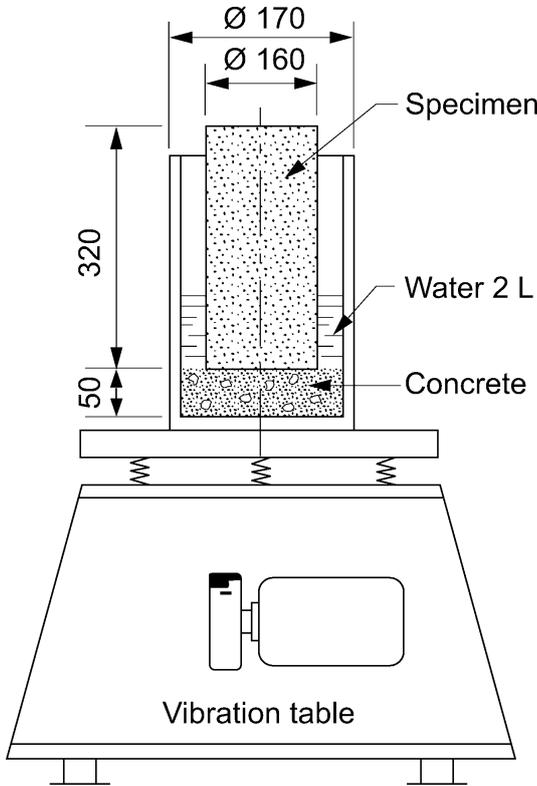
As erosion is sensitive to compaction, it is recommended that, where erosion is considered possible, testing be undertaken at a range of densities below the normal testing regime density.

The only Australian road agency erosion test method is Roads and Maritime test method T186 (Roads and Maritime 2012f) (Figure 8.13). Other test methods exist and have been used in Australia and internationally (refer Jameson 1995). The Roads and Maritime test method, in its draft form, refers to the collection of ‘fines’ from the eroded sample but does not specify a limiting size to the collected fines which are weighed to determine the erodibility of the sample. It is suggested that, unless further information is available, the fines used for the determination of erodibility be that proportion which passes the 2.36 mm sieve.

8.4.13 Leaching

Leachate concentration, particularly where chemical binders are being considered, can be determined in accordance with AS 4439.3-1997. However, this test method is based on a loose sample and results in a conservative test result compared to the leaching of compacted material in the roadbed.

Figure 8.13: Roads and Maritime erodibility test



Source: AustStab (2015).

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Appendix A Worked Example of Blending Materials to Achieve Specification Requirements

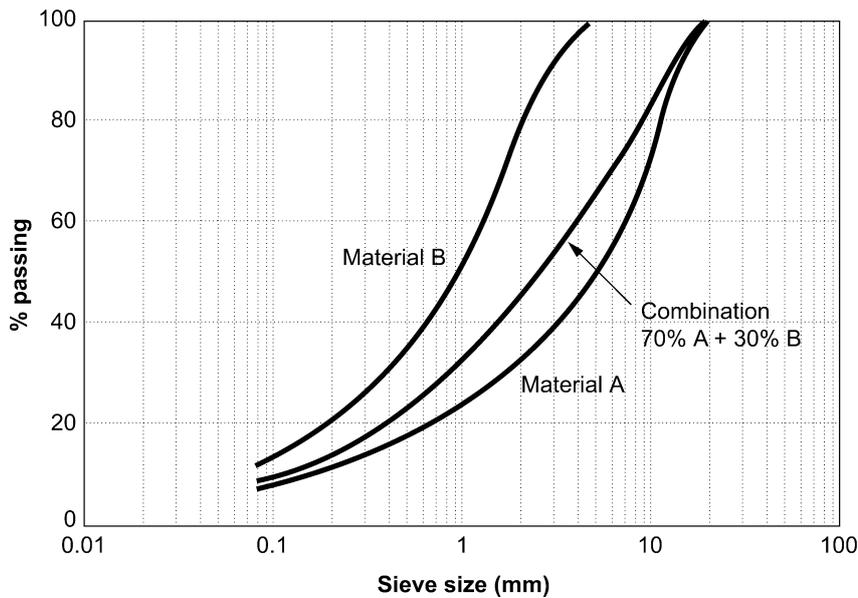
This Appendix presents a worked example in which 70% material 'A' (coarse product) is combined with 30% material 'B' (fine product) to achieve a combination PSD to meet a typical basecourse specification. Simple spreadsheets can be developed to perform these analyses as shown in Table A 1.

An example combination particle size analysis is shown in Figure A 1.

Table A 1: Example calculation – blending two materials

Material type	Mix proportions (%)	Sieve size (mm) and per cent finer by mass							
		0.075	0.300	1.18	2.36	4.75	9.50	19.0	26.5
PSD of Material A	70	8.0	14.0	27.0	35.0	47.0	74.0	99.0	100.0
PSD of Material B	30	12.0	27.0	58.0	86.0	100.0	100.0	100.0	100.0
Combination A +B	70/30	9.2	17.9	36.3	50.3	62.9	81.8	99.3	100.0

Figure A 1: Example combination particle size analysis



Austrroads' **Guide to Pavement Technology Part 4D: Stabilised Materials** discusses the types of stabilised pavement materials (including subgrades), the binders associated with various types of stabilised pavement materials, methodologies for the determination of the appropriate mix proportions in the manufacture of stabilised pavement materials, and specification considerations for manufacture and supply of stabilised materials.

Guide to Pavement Technology Part 4D



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