Guide to Pavement Technology Part 4D

Stabilised Materials

Austroads

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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 of 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 Austroads Guide to Pavement Technology has been developed from a complete revision of the Austroads Guide to Stabilisation in Roadworks.

Issues described in detail include:

- the types of stabilisation undertaken in improving pavement materials and subgrades
- the types of binders used in stabilisation
- the types of materials suited to particular binders
- the 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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- Department of State Growth Tasmania
- Department of Infrastructure, Planning and Logistics Northern Territory
- Transport Canberra and City Services Directorate, Australian Capital Territory
- Australian Government Department of Infrastructure and Regional Development
- Australian Local Government Association
- New Zealand Transport Agency.

Summary

Stabilisation may be defined as a process by which the intrinsic properties of a pavement material are altered by the addition of a stabilisation binder 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 in improving pavement materials and subgrades
- the types of binders used in stabilisation
- the types of materials suited to particular binders
- the laboratory determination of the type and quantity of binder required to achieve a particular type of stabilised material (mix design).

This part of the Austroads Guide to Pavement Technology represents a complete revision of the *Austroads Guide to Stabilisation in Roadworks*. It does not detail quality control aspects of manufacture or performance attributes of stabilised materials nor the safety aspects of using specific binders.

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1. Introduction

Part 4D of the Guide to Pavement Technology replaces, in part, the Austroads (1988) *Guide to Stabilisation in Roadworks*. It reflects the significant technical research that has been undertaken nationally and internationally in improving stabilisation technology since that time.

In particular this part of the Guide to Pavement Technology 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
- specification considerations for manufacture and supply of stabilised materials.

Topics referring to other aspects of stabilisation technology in other parts of the Guide to Pavement Technology are listed in Table 1.1.

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

Structural design of new stabilised pavement	Part 2: Design of New Pavements
Bituminous sealing of stabilised pavements	Part 3: Pavement Surfacings
Rehabilitation of pavements using in situ stabilisation	Part 5: Pavement Evaluation and Treatment Design
Structural design of in situ stabilised pavements	Part 5: Pavement Evaluation and Treatment Design
Stabilisation in unsealed pavements	Part 6: Unsealed Pavements
Specifications for construction of in situ stabilised pavements	Part 8: Pavement Construction Assurance
Construction of stabilised pavements	Part 9: Pavement Work Practices
Maintenance practices for stabilised pavements	Part 9: Pavement Work Practices

Table 1.2 lists road authority and industry associations websites from which pertinent publications on stabilisation technology such as Technical Notes, Guidelines, Work Tips and safety data can be obtained. Also included in the Guide is a bibliography of relevant publications.

Table 1.2: Key websites pertinent to stabilisation technology

Austroads	**www.austroads.com.au**
Australian Stabilisation Industry Association	www.auststab.com.au
PIARC	www.piarc.org
Materials Safety	www.msds.com.au

2. Purpose of Stabilisation

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

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

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

3. Overview

3.1 Manufacture of stabilised materials

Stabilised pavement materials can be manufactured using the following procedures.

3.1.1 Plant-mixed stabilised materials

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 must conform to traditional road authority specifications for particle size distribution, plasticity and source rock hardness. The stabilised material is delivered to the site in trucks and then spread, compacted, shaped and cured in preparation for the placement of the overlying layers, e.g. additional stabilised or unbound granular layers or bituminous surfacing.

3.1.2 In situ stabilisation

This is a mobile process, during which a stabilisation binder is added to an existing pavement and mixed with a purpose-designed road recycler. The quality of in situ stabilised materials is often variable and in some cases requires the addition of other unbound granular materials to correct deficiencies in particle size distribution 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.

3.1.3 Subgrade stabilisation

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

3.2 Stabilisation binders

The binders most commonly used in road stabilisation include:

- granular materials, including crushed rocks, aggregates and sand
- chemicals, including salts, organic and polymer compounds
- lime
- Portland cement
- cementitious blends, including lime and/or cement combined with slag and/or fly ash
- bituminous materials, including foamed bitumen and bitumen emulsions
- bituminous combinations, in which lime or cement is added as a secondary binder.

3.3 Categories of stabilised materials

In the past, the types of stabilised materials have been identified in terms of the type of binder used, e.g. cemented materials, lime stabilised materials, bitumen stabilised materials etc. However in this document an alternate categorisation is adopted based upon the type of material stabilised and its performance attributes associated with design, viz.:

- subgrade stabilised materials
- granular stabilised materials
- modified stabilised materials
- bound stabilised materials.

For modified and bound stabilised materials their associated properties can be obtained using various binders, quantities of binder or combinations of these. Table 3.1, summarises the types of stabilised materials, typical strengths achieved after stabilising, how they are commonly achieved and performance attributes associated with the classification.

Table 3.1: Types of stabilisation

Category of stabilisation	Indicative laboratory strength after stabilisation	Common binders adopted	Anticipated performance attributes
Subgrade	CBR[1] > 5% (subgrades and formations)	• Addition of lime • Addition of chemical binder	• Improved subgrade stiffness • Improved shear strength • Reduced heave and shrinkage
Granular	40% < CBR[1] < +100% (subbase and basecourse)	• Blending other granular materials which are classified as binders in the context of this Guide	• Improved pavement stiffness • Improved shear strength • Improved resistance to aggregate breakdown
Modified	0.7 MPa < UCS[2] < 1.5 MPa (basecourse)	• Addition of small quantities of cementitious binder • Addition of lime • Addition of chemical binder	• Improved pavement stiffness • Improved shear strength • Reduced moisture sensitivity, i.e. loss of strength due to increasing moisture content • At low binder contents can be subject to erosion where cracking is present
Bound	UCS[2] > 1.5 MPa (basecourse)	• Addition of greater quantities of cementitious binder • Addition of a combination of cementitious and bituminous binders	• Increased pavement stiffness to provide tensile resistance • Some binders introduce transverse shrinkage cracking • At low binder contents can be subject to erosion where cracking is present

Notes:

1. *Four day soaked CBR.*
2. *Values determined from test specimens stabilised with GP cement and prepared using Standard compactive effort, normal curing for a minimum 28 days and 4 hour soak conditioning.*

3.3.1 Subgrade stabilisation

In the context of this Guide, subgrade stabilisation implies the addition of a binder to a subgrade soil by in situ stabilisation. Other methods of improving subgrades, such as geofabrics or geogrids, are discussed in Part 4H of the Guide to Pavement Technology.

Stabilisation of subgrades is generally undertaken to provide a higher bearing capacity pavement layer, 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.

3.3.2 Granular stabilisation

Granular stabilised materials are those to which another granular material is added to correct a deficiency in the intrinsic material properties of the parent material. These materials include natural gravels, crushed rock and fine grained soils. They do not exhibit any tensile strength and resist traffic loading through a combination of cohesion and internal friction between particles.

Granular stabilisation alters the intrinsic properties of the parent material, e.g. particle size distribution, plasticity and stone hardness. 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 distress mechanisms for granular stabilised pavement materials are vertical deformation and shear.

3.3.3 Modified stabilisation

Modified stabilised materials are those to which small quantities of cement, cementitious, lime or, chemical binders are added to improve the performance attributes of the material whilst still maintaining the properties of an unbound granular material.

This stabilisation type is adopted when it is desirable to increase bearing capacity, stiffness and/or decrease moisture susceptibility at the same time maintaining flexible pavement characteristics. In addition, and for the same reasons, modification includes the treatment of subgrade soils by the addition of lime or cementitious binders.

The distress mechanisms of modified stabilised pavement materials are vertical deformation and shear.

3.3.4 Bound stabilisation

Bound stabilised materials are produced by the addition of stabilising binders (usually cementitious in nature) to granular materials in sufficient quantities to produce a material which has a significant tensile strength. It should be noted that other binders such as bitumen, when mixed with lime or cement, can also produce a bound material.

The 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 cracking in the pavement base layers needs to be controlled.

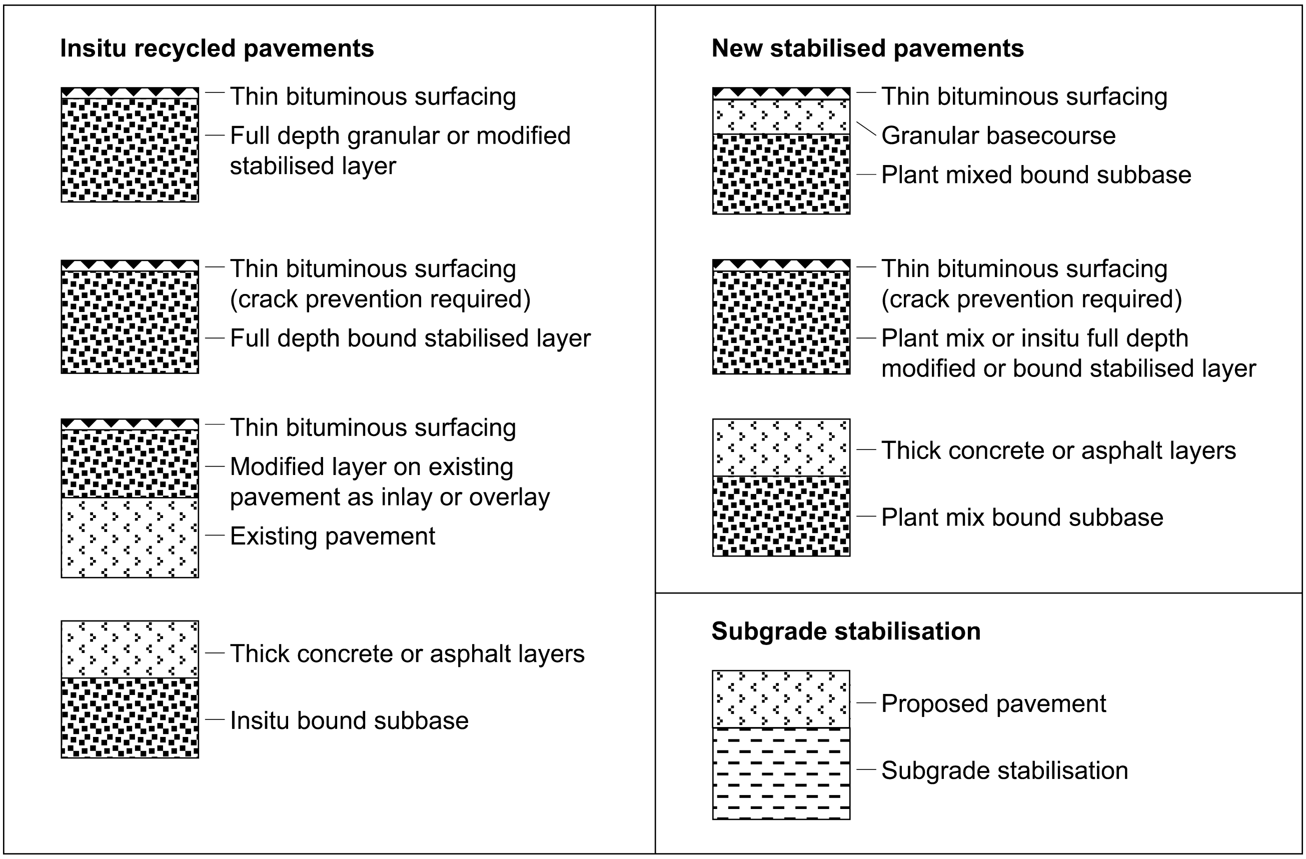
The distress mechanism for bound stabilised pavement materials is tensile fatigue manifest as cracking.

When a bound stabilised material is manufactured from low quantities of cement or cementitious binders and cracking has occurred, additional distress from erosion caused by ‘pumping’ and subsequent deformation or fatigue can occur.

3.4 Pavement configurations incorporating stabilised materials

Stabilised pavement materials can be incorporated in a variety of pavement configurations including composites of different materials. The common stabilised pavement configurations are shown in Figure 3.1.

Figure 3.1: Typical pavement configurations incorporating stabilised materials



4. Binders used in Stabilisation

4.1 Principal binders and combinations

Stabilisation binders used in road construction are manufactured to either specific Australian or New Zealand Standards or road authority 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.:

- granular materials including gravels and sands
- lime AS1672.1, TNZ M/15
- cement AS3972, NZS 3122
- hot bitumen AS2008, TNZ M/1 (for foaming)
- bitumen emulsion AS2341, TNZ/M/1
- slag AS3582.2
- fly ash AS3582.1
- chemicals there are no recognised standards.

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

- lime hydrated lime [$Ca(OH)_2$], quicklime [CaO] and lime slurry
- cement normal Portland (GP) cement and blended (GB) cements
- cementitious contains a combination of pozzolanic material and cement and/or lime:
  - lime and fly ash
  - lime and slag (ground granulated blast furnace slag)
  - lime and slag and fly ash
  - cement and slag
  - cement and lime
  - cement and lime and fly ash
  - cement and slag 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 C170 bitumen is typically used in stabilisation:
  - foamed bitumen
  - bitumen emulsion.

Bitumen stabilisation typically incorporates a supplementary binder, e.g. lime or cement:

- chemical: proprietary products, including lignin derivatives, polymers and salts.

4.2 Granular materials

The addition of additional granular material to an in situ granular material is termed granular stabilisation. It is associated with unbound granular pavement materials rather than subgrades. It involves the improvement of one material by blending it with one or more other granular materials. Alternately, granular stabilisation can be adopted as the first stage of stabilisation in which the intrinsic properties of the in situ material are changed to suit a second stage stabilisation process using other types of binders.

Granular stabilisation provides a direct means of altering the particle size distribution, plasticity or stone hardness to meet the general limits specified by Road Authorities in standard pavement material specifications. These material specifications have been in existence for many years and, as a result, they have a long history of satisfactory performance. Granular stabilisation is therefore an empirically-based process.

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, either on site or in a mixing plant.

Some typical applications of granular stabilisation are:

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

4.3 Lime

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:

$$\begin{array}{ccc} CaCO_3 + \text{heat} & \longrightarrow & CaO + CO_2 \\ \text{(calcium carbonate)} & & \text{(calcium oxide)} \\ \text{(limestone)} & & \text{(quicklime)} \end{array}$$

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.

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:

$$\begin{array}{ccc} CaO + H_20 & \longrightarrow & Ca\,(OH)_2 + \text{heat} \\ \text{(calcium oxide)} & & \text{(calcium hydroxide)} \\ \text{(quicklime)} & & \text{(hydrated lime)} \end{array}$$

(heat of hydration ~ 272 kcal/kg CaO)

For lime stabilisation, the quantity of calcium hydroxide is the active component that reacts with the subgrade or pavement 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) which needs to be taken into account in the mix design of stabilised materials.

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

	Hydrated lime	Quicklime	Slurry lime
Composition	$Ca(OH)_2$	CaO	$Ca(OH)_2$
Form	fine powder	granular	slurry
Equivalent $Ca(OH)_2$/unit mass	1.00	1.32	0.56 to 0.33
Bulk density (t/m^3)	0.45 to 0.56	1.05	1.25

4.4 Cement

There are a variety of commercially-produced cement types and blends in Australia and each has different properties and characteristics. The principal cement types available are as follows:

Type GP – general purpose Portland cement

Type GB – general purpose blended cement

General purpose cements are produced from a mixture of calcium carbonate, alumina, silica and iron oxide which, when calcined and sintered at high temperatures, result 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.

Special purpose cements are not commonly used for the stabilisation of subgrade soils or pavement materials in Australia.

4.4.1 General purpose Portland cement (GP)

Portland cement is defined in AS3972 (Portland and Blended Cements) as: ‘a hydraulic cement which is manufactured as an homogeneous product by grinding together Portland cement clinker and calcium sulphate and which, at the discretion of the manufacturer, may contain up to 5% of *mineral additions.*

4.4.2 General purpose blended cement (GB)

Blended cement is defined as ‘hydraulic cement containing Portland cement and a quantity comprised of one or both of the following:

- greater than 5% of fly ash or ground granulated iron blast furnace slag, or both
- up to 10% silica fume’.

Both types of cement are used in stabilisation but there is an increasing trend towards the use of blended cements because their increased working time allows the constructor more time to achieve better compaction and a smoother riding surface. In addition, early trafficking to induce closely-spaced microcracks can minimise the risk of larger, wider-spaced shrinkage cracks appearing later.

4.5 Cementitious

4.5.1 Pozzolanic materials

Cementitious binders 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 Portland cement or lime to form compounds possessing cementitious products. Pozzolanic additives include fly ash, iron and steel slags, and silica fume which may be combined with lime or cement to form cementitious binders.

Cementitious binders provide an alternative to GP cement, on the grounds of economy or for extended working time for compaction and finishing.

4.5.2 Iron and steel slags

Approximately 4 million tonnes of iron and steel slag are produced in Australia annually. The recycling of these by-products as road stabilising agents contributes significantly to national waste management strategies.

The various types of slag are:

- blast furnace slag, i.e. known as BF slag or BFS
- basic oxygen steel slag, i.e. known as BOS slag or BOS
- electric arc furnace slag i.e. known as EAF slag or EAF.

BOS and EAF is not used in road stabilisation due to their limited commercial availability and quantity.

A summary of the manufacturing processes and application of slags pertinent to road stabilisation is presented in Table 4.2 (see Australian Slag Association (ASA) 2002).

Table 4.2: Types of slag used in road stabilisation (modified from ASA 2002)

Slag source	Common nomenclature	Manufacturing process	Applications pertinent to stabilised pavements
Blast furnace iron slag	Rock slag or air cooled slag	Crushing and screening slag which has been slowly air cooled	Granular pavement layers which may become naturally bound over time
	Granulated slag or slag sand	Rapidly quenching molten slag with high pressure-high volume water sprays	Stabilisation binder Subbase layers which may become naturally bound over time
	Ground granulated blast furnace slag (or GGBFS)	Grinding granulated slag to cement fineness	Stabilisation binder
Basic oxygen steel slag	BOS slag or steel furnace slag	Crushing and screening slag which has been slowly air cooled	Granular pavement layers which may become naturally bound over time
Electric arc furnace steel slag	EAF slag or steel furnace slag	Crushing and screening slag which has been slowly air cooled	Granular pavement layers which may become naturally bound over time

Crushed slags used as granular pavement layers generally conform to the standard specifications (particle distribution, plasticity and hardness) of road authorities. Indications are that pavements constructed from these materials 20 years ago become a fully bound material (UCS >10 MPa) through natural cementation. However, this property of self cementation is particular to the slag source and should not be taken as a general property.

Generally in road stabilisation ground granulated blast furnace slag (GGBFS) is used. It is often manufactured pre-blended with lime as a proprietary product. It is commonly sourced from Wollongong in NSW. GGBFS should conform to AS3582.2.

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. It is treated as such in this Guide. Some ground slag materials already contain small amounts of free lime but the minimum lime content required for the reaction is one part lime for each 10 parts slag.

The most common combination of slag/lime blends is 85:15. In addition to Accelerated Loading Facility (ALF) trials (Jameson *et al.* 1995; Moffatt *et al.* 1998), which demonstrated the good performance of slag/lime blends in bound pavement stabilisation, it has been successfully used in NSW for at least the past 10 years.

4.5.3 Fly ash

Fly ash is a product of the power generation industry. The type of coal used, and the mode of operation of the plant, determines the chemical composition and particle size distribution. 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 should conform to AS3582.1 and be a ‘fine grade type’, i.e. that solid material extracted from the flue gases of a pulverised coal fed boiler that has at least 75% passing the 45 micron sieve and also 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. Chapman and Youdale 1982; Jameson *et al.* 1996*a* and *b*; Francis 1994; Symons and Poli 1996; Ash Development Association of Australia 2005).

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.

4.6 Bitumen

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

- foamed bitumen
- bitumen emulsion, either as cationic or anionic emulsion
- cut back bitumen (not used since the early 1990s); for further information refer to Austroads (1998).

In association with bitumen stabilisation secondary binders which are generally lime or cement are usually added to increase the stiffness of the mix.

Approximate equivalents between Australian and New Zealand bitumen grades are shown in Table 4.3.

Table 4.3: Australian and New Zealand bitumen classifications

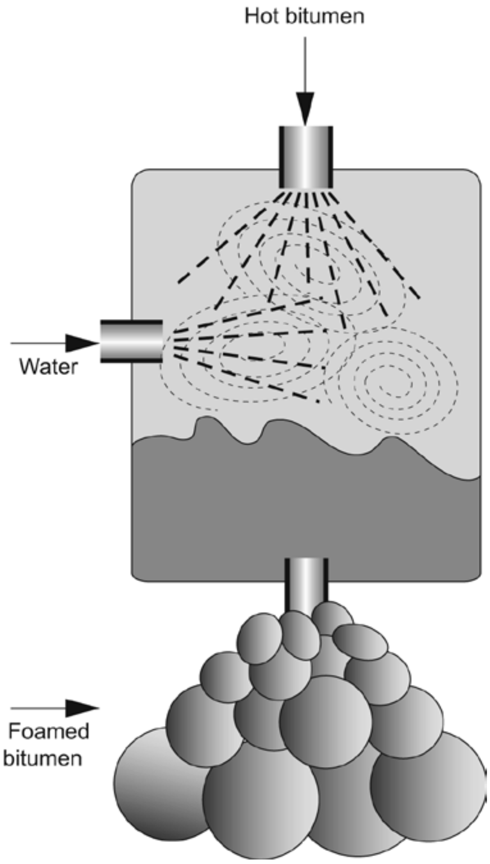
Australian classification	Minimum penetration	New Zealand classification
C50	130	180/200
C170	82	80/100
C320	40	60/70
C600	20	40/50

4.6.1 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 4.1.

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.

Figure 4.1: Manufacture of foamed bitumen



4.6.2 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 involves 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 AS1160, 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 emulsions may be readily mixed with damp soil to produce a good dispersion of bitumen throughout the soil.

4.6.3 Bitumen with secondary binders

The stabilisation of gap-graded granular materials and/or materials with smooth rounded grains can be improved by the addition of mineral filler, rock dust, fly ash, etc. Hydrated lime (1-2%) may also be used as a secondary binder to improve particle coating. Lime may also be used as a preliminary modifying treatment to render particular soils more amenable to receiving a bituminous stabilising binder.

The addition of lime has the effect of stiffening the bitumen and hence the mix as well as acting as an anti-stripping agent through the enhancement of the bonding of bitumen to fine particles. As the addition of cement increases the mix stiffness through hydration, it is not commonly used with foamed bitumen.

The bond between soil particles with bituminous binders can be improved by the use of surface active agents or anti-stripping additives. These agents usually improve the wet strength and water absorption resistance. They can either be mixed with the material before the addition of the bituminous binder, or combined with the bituminous binder before use. The proportion of these binders is usually only about 0.3 to 1.0% by mass of the bitumen.

4.7 Chemicals

The categories of the mainstream chemical binders, and their reaction with subgrades and pavement materials, are as follows:

Polymers:	PVA or PVC polymers and copolymers bond fine particles and impart hydrophobic properties to the soil. They are effective in sandy soils and in lime added for clays.
Organic:	These include tall oil pitch, sulphonated lignin and di-limonene which bind fine particles to interlock with larger aggregates. In addition they often have surfactant properties enhancing compaction by dilation of fine material when compacted with a vibrating roller. Their success is dependent upon both plasticity and particle size distribution. Cement or lime can be added as a secondary binder for increased stiffness.
Ionic:	An electro-chemical charge is imparted to clay platelets. The binders are very material dependent and slow reacting.
Salts:	The most commonly used salt is water-attracting (hydroscopic) magnesium chloride. They require moisture (humidity) to be effective. They also require frequent re-application following rainfall.
Biological:	In this process, microbes consume clay to excrete polymeric residue. High clay contents are required. These types of binders are not commonly available in Australia.

The main applications of chemicals in stabilisation are either as stabilisation binders or surface treatments for dust suppression on unsealed roads, viz.:

Dust palliatives

- water and wetting agents
- hygroscopic salts (e.g. calcium, magnesium or sodium chloride)
- natural polymers (e.g. ligno-sulphonate, molasses, tannin extracts)
- synthetic polymer emulsions (e.g. acrylate, polyvinyl acetate, polyvinyl chlorate)
- modified waxes
- petroleum resins
- tars and bitumens (e.g. prime, bitumen emulsion, cutback bitumen) (Foley et al. 1998).

Stabilisers

- synthetic polymer emulsions (e.g. acrylate, polyvinyl acetate, polyvinyl chlorate)
- sulphonated oils
- enzymes and biological agents
- bitumens (e.g. prime, bitumen emulsion).

4.8 Binder characteristics

The most common binders used in road pavement stabilisation are cementitious, bituminous or chemical products and added material. Table 4.4 presents an overview of the attributes associated with commonly-adopted binders. Typical binders associated with the types of stabilisation are listed in Table 4.5.

Table 4.4: Characteristics of various stabilising binders

Stabilisation binder	Stabilising action	Stabilisation effect	Applicable soil types
Cement & blended cement etc.	Cementitious inter-particle bonds are developed	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.	Not limited apart from deleterious components (organics, sulphates, etc. which retard cement reactions). Suitable for granular soils but inefficient in predominantly one-sized materials and heavy clays
Lime (including hydrated lime and quicklime)	Cementitious inter-particle bonds are developed but rate of development slow compared to cement. Reactions are temperature dependent.	Improves handling properties of cohesive materials. Low binder content (<2%): decreases susceptibility to moisture changes and improves strength. resulting in modified materials. High binder content: increases modulus and tensile strength, resulting in bound materials	Suitable for cohesive soils. Requires clay components in the soil that will react with lime. Organic materials will retard reactions.
Cementitious	Lime and pozzolans modify particle size distribution and develop cementitious bonds	Generally similar to cement but rate of gain of strength similar to lime. Also improves workability. Generally reduces shrinkage cracking problems.	As for cement stabilisation. Can be used where soils are not reactive to lime. Strength gain over years is significant.
Bitumen (either foamed or emulsion)	Agglomeration of fine particles.	Decreases permeability and improves cohesive strength. Decreases moisture sensitivity by coating fines.	Applicable to granular materials with low cohesion and low plasticity
Granular material	Mixing two or more materials to achieve planned particle size distribution.	Some changes to soil strength, permeability, volume stability and compactablity. Materials remain granular.	Poorly-graded soils, granular soils with a deficiency in some size(s) of the particle size distribution
Chemicals	Agglomeration of fine particles and/or chemical bonding. (see trade literature)	Typically increased dry strength, changes in permeability and volume stability	Typically poorly-graded soils and gravels

Table 4.5: Types of stabilisation and typical binder applicable

Binder	Subgrade	Type of stabilisation	
		Modified	Bound
Lime	yes	yes	no
Cement	yes[1]	yes	yes
Cementitious	no	yes	yes
Bitumen	no	yes	yes
Chemical	yes[2]	yes	no[3]

Notes:
1. In some situations cement has been successfully used after an initial lime treatment to improve the subgrade strength.
2. Not common but may be considered for the improvement of subgrade materials.
3. Sometimes used as a secondary binder in association with a primary cement or cementitious binder.

4.9 Preliminary binder selection

To gain a preliminary assessment of the type of stabilisation required for a particular pavement material, particle size distribution and Atterberg limits are commonly used.

The usual range of suitability of various types of stabilisation is based on the percentage of material passing the 0.425 mm sieve and the plasticity index (PI) of the soil, viz.

weighted plasticity index (WPI) = % passing 0.0425 mm x PI

This provides a guide for more detailed studies with particular materials and particular stabilising binders.

Table 4.6 provides initial guidance on the selection of a stabilisation type.

Table 4.6: Guide to selecting a method of stabilisation

Particle size	MORE THAN 25% PASSING 0.425 mm			LESS THAN 25% PASSING 0.425 mm		
Plasticity index	PI ≤ 10	10 < PI <20	PI ≥ 20	PI ≤ 6 WPI ≤ 60	PI ≤ 10	PI > 10
Binder type						
Cement and cementitious blends*	■	▨	□	■	■	■
Lime	▨	■	■	□	▨	■
Bitumen	▨	▨	□	■	■	□
Bitumen/ cement blends	■	▨	□	■	■	▨
Granular	■	□	□	■	■	▨
Polymers	■	■	□	■	■	□
Miscellaneous chemicals**	□	■	■	□	▨	■

Key	Usually suitable	■	Doubtful or supplementary binder required	▨	Usually not suitable

* *The use of some chemical binders as a supplementary addition can extend the effectiveness of cementitious binders in finer soils and higher plasticities.*

** *Should be taken as a broad guideline only. Refer to trade literature for further information*

Note that these binders may be used in combinations or as part of staged construction, viz.:

- bitumen and lime combined is commonly used as a binder in the manufacture of bound pavement materials
- lime stabilisation may be adopted to dry out materials and reduce their plasticity prior to the application of other binders.

5. Stabilisation Mix Design

The determination of the most appropriate binder and quantity required to meet the selected type of stabilised pavement material (i.e. granular, modified 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 subgrades.

In mix design it is important to recognise the fact that the cost of the binder is often in excess of 40% of the total cost of 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 laboratory tests associated with stabilisation mix design are presented in Appendix A of this Guide.

5.1 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 as 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
- particle size distribution 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
- construction issues, e.g. shape control (the lifting of levels or changing the crossfall 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. particle size distribution and plasticity).

5.2 Laboratory sample preparation

5.2.1 Particle breakdown during construction

During in situ stabilisation, the mixing action typically takes place at a rate of 150 to 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 under the mixing process, resulting in a finer particle size distribution. 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 by repeated compaction in the laboratory can usually take into consideration this effect in the field. When sampling, some practitioners have sought to use profiler attachments on skid steer machines as a way of simulating the particle size distribution of the material after mixing.

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

5.2.2 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 also be used for the laboratory testing program. If required, a Materials Safety Data Sheet should be provided.

An example standard for stabilisation of water supply is RTA QA Specification R75:

> *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 RTA T1004*
> *(b) 400 parts per million of sulphate ion in accordance with RTA T1014*
> *(c) 1% by mass of undissolved solids in accordance with AS 3550.4.*

5.2.3 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. Fresh supplies of binder should also be obtained prior to undertaking laboratory tests.

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 strengths but each may have a different performance characteristic (e.g. potential of cementitious binders to crack or aversion to capillary rise).

5.2.4 Laboratory compaction of test specimens

The compaction level for sample preparation and compliance varies across Australia. Most of the documented research work in Australia has been based on Standard compaction of samples at 100%. Whilst both Standard and Modified compaction should achieve similar outcomes, the level of compaction in the laboratory test program should be similar to the specified compliance level in the works. There is also no agreement regarding the conversion of test values for samples compacted using Standard or Modified compaction, and practitioners should always take note of the test compaction levels used in any research work. The default compaction level is 100% Standard.

5.3 Laboratory tests associated with stabilisation mix design

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

- To determine the most appropriate binder type in terms of compatibility with the parent material, i.e. does the binder work?
- To determine the optimum binder content to achieve the type of stabilised pavement required, i.e. how much binder is required?
- To provide parameters for the empirical or mechanistic 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 5.1 lists Austroads and national Standard test procedures appropriate to the evaluation of stabilised pavement materials. Note, however, that local road authorities may also have their own version of these test procedures.

Table 5.1: Laboratory test methods for stabilised materials

Test	Test method reference
General classification tests	
Particle Size Distribution, Atterberg Limits, (LL, PL, PI, LS)	
Soil Reactivity	AS1289.3.6.1, 3.1.2, 3.2.1,3.3.1, 3.4.1, NZS 4407/3112
Los Angeles Abrasion (LA), Crushing Value, Wet/Dry Strength variation	AS1141.23, .22, .21 NZS 4407
Compaction (Modified and Standard)	AS1289.5.1.1, 5.2.1, NZS 4407
Gyratory compaction of bituminous stabilised materials	Austroads AG:PT/T211
Test specimen preparation	
Stabilised pavement materials with cementitious binders	Austroads PT/T302
Stabilised pavement materials with lime	Austroads PT/T303
Stabilised subgrades with lime	Austroads PT/T304
Stabilised pavement materials with foamed bitumen	Austroads PT/T305
Stabilised pavement materials with bitumen emulsion	Austroads PT/T306
Stabilised pavement materials with chemical binders	Austroads PT/T307
Structural strength tests	
Californian Bearing Ratio (CBR)	AS1289.6.1.1, 6.1.2, NZS 4407
Unconfined Compressive Strength	Austroads PT/T308
Indirect Tensile Strength	Austroads PT/T314
Triaxial Shear (and Texas Triaxial)	AS1289.6.4.1 and RTA T171, NZS 4402
Compressive Resilient Modulus	Austroads AG:PT/T080
Compressive Permanent Deformation	Austroads AG:PT/T080
Indirect Tensile Resilient Modulus	Austroads PT/T313
Flexural Modulus of pavement materials with bituminous binders	Austroads AG:PT/T233
Wheel Tracking	Austroads AG:PT/T231
Binder interaction tests	
Capillary Rise, absorption and swell	Austroads PT/T309
Lime Demand	Austroads PT/T315
Drying Shrinkage of stabilised pavement materials	Austroads PT/T311
Working Time	Austroads PT/T312
Vertical Saturation	
Erosion	RTA T186
Leaching	AS4439.3
Cement content of stabilised pavement materials	Austroads PT/T310

A guide to the application of the appropriate laboratory test to the type of stabilisation being considered is presented in Table 5.2.

Table 5.2: Laboratory tests appropriate to stabilisation type

Appropriate tests	Modified	Bound
Classification	yes	yes
Lime demand	yes	no[1]
Shrinkage	no	yes[2]
Erosion	possible	no[1]
UCS	yes	yes
CBR	yes	no
Modulus	yes[2]	yes[2]
Wheel tracking	yes[2]	no[1]
Binder leaching	yes	no[1]
Fatigue	no	yes[2]
Working time	no	yes

Note:

1. *May be required where low binder contents are used or the UCS is less than 3.0 MPa.*
2. *Undertaken for large projects only or heavily trafficked high risk pavements.*

5.4 Granular stabilisation

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

Internal friction is generated primarily as a result of the characteristics of the coarser soil particles and the Particle Size Distribution (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 particle size distribution and plastic properties (e.g. VicRoads 1997). Strength testing is generally undertaken using the CBR test and/or triaxial shear test.

The resilient modulus of granular stabilised materials (used in mechanistic pavement design) can be determined using AS1289.6.4.1. Granular stabilised materials are considered to have a Poisson's ratio value between 0.30 and 0.45 and to be cross-anisotropic, with $E_v/E_h = 2$ for the purpose of pavement design (see Austroads 2004).

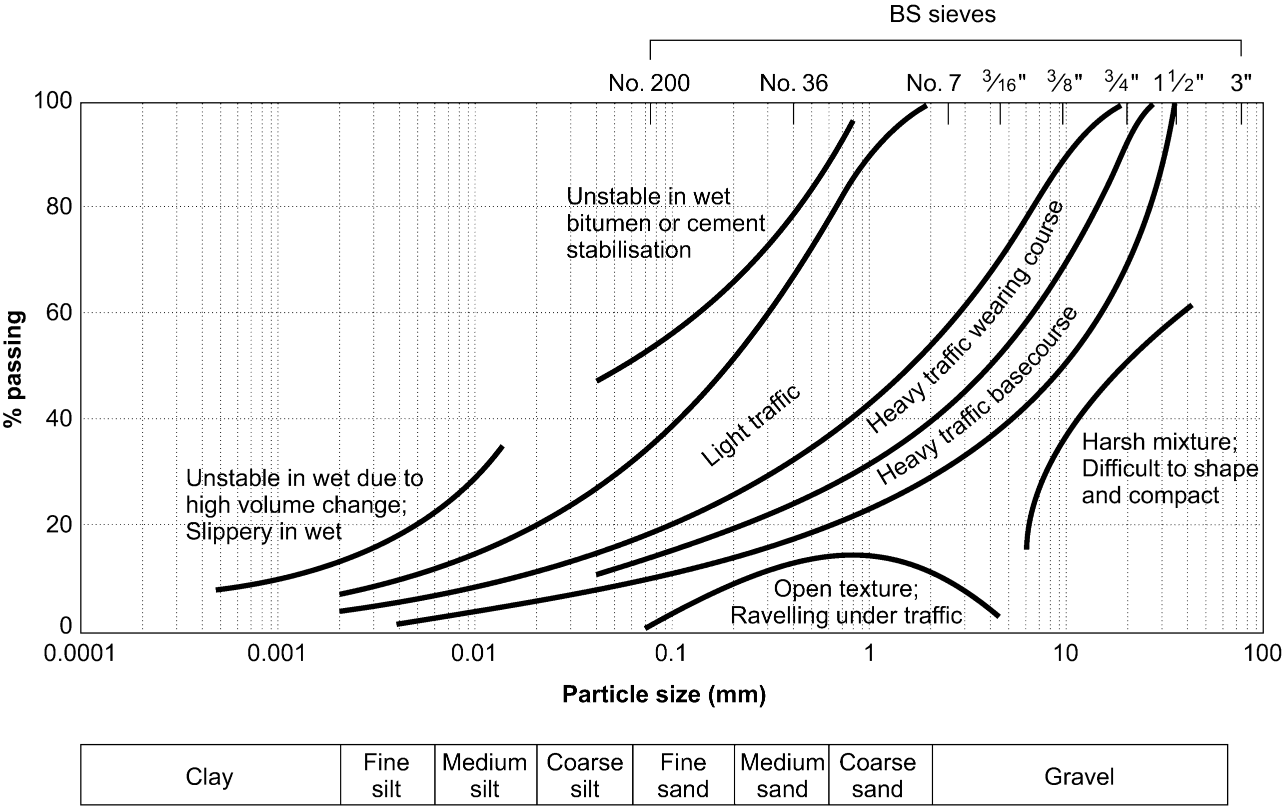
5.4.1 Correction of particle size distribution

The strength of a granular material is derived solely from the mechanical interlock of particles throughout the particle size spectrum. This is often referred to as the ‘maximum density particle size distribution principle’ in which successively smaller-sized particles fit into the remaining void space thereby reducing the void space to a minimum (maximising density).

The performance of a granular pavement layer is highly dependent upon the achievement of a high degree of compaction. This is best achieved through the material being suitably graded and the compaction moisture content (i.e. OMC) being consistent with depth. The use of stabilisation equipment, apart from being far more expedient, allows better distribution of compaction moisture content through the full depth of the granular layer compared to windrow grader mixing.

Performance of granular pavement materials is illustrated in Figure 5.1.

Figure 5.1: General characteristics of unbound granular materials



Standard specifications for unbound granular pavement materials are available from all State Road Authorities which are designed to suit the local jurisdiction conditions.

5.4.2 Correction of plasticity and volume stability

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:

Weighted Plasticity Index (WPI) = % passing 0.425 mm x Plasticity Index (PI)

Typically values of WPI for granular pavement layers specified by SRAs are shown in Table 5.3.

Table 5.3: Suggested limits for weighted plasticity index

Pavement layer	Weighted plasticity index
High quality basecourse	< 100
Standard quality basecourse	< 200
Subbase	< 400
Subgrade 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.

5.4.3 Correction of 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 harder aggregates provides protection for the soft aggregate, resulting in higher bearing capacity.

5.4.4 Numerical method of blending

Granular mix design is undertaken by altering the particle size distribution. This is achieved using simple proportion calculations of the percentage of the constituent materials passing respective sieves, viz.:

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

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

A worked example is provided in Appendix B of this Guide.

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. 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.

5.4.5 Subgrade and modified stabilisation with lime

Stabilisation of subgrade soils and pavements with lime is usually undertaken to:

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

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

The rate of strength gain is dependent on both temperature and lime content. Caution should therefore be exercised in accepting results of high-temperature accelerated testing without validating them 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 65°C (40°C for chemical binders).

Lime stabilised materials are usually evaluated at both 7 and 28 days.

High lime contents will not necessarily produce high early strengths. Figure 5.2 illustrates the variations in strength of lime-stabilised materials with time and lime content.

Increasing the lime content increases the optimum moisture content of the material being stabilised owing to the fine grained nature of hydrated lime. This effect is further increased by delaying compaction once the lime is added. The effect of lime content OMC and dry density is shown in Figure 5.3.

Figure 5.2: Variation in compressive strength as a function of lime content and time

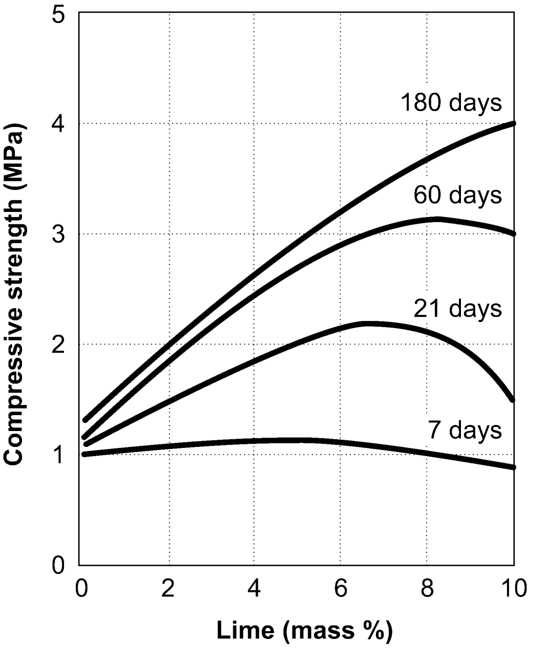
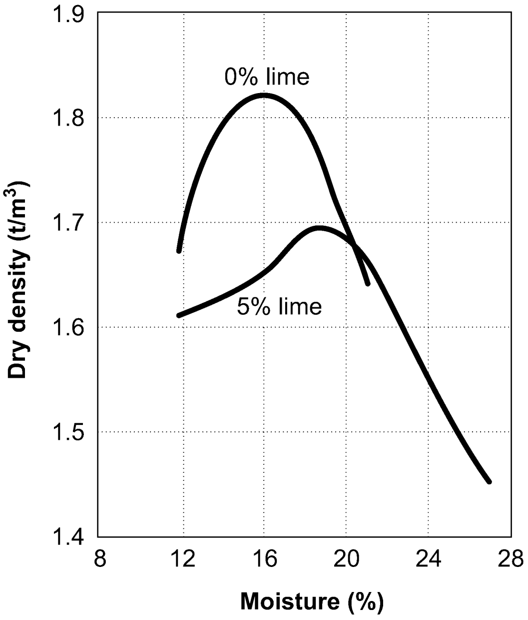


Figure 5.3: Effect of lime content on OMC and dry density



For lime to be effective, the material being treated must contain clay particles or pozzolanic materials that react 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.

The initial lime demand of the soil to be stabilised should be assessed to ensure that adequate lime is added to achieve an excess after initial reactions with the soil are completed. This will ensure that the stabilised design properties are achieved in the long term (Sherwood 1994; Little 1996).

The advantages of using lime instead of a cementitious binder increase with increasing plasticity and fines content. Generally, soils with a PI < 10 will respond better to cementitious binders. However, comparative testing is advisable.

If the PI is to be reduced and workability improved using lime modification, then sufficient lime should be added to ensure that the additional quantities do not result in further changes in PI.

pH testing is used to determine whether a soil is reactive to lime and also to estimate the approximate lime content. This is augmented by 28 day UCS testing to establish the optimum lime content. The optimum lime content occurs when the plot of UCS v. lime content peaks.

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

- sugars and reactive organic materials
- quality of water, and moisture content of material prior to stabilisation
- quality of lime (i.e. available lime index)
- uniformity of mixing and curing
- compaction
- clay content.

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.

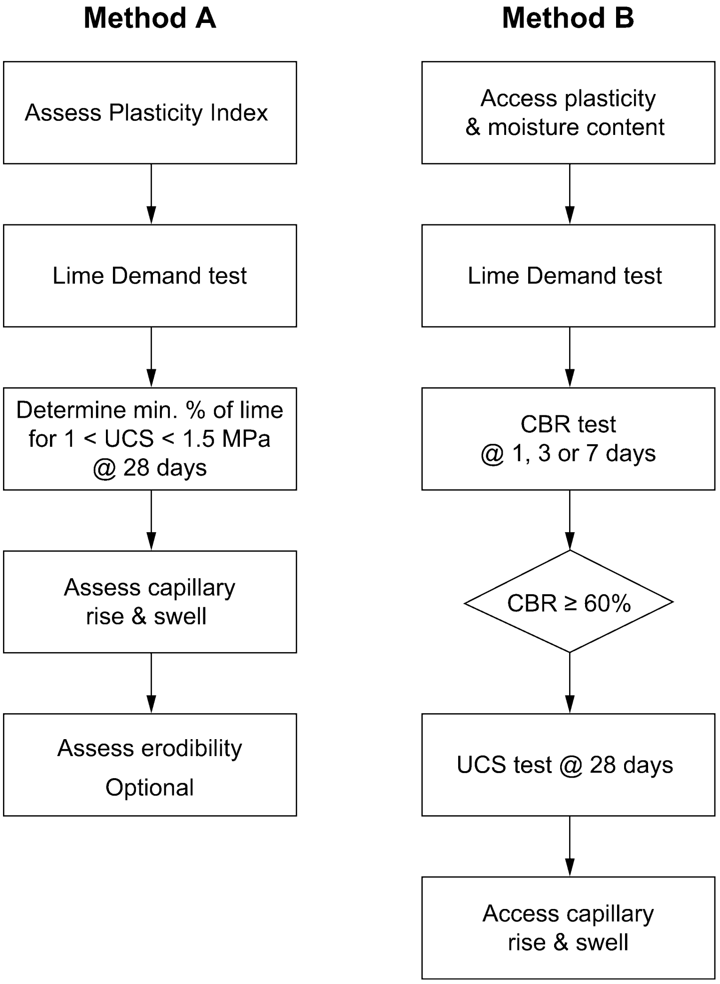
The use of quicklime to establish a working platform on a wet clay is a useful construction expedient. The process is based on the exothermic reaction of the lime as it hydrates reducing the moisture content of the soil.

The required lime content can be determined in two ways as indicated in Figure 5.4:

- In Method A the lime content is determined based on a desired UCS. A suggested UCS for subgrade stabilisation is between 0.5 MPa < UCS < 1.0 MPa whilst, for modification of pavement layers, the suggested UCS is 0.5 MPa < UCS < 1.5 MPa.
- In Method B the lime content is determined based upon CBR. This is used principally for subgrades where it is suggested that a minimum CBR of 5% is desired.

Modification with lime may also be adopted to reduce the plasticity of unbound granular pavement materials without adding significant strength. In this case plasticity indices are determined for various lime additions.

Figure 5.4: Selection of lime content



5.4.6 Subgrade and modified stabilisation with chemical binders

The consideration of proprietary chemical binders is generally based on the determination of their suitability to the parent material rather than the determination of the required application rates. Basic information is generally available from product literature together with field examples. In some cases quantitative measurement of performance or attribute improvement is available.

Chemical binders are generally separated into either dust palliatives or stabilisers and the following performance properties need to be considered:

- resistance to abrasion (effect of traffic and wind on treated surfaces)
- resistance to erosion
- resistance to leaching
- increased shear strength (all weather trafficability)
- long-term durability
- maintainability.

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.

A suggested appropriate brief protocol for mix design with chemical binders is as follows:

1. Review product literature, placing particular emphasis on the results of field trials (particularly independent trials) and quantitative stabilised pavement data.
2. Prepare test samples at the binder content recommended in the manufacturer's specification.
3. If the product is designed to provide water-proofing properties binder suitability and content confirmation by capillary rise, vertical saturation or permeability testing is recommended.
4. If the product is designed to provide increased strength characteristics binder suitability and content confirmation by CBR, UCS or resilient modulus testing is recommended.
5. If the product is designed to provide improved compaction characteristics binder suitability and content confirmation by laboratory compaction tests.
6. Dependent upon the outcomes of laboratory testing adjust the binder content as appropriate if the binder appears suitable.
7. If insufficient field performance data is available undertake a field trial, ensuring that an untreated (or proven treatment alternative) section is included as a base measure or 'control'. Alternatively, if the laboratory testing suggests that the product may be suitable, adopt it for the project and monitor performance for future application.
8. Monitor in-service performance over the length of the treatment life.

5.4.7 Modified stabilisation with cement and cementitious binders

The selection of a modified stabilised pavement must first be made on the basis of its suitability to its operating environment, e.g. in a wet location erosion may be an issue.

For cementitious binders, including cement and slag/lime and fly ash/lime blends, typically between 1% and 3% addition is required to achieve the structural characteristics of modified pavements. It is important, however, to recognise that there is an upper limit on strength which, if exceeded, may lead to a bound pavement which has different performance characteristics.

Depending upon the type of binder spreader or plant mix control used, binder contents < 5 kg/m^2 may be difficult to apply uniformly.

A suggested mix design process for modified pavements is as follows:

1. Select the binder or range of binders that may be suitable taking consideration of the following:
    a. Does the pavement material require primary granular, chemical or lime stabilisation to improve the stabilisation effect of the primary binder?
    b. Are slow-setting binders suitable if early trafficking is required?
2. Select 1% binder addition for initial assessment and adjust in accordance with laboratory testing taking due cognisance of practical spread rates possible.
3. Determine the strength (either UCS and/or compressive resilient modulus) within the range shown in Table 3.1.
4. Determine capillary rise and any free swell due to wetting up. Free swell should not exceed 0.5% for pavement materials.
5. On large projects shrinkage tests may be undertaken to evaluate the differences between cementitious binder types or the effects of the addition of a supplementary chemical binder.

5.4.8 Bound stabilisation with cement and cementitious binders

For cementitious binders, including cement and slag/lime and fly ash/lime blends, typically more than 3% addition is required to achieve the structural characteristics of bound pavements.

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

For slow-setting binders working time will need to be determined if immediate trafficking of the pavement during construction is anticipated.

Certain material constituents can be detrimental to the formation of bound materials. These include sugar and reactive organic compounds which can retard the hydration process and prevent the hardening of the binder. In some circumstances, soluble sulphate can cause expansion and subsequent disintegration of the hardened paste. The use of lime or chemicals as a secondary binder may rectify these situations.

Figure 5.5 illustrates the process for the determination of binder content. If there is no improvement in the stabilised material after increasing the binder content, it is suggested that another binder type be selected.

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.

5.4.9 Bound stabilisation with bituminous binders

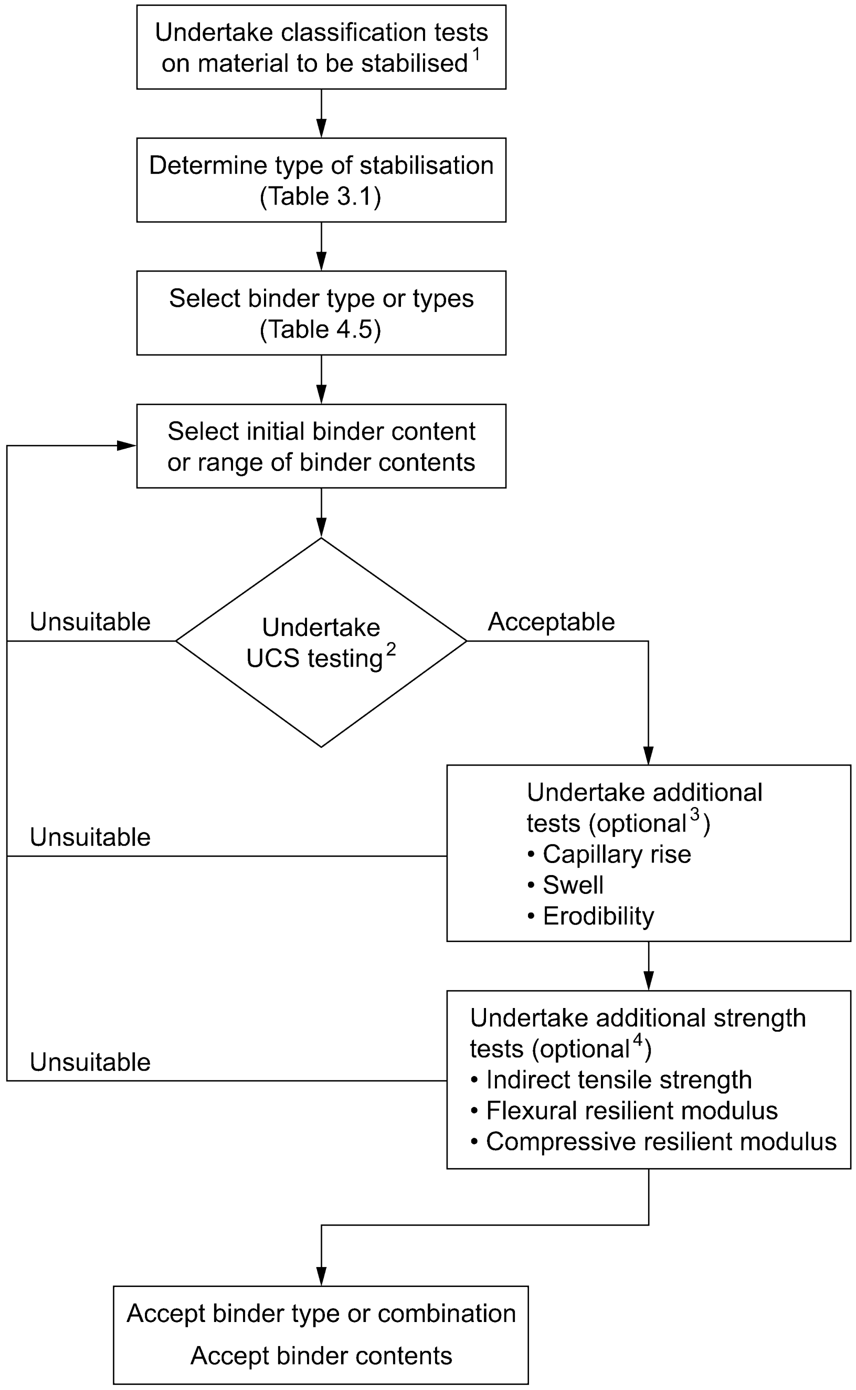
The application of bituminous binder (as a foam or emulsion) in stabilisation is highly dependent on the particle size distribution of the parent material. Recommended limits for bituminous binder in stabilisation are shown in Table 5.4 and Figure 5.6.

Table 5.4: Particle size distribution envelopes for bituminous binders

% Passing sieve (mm)	Zone A (Ideal particle size distribution)	Zone B (fine material)	Zone C (coarse material)
26.5	73-100	100	<73
19.5	64-100	100	<64
9.5	44-75	>75	<44
4.75	29-55	>55	<29
2.36	23-45	>45	<23
1.18	18-38	>38	<18
0.6	14-31	>31	<14
0.3	10-27	>27	<10
0.15	8-24	>24	<8
0.075	5*-20	>20	<5*

The 5% lower limit for the 0.075 mm sieve could be relaxed when secondary cementitious binders are used.
Where the PI of the material exceeds 12% a supplementary addition of lime is usually necessary to reduce the plasticity prior to foamed bitumen modification.*

Figure 5.5: Flow chart for determining the binder content of bound pavement material design



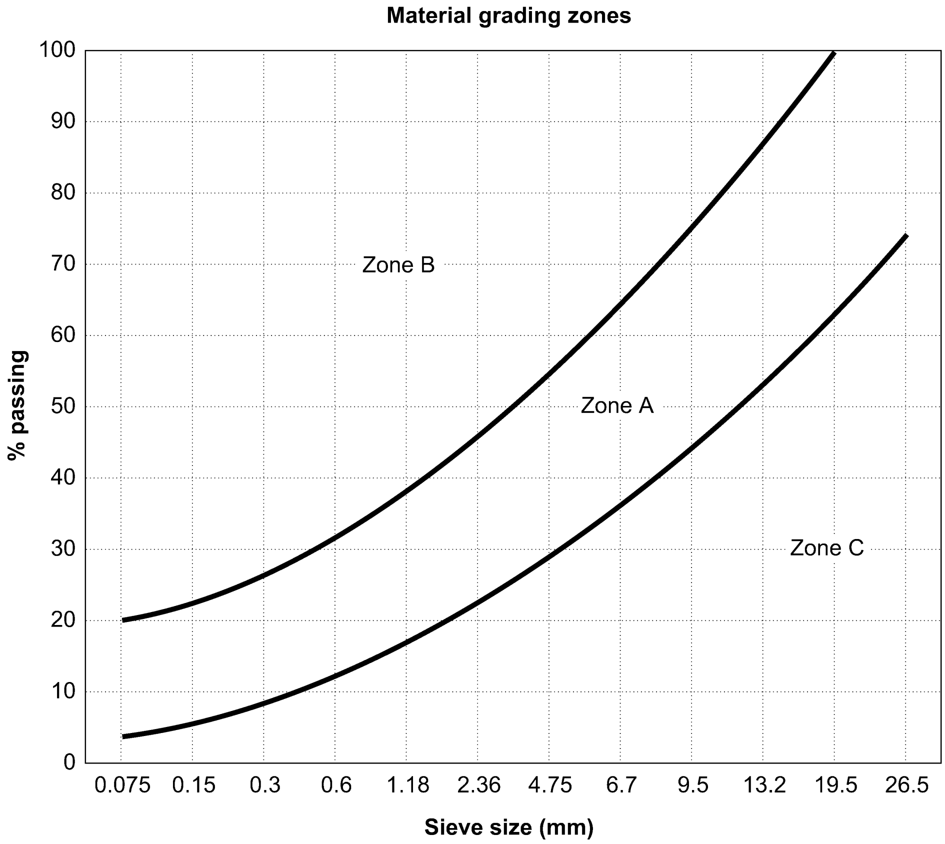
[1] The material to be stabilised may need particle size distribution to be corrected prior to testing or pretreatment to simulate particle breakdown during construction.

[2] UCS testing is undertaken at either the compaction specified or the default standard compaction. Accelerated curing may be considered in addition to gain in strength with time under standard curing.

[3] If it is proposed to use a material where there is no knowledge of its performance in the operating environment, then additional tests may be required to reduce the likelihood of moisture sensitivity, unacceptable cracking or erodibility.

[4] On large projects other forms of strength testing may be required to confirm mechanistic design assumptions.

Figure 5.6: Suitable PSD envelopes for bituminous binders (Zone A)

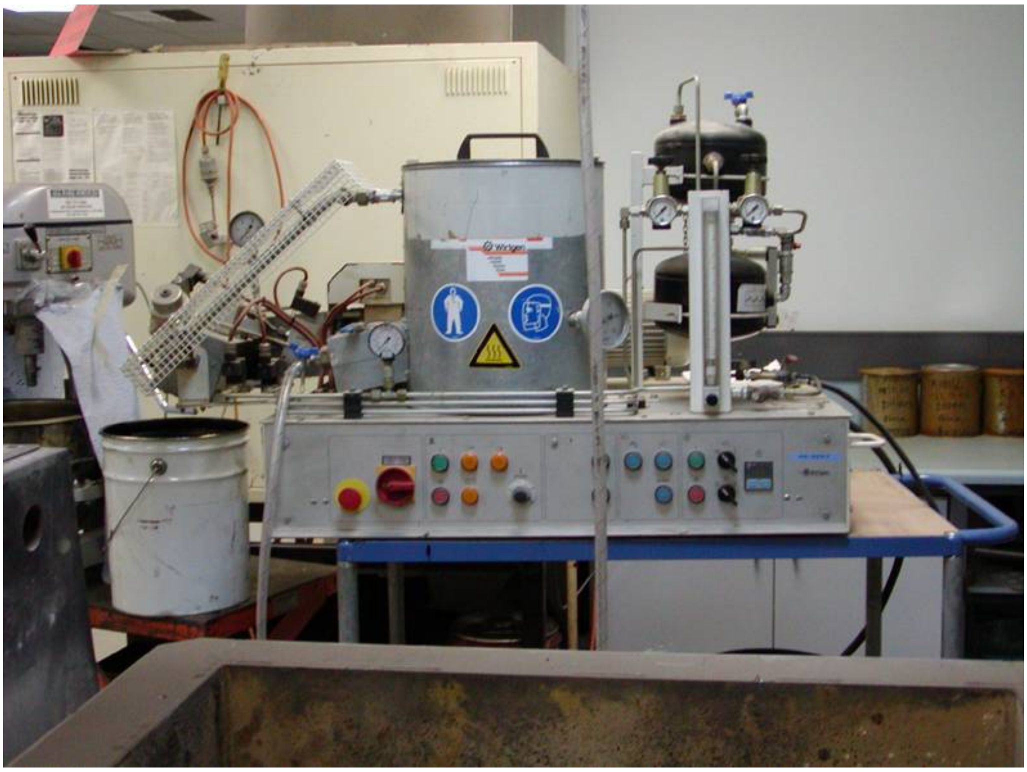


Foamed bitumen binder design

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. This testing can only be undertaken using specially-designed laboratory apparatus (e.g. see Figure 5.7) and experienced technicians.

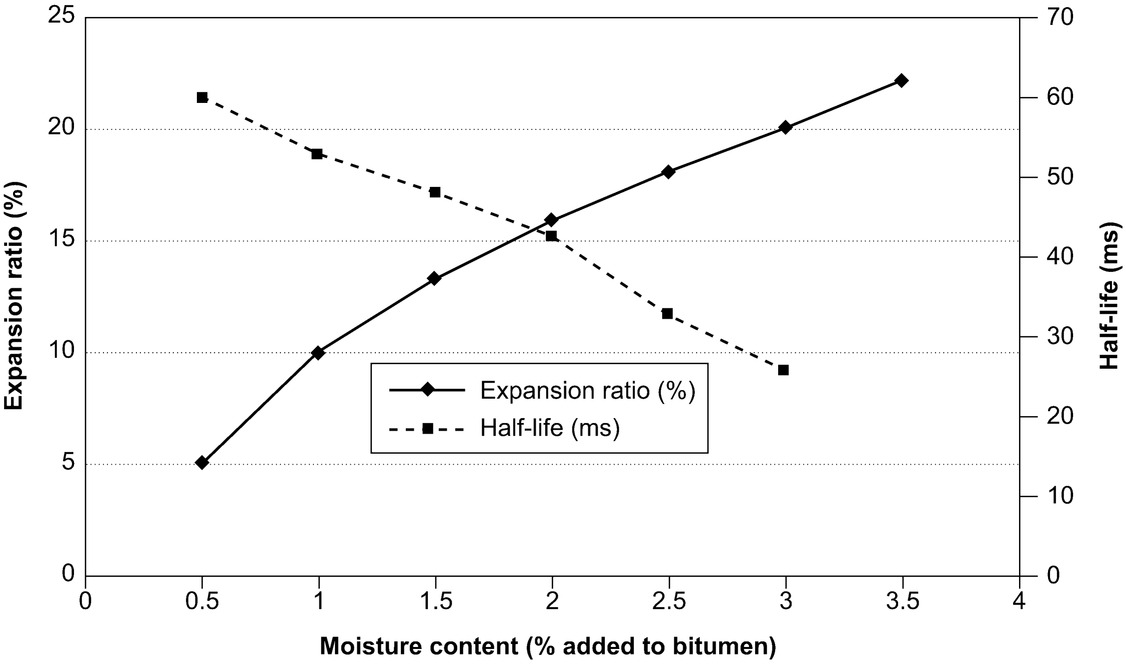
Binder design is undertaken to determine the half life of the foam, i.e. the time for the foam to settle to half its initial height. Generally a foaming agent is added to the water to enhance the expansion ratio, i.e. the increase in volume due to foaming, with a typical value being between 12 and 15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

Figure 5.7: Laboratory foam bitumen apparatus



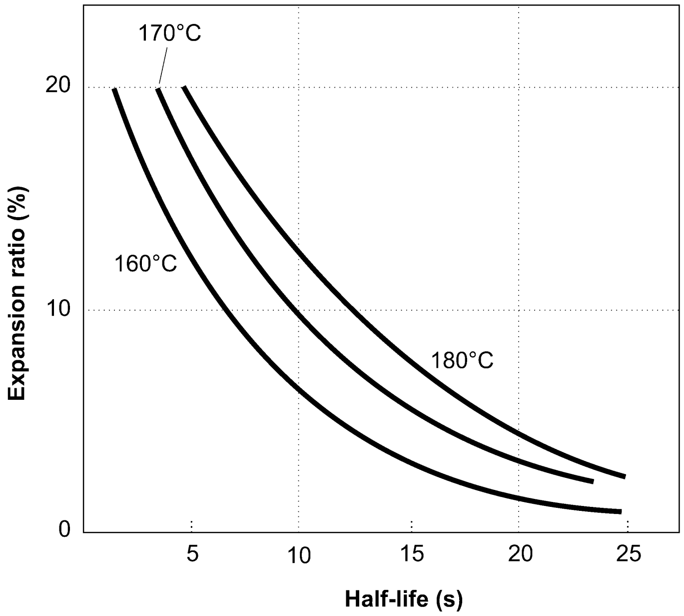
The influence of water content on the foaming characteristics of bitumen affects both the expansion ratio (increase in volume) and the half life (time for half the expanded volume to collapse) as shown in Figure 5.8 (Kendall *et al.* 2001).

Figure 5.8: Effect of water on foaming characteristics



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. This is illustrated in Figure 5.9, where the variation in water content from left to right along the horizontal axis is not significant enough to change the impact on half-life for different bitumen temperatures.

Figure 5.9: Effect on expansion ratio and half-life at different bitumen temperatures



Bitumen emulsion

Bitumen emulsions are manufactured to comply with Australian Standard AS1160 which allows for 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.

Most manufacturers produce all classes and grades. However, emulsions of the same class made by different manufacturers can react differently with the same soil. It is therefore important that the emulsion to be used in the field is identical to that used in the laboratory testing program.

Bitumen emulsion is normally manufactured with Class 170 bitumen, which is usually satisfactory for soils with lower fines content (0-10% passing the 75 μm sieve). For soils with higher fines content (15-25%) passing the 75 μm sieve, it may be necessary to use softer bitumen (Class 50) if difficulty in distribution is encountered. Under extreme conditions, emulsion containing oil may have to be used.

Application rates of 2% to 3% (by mass) of residual binder are commonly used. Lower rates of about 0.5% to 1% may be satisfactory for well-graded materials in dry climates (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.

If there is no other data available, a guide to the amount of emulsion required to form a heavily-bound material may be obtained from the following equation (Asphalt Institute 1989):

$$\% \text{ (by mass)} = 0.75\ (0.05A + 0.10B + 0.50C)$$

where:

A = % retained on 2.36 mm sieve
B = % passing the 2.36 mm sieve but retained on the 75 μm sieve
C = % passing the 75 μm sieve.

Bitumen and secondary binder combinations

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 stiffness, and early rut resistance, of the stabilised pavement
- reduce moisture sensitivity of the stabilised material.

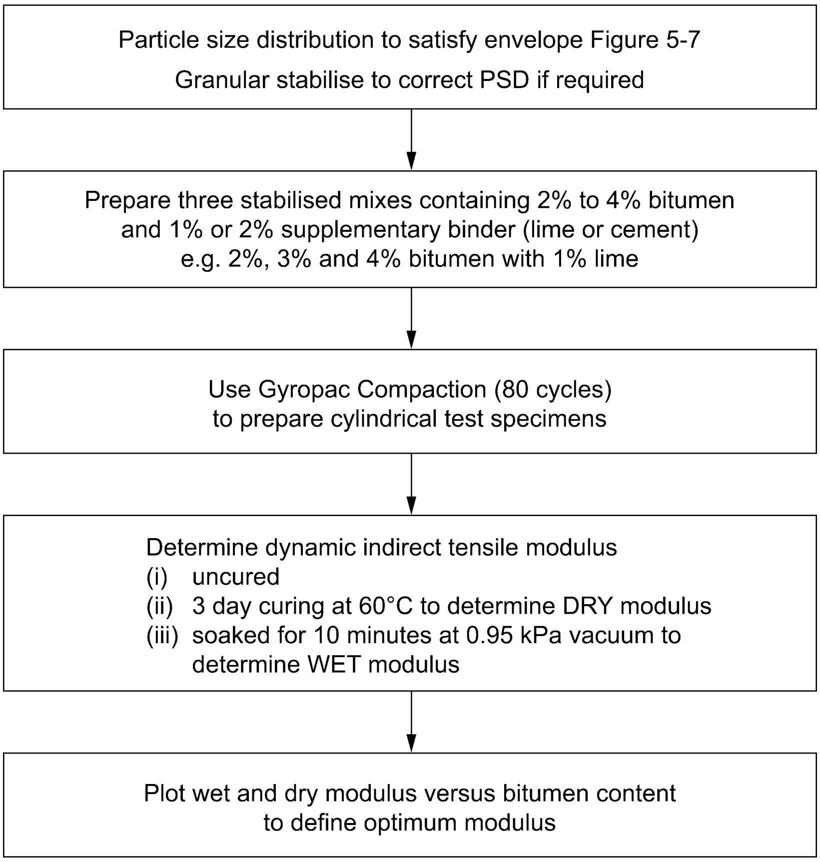
The secondary binder and the bitumen are mixed at the same time to achieve the required mix performance.

The general materials mix design for bituminous binders is shown in Figure 5.10.

In using this procedure, the following notes are pertinent:

- When selecting a bitumen for foaming, the expansion ratio should not be less than 15 times and have a half-life of 30 to 45 seconds. The 2% requirement for lime is a practical limit in the field. No lime may be required for lightly-trafficked roads if the PI of the material is very low. The requirement for no lime should be verified in the laboratory.
- The two approaches adopted for sample preparation are the Gyropac (80 cycles) or Marshall (50 blows). The Gyropac approach is a refinement of the work conducted by Maccarrone *et al.* (1994). Work conducted by Jones *et al.* (2000) has shown that the pavement material may be sensitive to the type of compaction. Where over-sized particles are found in the sample it is recommended that a 150 mm diameter mould be used.

Figure 5.10: Procedure for the design of bituminous stabilised materials



- Dynamic indirect tensile modulus testing is carried out on three samples with the average value reported. As the test is conducted in indirect tensile mode, it does provide a correlation with the resilient modulus of the sample material. The samples are oven cured at 60°C for 3 days and tested M_{dry}. After the samples are tested they are then soaked in water prior to testing to give M_{wet}. Two methods may be used for soaking the sample: submerged under water for 24 hours, or in a vacuum chamber for 10 minutes. The set that is uncured and tested just after compaction using the MATTA should have a modulus exceeding 700 MPa to ensure that the pavement can be opened to traffic after trimming.

- Testing should be conducted with the ratio of $M_{wet} / M_{dry} \geq 0.5$ because bituminous binders will not cure at excessive moisture contents. The moisture content needs to dry back to below 'optimum' for the material to gain strength, particularly for 'cut-back' and emulsified bitumen.

- On large projects additional strength testing may be undertaken including flexural fatigue and creep.

6. Laboratory Tests Associated with Stabilisation Mix Design

6.1 Particle size distribution (PSD) plasticity, reactivity

These standard classification tests identify the nature of the material to be stabilised in terms of the type of PSD – whether uniform, gap graded, single sized, fine or coarse – and also the presence of oversize stone. The suitability of the PSD as a granular material can be associated with standard PSD envelopes for granular pavement materials and also with the requirements of specific binders (see Table 4.6).

Plasticity and reactivity (seasonal heave and shrinkage) are generally associated with the need for their effect to be reduced by the addition of lime, chemical binders or by granular stabilisation.

6.2 Compaction

Laboratory compaction methods on stabilised materials are used for the following purposes:

- determination of MDD and OMC
- preparation of laboratory test specimens
- evaluation of compaction aid binders.

6.2.1 MDD and OMC determination

The laboratory compaction test determines the maximum dry density and optimum moisture content of the stabilised material from which laboratory test specimen density and moisture content are selected and/or compaction specification levels are set for the construction works.

For granular modification or the use of cementitious and chemical binders with granular materials dynamic compaction is adopted either using standard or modified compactive effort. For subgrade stabilisation, generally standard compaction is adopted however modified compaction may be specified in consideration of Authority protocols and the type of compaction equipment proposed.

For materials stabilised with bituminous binders gyratory compaction (80 cycles) or dynamic (Marshall) compaction is generally adopted however this test only determines maximum density as there is no concept of OMC.

6.2.2 Preparation of compacted test specimens of stabilised material for mix design

The preparation of compacted test specimens in the laboratory is used in the mix design process to determine the most appropriate type and quantity of binder required to achieve the type of stabilised material. In addition, mechanistic design parameters may be obtained from which the structural design of the pavement is based. It is therefore essential that the type of compaction adopted in the laboratory be specified in the works.

With in situ stabilised materials the use of modified dynamic compaction techniques in the laboratory may result in adverse changes in PSD due to breakdown of soft aggregates. This document recommends that standard compactive effort be used in these circumstances. Alternately pre-treatment methods to simulate field alteration may be undertaken using RTA test protocols T102 or T103.

For materials stabilised with bituminous binders gyratory compaction (80 cycles) or dynamic (Marshall) compaction is generally adopted.

6.2.3 Evaluation of ‘compaction aid’ binders

Compaction aid chemical binders increase MDD and/or reduce OMC or act as wetting agents. Either standard or modified compactive effort can be used in this circumstance to determine the shift in OMC and MDD provided by the binder.

6.3 Capillary rise

Capillary rise 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 to discriminate between binder types or contents for a specific pavement material. In addition, current research is studying the rate of capillary rise as well as the absolute value.

The Australian Standard test for capillary rise also determines the absorptivity and the swell of the specimen, thus providing additional data.

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.

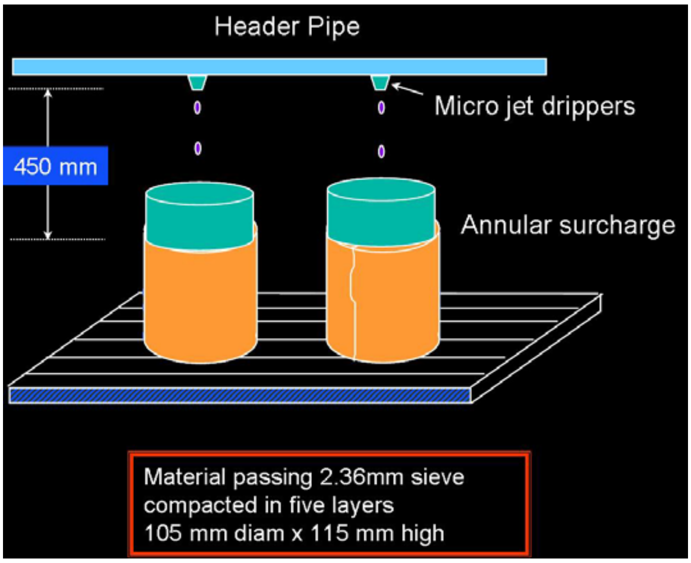
Remoulding a soil or pavement material in the laboratory may remove structural discontinuities, which, in the field, would greatly accelerate infiltration.

6.4 Vertical saturation

This is an empirical test used to determine the effectiveness of a binder by dripping water onto a compacted specimen of stabilised material (see Figure 6.1). The effectiveness of the binder is judged by monitoring the time for the specimen to collapse under an annular surcharge.

The method has been developed with particular reference to the evaluation of the suitability of various types of chemical binders.

Figure 6.1: DTEI SA saturation test



6.5 Lime demand

The aim of the lime-demand test is to identify the quantity of lime needed to satisfy cation exchange by reaching a pH level (i.e. alkaline level of pH 12.4) to produce long term reactions. Some soils 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 will use quicklime. Therefore, the application rate of quicklime needs to be converted from the rate of hydrated lime used in the testing.

6.6 California bearing ratio (CBR)

This is the most commonly used test associated with the design of granular pavements and subgrades. Tests are undertaken under a condition of both moulded moisture content (generally OMC) and after four day 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.

In the context of stabilised materials the test is most commonly associated with granular stabilisation of pavement materials and subgrade soils.

6.7 Unconfined compressive strength (UCS)

This is the most commonly used test for the determination of the strength associated with stabilised materials as it is linked to the type of stabilisation category, viz. modification or bound as detailed in Table 3.1.

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).

There are also useful relationships between UCS and resilient modulus for use in mechanistic pavement design, viz.:

$$E_r \text{ (MPa)} = f \times \text{UCS (MPa)}$$

where ‘f’ is a constant between 1000 and 1500 depending upon the quality of the unstabilised pavement material.

6.8 Resilient modulus

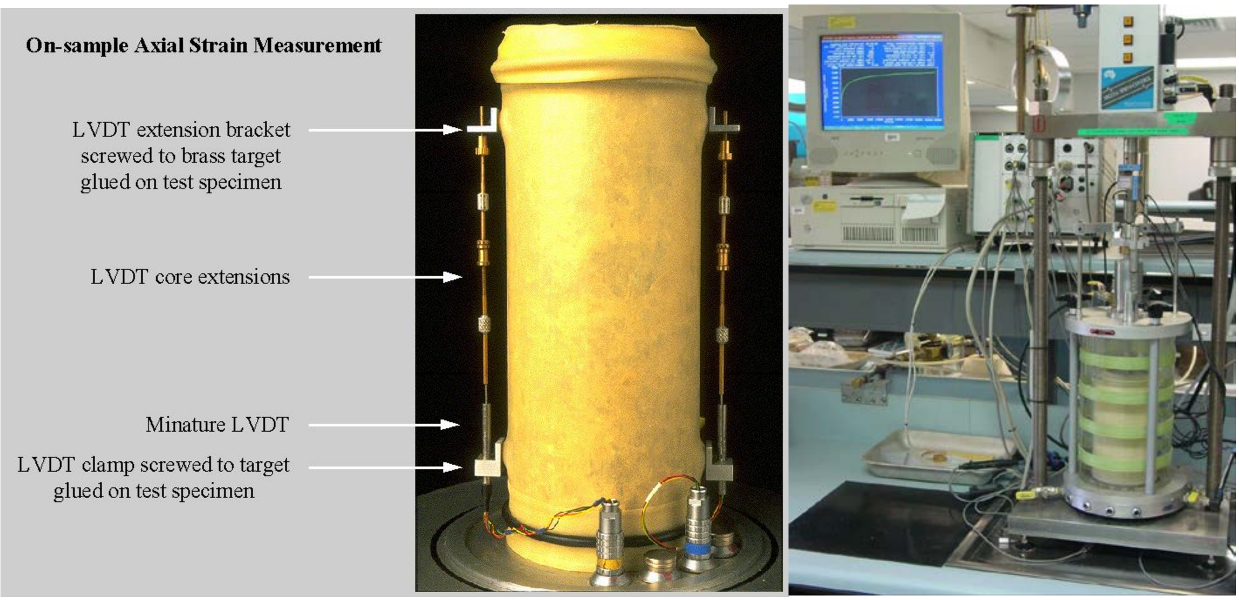
The determination of resilient modulus for stabilised materials is dependent upon the fatigue relationships used in design, viz.:

- granular stabilisation: determined from dynamic confined triaxial compression tests or UCS relationships
- modified stabilisation: determined from dynamic indirect tensile tests or UCS relationships
- bound stabilisation: determined from dynamic flexural tests or dynamic confined triaxial compression tests or UCS relationships.

The types of resilient modulus determination are:

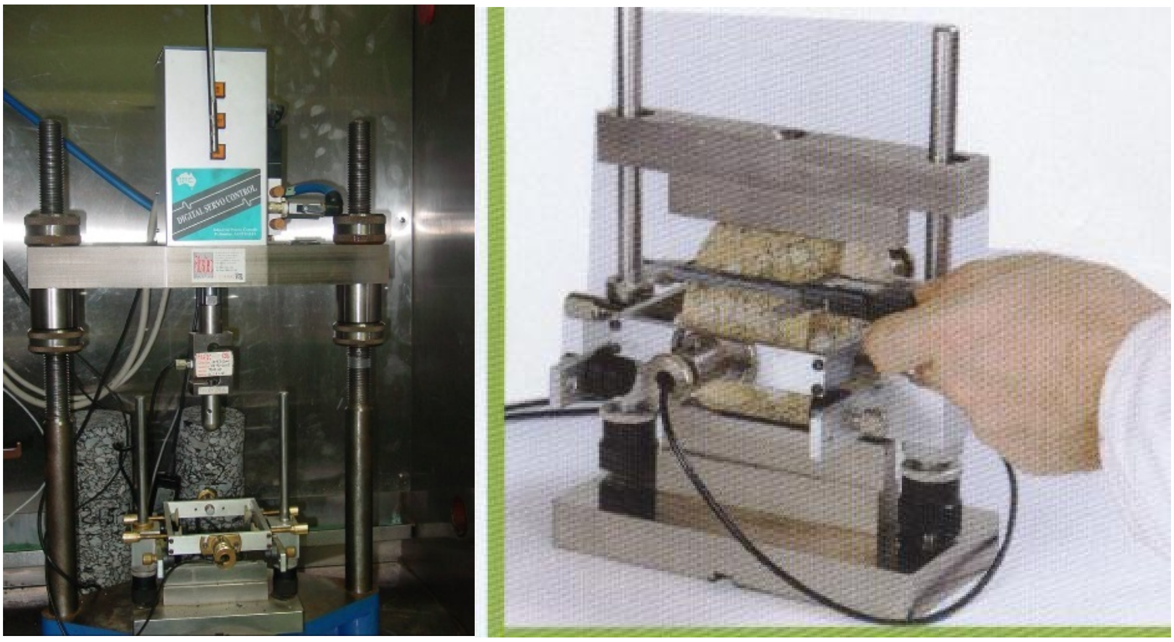
- Dynamic confined triaxial compression: determined using the UMATTA (Unbound MATerials Testing Apparatus) using ‘on sample’ or ‘off sample’ strain measurement (Figure 6.2).

Figure 6.2: Compressive resilient modulus test



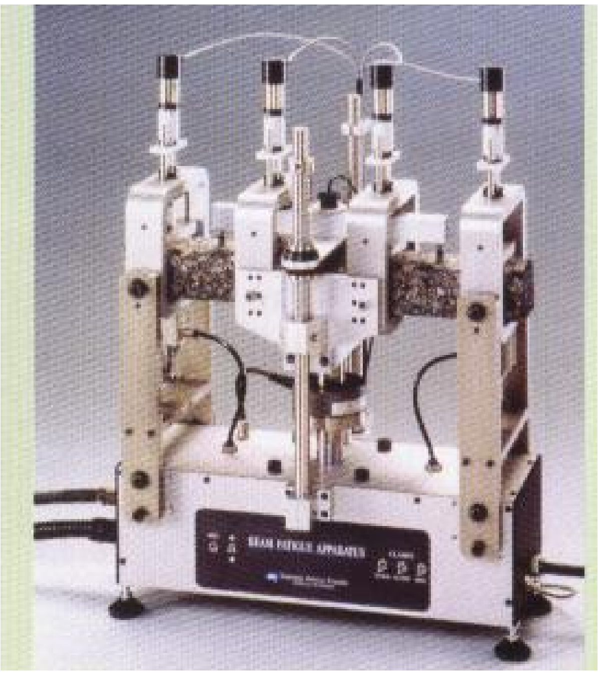
- Indirect tensile modulus: determined using the MATTA (MATerials Testing Apparatus) and carried out in accordance with AS2891 (Figure 6.3).

Figure 6.3: Dynamic indirect tensile modulus test



- Flexural resilient modulus: determined using the two-point loading flexure beam for all bound materials (Figure 6.4).

Figure 6.4: Dynamic flexural modulus test



Care must be exercised in adopting the test results into mechanistic design procedures and specialist advice from the testing authority should be sought.

6.9 Fatigue characterisation

The long term tensile fatigue performance of stabilised materials can be determined from either the dynamic flexural test or more commonly the dynamic confined trial compression test. This testing is undertaken at varying moisture content and density conditions to induce fatigue failure after many thousands of cycles.

6.10 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. Materials stabilised with bitumen emulsion or foamed bitumen also need to be compacted within a certain time to ensure uniform distribution of the bitumen binder. 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 is:

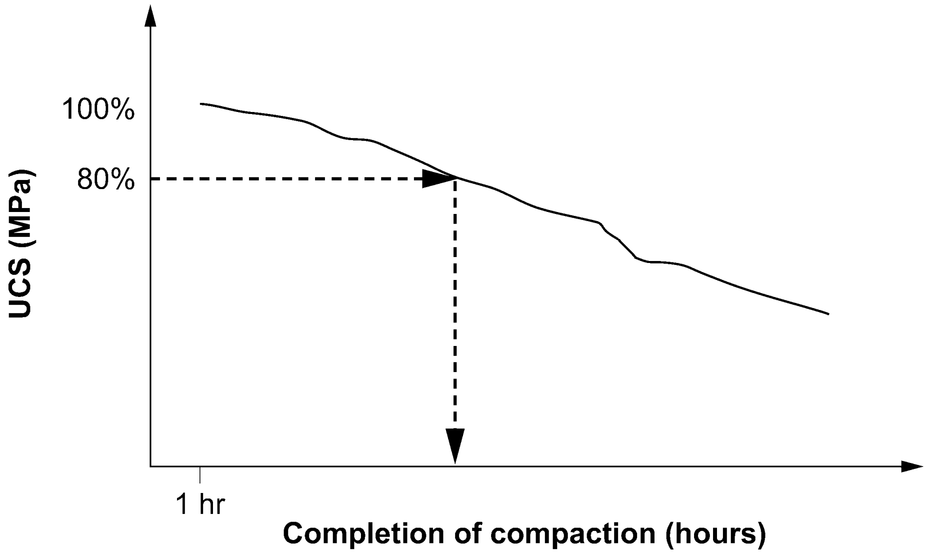
> *The nominated working time for any proposed mix is the time measured from the commencement of the addition of the binder to the compaction of the stabilised material, which corresponds to 80% of the mean value of three determinations of UCS, for samples compacted one hour after incorporation of the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

Working time is determined from preparing loose stabilised material sealed in an airtight container and cured for the designated time at which 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 6.5.

Figure 6.5: Diagrammatic representation for working time of cemented materials



No method for the determination of working time is yet available for bituminous binders and bituminous binders with secondary binder additions.

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.

6.11 Erodibility

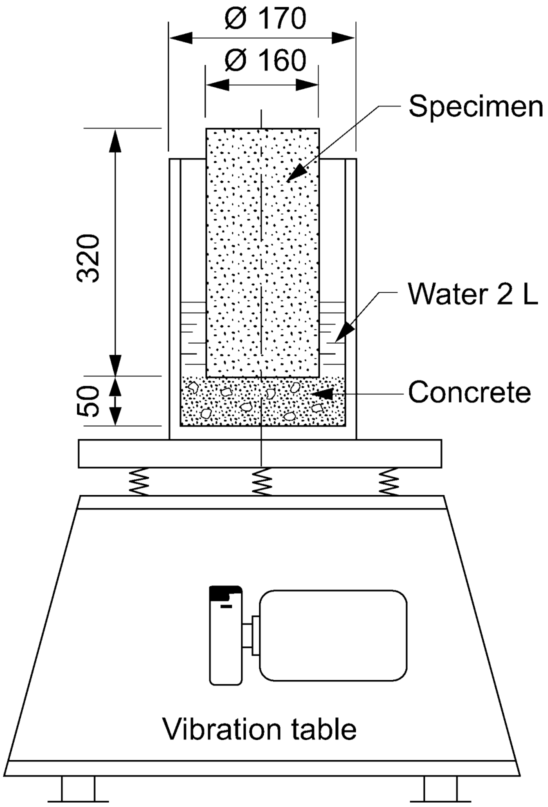
Erodibility is generally described as a loss of material; it is therefore reported in units of gram/minute to the nearest whole number.

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 performance of the pavement in the field. 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 known erosion test method practiced as a design tool in Australia is the RTA NSW Test Method T186 (Howard 1990; RTA NSW 1994) (Figure 6.6). Other test methods exist and have been used in Australia and internationally (see Jameson 1995). The RTA 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.

Figure 6.6: RTA NSW erodibility test



6.12 Leaching

Leachate concentration, particularly where chemical binders are being considered, can be determined in accordance with AS4439.3. However, this test method is based on a loose sample and would result in a conservative test result given that the material in any pavement layer would be compacted in the road.

7. Example Calculation for Granular Stabilisation

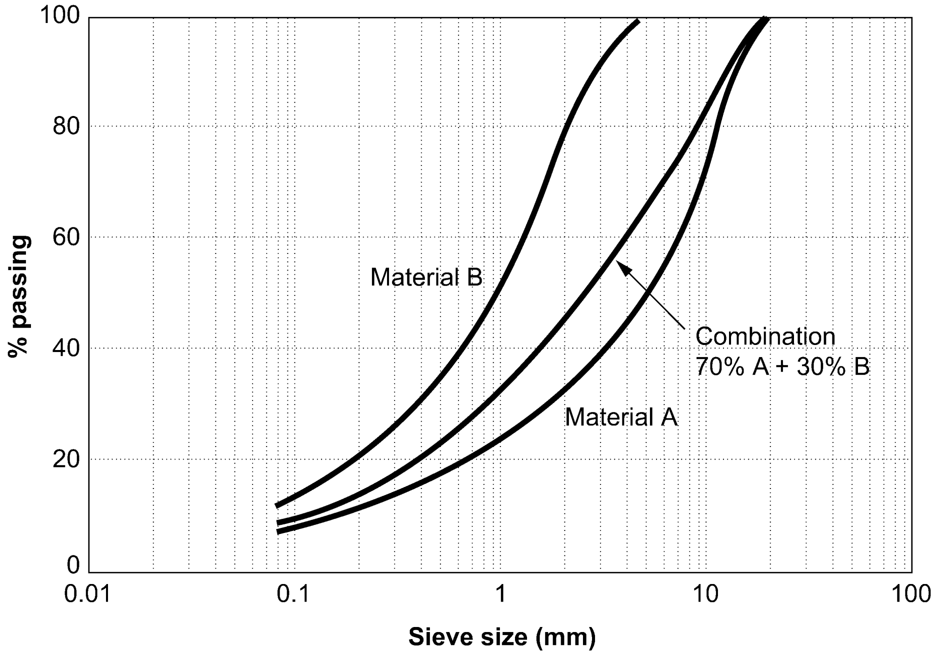
A worked example is shown below in which 70% material ‘A’ (coarse product) is combined with 30% material ‘B’ (fine product) to achieve a combination Particle Size Distribution to meet a typical basecourse specification. Simple spreadsheets can be developed to perform these analyses as shown in Table 7.1.

An example combination particle size analysis is shown in Figure 7.1.

Table 7.1: Example calculation – blending two materials

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

Figure 7.1: Example combination particle size analysis



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Austroads' **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

4D

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