

# **STABILISATION FOR NEW ZEALAND ROADS: A REVIEW**

**Transit New Zealand Research Report No. 64**



# **STABILISATION FOR NEW ZEALAND ROADS: A REVIEW**

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## **EXECUTIVE SUMMARY**

### **1. Introduction**

A review has been made of the international literature that was available up to 1996 on stabilisation for road pavements. The review also draws on New Zealand experience and practices in stabilisation.

This review indicates that stabilisation has provided economic and durable road pavements over more than three decades of use in New Zealand. Highlighted is the New Zealand approach of using relatively low levels of additives when stabilising with lime or cement to provide flexible stabilised pavements which have minimised potential for cracking.

### **2. Properties of Stabilised Soils**

Stabilisation offers the ability to improve the materials used in roads. These improvements include increased strength, and the ability to retain most of their strength when wet. Stabilisation with low rates of added lime or cement can decrease resistance to frost, while higher rates can increase a soil's resistance to frost.

### **3. Stabilisation as Option for Construction or Rehabilitation**

The many types of stabilisation include using heat, adding and mixing in aggregate, and adding and mixing in additives. The additives discussed in the review include lime (either as quicklime or hydrated lime), cement, bitumen, other chemicals, and proprietary products. As lime- and cement-stabilisation are mainly used in New Zealand, the review concentrates on these.

Stabilisation should always be considered as an option for the construction or rehabilitation of roads. It can be used in rural or urban areas. It can be used in any of the structural layers of the pavement, but requires a surfacing to withstand the traffic wear on all except very lightly trafficked roads. The use of lime- or cement-stabilisation for the repair of smaller areas of existing pavements, and for unsealed roads, is also included.

Stabilisation can be used to provide lower construction costs, reduced long-term maintenance costs, or longer lasting roads. Stabilisation also offers environmental benefits by allowing the use of materials already in place on the road, which reduces trucking needs, or by allowing the use of otherwise sub-standard or even waste materials.

When considering stabilisation, the advantages and disadvantages should be recognised and understood, and they should be related to the actual road situation and to the problems that are to be addressed or to the benefits that are sought.

#### **4. Testing for Stabilisation**

Testing of the materials to be used for road stabilisation is required to ensure that the advantages obtained from stabilisation can be achieved. The tests can be staged as preliminary testing, then as more comprehensive laboratory testing. Testing should indicate which type of stabilisation (e.g. lime, cement, or bitumen) is appropriate, and the required rate of the additive. The prime factor governing the test results for the selection of the type of stabilisation is the soil type. Lime is best for clays, silty clays or clayey gravels. Cement is best for gravels, sands, or silts, or a combination of these, but may be used in clays although mixing with heavy clays may be impractical. Bitumen needs clean well-graded or open-graded gravels or sands.

#### **5. Construction of Stabilised Layers**

Construction of lime-stabilisation and cement-stabilisation are covered in some detail in the review. While the construction of lime-stabilisation needs plenty of water and is very tolerant of wet conditions, both lime- and cement-stabilisation should be carried out when the soil is warm. The emphasis in the review is on mix-in-place stabilisation (i.e. on the road) using lime or cement. For this method, powerful purpose-built rotary mixers are usually used. Ancillary equipment to complement these include water spray truck(s), bulk lime or cement spreaders, graders and rollers. Expertise is required to supervise the construction. Aspects of safety and quality control are given.

Mixing materials for bitumen-stabilisation construction can be in a central plant or mix-in-place, but the usual New Zealand practice is to mix in a central plant. Depending on the soil type and form of the bitumen, specialist equipment may be required for the mixing. Comment on other additives for stabilisation includes the use of proprietary products.

#### **6. Future Scope**

The review recognises that there is scope in New Zealand for greater use of stabilisation, particularly using lime, cement or bitumen. This potential includes pavement construction and rehabilitation, as well as pavement repairs and unsealed roads.

While the equipment, science and understanding of stabilisation has developed over recent decades, a certain amount of art remains, with much to be learnt and refined. This is the challenge for the future.

## **ABSTRACT**

A review has been made of the international literature that was available up to 1996 and of New Zealand experience and practices for the stabilisation of materials for use in road pavements. The review indicates that stabilisation has provided economic and durable road pavements, and highlights the New Zealand approach of using relatively low levels of additives to provide flexible stabilised pavements.

The main additives currently used for stabilisation of New Zealand roads are lime or cement. The review concentrates on these and includes coverage of the reaction mechanisms, changes in properties caused by stabilisation, and suitable soils and testing. Construction is covered in some detail.

The use of lime or cement for pavement repairs, and in unsealed roads, is covered. These applications of stabilisation have potential for much wider use. Bitumen-stabilisation is also covered in broad terms, and again its potential for greater use is recognised. Comment is included on other additives for stabilisation, including the use of proprietary products. Aspects of safety and quality control are given.



# **PART A**

# **INTRODUCTION**

## **1. INTRODUCTION**





## **1. INTRODUCTION**

### **1.1 Scope**

This report reviews the use of stabilisation for the pavements of public roads in New Zealand. Carried out in 1995 and 1996, it reviews processes by which additives or heat are used to enhance the properties of natural subgrade and/or pavement materials.

Over thirty year's experience of stabilisation has now been gained on New Zealand roads. This experience has shown stabilisation to be a versatile and powerful technique in the construction and rehabilitation of roads. While economics are often the deciding factor in selecting stabilisation, the benefits are far wider than just those of the pavement being stabilised. These include conservation of aggregate resources, and reduced requirement for the trucking of materials with its attendant wear on other roads and emissions of carbon dioxide.

A number of methods of stabilisation are available, usually using additives to enhance the properties of subgrade or of pavement materials. Many types of stabilisation have been used in New Zealand including heat stabilisation, bitumen stabilisation, and stabilisation using proprietary products. By far the most widely used forms of stabilisation for New Zealand are lime- and cement-stabilisation. Accordingly most of this review will be devoted to stabilisation of subgrade and/or pavement materials with lime and/or cement. Other methods of stabilisation are covered briefly.

While this review offers guidance, it would be wrong to approach stabilisation with a fixed set of rules, because each job is different and so should be approached differently. Flexibility and expertise is needed to consistently obtain good results (BR Webster, pers.comm. 1994).

With all forms of stabilisation adequate compaction, good pavement drainage, and good design remain as vitally important as for all other types of pavement construction. While stabilisation is a powerful and tolerant technique that offers economies in many situations, it should be used wisely. It should not be used as a substitute for good practice.

### **1.2 Purpose of Stabilisation**

Stabilisation is commonly considered as a means of rectifying a deficiency in a soil material. Stabilisation is also a means by which the engineer can improve a situation by altering the properties of the materials to optimise their benefits.

Stabilisation therefore should be extended beyond its use for remedial treatment of materials, to be considered as a general tool for pavement design, construction and maintenance, and to widen the range of materials that can be applied to pavement construction (NAASRA 1986).

Table 1.1 Application of stabilisation (after NAASRA 1986).

| Type of Stabilisation   | Process  | Effects   | Applicable Soil Types*  |
|---|--|---|---|
| Mechanical  | Mixing of two or more materials to achieve planned particle size distribution  | Changes soil strength, permeability, volume stability.  | Poorly graded soils, granular soils with deficiency in some particle sizes.   |
| Lime  | Chemical alteration of clay minerals. Cementitious interparticle bonds develop but rate of development is slow, relative to cement | Improves handling properties of cohesive material. <ul style="list-style-type: none"> <li>• Low additive contents: decreases susceptibility to moisture changes, improves strength;</li> <li>• High additive contents: also increases modulus and tensile strength.</li> </ul>                  | Suitable for cohesive soils. Requires clay components in soil that will react with lime.<br><br>Organic material will retard or prevent effective reactions. Sulphates can cause swelling.  |
| Cement  | Cementitious interparticle bonds are developed   | <ul style="list-style-type: none"> <li>• Low additive contents: decreases susceptibility to moisture changes, improves strength;</li> <li>• High additive contents: increases stiffness and tensile strength significantly;</li> <li>• Decreases susceptibility to moisture changes.</li> </ul> | Not limited; some deleterious components (organic, sulphate etc.) can retard or prevent effective cement reactions. Suitable for granular soils but inefficient in predominantly one sized materials. Expensive and difficult to mix in cohesive soils. |
| Lime plus Pozzolan, e.g. Fly ash, Pulverised blastfurnace slag  | Lime and pozzolan modifies particle size distribution and develops cementitious bonds  | Generally similar to cement but rate of gain of strength similar to lime; also improves workability.  | As for cement stabilisation; can be used when soils are not reactive to lime.   |
| Bitumen and Tar (including foamed and high impact bitumen, cutback bitumen, tar, or bitumen emulsion) | Agglomeration of fine particles  | Waterproofs; also improves interparticle cohesive strength.   | Applicable to granular low cohesion, low plasticity materials.  |

\* Use is always constrained by the properties of raw materials.  
NAASRA - National Association of Australian State Road Authorities.

The properties that are usually altered by stabilisation include:

- strength
- stiffness
- volume stability
- sensitivity to changes in moisture content
- permeability
- durability
- workability

Most stabilisation treatments will alter many or all of these properties at the same time, as indicated in Table 1.1. The correct use of stabilisation requires identification of the properties that will be altered, and whether each alteration is an improvement or otherwise.

### **1.3 Types of Stabilisation**

Many methods of stabilising road materials are available, but the most common types are:

- lime
- cement
- bitumen
- mechanical
- chemical

Before deciding the type of stabilisation method to use, adequate laboratory testing should be carried out on the materials to be used on the road.

### **1.4 Stabilisation of Different Soil Types**

The applicability of the different types of stabilisation to a range of sizes of soil particles is indicated in Figure 1.1 (from Ingles and Metcalf 1972). A variation on this approach is shown in Figure 1.2 (from NAASRA 1986) in which the generalised suitability of the types of stabilisation is based on the percentage of material passing the 75µm sieve and the plasticity index of the soil.

Soils have a wide range of properties. Consequently the reaction of a specific soil with any particular stabiliser cannot be determined by simple observations, or by defining the soil type. Instead, some form of laboratory testing should be carried out to determine the response of the particular soil with the stabilising agent(s) being considered.

# A. INTRODUCTION

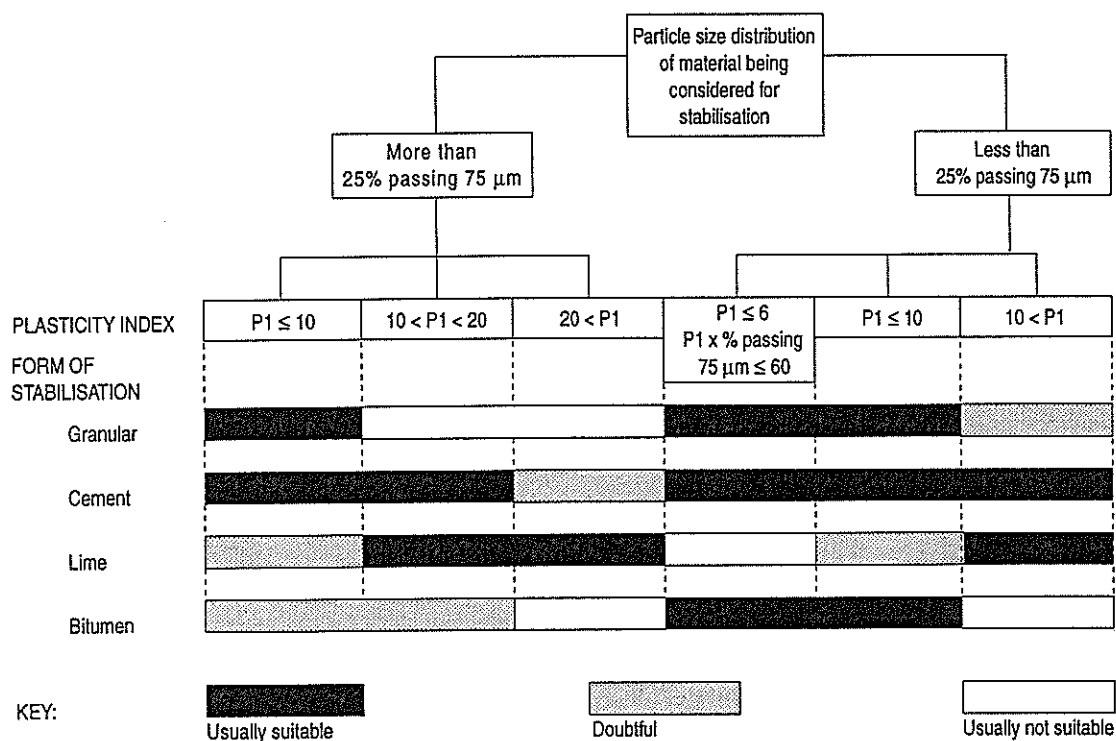
Figure 1.1 Applicability of stabilisation methods based on soil particle size (from Ingles and Metcalf 1972).

| Designation                      | Fine clays | Coarse clays | Fine silts | Coarse silts | Fine sands | Coarse sands |
|----------------------------------|------------|--------------|------------|--------------|------------|--------------|
| Soil, particle size (mm)         | <0.0006    | 0.0006—0.002 | 0.002—.01  | 0.01—.06     | 0.06—.4    | 0.4—2.0      |
| Soil, volume stability           | V. poor    | Fair         | Fair       | Good         | V. good    | V. good      |
| Type of stabilisation applicable | Lime       |              |            |              |            |              |
|                                  | Cement     |              |            |              |            |              |
|                                  | Bitumen    |              |            |              |            |              |
|                                  | Granular*  |              |            |              |            |              |
|                                  | Heat       |              |            |              |            |              |

Range of maximum efficiency Effective, but quality control may be difficult

\*i.e. improvement of soil grading by mixing-in gravels, sands or clays as appropriate

Figure 1.2 Applicability of stabilisation methods based on plasticity index and percent passing 75µm sieve (from NAASRA 1986).

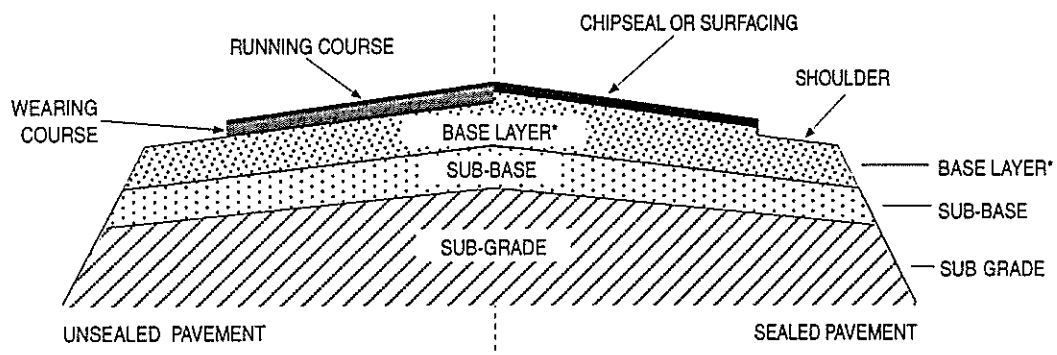


## 1.5 Stabilisation of Different Pavement Layers

When the upper portion of a subgrade is stabilised in-situ by lime and/or cement the resulting layer is referred to as sub-base, and the original subgrade becomes the underlying material (NRB 1989).

Terminology for the pavement layers is shown in Figure 1.3. The pavement layers, with the benefits to each from stabilisation, are described in Sections 1.5.1-1.5.3.

Figure 1.3 Terminology applied to the layers of a road pavement (after Ingles and Metcalf 1972, Ferry and Major 1991).



\*Graded granular material in base layer referred to as basecourse.

### 1.5.1 Subgrade

Stabilisation of the subgrade can offer these benefits:

- Provides a working platform to allow access for construction.
- Reduces the required thickness of the overlying pavement layers.
- Improves the quality of the overlying pavement layers by allowing better compaction.
- Reduces or eliminates intrusion of the subgrade material into the sub-base and basecourse.
- Provides a relatively moisture-tolerant support for the pavement.

These benefits can result in cost and construction time advantages. The cost advantages are likely to extend to future maintenance and rehabilitation cost savings, quite possibly well beyond the "20 year design life" of the pavement.

#### **1.5.2 Sub-base**

Stabilisation of the sub-base, using either imported or in-situ materials, can offer these benefits:

- Provide a working platform over a weak subgrade.
- Reduce the required thickness of the base layer.
- Reduce or eliminate intrusion of subgrade material into the base.
- Provide a relatively moisture-tolerant pavement layer.

#### **1.5.3 Base**

Stabilisation of the base layer can offer these benefits:

- Compensate for deficiencies in the basecourse aggregate.
- Reduce the sensitivity of the basecourse to moisture.
- Increase tolerance of the basecourse to heavy axle loads, high tyre pressures, or "super single" tyres.
- Increase the life of the wearing surface by reducing pavement deflection or "punching-in" of the sealing chip.

Bitumen adhesion to fine-grained cement-stabilised and lime-stabilised bases may be a problem and requires special attention.

#### **1.5.4 Wearing Course**

Generally cement, lime and bituminous stabilisation will not provide a surface that is sufficiently resistant to traffic for their use as a permanent wearing course on sealed roads. However they may be suitable as a temporary wearing course for repairs on sealed roads, or as wearing courses on unsealed roads.

For unsealed roads, stabilised bases or wearing courses are usually more wear-resistant than unbound aggregate but they will still require maintenance. As fine cracking is less of a concern for unsealed roads the stabiliser content may be increased slightly to provide increased wear resistance.

#### **1.5.5 Shoulders**

When a stabilised basecourse is used it is normal to extend the stabilised layer well into the shoulders. Once cemented the surface of such shoulders can be difficult to maintain or repair, so sealing should be considered.

Apart from surface maintenance, care is needed if using thick layers of stabilised material in shoulders where the adjacent sealed pavement is unstabilised, as the stabilised shoulders may prevent moisture from moving out of the unstabilised material.

## **1.6 New Zealand Experience**

Although first used in New Zealand in 1943, cement-stabilisation for roads did not become widespread until the mid 1960s. Lime-stabilisation for roads became popular later, in the 1970s (Plate 1).

Most of these early stabilised pavements performed very well. However for a period around the mid 1960s, extensive cracking became evident in a number of stabilised pavements (NRB RRU 1979). This cracking was usually related to cement-stabilisation using heavier application rates that produced strongly cemented materials.

This experience and the widespread use of thin seal coats have both led to the current situation (1995) in New Zealand where light levels of stabilisation using relatively low percentages of lime or cement are preferred.

Use of stabilisation has tended to be concentrated in some areas of New Zealand more than others. A 1985 survey of stabilisation practices in New Zealand (NRB RRU 1985) showed:

- 41% of roading authorities had used some form of stabilisation over the past five years.
- Lime was the most commonly used stabilising agent, with cement being the next most widely used (Table 1.2).
- The bulk of New Zealand's stabilisation works was carried out in the northern and eastern areas of the North Island.
- The "mix in-place" method was the most common form of stabilisation.
- Most roading authorities using lime and/or cement considered they had achieved "variable" or "excellent" results.
- For stabilising agents other than lime or cement the success rate was either "excellent" or "poor" (i.e. worked well or not at all).
- For unsealed roads, reductions in rutting, dust and aggregate loss were seen as significant improvements after stabilisation.

Table 1.2 Some annual tonnages of lime and cement used for pavement stabilisation in New Zealand (Burt 1984, Bourne-Webb 1992).

| Agent  | 1983/84<br>Tonnes* | 1989<br>Tonnes† | 1990<br>Tonnes† |
|--------|--------------------|-----------------|-----------------|
| Lime   | 13,500             | 28,000          | 21,000          |
| Cement | 3,992              | 7,000           | 6,000           |

\* 1983/84 construction season

† Calendar year

## 1.7 Explanation of Parts B and C of Review

By far the most common forms of stabilisation used in New Zealand are cement and lime. Part B of this Review is devoted to cement-stabilisation and lime-stabilisation.

Other stabilisers (e.g. bitumen) are also available. These are considered in Part C of this Review.

A Glossary of terms used in this report is given as Appendix 5 in the report.



1. *Introduction*

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Plate 1. Spreading fine aggregate during mechanical stabilisation, to be followed by cement-stabilisation.



Plate 2. Stabilisation train: sequence (right to left in photo) is bulk cement spreader, water cart, rotary mixer, roller.





# **PART B**

## **LIME AND/OR CEMENT STABILISATION**

- 2. PROPERTIES OF STABILISED SOILS**
- 3. MATERIALS**
- 4. SELECTING THE STABILISATION OPTION**
- 5. TESTING FOR STABILISATION**
- 6. DESIGN OF STABILISED PAVEMENTS**
- 7. CONSTRUCTION OF STABILISED LAYERS**
- 8. CONSTRUCTION OF LIME-STABILISATION**
- 9. CONSTRUCTION OF CEMENT-STABILISATION**
- 10. STABILISATION IN PAVEMENT REPAIRS**
- 11. STABILISATION IN UNSEALED ROADS**
- 12. QUALITY CONTROL OF CONSTRUCTION**



## **2. PROPERTIES OF STABILISED SOILS**

### **2.1 Introduction**

For cement- or lime-stabilisation the cement (generally ordinary Portland cement (OPC)) or the lime is mixed with the soil. The cement-soil or lime-soil mixture then reacts to form a material that has different properties to the original material.

The mixing may be either in a central plant, or in-place on the road. The soil may be any material ranging from a heavy clay to a clean aggregate.

### **2.2 Soil Reactions**

#### **2.2.1 Lime-Soil Reactions**

The reaction of soil with lime is normally divided into two distinct phases:

##### **(1) Flocculation**

As soon as the lime comes in contact with the clay in the soil, particles in the soil undergo cation-exchange. The calcium cations from the lime displace cations such as sodium or hydrogen from the clay. This modifies the surface properties of the clay particles which then attract each other, i.e. flocculate. It also reduces the ability of the clay particles to attract and bind water particles (Arabi and Wild 1989).

While this soil clay-lime reaction can be immediate the overall process is slowed because the available lime has to diffuse through both the soil system and the initial cementitious products to the reaction sites.

This cation-exchange process has a number of effects on the soil (Figure 2.1):

- Increased volume stability related to the reduced ability of the clay particles to attract and bind water,
- Increased permeability related to the flocculation providing larger voids in the soil structure,
- Increased plastic limit,
- Decreased plasticity index,
- Increased workability relating to the flocculation and reduced plasticity,
- Increased strength,
- Altered compaction properties,
- Decreased frost resistance relating to the increased permeability.



## (2) Cementation

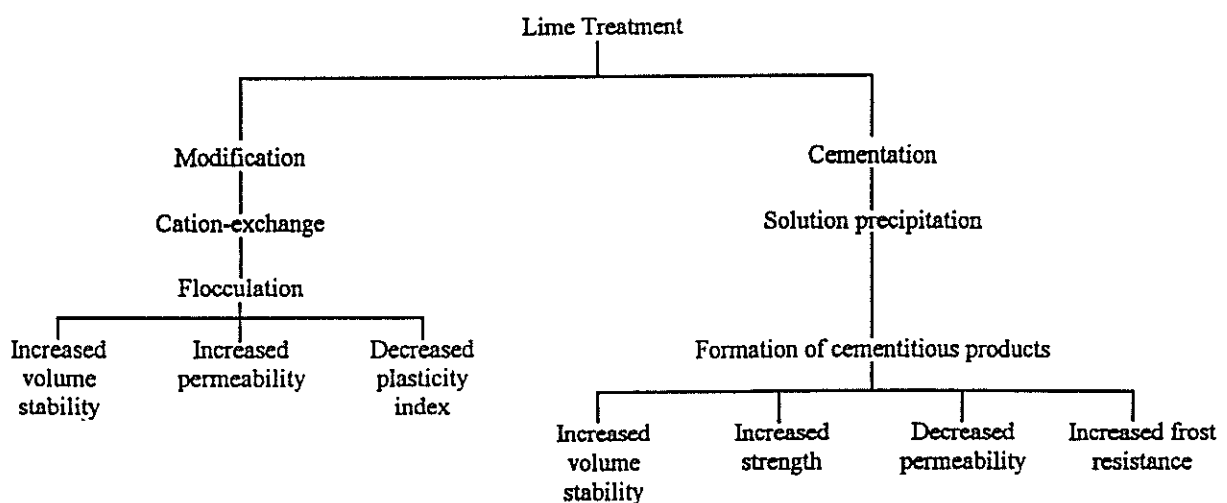
In addition to flocculation another very slow chemical reaction occurs. While there is some debate on the mechanism details, it is generally accepted that a crystallisation or solution precipitation mechanism occurs whereby calcium (from the lime) and aluminosilicates are slowly taken into solution and re-precipitated as hydrated cementitious products, particularly at the contact points between the clay particles. The principal cementitious product is a calcium silicate hydrate gel which is believed to be the major contributor to the long-term strength gain because it forms an interlocking gel-clay microstructure.

The actual reaction products formed depend on the type of clay, the temperature, the lime content, and the curing time. The rate of formation of the cementitious products is very sensitive to temperature, which makes stabilisation more effective in hot climates. Also the cementation reaction requires moisture, and strength development ceases if the water content falls below a critical level. The rate of cementation is also dependent on soil reactivity which is determined mainly by soil type (Arabi and Wild 1989, Wild and Arabi 1989, Sameshima and Black 1982).

This cementitious reaction has a number of additional effects (Figure 2.1):

- Increased volume stability,
- Increased strength,
- Decreased permeability relative to the flocculated soil: this relates to the cementitious products occupying an increasing portion of the voids in the soil structure,
- Increased frost resistance (see Section 2.5.7).

Figure 2.1 Progressive changes that occur during lime-soil reactions (after Arabi and Wild 1989).



Hydrated lime by itself does not react with water and set. It reacts with the clay soil, and the reaction products impart the strength gains (Gutschick 1978). Consequently addition of too much lime can leave unreacted particles of lime which have no cohesive strength, resulting in a decrease in strength.

### **2.2.2 Cement–Soil Reactions**

Cement itself reacts with water and sets. On adding cement to a soil the primary reactions are the hydration reactions of the cement with the water in the soil. This leads to the formation of cementitious gel material (calcium silicates and alumina hydrates). The cement hydration reactions themselves generate hydrated lime, at a proportion of 30–35% by mass of the cement. A significant proportion of this hydrated lime is generated during the first few days (ICR Brown, Milburn NZ Ltd, pers.comm. 1994), and it can cause secondary reactions.

For granular soils with little or no clay the cementitious gel binds to the soil particles. This is similar to the effect of cement in concrete in which the cement gel fills most of the voids, and in stabilised soils the cement gel fills only some of the voids between the aggregate particles.

For soils containing an appreciable amount of clay the action of cement is different. In this case the particles of cement are large relative to the clay particles. The cementitious gel forms a lattice which tends to enclose agglomerations of clay. The hydrated lime released by these hydration reactions is available for cation-exchange with the clay particles, as for lime-stabilisation (NAASRA 1986, Tait 1981).

Because the cement hydration reactions require moisture, hydration and consequent strength development cease permanently if the stabilised material ever becomes too dry.

The degree to which soil properties are affected depends on a number of factors including the amount of cement, and the nature of the soil. Progressively increasing the cement content progressively changes the properties of the stabilised soil. While the continuity of properties is emphasised, two categories of cement-stabilised material have been defined:

#### **(1) Modified Material**

At low contents (1–3%) of cement (see Section 2.3 of this report), the cementitious bonds are likely to be weak relative to applied traffic and thermal stresses, and may be discontinuous. The modified material is likely to have:

- Reduced moisture susceptibility,
- Increased shear and bearing strength,
- No significant tensile strength,
- Decreased permeability, although permeability may increase in clayey soils.

(2) **Cemented Material**

Increased amounts of cement lead to increasing continuity and strength of the cementitious bonds. In addition increasing amounts of hydrated lime are released allowing cation-exchange with any clay particles they can reach directly, or by diffusion through the soil lumps. The result depends on the soil type and the effects may include development of materials with significant tensile strengths. The cementing reaction has a number of effects:

- Increased shear, bearing and tensile strength,
- Decreased permeability,
- Increased shrinkage in granular soils and decreased shrinkage in clayey soils are likely,
- Increased durability to wet/dry cycles,
- Increased frost resistance.

## **2.3 Degrees of Stabilisation**

From the preceding discussions the properties of a lime-stabilised or cement-stabilised material are seen to vary progressively with increasing lime or cement content:

- Low quantities (e.g. 1–3%, depending on soil type) of lime or cement produce flocculation and may be light bonding to create a material with little or no tensile strength. Such materials are called "modified".
- Medium quantities of lime or cement (e.g. 3–8%, depending on soil type) produce increased bonding and create a material with significant tensile strength. Such materials are called "cemented".
- High quantities of cement (e.g. >8%), when mixed with clean and graded aggregates produce stiff materials with very significant tensile strength. Such materials are called "lean concrete" or "concrete".

The variation in properties imparted by varying the lime or cement contents produces materials that are suitable for different uses. A spectrum of both properties and stabilised materials is available, from a natural soil with no lime or cement at one end to concrete at the other end of the spectrum. Table 2.1 shows the degrees of stabilisation that may be achieved.



## 2. *Properties of Stabilised Soils*

Factors which influence the degree of stabilisation achieved include:

- The type and nature of the soil,
- The amount of lime or cement, although with lime the limit on achievable strength is much less than with cement,
- The degree of hydration of the lime or cement – this relates to the duration of hydration in the presence of adequate water, and the temperature,
- The degree of compaction.

The demarcation between modified and cemented is arbitrary and has been defined (determined at maximum dry density, standard compaction, and moist cured for 7 days) in NAASRA (1986) and NRB (1989) manuals as either:

- 80 kPa tensile strength (indirect tension),
- 800 kPa unconfined compression strength (UCS).

In New Zealand the usual stabilisation practice is to create a stabilised material that is highly modified to lightly cemented, i.e. near the arbitrary modified–cemented boundary shown in Table 2.1.

Table 2.1 Degrees of stabilisation of a lime-soil or a cement-soil mixture  
(adapted from Tait 1981, NAASRA 1986, Hudson and Simmons 1991, Williams 1986; DP Barnard, pers.comm. 1996).

| PROPERTY OF STABILISED SOIL               | DEGREE OF STABILISATION  |          |                        |               |          |
|---|--|----------|------------------------|---------------|----------|
|   | SOIL   | MODIFIED | CEMENTED               | LEAN CONCRETE | CONCRETE |
| Tensile Strength                          | Less than 80 kPa   |          | More than 80 kPa       |               |          |
| Compressive Strength                      | Less than 800 kPa  |          | More than 800 kPa      |               |          |
| Lime Content<br>- Range<br>- Indicative   | <div><div></div><div>INCREASES</div><div>0%3%</div></div>      |          |                        |               |          |
| Cement Content<br>- Range<br>- Indicative | <div><div></div><div>INCREASES</div><div>0%3%8%20%</div></div> |          |                        |               |          |
| Technology Base                           | TECHNOLOGY OF SOILS  |          | TECHNOLOGY OF CONCRETE |               |          |

## 2.4 Appropriate Degrees of Stabilisation for Different Pavement Layers

Not all degrees of stabilisation are suitable for use in all parts of the pavement. For example rehabilitation of a poor quality basecourse to achieve a highly cemented base material carries a high risk of the base cracking, caused by load-induced stresses and/or shrinkage and temperature stresses.

The general applicability of the various degrees of stabilisation to the different pavement layers is shown in Table 2.2 (adapted from Tait 1981).

Table 2.2 Applicability of stabilisation to different pavement layers  
(adapted from Tait 1981).

| PAVEMENT LAYER  | Degree of Stabilisation | Stabilisation Applicable? |
|-----------------|-------------------------|---------------------------|
| SUBGRADE        | Modified                | Yes                       |
|                 | Cemented                | Yes                       |
|                 | Lean Concrete           | No                        |
| SUB-BASE        | Modified                | Yes                       |
|                 | Cemented                | Yes                       |
|                 | Lean Concrete           | Yes                       |
| BASE            | Modified                | Yes                       |
|                 | Cemented                | Yes                       |
|                 | Lean Concrete           | Yes                       |
| WEARING SURFACE | Modified                | No                        |
|                 | Cemented                | No                        |
|                 | Lean Concrete           | No                        |
|                 | Concrete                | Yes                       |
| REHABILITATION  | Modified                | Yes                       |
|                 | Cemented                | No*                       |
|                 | Lean Concrete           | No*                       |

\*unless overlaid or treated with other specific measures to slow the rate of reflective cracking.

## 2.5 Effect of Degree of Stabilisation on Engineering Properties

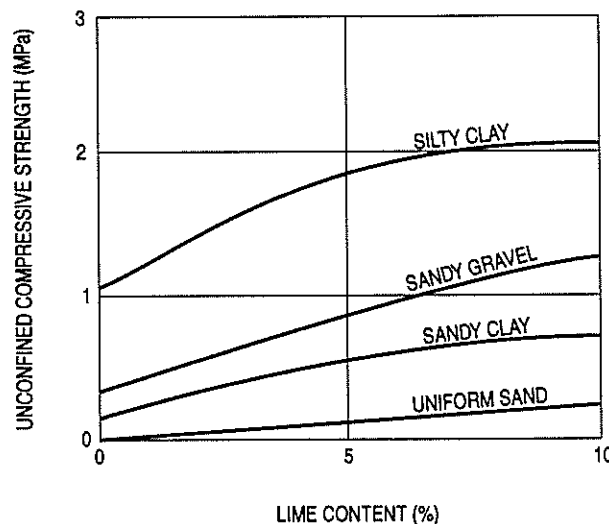
The general engineering properties of a stabilised soil are outlined in this Chapter 2. They are illustrative only, and laboratory testing of the actual soil will be necessary before commencing any stabilisation project.

### 2.5.1 Strength

Generally strength increases with increasing lime or cement content for any given soil (Figure 2.2 from Metcalf 1977). With lime the addition of more lime above a certain percentage results in a strength decrease.

For modified soils the shear and bearing strength increases with increased lime or cement content with little to no increase in tensile strength. With cemented soils a further increase in shear and bearing strength occurs, with development of significant tensile strength and increases in stiffness.

Figure 2.2 Generalised effect of lime content on the UCS of four different soils (from Metcalf 1977).



### 2.5.2 Plasticity

Lime reduces plasticity and increases the plastic limit and possibly the liquid limit, all of which improve the engineering properties. Cement can provide similar benefits but it is not usual to use cement in soils with medium to high plasticity. The effect of lime on liquid limit and plastic limit, and on plasticity index, is illustrated in Figure 2.3 (from Metcalf 1977).

The value of increasing the plastic limit is illustrated in Figure 2.4 by considering a clay with a water content of 35%. With no lime the plastic limit is 25%. Thus its water content is 10% above its plastic limit ( $x$  in Figure 2.4) so the clay is wet and sticky. If 2% lime is added, the plastic limit increases to 40%. As this is 5% above

its water content ( $\gamma$  in Figure 2.4), the clay will be comparatively dry (Sherwood 1994).

### **2.5.3 Volume Stability**

Even small amounts of lime or cement can have profound effects on the volume stability of expansive materials. For example, they can greatly reduce moisture-induced swelling and shrinkage (NAASRA 1986).

With cemented soils, particularly with cement-stabilisation in granular materials, shrinkage on drying and temperature-induced contraction can result in cracking.

### **2.5.4 Durability**

The chemical reaction products of lime or cement with soil and water are semi-permanent in that they are not reversed by the presence of water. However external influences such as traffic loadings, wetting/drying, and frost can cause physical damage to the stabilised material if design, construction or maintenance is inadequate.

Stabilised soils resist alternating wetting and drying, or freezing and thawing, to an increasing extent as the cement content is increased. In the US where low temperatures in winter cause regular freezing of pavements, the ability to achieve standard levels of durability is one of the design factors for stabilised materials (Tait 1981).

### **2.5.5 Permeability**

The permeability of soils is affected in different ways by stabilisation. For example, the permeability of clays modified or lightly cemented by lime-stabilisation tends to increase and be higher than for the untreated soil, whereas the permeability of soils cemented by lime-stabilisation tends to decrease (Arabi and Wild 1989).

The permeability of cement-stabilised soils generally decreases, but can increase in clayey soils (Ingles and Metcalf 1972).

Permeability generally (but not always) decreases with increasing time after stabilisation, but can increase with time when periods of low temperatures occur soon after stabilisation (Arabi and Wild 1989).

### **2.5.6 Water**

Prolonged exposure to water produces only slightly detrimental effects. This is shown by US data for cured lime-soil samples where the ratio of soaked to unsoaked UCS is high being 0.7 to 0.85 (TRB 1987).

Cyclic wetting and drying may cause more significant strength reduction. However the major consideration about durability of lime-soil or cement-soil mixtures is resistance to cyclic freezing and thawing (TRB 1987).

## 2. *Properties of Stabilised Soils*

Figure 2.3 Effect of lime on plasticity index  
(from Metcalf 1977).

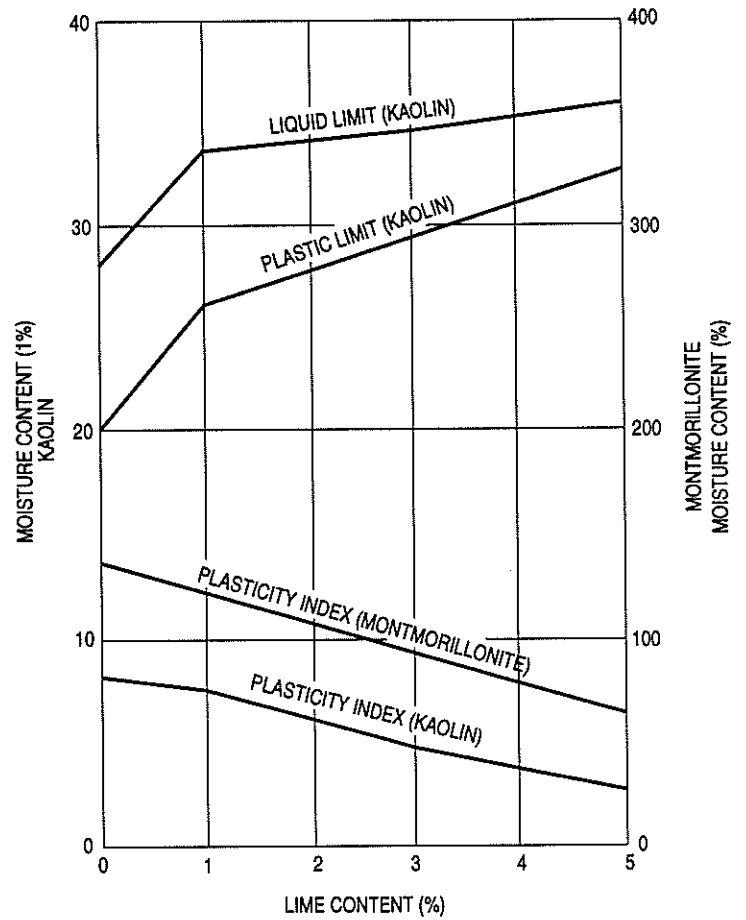
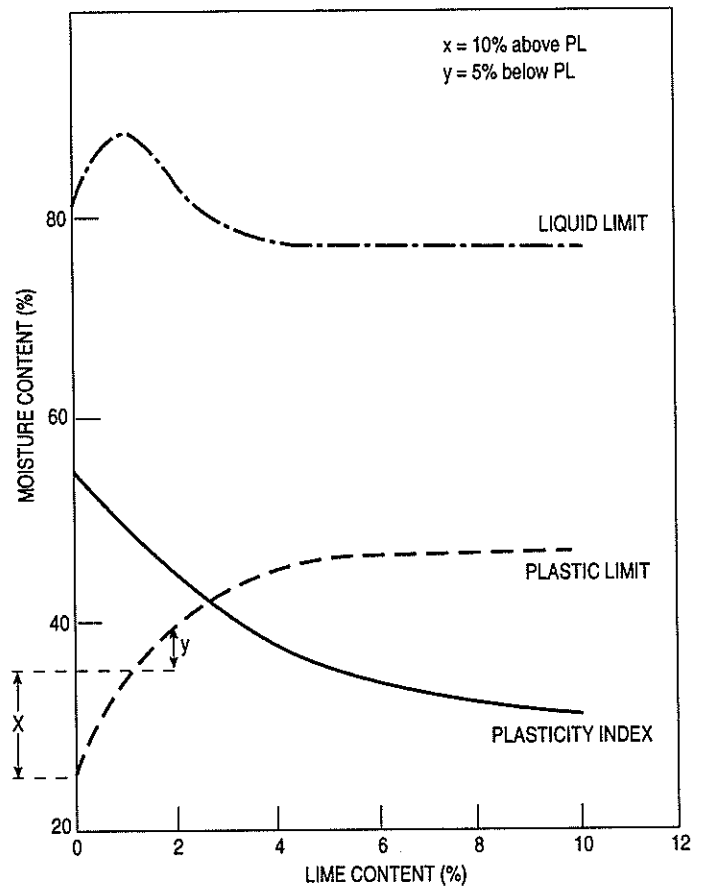


Figure 2.4 Effect of lime on the plasticity properties of a clay with 35% water content  
(from Sherwood 1994).



### **2.5.7 Frost**

Freezing of the pavement may in some circumstances lead to frost heave. The mechanism that creates frost heave is the drawing of water into the frozen pavement, so permeability of the pavement materials is an important factor.

Thawing of a frozen pavement tends to trap water within the upper layers because the lower layers are still frozen and prevent drainage downwards. This creates very weak pavements which are vulnerable to rapid deterioration. Permeability of the pavement materials is again an important factor as it influences the amount of moisture drawn in during the initial freezing.

Numerous cycles of freezing and thawing during a cold period can draw more water into the pavement than is drawn in by only one cycle of freezing and thawing.

As the ease of migration of moisture from the unfrozen to the frozen layers depends on the permeability of the layers, adding small amounts of lime (particularly if short curing periods are used) can increase the permeability and so produce greater distress than in the untreated soil.

On the other hand, adding enough lime to achieve both modification and cementation of the soil can improve frost resistance by reducing permeability (Arabi and Wild 1989). The increased tensile strength of the soil will also be beneficial in reducing heave, and increasing strength during thaw.

Cement-modified materials may have marginally improved frost resistance. Cement has been widely used in the US to produce materials that are resistant to frost, but these materials are strongly cemented with UCS of 3 MPa or higher (NAASRA 1986).

Frost shattering results from expansion of water as it freezes in the voids of a pavement material. All unbound, modified or weakly cemented pavement materials are at risk from frost shattering if they are wet (high water content), are loose or friable (low density), and are exposed to sub-zero temperatures (Sherwood 1994).

### **2.5.8 Cracking**

In practical terms cracking in pavements is not a problem with lime- and cement-modified materials, given adequate pavement strength.

Cracking can be a problem with strongly cemented materials having higher tensile strength and stiffness (see Section 6.1). This cracking primarily occurs in cement-stabilised materials (Hudson and Simmons 1991, Tait 1981, NAASRA 1986) and relates to:

- Cracking from dry shrinkage and temperature contraction, and/or
- Load-induced cracking.

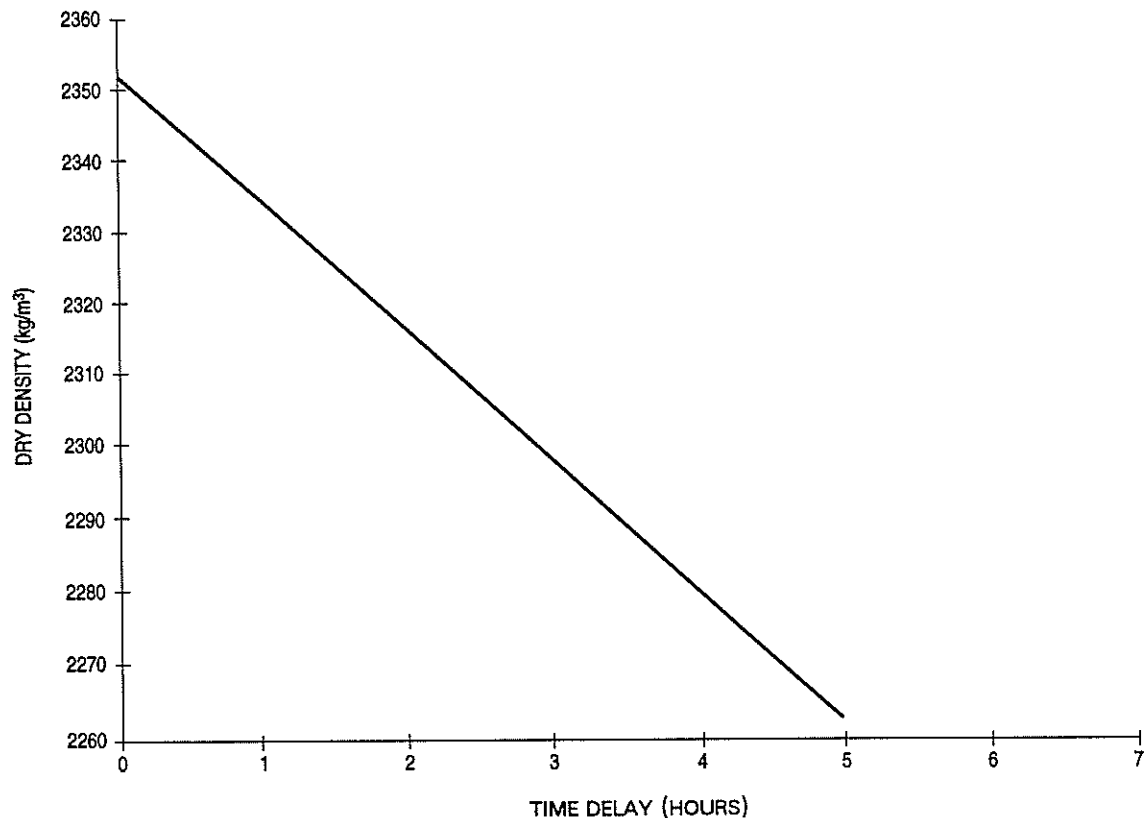
Some lime-soil mixtures display autogenous healing properties, which may restore the stability of the soil in warmer weather after freezing or freeze-thaw cycles (TRB 1987).

### 2.5.9 Compaction

Good compaction to achieve a high density is most important for stabilised materials. With an increase in density, both strength and bearing capacity increase (Metcalf 1977).

A time delay between mixing and compaction of the materials can decrease the density achieved and hence strength. This decrease in strength may be particularly marked if a high rate of formation of cementitious products has produced a cemented material (Figure 2.5).

Figure 2.5 Delay between mixing and compaction: effect on density using cement with no retarder (after Symons and Collins 1991).



The compaction properties of the soil alter with the addition of lime or cement, particularly with lime-stabilisation of clays. These properties vary with clay types but maximum dry density tends to be reduced. Also the optimum water content for maximum dry density increases with increasing lime content.

#### **2.5.10 Layer Thickness Equivalence to Unbound Materials**

Modified materials are regarded as equivalent to unbound materials for the purposes of pavement thickness design (NRB 1989).

The use of equivalency factors for design is convenient and expedient, but it is not a theoretically rigorous approach (Rollings et al. 1992).

For cemented, and stronger, stabilised materials specific design should be undertaken. However Tait (1987) suggests that a lime- or cement-stabilised layer is equivalent to 1.2 times its thickness of granular basecourse.

## **2.6 Rates of Reaction**

### **2.6.1 Lime-Stabilisation**

Lime has an almost instantaneous effect on the plasticity and strength of a clay (Metcalf 1977). After the initial reactions have taken place, the rate of strength gain of lime-stabilised materials is considerably less than that of cement-stabilised materials.

The rate of strength gain of lime-stabilised materials is temperature sensitive, more so than it is for cement-stabilised materials. Some evidence (NAASRA 1986) shows that in lime-stabilisation the physical form of the cementitious products is sensitive to their curing temperatures. Some caution is needed therefore in accepting the results of high temperature testing (e.g. 50°C) without some verification at ambient temperatures (NAASRA 1986). Low temperatures affect the rate of reaction, with little reaction occurring below 10°C and virtually no reaction below 5°C (Hudson and Simmons 1991, TRB 1987).

### **2.6.2 Cement-Stabilisation**

After rapid gains in strength in the first one to two days, cement-stabilised materials continue to gain strength slowly, provided that curing is sustained (NAASRA 1986). Temperature affects the rate of reaction. While cement will continue to react at a slightly lower temperature than lime, virtually no cement reaction occurs below about 5°C (Hudson and Simmons 1991, ACI 1990).

Figures 2.6 to 2.8 illustrate some of these trends and compare the effects of lime and cement.



Figure 2.6 Initial strength gain with time (after Metcalf 1977).

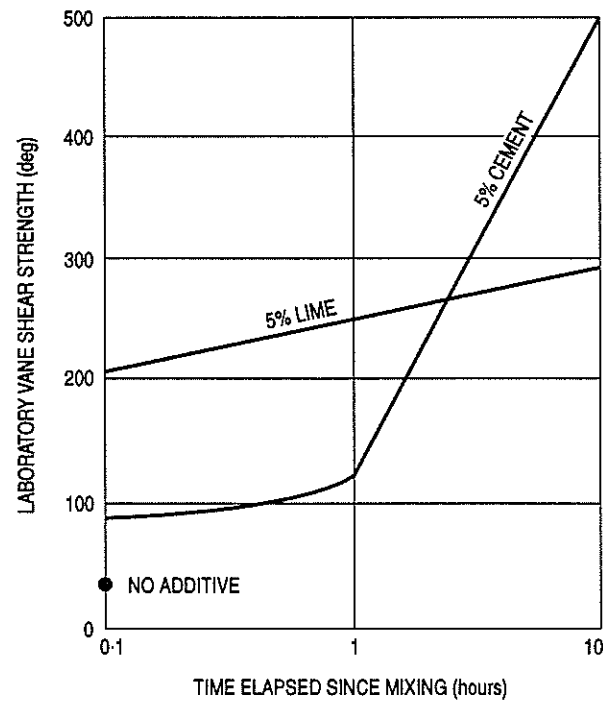


Figure 2.7 Effect of curing temperature on lime- and cement-stabilised soils after curing for 7 and 56 days (after Metcalf 1977).

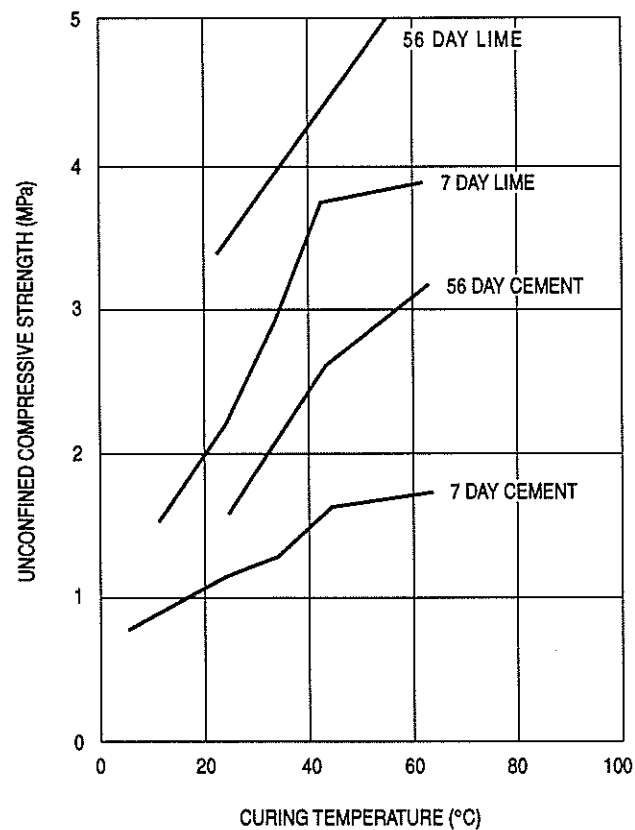
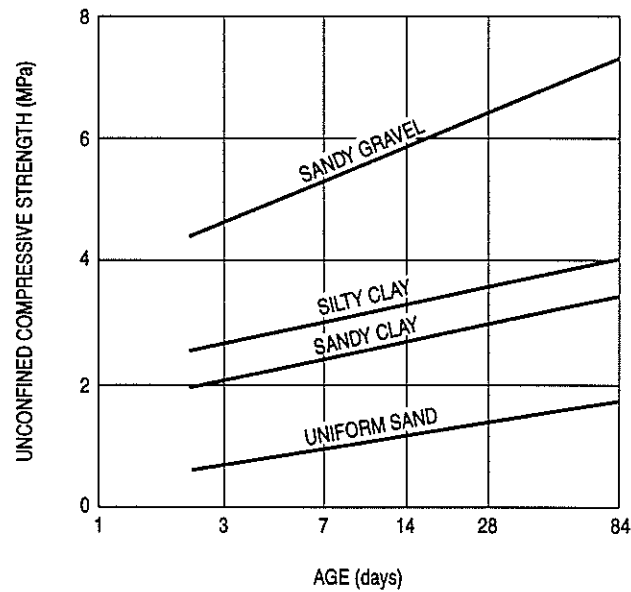


Figure 2.8 Effect of time of curing on four different soils that have been cement-stabilised (from Metcalf 1977).



## 2.7 Long-Term Strength Gain

Field data indicate that, with some lime-soil mixtures, strength continues to increase with time up to and in excess of 10 years (TRB 1987). The NRB (1989) Manual recognises this and has incorporated an allowance for it into the design charts for stabilised sub-bases.

### 3. MATERIALS

#### 3.1 Types of Lime

The term "lime" tends to be used rather loosely to mean agricultural lime, quicklime (burnt lime), or hydrated lime (slaked lime). For the purpose of this review lime is to be read as quicklime or hydrated lime, unless stated otherwise (Table 3.1).

Table 3.1 Properties of lime types (after NAASRA 1986; M Eberlin, pers.comm. 1994).

| Lime Type                        | Hydrated Lime     | Quicklime    | Slurry Lime*      |
|----------------------------------|-------------------|--------------|-------------------|
| Composition                      | $\text{Ca(OH)}_2$ | $\text{CaO}$ | $\text{Ca(OH)}_2$ |
| Form                             | Fine powder       | Granular     | Slurry            |
| Bulk Density ( $\text{kg/m}^3$ ) | 450 to 560        | 1,050        | 1,250             |

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

##### 3.1.1 Agricultural Lime

Agricultural lime (calcium carbonate) is simply crushed limestone. It is of little value as a stabiliser and is not discussed further.

##### 3.1.2 Quicklime

Quicklime (or burnt lime) is calcium oxide, manufactured by heating calcium carbonate (natural limestone) in a kiln until the carbon dioxide is driven off.

Commercially supplied quicklime is not usually 100% calcium oxide because of impurities, incomplete heating, or carbonation. Its quality can be judged by its grade which is the statistical percentage of active ingredient (TNZ 1986).

##### 3.1.3 Hydrated or Slaked Lime

Hydrated lime or slaked lime is calcium hydroxide and is produced by slaking quicklime with water. Commercially supplied hydrated lime is not 100% pure, and its quality can be judged by its grade which is the statistical percentage of active ingredient (TNZ 1986).

Hydrated lime is generally more expensive to truck to the site than quicklime as it is bulkier and, for any given amount of active lime, heavier. Thus hydrated lime is usually made at the site by slaking quicklime with water, either from the water cart or from the soil itself. Hydrated lime is a fine powder and tends to blow around easily, unlike quicklime which is granular. After mixing with the soil, it is the hydrated lime, rather than the quicklime, that reacts with the soil and stabilises it.

### **3.1.4 Lime Slurries**

A lime slurry is a suspension of hydrated lime in water. Lime slurries can be prepared by adding water to hydrated lime but they may also be produced as an industrial by-product, e.g. from the manufacture of acetylene.

Adding lime to water containing common soluble salts causes the calcium to crystallise and precipitate as "scale". (This is the reaction used to "soften" calcium-rich or "hard" water.) In the manufacture and application of lime slurry this reaction causes scaling of pipes, pumps, valves, etc. This scaling problem can be minimised by agitating the mixture during slurry manufacture, so that crystallisation occurs on the suspended solids which remain in suspension (Beals 1976).

A typical slurry mix consists of one tonne of hydrated lime to 1900 litres of water. At higher concentrations of hydrated lime, pumping and handling the slurry through the spray bars is difficult. One tonne of hydrated lime to 1500 litres of water is the highest concentration that is still pumpable (NLA 1991).

Slurry storage tanks must be agitated to maintain the suspension.

Water used to make slurries should be of potable standard (Section 3.5 of this report). While it is desirable that the level of soluble salts in the water used to make slurries is low to reduce scaling, in many locations this will be impracticable.

## **3.2 Types of Cement**

Any cement may be used for stabilisation, but traditionally in New Zealand "ordinary Portland cement" (OPC) has been the most widely used. Neither of the two New Zealand cement companies now manufacture OPC, and instead both now manufacture "type General Purpose cement" (type GP). Type GP, according to NZS 3122:1995, contains up to 6% of mineral and processing additions, in comparison to the 0.1% of additions allowed for OPC under the previous standard (NZS 3122:1990). This change from OPC to type GP has not altered the performance properties of the cement, so there is no technical need to adjust any stabilisation specifications (Gaerty 1996, DP Barnard, pers.comm. 1996).

Blended cements (type GB) are also permitted under NZS 3122:1995. They are blends of Portland cement with more than 5% of fly ash and/or granulated iron blast furnace slag (Gaerty 1996). A blended cement is slower setting which may assist in achieving full compaction, and be more tolerant of construction delays.

A cement with a retarder is also slower setting and may assist in achieving full compaction, and is more tolerant of construction delays (see Section 9.2.8 of this report).

Rapid hardening cements with added calcium chloride may be useful in organic soils. Sulphate-resisting cements are rarely used (Metcalf 1977), and may not be beneficial (Arabi and Wild 1989) at least in clay soils containing sulphates.

### **3.3 Percentage of Lime or Cement**

Lime and cement contents are normally specified as a percentage by mass. As the percentage of lime can be defined in a number of ways (e.g. % active ingredient, % quicklime, % hydrated lime), the definition that is used should be made clear in all specifications, and in laboratory test reports.

A suggested definition of the percentage is:

- Lime - Mass of CaO as a percentage of the dry mass of the soil.
- Cement - Mass of cement as a percentage of the dry mass of the soil.

The contractor, and perhaps the supervisor, will need to relate this percentage to both density and layer thickness to calculate the application rate of the lime or cement in kg/m<sup>2</sup>. The relevant density then is the dry density of the soil after compaction.

A simplified form of this definition can be applied, as an approximation, if using a high grade quicklime (e.g. Grade 94%), and if dry densities near 100% New Zealand standard compaction are to be achieved. Then the definition to calculate lime or cement application rates simplifies to:

- Lime - Mass of quicklime as a percentage of maximum dry density of the soil under New Zealand Standard Compaction.
- Cement - Mass of cement as a percentage of maximum dry density of the soil under New Zealand Standard Compaction.

The simplified forms of the definition above allow calculation of lime application rates based on laboratory test results.

As the New Zealand Standard Compaction test is carried out only on material passing the 19mm sieve, a correction factor may be needed for soils with a significant portion of particles larger than 19mm diameter.

### **3.4 Stabilisation of Different Soil Types**

Any but the most organic soil can be stabilised. The applicability of the different types of stabilisation to a range of soil particle sizes is indicated in Figures 1.1 and 1.2, in Chapter 1 of this report.

Properties of soils can range widely. Consequently the reaction of a soil with any particular stabiliser cannot be determined just by looking at the soil, or by defining the soil type. Some general trends are:

- **Clays.** Clays are usually suitable for lime-stabilisation. In general, clay soils containing montmorillonite and halloysite will react better to lime-stabilisation than those containing kaolinite and illite (Dunlop 1977).

Cement can be used to stabilise clays, but in heavy or wet clays (e.g. those with liquid limit >50 and plasticity index >18) the high application rates of cement and the extreme difficulties in mixing tend to make cement-stabilisation expensive and impractical. Pre-treatment with lime before cement-stabilisation can assist by both reducing the percentage of cement required, and making mixing easier (Ingles and Metcalf 1972).

- **Volcanic Soils.** Stabilisation of some volcanic soils can be difficult and provide very limited strength gains, especially for the brown volcanic ash soils occurring around central North Island. They may respond adequately to higher than normal dose rates (e.g. 6% of cement). A check on the response to lime or cement should be carried out to ensure their suitability or otherwise (Dunlop 1977, Tait 1981, ICR Brown, pers.comm. 1994). For further information on this topic refer to Nguyen (1975).
- **Sands.** Clean sands can be very suitable for cement-stabilisation, although poorly graded or single sized sands can be difficult to compact and require higher cement application rates which create the risk of cracking.

Silty sands and clayey sands may be suitable for stabilising with either lime or cement, or a combination of both. At low application rates of either lime or cement however, the cohesive forces may be lost and the strength of the stabilised material reduced to below that of the natural sand.

- **Gravels.** Gravels can be very suitable for cement-stabilisation, but increasing the cement application rates can lead to the development of shrinkage, temperature and/or fatigue cracking problems.
- **Organic Soils.** Organic soils do not stabilise well, if at all. An organic content of about 2% is generally regarded as the maximum.

Sugars and reactive organic compounds can retard the hydration reactions and even prevent the development of cementitious bonds with both lime or cement. If stabilisation is to be attempted adequate laboratory testing is essential. Laboratory testing will usually include determination of the percentage of organic matter and the pH of the stabilised material, but these tests may not in themselves indicate a poorly reacting material (NAASRA 1986, Hudson and Simmons 1991, ACI 1990).

- **Soils Containing Sulphates.** Sulphates in soils can retard or prevent the proper hydration of cement in cement-stabilisation. In some circumstances, soluble sulphates can disrupt the cementing paste after it has hardened. As the adverse effects may include a reaction with the clay in the soil, and not with the stabilising agent, little benefit may be gained from using sulphate-resisting cement or lime. More benefit may be gained from increasing the cement content (ACI 1990).

In lime-stabilisation, sulphates that are present in sufficient quantities in the soil can cause a secondary reaction which, in the presence of water, can cause swelling and disruption of the compacted mix (Ingles and Metcalf 1972, Dunlop 1977, Arabi and Wild 1989, Hudson and Simmons 1991).

Such sulphate-containing soils are rare in New Zealand but they do occur, e.g. in some of the volcanic areas of the central North Island.

- **Saline Soils.** In some saline soils sulphate ions are present which react adversely with the cement (Bell 1983).

### 3.5 Water Quality

As a general rule, the water used for stabilisation should be of a potable standard. Sea water has been used for lime- and cement-stabilisation, but the presence of chlorides may speed the reaction of cement and so inhibit compaction. Also the build-up of surface salts can interfere with the adhesion of future seal coats (ACI 1990, NAASRA 1986).

Highly organic waters, or waters containing sulphates or chlorates should not be used as they can retard the hydration reactions or even react with the clay or hardened cement paste.

## **4. SELECTING THE STABILISATION OPTION**

### **4.1 General**

As discussed in Section 1.5 of this report, stabilisation can be used in any of the pavement layers. Stabilisation can be used in new pavement construction, reconstruction, or for rehabilitation of an existing pavement. Before deciding to use stabilisation in any of these situations it must be established if it is the appropriate technique. This decision is determined by analysis of the problem or deficiency that stabilisation is to address, and of the available materials, of external considerations and economics.

### **4.2 New Construction and Reconstruction**

For new construction and reconstruction over weak subgrades, stabilisation offers a once-only opportunity to improve the subgrade.

Lime-stabilisation may also be used to dry out wet plastic clays to provide a working platform for further construction work, as well as subsequently providing additional strength to the pavement. Chapter 1 of this report outlines other benefits of stabilisation.

### **4.3 Rehabilitation**

#### **4.3.1 Problem Diagnosis**

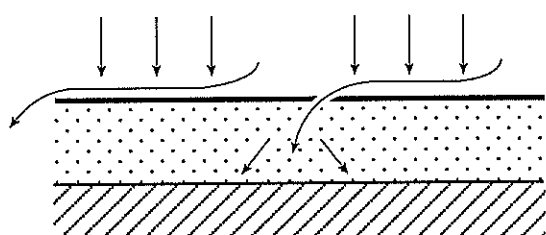
Six basic types of pavement distress (Figure 4.1) are identified in the NRB (1989) Manual. Once a section of pavement is selected for possible rehabilitation the defects should be defined and the mechanisms causing the defects should be postulated. Drainage and surface waterproofing are of fundamental importance for any maintenance or rehabilitation option, including that of stabilisation.

Considering each of the six distress modes shown in Figure 4.1, stabilisation can offer benefits to help prevent re-occurrence of five of the modes:

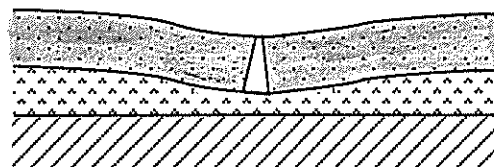
- Loss of waterproofness, Figure 4.1(a).  
Stabilised pavement materials themselves tend to be less affected by moisture than unbound materials. However it is still good practice to maintain a waterproof bituminous surfacing and good drainage on sealed roads incorporating stabilised layer(s). The stabilised material can prevent unbound layers over or beside the stabilised layers from draining. Therefore maintenance of the waterproofness of surfacing is vital to these layers.



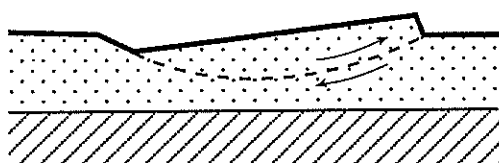
Figure 4.1 Pavement distress types (from NRB 1989).



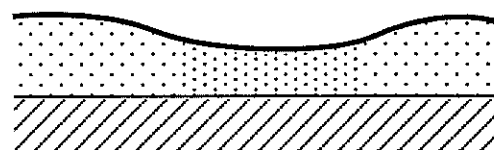
(a) LOSS OF WATERPROOFNESS



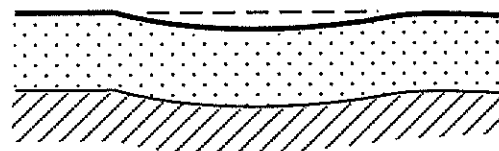
(d) LOSS OF STRUCTURAL SLAB ACTION (CRACK)



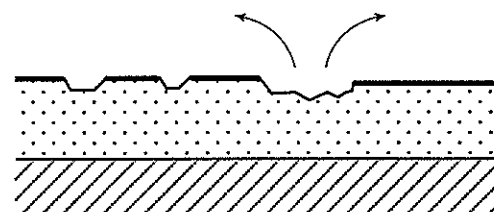
(b) WITHIN PAVEMENT SHEAR



(e) WITHIN PAVEMENT DENSIFICATION

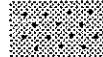



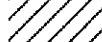
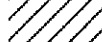
(c) FOUNDATION DEFORMATION



(f) SURFACE ATTRITION

**Key:**

— Bituminous surfacing  
 Concrete or asphaltic concrete

 Basecourse  
 Sub-base  
 Subgrade

#### 4. *Selecting the Stabilisation Option*

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- Within pavement shear, Figure 4.1(b).  
Stabilisation can improve the strength and reduce the sensitivity of pavement materials to moisture, so allowing the materials to resist shear.
- Foundation deformation, Figure 4.1(c).  
Stabilisation of the subgrade can strengthen that layer of the foundation. Stabilisation of sub-base or base layers may give increased stiffness which also decreases subgrade strains.
- Loss of structural slab action, Figure 4.1(d).  
Stabilisation of the sub-base can resist sub-base erosion from "pumping" and can improve sub-base strength. Support of the base increases and so the risk of cracking is reduced.
- Within pavement densification, Figure 4.1(e).  
Proper compaction of all the material is essential for all pavement layers, including the stabilised layers. Stabilisation of the subgrade can provide a firm platform allowing higher degrees of compaction to be attained in overlying layers.
- Surface attrition, Figure 4.1(f).  
For unsealed roads, stabilisation of the wearing course can substantially reduce the rate of surface attrition. For sealed roads, stabilisation generally does not increase resistance to surface attrition, and may decrease it. For example the cement-stabilisation of fine-grained materials, such as pumice, can increase the risk of surface attrition because the dust on the stabilised surface hinders bitumen adhesion.

##### **4.3.2 Preliminary Assessment of Alternatives**

The defects affecting the pavement and the mechanisms causing them should be the basis for defining the rehabilitation options. More than one option should be considered for each rehabilitation project (NRB 1989), and stabilisation should always be considered as one of the options.

##### **4.3.3 Available Stabilisation Options**

For pavement rehabilitation a number of stabilisation options are available. The following are a few which may be used (Dunlop 1977):

- The existing basecourse and seal are stabilised in-situ to a depth of 100mm to 250mm, and resurfaced.
- The existing basecourse is windrowed onto the side of the road then the sub-base or subgrade is stabilised in-situ. The basecourse can then be re-laid and either stabilised or left unbound.

- Lime slurry can be injected under pressure, or just left to soak in, through holes in the pavement (or a permeable sub-base) to stabilise the foundation (or surface of the subgrade).
- The existing basecourse and seal can be removed, treated in a pug-mill type mixer with lime or cement, then re-laid on the road, and resurfaced.
- Selected aggregates (to improve the particle size distribution, i.e. mechanical stabilisation) or clay (to improve the reactivity to lime) can be added to the existing basecourse before in-situ stabilisation and resurfacing.
- The existing pavement can be overlaid, then the overlay material can be stabilised (and possibly some of the existing basecourse) in-situ, then surfaced.

#### **4.3.4 In-Situ Materials**

If stabilisation is one of the preferred rehabilitation options the designer needs some general data to decide if to proceed with stabilisation. In addition to deductions of the symptoms and mechanisms causing the distress, these should include a preliminary assessment of:

- Type, quality and variability of in-situ materials,
- Any previous stabilisation of in-situ materials,
- Thickness, and variability of thickness, of existing pavement layers,
- Subgrade strength and type,
- Existing drainage,
- Local quarries or borrow areas, of the range of roading materials they produce, and of the responses of each of these materials to stabilisation.

These preliminary assessments could be based purely on close examination of the site and local knowledge. Small test pits dug through the existing pavement and/or specific tests (e.g. Benkelman Beam) may be needed to complement this knowledge.

#### **4.3.5 Advantages**

Rehabilitation using stabilisation can offer many advantages, some of which have been already outlined. A few of these advantages are:

- In urban areas it can reduce or eliminate the need to raise the pavement surface above the kerb and channelling.
- Rehabilitation outside the normal construction season is not good practice but, if it must be done then, stabilisation can allow an early start to the construction season in spring, once temperatures start to rise.

#### *4. Selecting the Stabilisation Option*

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- In-situ stabilisation of existing material may reduce or eliminate the need to truck soil away or aggregate in.
- In a poorly drained situation the moisture tends to have less affect on stabilised materials than on unbound materials.

### **4.4 Considerations Affecting Decision**

#### **4.4.1 Climate and Drainage**

Poorly drained clay soils may become untraffickable when wet. Almost immediately after lime-stabilisation, such soils will generally be strengthened enough to enable passage of construction vehicles. Cement-stabilisation can produce some of these benefits in poorly drained silts as well, but it takes a longer time to have effect.

Weather can affect construction techniques. Dry dusty conditions may favour use of a lime slurry (Dunlop 1977; also see Sections 8.2.6 and 8.2.7 in this report). Cool weather may favour cement instead of lime.

The temperature of the soil influences the reaction rate of lime or cement. As reaction rates are very slow at low temperatures, stabilisation should not be undertaken when the ground temperature is below 10°C (see Section 2.6 of this report). At temperatures above 20°C, the time delay between mixing and compaction in cement-stabilisation should be reduced.

#### **4.4.2 Location and Size of Project**

Remoteness of job sites can increase the cost of establishing the stabilisation plant on-site, and the costs of supervision. In remote sites, planning stabilisation of larger areas, or the use of as much locally available equipment as practical, are preferred.

#### **4.4.3 Availability of Plant**

Adequate plant must be available for the job (Plate 2). In addition to roller(s), water cart(s) and grader, a heavy duty rotary mixer is required. A mechanical spreader for the lime or cement is desirable, although bags laid to a grid pattern can be used on smaller jobs. See Sections 8.2.5 and 9.2.5 in this report.

The rotary mixing machine and the roller(s) must be adequate to handle the required stabilised layer thickness. The rotary mixing machine should also be matched to the type of soil to be stabilised. See Section 7.3 in this report.

#### **4.4.4 Dust and Steam Produced During Stabilisation**

Stabilisation can create clouds of steam and dust. The dust can be dry soil particles, cement or hydrated lime blown by the wind. This can result in adverse comment from motorists and local residents. Good job planning to enable rapid execution of

stabilisation will help stem comment, as will the wise use of the water cart, or the use of lime slurry to reduce the dust and steam.

#### **4.4.5 Effects of Frost During Pavement Lifetime**

A pavement that is subjected to frost during its lifetime cannot be treated with small quantities of lime or cement to eliminate a material's susceptibility to the frost. Treatment may make it worse (see Section 2.5.7 in this report). Specific design of stabilised material in these situations is required, particularly if the stabilised material is to be used in the upper layers of the pavement.

Stabilised materials need to have cured to a certain degree before they have frost or freeze-thaw resistance. Hence stabilisation in frost situations must be carried out well in advance of winter frosts (Smits 1986). The necessary period that has to elapse before the frosts occur is closely related to each particular soil type (Smits and Crundwell 1989).

Tests for the evaluation of the freeze-thaw resistance of stabilised materials are available in New Zealand (e.g. the RRU/CL Frost Test described in Smits 1986).

#### **4.4.6 Shrinkage**

A combination of high shrinkage and high tensile strengths can cause widely spaced (e.g. 0.5m to 5m) block cracking in cement-stabilised bases. While such cracking usually does not lead to early structural deterioration in well designed pavements, it reduces the quality of the ride (NAASRA 1986). It is usual New Zealand practice to limit strengths of stabilised materials, particularly cement-stabilised bases, by limiting the cement application rate. (See Section 2.3 in this report.)

Modification of clays by lime-stabilisation usually reduces shrinkage. At low lime contents (e.g. 1.5–3%) the risk of undesirable shrinkage cracking is low (NAASRA 1986).

#### **4.4.7 Thickness of Stabilised Layer**

For construction and reconstruction the thickness of a lime- or cement-stabilised sub-base, and for modified bases, can be obtained from the NRB (1989) Manual.

For rehabilitation of existing base layers the thickness of the stabilised layers to be attained typically ranges between a minimum of 150mm and a maximum of 200mm to 250mm (compacted thickness).

If the layers are thicker than about 200mm to 225mm, compaction of a single lift to the full depth of stabilised layers tends to be difficult and requires appropriate compaction equipment. A firm foundation to compact against is also needed.

While layers thicker than about 200mm to 225mm have traditionally been constructed in two lifts, recent Australian experience has included stabilising of layers up to 350mm thick (the target compacted thickness) in a single lift. Special rotary mixers, heavy compaction equipment, and slow setting binders capable of providing at least 12 hours of working time are required (CACA 1993a, 1993b, Dash and Bilaniwskyj 1994). Even so Dash and Bilaniwskyj reported that full density was not obtained at the bottom of the stabilised layer. Presumably this lower density is also influenced by the foundation/subgrade strength.

#### **4.4.8 Existing Services**

For rehabilitation of roads the presence of existing buried services should be checked. If services do exist close to the surface their depths should be determined accurately. If the services lie within the scarification and pulverisation zone for stabilisation then the service should be lowered, abandoned or removed; or lesser depths of stabilisation should be considered at these places. In some cases the presence of existing services will make stabilisation impractical or uneconomic.

### **4.5 Assessment of Alternatives and Decision to Proceed**

The points raised in this Chapter 4 will indicate if the lime- or cement-stabilisation option is likely to be technically viable and appropriate. Economics then becomes the next important consideration. A preliminary estimate can be made of the construction costs, and of likely future maintenance costs, for each of the technically attractive options to shortlist the preferred options. Options could include stabilisation if appropriate.

The pavement construction or rehabilitation option(s) can then be selected, and the project can proceed to detailed testing and design.

Stabilisation has other advantages that are difficult to quantify in economic terms at the time of project assessment. The conservation of quality aggregate resources is one advantage. Another is the long-term performance because stabilised materials may require less maintenance than unbound materials in the longer term, particularly in situations with poor drainage or weak subgrades.



## **5. TESTING FOR STABILISATION**

### **5.1 General**

In addition to data required for the design of unbound pavements such as traffic loading and subgrade strength, and the considerations discussed in Chapter 4 of this report, a designer of a stabilised pavement needs to consider matters such as the following:

- Is the soil reactive?
- If so, what is the appropriate stabilising agent(s)?
- What mix design (e.g. lime- or cement-soil-water mix ratio) is appropriate?
- What laboratory testing should be carried out?
- What sampling is required?

Soils are variable in their reaction to lime or cement so laboratory testing to determine their reaction should always be carried out. The results from this laboratory testing should precede pavement design, even if recourse to further laboratory testing is found necessary after carrying out preliminary design (Dunlop 1977).

### **5.2 Sampling**

The success of both the testing programme and the stabilisation works depends on the sampling. Key practical points in sampling include:

- Samples are to be of the actual soil that is to be stabilised.
- Size of each sample is to be sufficient for all envisaged tests.
- Number of samples are to be increased as variability of soil within the area to be stabilised increases.
- Samples are to be immediately sealed in waterproof airtight bags, and never allowed to dry out.
- Separate samples are not to be mixed, unless mixing will replicate what really will happen during construction.
- Labels are to be put on each sample.

The sample size required will depend on the particle size of the soil and on the tests to be performed. Table 5.1 indicates the mass of sample to be submitted to the laboratory for each of the tests.



Table 5.1 Guide to masses of sample to be submitted for each of the laboratory tests (after Rowe 1985 and NZS 4402:1986).

| Type of Test   | Soil Group* |        |        |
|--|-------------|--------|--------|
|  | Fine        | Medium | Coarse |
| Quick Response †   | 10kg        | 10kg   | 10kg   |
| Water Content ‡  | 50g         | 2kg    | 6kg    |
| Dry Density/Water Content Relationships:<br>NZ Standard or Heavy Compaction† | 25kg        | 25kg   | 25kg   |
| NZ Vibrating Hammer Compaction†  | 80kg        | 80kg   | 80kg   |
| Linear Shrinkage ‡   | 4kg         | 4kg    | 4kg    |
| CBR ‡  | 6kg         | 7kg    | 7kg    |
| UCS ‡  | 6kg         | 9kg    | 9kg    |
| Tensile Strength ‡   | 6kg         | 9kg    | 9kg    |
| Durability ‡   | 6kg         | 9kg    | 9kg    |
| Cement or Lime Content of a<br>previously stabilised soil ‡                  | 400g        | 4kg    | 8kg    |

**Notes:**

- \* Soil groups are those described in NZS 4402:1986:clause 1.5:
- Coarse: Soils containing not less than 90% passing a 63mm sieve.
  - Medium: Soils containing not less than 90% passing a 20mm sieve.
  - Fine: Soils containing not less than 90% passing a 2mm sieve.
- The groupings for medium and fine may be based on the 19mm and 2.36mm sieves respectively without seriously affecting the results (NZS 4402:1986).
- † Masses are for each set of tests.
- ‡ Masses are for each test.
- CBR = Californian Bearing Ratio; UCS = Unconfined Compressive Strength

### 5.3 Quick Response Test

This test (Rowe 1985, Test 2) is designed for speed rather than accuracy. The test indicates if the soil will react favourably with lime or cement and thus determines whether further tests are needed.

Two sub-samples are prepared from each sample. One is mixed with about 2.5% lime or cement. Each specimen is compacted into a mould, demoulded and cured for approximately 48 hours. The UCS of each specimen is then measured. Broken pieces from each specimen are soaked in water to test slaking properties.

The soil may be regarded as responsive to lime or cement if the UCS of the treated specimen is higher or if it slakes at a slower rate (Rowe 1985) than the untreated soil.

If the response to both lime and cement is nil it is likely that no further testing is warranted. If the response is positive then laboratory testing should follow.

## **5.4 Laboratory Tests**

### **5.4.1 General**

A wide range of laboratory tests can be used, but the objective is to establish the stabilising agent, lime or cement, that is appropriate for the particular soil, and the percentage to use. Laboratory testing also may include determination of shrinkage, plasticity, strength, modulus, and/or durability value(s) for the stabilised soil.

The testing programme ideally should include assessing the specific deficiencies that stabilisation of the material is intended to rectify (Cornwell 1979) and to assess the properties that the stabilised material is to attain. Considerations such as those outlined in Chapter 4 of this report, construction needs such as early trafficking, and the amount of cementation to be achieved (i.e. modified, cemented or highly cemented) should also be considered.

Much of the stabilisation carried out in New Zealand is aimed to achieve a modified or weakly cemented material. For this the quality of the lime- or cement-stabilised material is usually assessed on the basis of strength tests made on the compacted stabilised material after it has been allowed some time to harden (Sherwood 1994).

Therefore the strength test most commonly used in New Zealand is the CBR (Californian Bearing Ratio) test, although the UCS test is sometimes used if a cemented material is sought.

Although widely used, both the CBR and the UCS tests have been criticised as neither test reflects the manner in which the stabilised layer is stressed in the pavement. They are used because of their relative ease (Sherwood 1994), ready availability of the test equipment, and familiarity with the tests. In addition, correlations between CBR, resilient modulus and allowable tensile stress are offered in NRB (1989) for lightly cemented (resilient modulus less than 1,000 MPa), reactive, fine-grained soils.

Most laboratory testing programmes should include density/water content test(s), and strength tests such as CBR. These tests should be carried out on lime-soil or cement-soil mixtures. Several samples of the soil mixed with different percentages of lime or cement are usually tested for strength after curing (e.g. 7 days curing at 20°C (Rowe 1985)).

The dry density/water content tests provide information on optimum water content for subsequent laboratory tests, and may provide information on optimum water content and maximum dry density for supervision or control of construction.

Rowe (1985) includes the following laboratory tests for stabilisation design in the RRU Technical Recommendation TR7:

- Quick response,
- Water content,
- Dry density/water content relationship - New Zealand standard compaction,
- Dry density/water content relationship - New Zealand heavy compaction,
- Dry density/water content relationship - New Zealand vibrating hammer compaction,
- Linear shrinkage,
- CBR,
- UCS,
- Tensile strength using the splitting test,
- Tensile strength using the double punch test (not to be used to obtain the resilient modulus) (NRB 1989),
- Wetting and drying durability,
- Cement or lime content of stabilised soils.

In addition the Atterberg limits tests (liquid limit, plastic limit, plasticity index) on both the unstabilised and stabilised materials can be useful.

#### **5.4.2 Sample Preparation**

Mixing a number of samples together to produce a combined sample should not be allowed unless:

- That sample is clearly representative of what will occur during construction,
- or
- Previous tests have shown that the combined sample will be representative of the individual samples (Rowe 1985).

Usually laboratory tests are performed only on the material passing the 19mm sieve. If a large proportion of the soil is coarser than 19mm then the test results will be affected, compared to those obtained from the whole soil.

Although no defined procedure is available for these coarse materials, the less than 19mm fraction could be tested (e.g. in accordance with Rowe 1985). Then the design lime or cement content could be lowered slightly on the basis that the coarser than 19mm material has little lime or cement requirement (NAASRA 1986).

### 5.4.3 Mixing

It is important that the soil sample is mixed with the lime or cement at the in-situ or near-optimum water contents, and not air-dried to a low water content (Dunlop 1977) or oven-dried.

### 5.4.4 Quality of Lime and Cement

All lime or cement used for mixing with the soil should be fresh, and not have been exposed, for any period of time, to moisture or to the air. For laboratory testing of lime-soil mixtures it is usual to use hydrated lime, which can carbonate even while stored within the manufacturer's paper bags. The details of manufacture of these paper bags varies from manufacturer to manufacturer (PW Rolls and M Eberlin, pers.comm. 1996; BR Webster, pers.comm. 1994), so the rate of carbonation of the hydrated lime will also vary.

Hydrated lime or cement should only be used from newly purchased and just-opened bags. If the lime or cement is of doubtful quality it should be discarded. For lime the grade can be checked by testing for its Available Lime Index.

### 5.4.5 Percentage of Lime

Usually laboratory testing for lime-stabilisation uses hydrated lime. The laboratory report should clearly state in its test results the basis of the quoted percentages of lime, e.g. CaO content, or quicklime of grade 93%.

The percentage can be converted from hydrated lime ( $\text{Ca(OH)}_2$ ) to CaO by the ratio of their molecular weights, and by allowing for the grade of the hydrated lime.

#### **Example: To determine lime content by percentage**

*Laboratory tests were carried out on a lime-soil mixture containing 3% by dry mass of hydrated lime. What percentage CaO is that?*

From the lime manufacturer's data the grade of the hydrated lime is 90%. The hydrated lime is fresh from the manufacturer and has been kept inside two sealed plastic bags for the few days it has been stored at the laboratory so it is assumed not to have carbonated.

$$\begin{aligned}\text{Molecular Weight of Hydrated Lime, Ca(OH)}_2 &= 40 + 2(16+1) \\ &= 74\end{aligned}$$

$$\begin{aligned}\text{Molecular Weight of Calcium Oxide, CaO} &= 40 + 16 \\ &= 56\end{aligned}$$

$$\begin{aligned}\text{Therefore percentage CaO} &= 3 \times \frac{56}{74} \times \frac{90}{100} \\ &= 2\%\end{aligned}$$

#### 5.4.6 Test Specimen Sizes

There are practical advantages when carrying out dry density/water content and strength tests if compaction of the specimens can be achieved by a drop hammer method, such as that used to determine New Zealand Standard Compaction. This favours a 127mm high x 152mm diameter CBR mould rather than a concrete UCS mould of say 200mm high x 100mm diameter. Strength testing on the CBR moulded specimen can then be CBR, tensile or UCS. However using a CBR moulded specimen for UCS testing does give a result that is not comparable with the New Zealand standard concrete UCS test results. While not of major consequence, Table 5.2 gives approximate correction factors for this.

Proctor moulds 115.5mm high x 105mm diameter may also be used with drop hammer compaction. The number of blows per layer for New Zealand standard compaction would then be 27, rather than the 62 for a CBR mould. However for some granular soils vibrating hammer compaction may be more suitable (NZS 4402:1986).

Table 5.2 Approximate correction factor for the different sizes and shapes of test specimens (after Sherwood 1994).

| Specimen Size                    | Correction Factor<br>To convert specimen UCS to UCS of a<br>200mm x 100mm diam. cylinder, multiply by |
|----------------------------------|---|
| 127 x 152mm diam. cylinder (CBR) | 0.77  |
| 200 x 100mm diam. cylinder (UCS) | 1.00  |
| 100mm cube                       | 0.77  |
| 150mm cube                       | 0.80  |

#### 5.4.7 Pre-treatment

If a soil is plastic and the clay lumps are difficult to reduce in size in preparation for testing, then pre-treatment with up to 50% of the lime should enable easier pulverisation, and allow the remainder of the lime to be mixed more readily.

The time lapse between adding the first portion of the lime and the second can be estimated by pre-treating a sample(s) and sealing it in a plastic bag. The time needed for the sample to become friable is an indication of the most suitable time lapse between pre-treatments (Rowe 1985).

If pre-treatment is beneficial in the laboratory, then it is a good indication that pre-treatment will also be beneficial during construction.

#### **5.4.8 Conditioning**

Often the effect of lime, and sometimes of cement, may be increased by conditioning, where the lime-soil or cement-soil mix is left uncompacted for a period, then remixed and compacted.

Compaction of the lime-soil and cement-soil should be completed within 2 hours, and ½ hour respectively, of initial mixing (Rowe 1985).

### **5.5 Dry Density/Water Content Tests**

The compaction characteristics of the lime-soil or cement-soil mix change once lime or cement is added to the soil. This change can be very pronounced for some soil types, e.g. montmorillonite clays (Lockett and Moore c.1981). Increasing quantities of lime or cement can bring about further change but usually the change is not as great as that after adding the first few percent of lime or cement.

The compaction characteristics of a lime-soil or cement-soil mix can also change with time after mixing.

A density/water content relationship for the lime-soil or cement-soil mixture is desirable for selection of the water content for laboratory strength tests, and for construction control. This relationship should be produced in the laboratory for a particular lime-soil or cement-soil mixture, using the lime or cement content near to that proposed for construction, and allowing the time delay between mixing and compaction to approximate that which will occur during construction.

In selecting the compactive effort which should be used for the preparation of laboratory samples, the design must consider the likely level of density that will be achieved during construction. For example the compaction of a stabilised sub-base on a weak subgrade will produce low densities, particularly in the lower portion of the layer, no matter how much compactive effort is applied (Dunlop 1977).

### **5.6 Strength Tests**

The original CBR procedure related all materials back to a well-graded non-cohesive crushed rock which was considered to have a CBR value of 100. Therefore the significance of a CBR value that is considerably higher than 100 is questionable, particularly for cement-stabilised soils with high strengths (Ingles and Metcalf 1972). The CBR test seems best used for materials with CBR values up to about 150 (NITRR 1986). Above this value the UCS test would be more appropriate.

### 5.6.1 Curing

The water content of the mix, the time allowed for curing, and the temperature at which the lime-soil or cement-soil mix is cured, all markedly affect the laboratory measured strengths.

Rowe (1985) specifies the following curing conditions for the CBR, UCS, linear shrinkage, splitting tensile, and double punch tests for both lime-soil and cement-soil mixtures:

- 7 days, at
- $20^{\circ} \pm 2^{\circ}\text{C}$ , with
- the specimen sealed against moisture loss, e.g. inside two sealed plastic bags.

The 7 days and  $20^{\circ}\text{C}$  is somewhat arbitrary and, given the faster initial strength gain of cement, the 7 day strength of lime-soil may not be comparable with the 7 day strength of cement-soil (Sherwood 1994). For this reason NRB (1989) recommends 14 days cure for lime-soil mix and 7 days cure for cement-soil mix.

Further research is needed on curing. For example, if a UCS or CBR test is to be carried out on a soaked specimen, the test methods do not define the periods to cure and then to soak before carrying out the strength test.

### 5.6.2 Subgrade and Sub-base

The sub-base is the pavement layer overlying the subgrade (or transition layer if used), and beneath the base layers (Figure 1.3 in Chapter 1 of this report). If the subgrade is stabilised with lime or cement, the resulting layer is referred to as a "sub-base", and the term subgrade is then applied to the underlying naturally occurring material (NRB 1989).

For a lime- or cement-stabilised "modified sub-base" (tensile strength  $<80$  kPa), the appropriate test would be CBR (Dunlop 1977, Tait 1981, NRB 1989).

For a lime- or cement-stabilised "cemented sub-base" (tensile strength  $>80$  kPa, but resilient modulus  $<1,000$  MPa), then the following data are needed for design (NRB 1989):

- Resilient modulus: Test by the Resonance method (ASTM C215-91),  
or Indirect Tension (ASTM D4123-82),  
or UCS method (e.g. ASTM C469-94).  
Do not use the double punch test.
- Tensile strength. Test by the Indirect Tension method (ASTM D4123-82),  
or the double punch method (Dunlop 1977).

## **5.     *Testing for Stabilisation***

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- Fatigue factor. The fatigue factor is used to reduce the allowable tensile stresses on the cemented layer. It may be calculated from the pavement loading. (Refer NRB 1989.)

However for fine-grained soils with a stabilised CBR less than 10, that exhibit more than a three-fold gain in CBR strength by lime- or cement-stabilisation, and with resilient modulus less than 1,000 MPa, the appropriate test is soaked CBR. The resilient modulus and tensile strength can then be estimated from the CBR, and no allowance need be made for fatigue (NRB 1989).

### **5.6.3   Base**

As for the sub-base, where lime- or cement-stabilisation is used to produce a modified material the appropriate test is CBR.

For rehabilitation of an existing distressed pavement by in-situ lime- or cement-stabilisation, the appropriate test is CBR (NRB 1989).

For stiff lime- or cement-stabilised cemented material (modulus >1,000 MPa) the appropriate tests include some combination of:

- UCS
- Linear Shrinkage
- Resilient Modulus
- Tensile Strength

## **5.7     Durability Tests**

### **5.7.1   General**

In some parts of New Zealand the stabilised layer will be subjected to either freezing and thawing, or wetting and drying, or both. Neither the CBR test nor the UCS test used alone fully reflects the freeze-thaw and/or wet-dry resistance of a stabilised material. Durability tests are therefore used to examine the ability of laboratory test specimens to resist exposure to such factors (Sherwood 1994).

However, in Central Otago, inland Canterbury, and the Central North Island, frost and freeze-thaw conditions prevail. Likewise unsealed roads, and parts of roads such as unsealed shoulders, become saturated after heavy rain, yet dry out in the hot weather. Therefore whether durability tests should be applied for the particular location under consideration is a decision to make (Dunlop 1977).



### **5.7.2 Frost**

In the areas where freezing ground temperatures occur, freeze-thaw damage may result (Cheung and Dongol 1996). The damage is generally characterised by volume increase and strength reduction.

Laboratory tests that may be used to determine freeze-thaw resistance include:

- ASTM D560-89 for cement-soil,
- Dunlop (1977), Appendix G, for lime-soil,
- Cheung et al. (1992, 1995), for lime-soil or cement-soil.

These tests subject the cured lime-soil or cement-soil samples to repeated freezing and thawing cycles with rubbing or brushing of the specimen between each cycle. The tests then measure water content and volume (swell or shrinkage) changes, and/or weight loss caused by these cycles. As the US climate can be very cold in winter, the ASTM tests are generally too severe for the New Zealand climate. (Refer ACI 1990, TRB 1987, Smits 1986, Cheung et al. 1992, Cheung and Sutherland 1995.)

### **5.7.3 Wetting and Drying**

Soaking a stabilised material in water usually reduces its strength slightly (see Section 2.5.6 of this report). Wetting and drying tests determine the strength reduction or weight loss produced by repeated cycles of wetting/drying and rubbing/brushing of the hardened stabilised soil specimens. Volume changes may also be measured.

Laboratory tests that may be used to determine wetting and drying durability include:

- ASTM D559-89, for cement-soil.
- BS 1924:Part 2:1990.
- RRU Technical Recommendation TR7 (Rowe 1985), for lime-soil or cement-soil. It is based on ASTM D560-89.
- Dunlop 1977, Appendix F, for lime-soil.

(Refer also ACI 1990, TRB 1987.)

## 6. DESIGN OF STABILISED PAVEMENTS

### 6.1 General Design Considerations

The pavement has to carry traffic loads without undergoing permanent distortion within itself, and to distribute these loads so the subgrade is not over-stressed. Stabilised layers used in the pavement also have to contribute to these requirements and not show significant non-elastic deformation or significant cracking.

Cracking in the stabilised layer(s) can be caused by tension forces arising from one or more of the following causes:

- Shrinkage,
- Temperature changes causing expansion and contraction,
- Traffic loads inducing fatigue stresses.

Section 2.3 of this report discussed the spectrum of properties available with stabilisation. From the original soil, with increasing degrees of stabilisation the spectrum ranged from modified, to cemented, to lean concrete, and ultimately with clean aggregates and high cement contents to concrete.

Materials modified by lime- or cement-stabilisation distribute traffic loads to the subgrade without developing significant tensile stresses. Such materials have little risk of traffic load-induced cracking and, provided adequate material selection and normal good construction practices are followed, have little risk of shrinkage- or temperature-induced cracking.

A cemented material distributes traffic loads to the subgrade partly by its tensile strength. If the tensile strength of the cemented material is exceeded, it is at risk from traffic load-induced cracking, in addition to shrinkage- and temperature-induced cracking.

The stiffer the stabilised material is relative to the surrounding pavement layers the greater the proportion of traffic loads it will attract. The chances of traffic load-induced fatigue cracking occurring are therefore greater in cemented materials unless stiffness, strength, layer thickness, traffic load, subgrade strength, etc. are balanced.

In pavement design terms stiffness is measured by the modulus. The higher the modulus the greater is the load-spreading ability of that material, but the greater the proportion of traffic load that material attracts. Thus stabilisation of a pavement layer to create a stiff, cemented, higher modulus layer can reduce stresses on the subgrade but may attract more load than the stabilised material itself can carry. As a result it cracks.

Also the stronger the stabilised layer the wider the spacing will be between shrinkage- and/or temperature-induced cracks. Hence any cracking is likely to be wider and more significant in strongly cemented materials than in lightly cemented materials.

While no fixed correlation exists between strength and modulus (the correlation depends on individual materials), NRB (1989) seeks to differentiate between lower risk and significant risk of cracking materials by means of resilient modulus. Materials with a resilient modulus of less than 1,000 MPa are assumed to have a low risk of cracking provided good selection of materials, thickness design and construction practices have been followed.

Materials with higher resilient moduli ( $>1,000$  MPa) are at risk from traffic load-induced cracking and require specific design to avoid cracking-related problems in-service.

## **6.2 Design Criteria for Stabilised Subgrades and Sub-bases**

### **6.2.1 Modified Materials**

Thickness design for modified materials is the same as that for unbound materials. Materials are defined as modified if their tensile strength is less than 80 kPa.

Modified materials do not provide significant tensile strength. Thus, as for unbound materials, the prime aim of design of the appropriate pavement thickness is to reduce stresses and strains on the subgrade to an acceptable level. Thickness design for modified materials follows the same procedures and uses the same design charts as for unbound materials (e.g. Chapter 7, Sections 3 and 4 in the NRB (1989) *State Highway Pavement Design and Rehabilitation Manual*, hereafter called NRB (1989) Manual).

### **6.2.2 Lightly Cemented Materials**

Thickness design for cemented materials with low resilient modulus ( $<1,000$  MPa) should be calculated from the design charts for stabilised sub-base given in NRB (1989) Manual. At least the modulus and tensile strength of the stabilised material should be determined by testing. However for fine-grained soils resilient modulus and tensile strength can be calculated from CBR (refer NRB (1989) Manual) if:

- the unstabilised material has a CBR of less than 10, and
- the CBR increases by more than 3 times on stabilisation.

Cemented materials can carry significant tensile stresses. Pavement thickness design therefore needs to consider two prime criteria for traffic load-induced stresses when using cemented materials:

- Subgrade stresses and strains,
- Tensile stresses within the cemented layer.

Two thicknesses of aggregate cover are obtained from the stabilised subgrade thickness design charts given in the NRB Manual (Chapter 7, Section 5), one for each of these criteria.

### **6.2.3 Upside Down Pavements**

"Upside down" pavements have a stiff and strong sub-base overlain by an unbound basecourse and bituminous surfacing. They are upside down in that the strongest material is near the bottom of the pavement, which is opposite to usual practice.

Stiff cemented sub-bases (with resilient modulus  $>1,000$  MPa) offer an upside down configuration. This configuration offers the advantages of thermal insulation to reduce temperature-induced volume changes of the cemented layer, and associated cracking and crack movement. This configuration also has the advantage that any cracks in the cemented layer are not reflected to the surface, or are reflected very slowly, through the overlying unbound basecourse. The cemented layer has two life phases. The first life phase is before traffic-induced fatigue cracking of the cemented layer. The second life phase is after such cracking.

Insufficient is known about these life phases so the NRB (1989) Manual does not offer design guidance, but encourages specific proposals. AUSTROADS (1992) and MRD (1990) offer some design guidance.

Good drainage of the unbound base layer is very important for the long-term performance of upside down pavements.

## **6.3 Design Criteria for Stabilised Bases**

### **6.3.1 Modified Materials**

Well-graded granular basecourses stabilised with cement or lime to produce a modified material can be used as a premium basecourse. For high traffic loadings the material should be near to the TNZ M/4:1995 specification for crushed basecourse (TNZ 1995), except that the specification may be relaxed to permit a higher proportion of fines in the material. For lightly trafficked roads further relaxation of the aggregate requirements is possible.

Lime- or cement-stabilised modified basecourses have generally performed very well in practice, although uniformity of mixing and control of the water content at the time of compaction is needed (Banks and Bargh c.1981).

Appropriate specification of lime- or cement-stabilised basecourses requires further investigation. Many of the normal unbound basecourse specification requirements, such as crushing resistance and particle size distribution characteristics, are unchanged. Increased quantities of fines may be permitted and usually the sand equivalent requirement is deleted. Acceptance control of lime-soil or cement-soil mixing and reaction is more difficult and a low plasticity index is not a reliable guide.

Instead an increase in plastic limit or soaked CBR appears more useful for acceptance control (Banks and Bargh c.1981, Bell et al. 1986).

Fine and clayey gravels, silty sands, and sands which are modified by lime or cement may break down under traffic even when they meet normal design criteria (i.e. CBR 80-120) (NAASRA 1986).

### **6.3.2 Cemented Materials**

Lime- or cement-stabilised cemented materials with low resilient modulus (<1,000 MPa) are generally not suitable for use as a basecourse. Because they have insufficient tensile strength they would break down to their components, so it is better to design it as a modified material from the start (NRB 1989).

Stiff-cemented basecourses (resilient modulus >1,000 MPa) are required to be stiff, but not so stiff that cracking of significant width would occur as a result of shrinkage and temperature stresses. Control of shrinkage is also important. The NRB (1989) Manual does not provide design charts, but AUSTROADS (1992) and MRD (1990) offer some design guidance for Australian conditions.

### **6.3.3 Rehabilitation**

For mix-in-place lime- or cement-stabilisation of an existing pavement showing shallow shear distress that requires rehabilitation, and where substantial increases in traffic loading are not envisaged, the design requirement is that:

- The basecourse CBR will improve with stabilisation, and
- The stabilised basecourse will have a laboratory soaked CBR of not less than 80 and not more than 120.

## **6.4 Design of Stabilised Mixes**

### **6.4.1 Objectives of Mix Design**

The objectives of the design of stabilised mixes are to select an appropriate soil, or combination of soils, and to establish whether to use lime or cement, or both, to stabilise the soil, and in what proportions. For in-situ stabilisation the characteristics of the soil are essentially fixed, unless a second soil is added or mixed in. Then, usually, the primary objective of the mix design procedure is to establish an appropriate content of lime or cement for construction of the stabilised layer.

Secondary objectives are to obtain maximum dry density, and optimum water content. Information may also be sought on the variation of its strength etc., with variation in the lime content, or cement content, or density. Conditioning and pre-treatment may also be assessed.

Table 6.1 Approximate lime contents for a number of different soil types (from Metcalf 1977).

| Soil Type                | Lime Requirement for Modification (% *) | Lime Requirement for Cementing (% *) |
|--------------------------|---|--------------------------------------|
| Basecourse †             | 1-2                                     | Not recommended                      |
| Well-graded clay gravels | 1-2.5                                   | ~ 3                                  |
| Well-graded sands ‡      | Not recommended                         | Not recommended                      |
| Poorly graded sands ‡    | Not recommended                         | Not recommended                      |
| Sandy clay               | Not recommended                         | ~ 4                                  |
| Silty clay               | 1-2.5                                   | 1.5-3                                |
| Heavy clay               | 1-3                                     | 2.5-6                                |
| Very heavy clay          | 1-4                                     | 3-6                                  |
| Organic soils            | Not recommended                         | Not recommended                      |

**Notes:**

- \* Percent is CaO as a percentage of the dry weight of soil.
- † Lime is only effective if fines are plastic.
- ‡ Lime is used in bitumen-stabilisation to promote adhesion of sands with some bitumens. Refer Chapter 13 of this report.

Table 6.2 Approximate cement contents for a number of different soil types (from Metcalf 1977).

| Soil Type                | Cement Requirement (%) |
|--------------------------|------------------------|
| Basecourse *             | 0.5-1.5                |
| Well-graded clay gravels | 1-3                    |
| Well-graded sands        | 2-4                    |
| Poorly graded sands †    | 4-6                    |
| Sandy clay               | 4-6                    |
| Silty clay               | 4-8                    |
| Heavy clay ‡             | 5-12                   |
| Very heavy clay §        | 6-15                   |
| Organic soils ¶          | 6-15                   |

**Notes:**

- \* Cement may be used as a construction expedient to aid "set up" on compaction, to reduce sensitivity to compaction water content, and to prevent ravelling under construction traffic. Also to increase resistance to heavy axle loads.
- † Compaction may be very difficult, and segregation of the cement may occur.
- ‡ Mixing may be difficult. Lime-stabilisation may be more appropriate.
- § Mixing likely to be very difficult. Lime pre-treatment may help. Lime-stabilisation may be more appropriate.
- ¶ Pre-treatment with lime or addition of 2% calcium chloride may help.

Laboratory trial mixes should be made to determine the lime or cement content to meet the selected criteria, at densities and water content levels considered to be realistic in the field.

Experience with the local soils and local conditions helps most when selecting lime or cement contents for the laboratory tests. Alternatively the percentages listed in Tables 6.1 and 6.2 may be used as a guide on which to base laboratory testing.

For typical New Zealand roading uses, much greater than 4% lime would not be commonly used, and the usual lime range is between 1% and 3%.

The laboratory tests to use have been outlined in Chapter 5 of this report, while some of the criteria that are to be achieved are indicated in this Chapter 6.

#### **6.4.2 Adjusting Laboratory Results to Field**

More variability occurs in spreading the lime or cement, pulverisation, mixing, compaction and curing, in the field than in the laboratory tests. This variability should be included in the mix design.

To nominally address this variability, an addition of 0.5% of lime or cement is usually made in the field to that required by the laboratory testing (Dunlop 1977).

### **6.5 Design for Fatigue**

When a load is applied to a cemented material the initial response is elastic. This elastic response is to a point around 35% of the stress at break, or 25% of the strain at break. As the load is increased the response becomes elasto-plastic and micro-cracking develops (Litwinowicz 1982). Repeated loading into the elasto-plastic region causes propagation of the micro-cracks.

To control this crack development a fatigue factor,  $f$ , is usually applied in the design to reduce allowable tensile stresses as the design number of heavy load repetitions increases. NRB (1989) proposes the following calculation to allow for the fatigue factor:

$$f = \frac{18.25 - \log N}{18.75}$$

where  $N$  is the design traffic loading.

For fine-grained soils (original CBR <10) with a three-fold or more increase in CBR with stabilisation,  $f$  is taken as equal to 1 (NRB 1989).

As fatigue testing is time consuming and expensive it is rarely, if ever, carried out in New Zealand for pavement design.

### **6.6 Economics of the Stabilisation Option**

The economics of stabilisation should be reviewed after the pavement design has been completed, as then factors affecting construction costs are more accurately known. These factors include the thicknesses of the pavement layers and the application rate proposed for the lime or cement.

Several stabilisation options may need to be costed to select the most economic balance between lime or cement application rate, stabilised layer thickness, and possibly density, and unbound layer thickness. Non-stabilisation options may also be included in the evaluation.

Comparison of the construction costs of these options will allow selection of the lowest cost pavement option.

Compared to non-stabilised layers, stabilisation often has advantages additional to those shown merely by the construction costs. These may include some combination of the following:

- Reduced sensitivity to moisture, and hence more tolerance to pavements with mediocre drainage and/or weak subgrades.
- Possibly reduced long-term maintenance costs.
- Prevention or inhibition of contaminating the granular sub-base or basecourse layers with fines that have been pumped up from the subgrade.
- Prevention of losing granular sub-base particles into weak subgrades.
- Speed of construction, particularly for new pavements over very weak subgrades.
- Provision of an "anvil" to allow greater density to be achieved in overlying sub-base and basecourse layers.
- Improved frost resistance.
- Ability to strengthen the pavement while maintaining the existing pavement surface levels.
- Reduction of trips to truck out existing materials and truck in new materials. This reduces wear on other roads and reduces CO<sub>2</sub> emissions.
- Conservation of aggregate resources.



## 6.7 Examples

### 6.7.1 Selection of Stabilising Agent

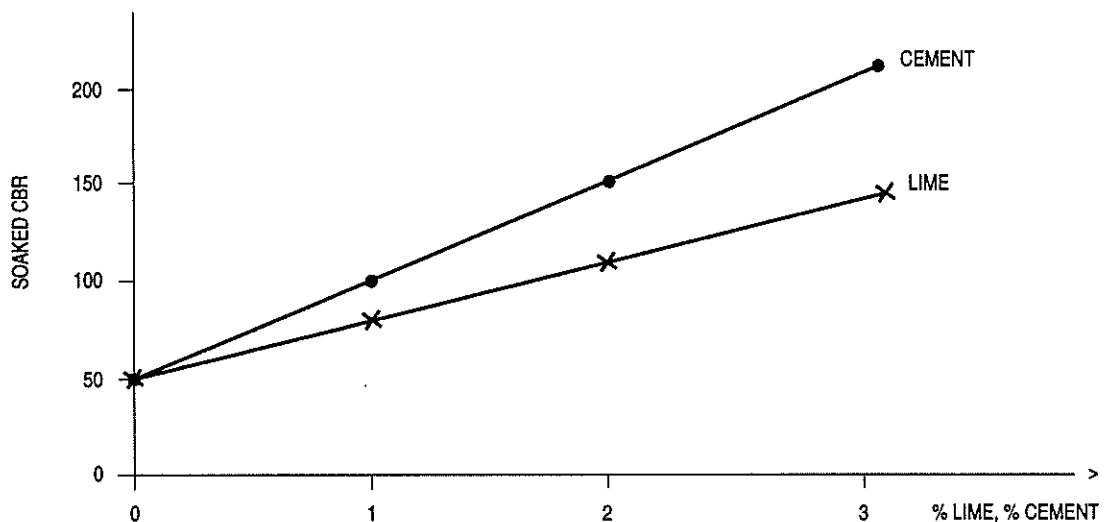
*An old existing pavement shows shallow shear and flushing of the seal coat in the wheel paths. Therefore it is proposed to rehabilitate the pavement using in-situ lime- or cement-stabilisation. Should lime or cement be used, and at what percentage?*

A check of the records, a walk-over inspection, and some test pits show:

- The pavement is of unbound aggregate with a chipseal surface.
- The edge drainage is good.
- The seal coat is a six year old Grade 3.
- Traffic loadings are not expected to increase sharply.
- The aggregate is quite well-graded but contains some plastic clay. The thickness of the aggregate layer ranges between 275mm and 400mm.

Laboratory sampling, then testing, of the basecourse for reaction to lime or cement gives the results shown in Figure 6.1.

Figure 6.1 Laboratory test results for selection of lime- or cement-stabilising agent. The percentage lime has been calculated by the laboratory as percent calcium oxide to dry mass of soil.





The delivered costs of quicklime and cement are very similar. The local contractors are experienced in both lime-stabilisation and cement-stabilisation. Construction will be carried out in summer so the ground temperature can be expected to be around 15°C to 20°C. Traffic volumes are only moderate so the stabilised surface can be trafficked for a period before first coat sealing.

The laboratory test results with the pavement material shows that cement reacts better than lime, and 1% cement gives a soaked CBR of 100 which is between the desirable CBR limits of 80 to 120.

Therefore the cement content indicated by laboratory tests is 1%.

To allow for construction variability arising during construction, 0.5% cement is added to this (see Section 6.4.2 of this report).

**Therefore selected cement application rate (% by dry mass) = 1.5%.**

### 6.7.2 Thickness Design for Stabilised Sub-base

*For construction of a new road the subgrade material is silty clay and responds well to lime, giving a CBR for the lime-stabilised material of 25 at a density that can be achieved in the field. The response to cement is not as good as the response to lime.*

|                                    |                            |
|------------------------------------|----------------------------|
| Design Traffic Loading             | = 5 x 10 <sup>5</sup> EDA* |
| Subgrade Design CBR (unstabilised) | = 4                        |

From recent tenders, costs† for estimate purposes are:

|  |   |
|--|---|
| Basecourse, supply and construct                         | \$50/m <sup>3</sup> (compacted measure) |
| Granular sub-base all passing 40mm, supply and construct | \$40/m <sup>3</sup> (compacted measure) |
| Lime-stabilisation<br>150mm-thick layer                  | \$4.25m <sup>2</sup>                    |
| Lime-stabilisation<br>200mm-thick layer                  | \$5.60m <sup>2</sup>                    |

Calculations:

|                                   |             |
|-----------------------------------|-------------|
| From NRB (1989) Resilient Modulus | E = 10 CBR  |
|                                   | = 10 x 25   |
|                                   | E = 250 MPa |

---

\* EDA Equivalent Design Axle.

† Costs are in NZ\$ (1995) and are indicative only.

From NRB (1989) Allowable Tensile Stress

$$S_t = \frac{E}{1.4}$$

$$= \frac{250}{1.4}$$

$$S_t = 180 \text{ kPa}$$

For this material no allowance for fatigue is required, i.e.  $f = 1$ .

Allowable tensile stress ( $S$ ) in the stabilised layer:

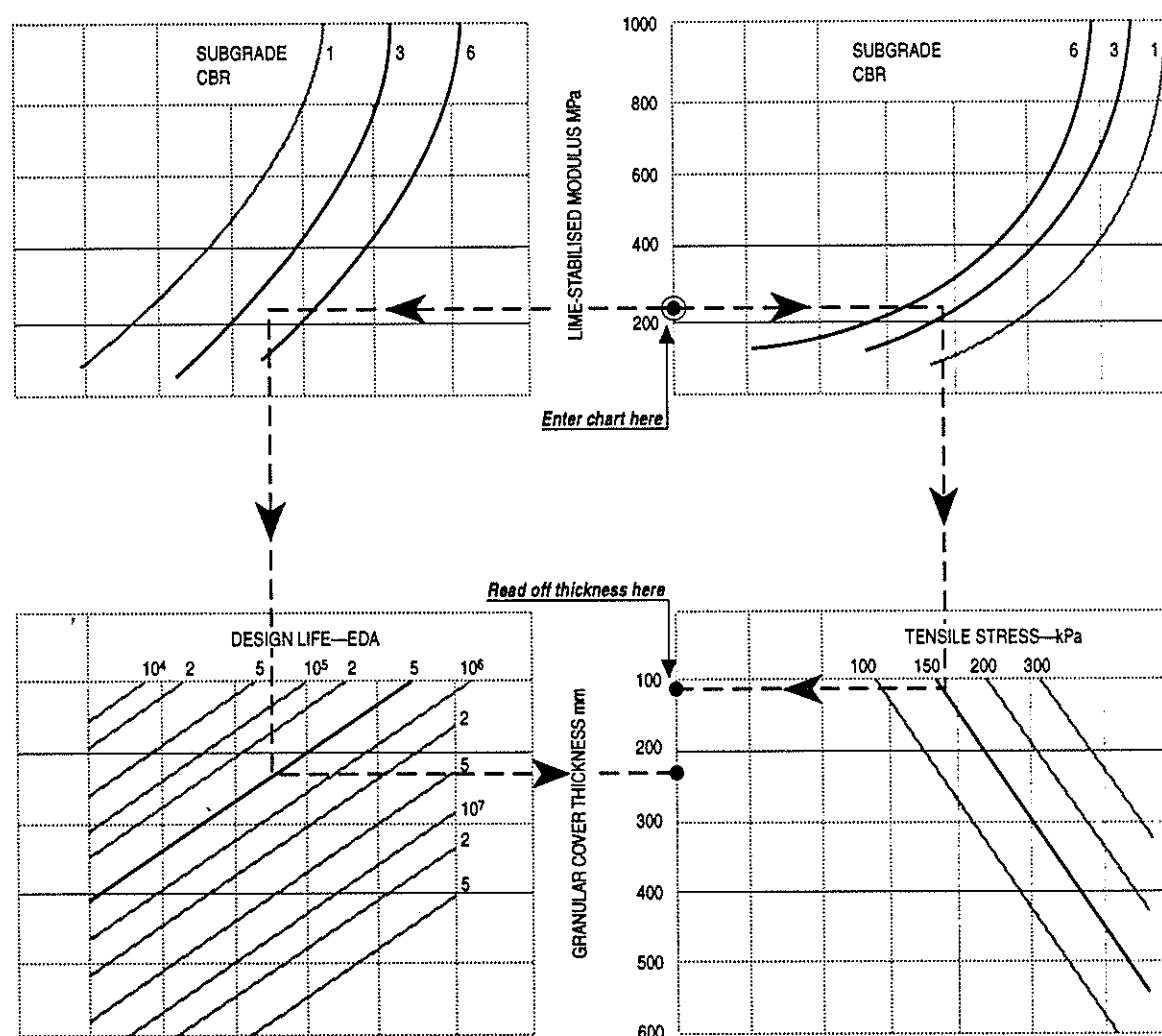
$$S = f S_t$$

$$= 1 \times 180$$

$$S = 180 \text{ kPa}$$

The above results are then applied to the design charts (Figures 9 and 10 in the NRB (1989) Manual) for cement- or lime-stabilised sub-bases that are 150mm thick and 250mm thick respectively.

Figure 6.2 Design Chart for granular cover required over a 150mm thick cement- or lime-stabilised sub-base (details in Figure 9 in NRB (1989) Manual).



## 6. Design of Stabilised Pavements

From these charts (Figure 6.2 in this report, and Figure 9 in NRB (1989) Manual), the thicknesses of the granular cover layers required are:

| Criteria  | Granular cover (mm) required over stabilised sub-base |                |
|---|---|----------------|
|   | if 150mm thick  | if 250mm thick |
| Subgrade Strain   | 230   | 120            |
| Sub-base Tensile Stress                                   | 60  | 0              |
| <b>Minimum Granular Cover based on Governing Criteria</b> | <b>230</b>  | <b>120</b>     |

Thus the minimum required granular cover thickness is 230mm for a 150mm-thick stabilised layer, 120mm for a 250mm layer, and so by extrapolation 175mm for a 200mm-thick layer. (This exceeds the minimum of 150mm required by Section 5.3 of NRB (1989) Manual.)

Using the unbound pavement design charts in the NRB (1989) Manual, with subgrade CBR 4 and traffic loading  $5 \times 10^5$  EDA, the thickness of 230mm can be made up of 140mm of primary basecourse and 90mm of sub-base. The granular thicknesses of 120mm and 175mm are too thin to allow for a granular sub-base so the entire granular layer thickness should be primary basecourse.

Pavement options and costs excluding drainage, subgrade preparation, seal coats, then are:

| Costs \$/m <sup>2</sup> * |                     |  |   |                        |
|---------------------------|---------------------|--|---|------------------------|
| 140mm                     | Basecourse          | $0.14\text{m} \times \$50/\text{m}^3$      | = | 7.00                   |
| 90mm                      | Granular Sub-base   | $0.09\text{m} \times \$40/\text{m}^3$      | = | 3.60                   |
| 150mm                     | Stabilised Sub-base | $\$4.25/\text{m}^2$                        | = | 4.25                   |
|                           |                     |  |   | <hr/>                  |
|                           |                     |  |   | \$14.85/m <sup>2</sup> |
| 175mm                     | Basecourse          | $0.175\text{m} \times \$50/\text{m}^2$     | = | 8.75                   |
| 200mm                     | Stabilised Sub-base | $\$5.60/\text{m}^2$                        | = | 5.60                   |
|                           |                     |  |   | <hr/>                  |
|                           |                     |  |   | \$14.35/m <sup>2</sup> |
| 120mm                     | Basecourse          | $0.12\text{m} \times \$50/\text{m}^2$      | = | 6.00                   |
| 250mm                     | Stabilised Sub-base | $\$5.60/\text{m}^2 \times \frac{250}{200}$ | = | 7.00                   |
|                           |                     |  |   | <hr/>                  |
|                           |                     |  |   | \$13.00/m <sup>2</sup> |

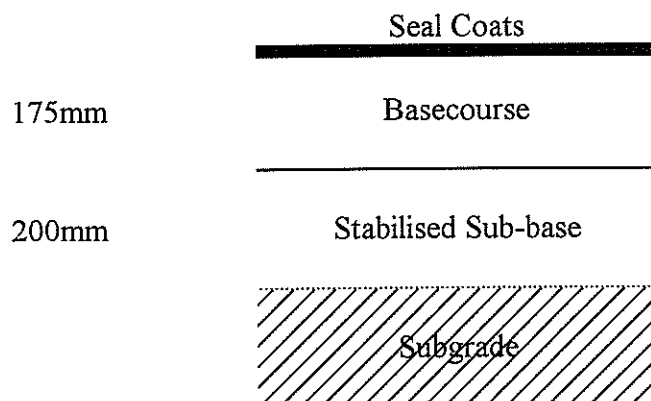
\* Costs are in NZS (1995) and are indicative only.

These cost calculations indicate that the pavement with a 250mm-thick stabilised sub-base has the lowest cost.

However it is difficult to achieve full compaction right to the bottom of a 250mm-thick layer, even if the weaker subgrade is dry and hence strong enough to provide a good anvil at the time of compaction.

Even with a strong subgrade at the time of construction and with appropriate rollers, a more practical maximum thickness of sub-base to ensure full compaction is 200mm.

Therefore the pavement option selected is:



## 7. CONSTRUCTION OF STABILISED LAYERS

### 7.1 Methods

While both lime- and cement-stabilisation generally have a good performance history, failures in stabilised materials usually are more pronounced than those in unstabilised materials. Particular care is needed first to ensure well compacted subgrade and good drainage, and then, in the stabilisation construction, to ensure thorough mixing of the soil with the lime or cement in the intended proportions, good compaction, and good surface preparation (NAASRA 1986, Dunlop 1977).

The two principal methods of lime- or cement-stabilisation are:

- Mix-in-place
- Plant-mix

The **mix-in-place method** is by far the most popular. It involves mixing the soil in-place on the site with lime or cement and water. Usually specialised self-propelled pulverising and mixing machines are used. Construction begins with the preparation and shaping of the soil or material to be stabilised. Pulverisation, spreading and mixing the lime or cement and water with the soil, and compaction and curing of the stabilised layer, follow. All these operations are carried out in-place on the site.

The **plant-mix method** uses a stationary mixing plant to proportion and mix the lime or cement, water and aggregate(s). The plant usually employs continuous mixing. The mixed material is transported to the site in trucks and laid, usually with a self-propelled paver, then compacted and cured.

The choice of mix-in-place or plant-mix is usually based on cost, equipment availability, and the type of material to be stabilised. Plant-mix can produce a material with a more uniform quality, but mix-in-place is usually faster and less expensive, and uses more readily available equipment. Granular materials are generally required for plant-mixing, while a wide range of soils including clays can be mixed-in-place.

### 7.2 Construction Sequence

While they will vary depending on the job type, the basic steps in the stabilisation construction sequence include:

- Prepare edge drainage.
- Prepare formation and shape.
- Scarify (or rip) (Plate 3).
- Remove or break any large lumps (Plate 4).

- Pulverise.
- Adjust the water content.
- Compact lightly.
- Spread the lime or cement.
- Slake, if using quicklime.
- Mix and pulverise, or mix.
- Compact fully.
- Trim to shape.
- Traffic and cure.
- Seal with bitumen.

### 7.3 Mix-In-Place Equipment

Mix-in-place equipment travels over the area to be stabilised. Equipment should be selected that will adequately pulverise the soil and uniformly mix it with measured quantities of lime and water to a controlled depth (NAASRA 1986). The strength gain from stabilisation is strongly dependent on the degree of pulverisation (Ingles and Metcalf 1972).

The selection of the most suitable stabilisation mix-in-place equipment will depend on:

- Type of material to be stabilised,
- Size and location of the stabilisation job,
- Depth of stabilised layer,
- Availability of equipment and trained staff,
- Standard of work required.

Of these, the type of material is the most important factor in selection of mixing equipment. Materials with higher fines content and higher plasticity are more difficult to pulverise and mix (ACI 1990).

The following equipment can be used for mix-in-place stabilisation:

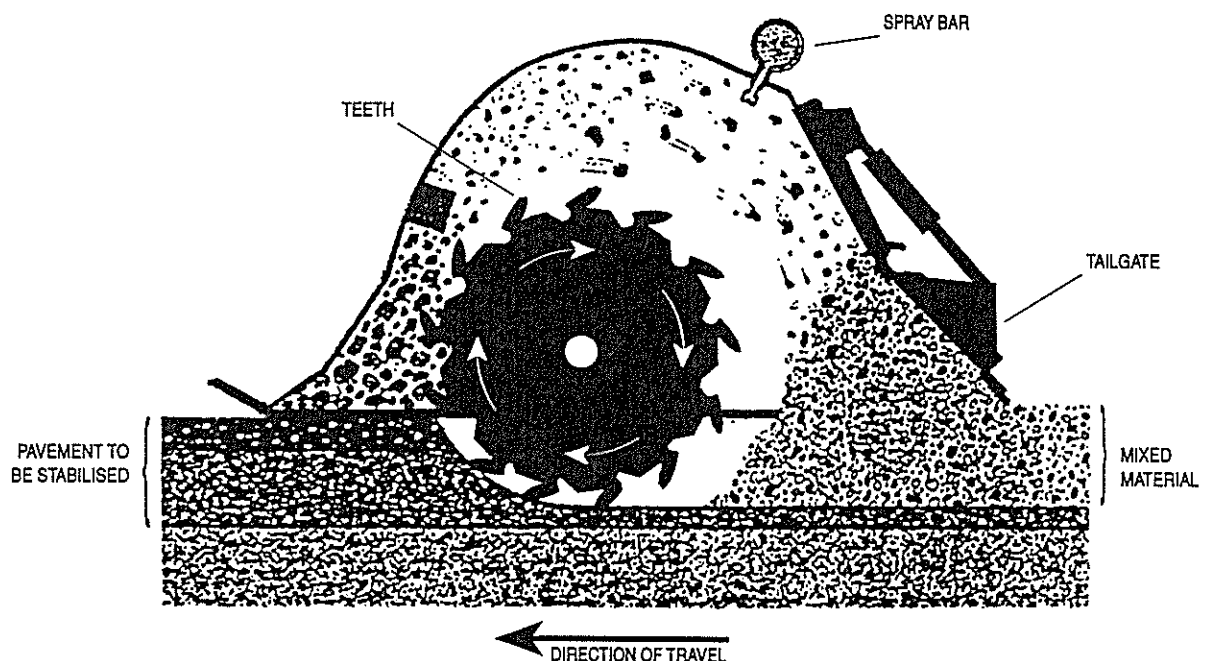
- **Graders** can be used for scarifying and shaping. Graders can also be used for mixing lime with some soils, but generally they are not suitable for heavier clay soils. As a guide, Bulman (1972) suggests a limit to soils suitable for grader mixing, based on soil plasticity index (PI) and percentage fines passing a 425µm sieve, of:

$$(PI) \times (\% \text{ finer than } 425\mu\text{m}) < 1,000$$



- **Disc Harrows**, disc ploughs and giant discs can be used for scarification, preliminary pulverisation or for preliminary mixing of the lime with the soil. Ripping of the soil may be required first.
- **Agricultural Rotary Hoes** generally require a number of passes to pulverise the soil, but have been used successfully for maintenance repairs and some minor road construction. They are not recommended for soils with medium to high plasticity (e.g. clays with  $PI > 20$ ; Bulman 1972) as they do not pulverise the soil finely, and attain relatively poor mixing uniformity (ACI 1990).
- **Multi-Rotor Mixers** are designed to do all the mixing in one pass, although pre-treatment of heavy clays with half the prescribed quantity of lime may be necessary before final mixing. Control of pulverisation and mixing is obtained by varying the forward speed, with slower speeds giving better pulverisation and mixing. Water can usually be added through a spray bar in the mixing chamber. Mixing depths normally range from 150mm to 250mm with mixing widths ranging from 1.5m to 3.5m.
- **Single-Rotor Mixers** are now almost exclusively used in New Zealand for stabilisation (Figure 7.1). These are capable of doing the pulverisation and mixing in one pass, although some machine manufacturers recommend two passes (Ritchie 1993). Heavy clays may require pre-treatment with half the prescribed quantity of lime before final mixing.

Figure 7.1 Diagram showing mixing and pulverisation with a single-rotor mixer in the mix-in-place process (after Mueller 1991).



Control of pulverisation and mixing is obtained by varying the forward speed which is usually independent of rotor speed, and by the degree of tailgate opening. Water can usually be added through a spray bar in the mixing chamber. Mixing depths normally range from 150mm to 250mm, although some modern mixers can mix to 400mm compacted depth (CACA 1993b).

The shape of the teeth on the rotor influences the pulverisation and mixing ability, and needs to be suited to the type of soil being stabilised. Generally bullet-shaped teeth are suited to aggregates, while hockey stick-shaped teeth are better suited to clays but may also suit aggregates.

For single-rotor mixers pulverisation can be improved by:

- Slower forward speed of the mixing machine while maintaining rotor speed,
- Closing the flap on the back of the mixing chamber,
- Making more passes with the mixing machine,
- Replacing worn mixer teeth,
- Conditioning (two stage mixing) of material,
- Pre-treating highly plastic soils with lime,
- Adjusting the water content of the soil.

Job productivity will be governed by the capacity of the lowest output plant item in the stabilisation plant train. For maximum productivity the performance of all the plant in the train should be matched. For lime-stabilisation in hot weather, the water cart frequently will be the limiting item, and two or more water carts may be needed to speed up the job.

## **7.4 Plant-Mix Equipment**

The two basic types of plant-mixers are: pugmill mixers for either continuous flow or batch mixing, and rotary-drum mixers. The most common plant-mix equipment is the continuous flow pugmill mixer. It is usually used for mixing granular materials (ACI 1990).

## **7.5 Ancillary Equipment**

### **7.5.1 General**

Bringing in the necessary equipment for lime- or cement-stabilisation can be expensive. In remote areas larger jobs, or a combination of smaller stabilisation jobs, are favoured.

There are advantages in efficiency if the individual plant items are matched to each other, e.g. rollers should match the same width and depth as the rotary mixer. The capability of both rotary mixer and roller(s) must be adequate to achieve the required stabilised layer thickness.

#### **7.5.2 Rippers and Grid Rollers**

Scarification or ripping the existing pavement is usually carried out with a large grader equipped with rear-mounted hydraulic rippers. If the existing pavement is known to have many seal coats or asphaltic concrete layers, a D6 or equivalent tractor is more appropriate, at least for the initial ripping.

Ripping may produce large lumps of material. To break up these lumps, multiple passes of a grid roller or a pad foot roller, or windrowing followed by passes of a heavy (e.g. 13 tonne) vibrating roller with a smooth steel drum, may be effective (Simmons 1991). The possibility of damage to buried services, weakening of the subgrade, or disturbance to nearby buildings should be considered before using such a heavy vibrating roller.

#### **7.5.3 Water Distribution**

For cement-stabilisation, water can be added by spray bar(s) attached to the water cart, or by jets within the mixing chamber of the rotary mixer. Adding water directly into the mixing chamber encourages uniform application of the water, avoids run-off, and may save an extra mixing pass (Kennedy and Clark 1990), but can create practical problems. These problems may include wet spots should the hoe's forward movement stall, and burnt clutches on the feeder tank truck when carrying out stabilisation on steep roads.

Lime-stabilisation using quicklime requires large quantities of water for slaking, and thus requires large water carts with pressurised spray bars. A side-arm spray bar avoids driving the water cart over the slippery slaked or partially slaked lime. A quick fill system for the water carts can speed up the larger lime-stabilisation jobs. For example, a large storage water tank should have tall enough legs for the water cart to drive under it for filling by gravity (Simmons 1991). Two or more water carts may be needed for job continuity on lime-stabilisation.

#### **7.5.4 Spreaders**

For larger jobs, lime or cement is usually supplied and spread in bulk. It is important for the spreader to achieve a uniform distribution of the lime or cement, and in the proper proportions (see Section 12.5 of this report).

*Spreaders for Quicklime.* Provided that adequate safety precautions are taken, then quicklime can be applied to the road using agricultural spreaders, box spreaders, roller spreaders (Plate 5), or tailgate spreading. Bagged quicklime may be used on smaller jobs.

*Spreaders for Hydrated Lime.* Dust can be a problem when spreading hydrated lime. Bulk hydrated lime can be spread with purpose-made mechanical spreaders, such as those incorporating a rotary vane geared to the wheels to distribute the lime uniformly. The lime hoppers should be specially designed to prevent arching of the lime (i.e. the lime "locking" or "bridging" between opposite sides of the hopper and not feeding to the spreader).

*Spreaders for Lime Slurry.* The hydrated lime and water slurry is mixed either in a central mixing tank, jet mixer, or in a tank truck. The slurry is spread over the scarified road by a tank truck equipped with a spray bar.

The lime in the slurry is prevented from settling by using pumps or air jets on the tank truck, or by adding the slurry directly from the agitated storage tank to the tank truck immediately before each spray run (TRB 1987).

*Spreaders for Cement.* Bagged cement may be used for smaller jobs. Generally cement is supplied in bulk then spread using purpose-made mechanical spreaders. Cement can be moved pneumatically or by gravity from the bulk cement truck to the spreader. Some spreaders can be supplied by bagged cement. The spreaders may use a screw or rotary vane, possibly geared to the road wheels, to distribute the cement uniformly on the road.

Because of the variable handling properties of cement, depending on its degree of aeration, mechanical spreaders require careful design and calibration. Hoppers should be specially designed to prevent arching of the cement.

## **7.6      Compaction**

A high degree of compaction of all layers of the pavement is essential to obtain the full benefits of stabilisation. Inadequate compaction of the stabilised layer weakens it, and may lead to early failure of the pavement. Achieving a high degree of compaction in the stabilised layer is one of the most important factors in stabilisation.

The degree of compaction will be influenced by the characteristics and water content of the material being stabilised, the compaction equipment, the strength of the underlying layer(s), and the time elapsed since mixing the lime or cement with the soil. A suggested target for compaction (NLA 1991) is:

- Bases compacted to at least 102% of New Zealand Standard Compaction.
- Subgrades and sub-bases compacted to at least 100% of New Zealand Standard Compaction.

Vibrating rollers are most suitable for compacting stabilised granular materials. For stabilised heavy clays, sheepfoot rollers may be used first, followed by pneumatic tyred rollers, or vibrating rollers used first followed by pneumatic tyred rollers.

The strength of the underlying subgrade influences the compaction of the stabilised layer. Over weak subgrades heavy high-amplitude vibrating rollers may not be effective, and instead more thinner lifts, compacted with static rollers or light low-amplitude vibrating rollers, may be more effective.

Single lifts of 200mm to 225mm compacted thickness are usually considered the practical maximum thickness that can be fully compacted. Such lifts require heavy vibrating rollers and a firm subgrade to achieve full compaction.

Recent Australian trials using slow-setting blended cements suggest that, in one lift, layers up to 350mm compacted thickness can be constructed for rehabilitation of rural highways (CACA 1993a, 1993b, Porter 1992, Dash and Bilaniwskyj 1994). A typical compaction train for a 350mm lift would include:

- A 15 tonne, single drum, vibrating, padfoot roller with pads at least 200mm proud of the drum, making approximately 12 passes,
- A grid roller (optional, used to remove padfoot imprints),
- An 8 tonne, smooth drum, vibrating roller, making 6 to 8 passes,
- A pneumatic tyred roller.

Even so, indications were that the bottom of the 350mm stabilised layer was not fully compacted (Dash and Bilaniwskyj 1994).

### **7.7 Drainage**

Edge drainage, in accordance with good roading practice, should be provided before stabilisation. Edge drainage should be located beyond the width of the proposed seal edge to allow room for construction. For example the road base should extend beyond the surfacing, and the sub-base should extend beyond the base (Dunlop 1977, Tait 1981). Edge drainage and shaping of the formation before stabilisation should either improve or control subgrade strength and thus enable passage of the heavy construction machinery (Jack 1991).

### **7.8 Preparation of Formation and Shaping**

For mix-in-place stabilising, the road should be close to its final shape before stabilisation because rotary mixers will not smooth out depressions, or correct cross-section profiles (Simmons 1991). Rotary mixers are intended to lift the material in a vertical plane, pulverise the soil, mix the water and lime or cement, then drop it back in its original position.

Therefore addition of granular material that is then compacted may be required to fill depressions, or to correct profiles before rehabilitation by stabilisation.

## **7.9 Services and Service Covers**

Existing services buried close to the surface of the road should be either lowered to below the depth of scarification and pulverisation, or removed, before stabilisation commences. Heavy plant on weak pavements or weak subgrades can displace kerb and channel or buried services sideways (Jack 1991).

Existing buried services close to the surface of the road but outside the traffic lanes (i.e. in lightly trafficked areas) may not need to be disturbed as the depth of stabilisation immediately above them can be reduced instead.

Most mix-in-place stabilisation equipment is difficult to work hard up against serviceholes and service covers (Plate 6). This difficulty is overcome by using a grader, backhoe or hydraulic excavator to move the material away from the service cover before mixing. After mixing the material is re-spread against the service cover (CACA 1988).

## **7.10 Stabilisation with Lime plus Cement**

For some materials, e.g. sandy silty clays, stabilising with both lime and cement can be the most economic stabilising option. If so, laboratory testing will demonstrate if it is economic.

Construction procedures follow those for lime-stabilisation and cement-stabilisation, but the lime-stabilisation is always carried out before the cement-stabilisation. Mixing the cement into some soils may be easier if a time delay (e.g. up to a week according to NLA 1991) between the two operations is allowed.

## **7.11 Storage**

Dry storage must be provided for both quicklime and cement. Should quicklime get wet in storage the heat generated by the hydration reaction could create a fire danger.

Doors and windows of the storage room should not be opened unnecessarily. Even when stored inside, quicklime should be covered with a tarpaulin to reduce the rate of carbonation (see Section 7.12 below).

Any quicklime or hydrated lime which has been stored for more than three weeks should be tested for available lime index before it is used (Sherwood 1994).

### **7.12 Carbonation**

The carbon dioxide in the atmosphere can cause hydrated lime, and to a lesser extent quicklime, to carbonate (i.e. to convert back to calcium carbonate) during storage before it is used for soil stabilisation. Even hydrated lime or quicklime stored in unopened paper bags can carbonate, although the polythene- or wax-lined bags used by some manufacturers should greatly reduce the rate (M Eberlin and PW Rolls, pers.comm.1996).

The rate of carbonation for stored quicklime is suspected to relate to the humidity of the atmosphere, presumably because the humidity hydrates the lime. The resulting hydrated lime being a fine powder has a much greater surface area exposed to the air than lumps of quicklime. The rate of carbonation therefore increases.

After spreading and slaking, hydrated lime is particularly vulnerable to carbonation, especially in cool damp weather. Lime should be mixed into the soil as soon as practical after slaking commences, e.g. within half an hour (BR Webster, pers.comm.1995).

Lime-stabilised and cement-stabilised materials can also carbonate. This almost invariably results in a loss in strength. In extreme circumstances up to 50% of the strength may eventually be lost.

In carbonation of lime-stabilised or cement-stabilised materials the free hydrated lime carbonates initially, so it is no longer available for reaction. Subsequently destruction of the already formed cementing compounds is possible. Further research is needed to fully understand carbonation of lime- or cement-stabilised materials.

The best methods for reducing or preventing carbonation of stabilised materials are generally those that retain moisture in, and exclude air from, the stabilised layer (Netterberg et al. 1987, Sampson et al. 1987) such as:

- Bituminous sealing or priming as soon as possible,
- Curing under plastic sheeting,
- Curing under a cover of moist sand,
- Constructing overlying aggregate layers as soon as possible.

### **7.13 Ground Temperature**

Both lime-stabilisation and cement-stabilisation are temperature-dependent. As the ground temperature increases so does the rate of reaction. Consequently it is desirable to carry out stabilisation in the summer when the ground temperature is about 20°C (Dunlop 1977).

The following are offered as guidance (ACI 1990, NLA 1991, Dunlop 1977, Hudson and Simmons 1991):

- In no case should lime- or cement-stabilisation be carried out when the ground temperature is below 3°C.
- For lime-stabilisation the ground temperature should be 10°C or higher.
- For cement-stabilisation the ground temperature should be 5°C or higher.
- One rule of thumb for stabilisation is "air temperatures should be at least 5°C in the shade and rising" (NLA 1991, ACI 1990).

#### **7.14 Proprietary Sealing Binders**

For first coat sealing of lime- or cement-stabilisation, the usual practice is to use a binder of 180/200 penetration grade bitumen with kerosine cutter and adhesion agent. The major requirements to achieve a good bond of the binder to the stabilised material are a clean dust-free surface, and an amount of cutter that is appropriate to the cleanliness and temperature of the surface.

While a normal cutback binder usually works, chemically such a binder prefers to bond to acidic stones, rather than to bond in the alkaline environment of lime- or cement-stabilisation. While a clean dust-free surface is the key factor, the recognition that a cutback bitumen has such chemical preferences may be useful should any unexplained seal adhesion problems occur.

Special Road Oil (SRO) (refer NRB 1968) was a doped cutback with an affinity for alkali materials, and overcame these chemical preferences. However it is understood that SRO is no longer available.



7. *Construction of Stabilised Layers*

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Plate 3. Scarifying an urban street ...



Plate 4. ... then breaking the lumps of old asphaltic concrete.







7. *Construction of Stabilised Layers*

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Plate 5. Spreading chips of bulk lime by truck-mounted roller spreader.  
Photo: courtesy of Taylors Lime Ltd ,



Plate 6. In-place-mixing of cement for rehabilitation of an urban street.  
Workman marks service cover.





## **8. CONSTRUCTION OF LIME-STABILISATION**

### **8.1 General**

The construction procedures followed for lime-stabilisation are similar whether a modified material or a cemented material is required. They differ only in detail. Generally stabilisation to produce a cemented material requires more thorough processing and job control than that required for a modified material. The two procedures are not described separately in this Chapter 8.

As with any road construction process, care must be taken in the construction phase of stabilisation, and experienced supervisors should be available on site.

### **8.2 Mix-In-Place Method**

#### **8.2.1 General**

Mix-in-place is by far the most popular construction method. It is usually faster and less expensive than the plant-mix method, particularly for rehabilitation.

#### **8.2.2 Scarification**

Some form of scarifying (ripping), either by grader or bulldozer tynes, is usually carried out before pulverisation. The depth of scarifying should be slightly deeper than that required for producing the final depth of the stabilised layer. In materials that are difficult to rip (e.g. when rehabilitating old pavements with many layers of existing chipseal), the first few passes of the ripper should reach to half the depth of the final stabilised layer thickness, with subsequent ripping passes slightly offset and reaching to the full depth.

Modern powerful rotary mixers do not require pre-ripping of the existing pavement. Even with such machines however, it has advantages by reducing wear or even damage to the rotary mixer (Simmons 1991). Pre-ripping is particularly important in case large rocks are encountered, or for the rehabilitation of existing pavements in urban areas in case unexpected service covers are encountered.

After ripping, large lumps (e.g. over 100mm diam.) may be able to be broken by multiple passes of a grid roller, or windrowed, and then crushed by a heavy vibrating roller (Simmons 1991). Alternatively the larger lumps can be removed from the site.

#### **8.2.3 Pulverisation**

The degree of pulverisation achieved before compaction greatly influences the quality of the lime-stabilised layer.

The requirement for soil pulverisation is not as stringent using lime as it is with cement, because in time the lime will diffuse through the soil to some extent. A usual requirement (NLA 1991) for pulverisation is:

- All stone, old asphaltic concrete, and old chipseal are to be broken to pass a 50mm sieve.
- 100% of any cohesive material is to pass a 25mm sieve, and at least 60% is to pass a 4.75mm sieve.

If this cannot be achieved for cohesive soils, other treatments may be necessary, e.g. pre-treatment of the soil, or conditioning, or additional passes of the rotary mixer, or the use of a different rotary mixer.

Usually the final pulverisation, and frequently all pulverisation, is done in conjunction with mixing.

#### **8.2.4 Water**

The lime-soil reaction requires water, as does diffusion of the lime into any lumps of clay. Any water required can be applied either through a pressurised spray bar in the mixing chamber of the rotary mixer, or from a separate water cart. For wet soils mixing the unslaked or partly slaked quicklime into the soil may be better if carried out first, and then mix in extra water if it is required (see Section 8.2.7 of this report).

If slaking of the quicklime is incomplete before mixing, more water is needed and the further slaking will draw available water out of the soil.

To stabilise cohesive materials with lime, water should be added and mixed into the soil to raise the water content of the lime-soil mixture to 2% above optimum water content (McDonald 1991a-d, NLA 1991). Generally during summer months, it is difficult to over-water lime-stabilisation. Therefore excess water is generally better than less water. However excess watering is not recommended in cool damp conditions when there is little evaporation.

With cohesive soils the soil-lime mixture must not be dry, i.e. below optimum water content, at the time of compaction.

For the lime-stabilisation of basecourse-type materials, water should only be added to bring the lime-basecourse material to optimum water content (NLA 1991, Simmons 1991).

#### **8.2.5 Spreading Dry Lime**

Dry lime can be spread from bags, or in bulk, as quicklime or as hydrated lime. The primary objective of spreading is to achieve uniform distribution of the lime in the required proportions (Table 8.1).



Bagged lime is usually used only for small jobs and for drying isolated wet spots. Bags are laid on the road in a grid pattern that has been pre-calculated to give the required application rate (Appendix 4 in this report). The bags are cut open with a knife and emptied into piles, which can then be spread to a uniform depth by hand rake.

Spreading dry bulk hydrated lime should be avoided in windy conditions to reduce problems of blowing dust, particularly in residential areas.

If higher application rates of quicklime are required, two applications of lime, with each application slaked and mixed separately, may be necessary. This is to facilitate slaking. The maximum application rate depends on the size of the quicklime particles. As an indication the maximum rate would be 12 to 14kg/m<sup>2</sup> for coarse particles (e.g. >9.5 mm chips, TRB 1987), and 10kg/m<sup>2</sup> for fine particles (e.g. sand sized) of quicklime.

#### **8.2.6 Lime Slurry**

Lime can be mixed into a slurry with water and the lime slurry can be spread from tank trucks by gravity or pressure spray bars.

Lime slurry has advantages where dust is a problem, e.g. in residential areas. It can have advantages on dry soils as lime spreading and water spraying are combined in the one operation. It has advantages in hot dry weather when adding enough water to slake the quicklime can be difficult, and it also generally promotes more uniform lime distribution.

The slurry method has disadvantages when used on wet soils, and during wet weather. Other disadvantages are the cost of the slurry plant, the extra time required to prepare the slurry, and the larger number of spreader passes required (NLA 1991).

#### **8.2.7 Slaking**

If lime is supplied as quicklime it needs to be slaked with water before it can be used to stabilise the soil.

The slaking method that is appropriate depends on the particular quicklime. For example, lime chips require different treatment to sand-sized lime particles. Also different limestones and kiln types produce quicklimes with different needs for slaking and these differences in lime can vary over the years even from the one manufacturer.

The comments in this Section 8.2.7 give general guidance but the manufacturer of the lime should be consulted on the best method of slaking their lime (BR Webster, pers.comm. 1994).

Table 8.1 Weight (kg) of quicklime after it has been spread on a 1m<sup>2</sup> canvas square, for quicklime containing 90% available lime.

| Compacted Layer Thickness (mm) | Dry Density of Soil After Compaction (kg/m <sup>3</sup> ) | Lime Application Rate *                 |       |       |       |       |       |       |
|--------------------------------|---|---|-------|-------|-------|-------|-------|-------|
|                                |   | 1%                                      | 2%    | 3%    | 4%    | 5%    | 6%    | 7%    |
|                                |   | Weight / unit area (kg/m <sup>2</sup> ) |       |       |       |       |       |       |
| 100                            | 1600  | 1.78                                    | 3.56  | 5.33  | 7.11  | 8.89  | 10.67 | 12.44 |
|                                | 1800  | 2.00                                    | 4.00  | 6.00  | 8.00  | 10.00 | 12.00 | 14.00 |
|                                | 2000  | 2.22                                    | 4.44  | 6.67  | 8.89  | 11.11 | 13.33 | 15.56 |
|                                | 2200  | 2.44                                    | 4.89  | 7.33  | 9.78  | 12.22 | 14.67 | 17.11 |
| 125                            | 1600  | 2.22                                    | 4.44  | 6.67  | 8.89  | 11.11 | 13.33 | 15.56 |
|                                | 1800  | 2.50                                    | 5.00  | 7.50  | 10.00 | 12.50 | 15.00 | 17.50 |
|                                | 2000  | 2.78                                    | 5.56  | 8.33  | 11.11 | 13.89 | 16.67 | 19.44 |
|                                | 2200  | 3.06                                    | 6.11  | 9.17  | 12.22 | 15.28 | 18.33 | 21.39 |
| 150                            | 1600  | 2.67                                    | 5.33  | 8.00  | 10.67 | 13.33 | 16.00 | 18.67 |
|                                | 1800  | 3.00                                    | 6.00  | 9.00  | 12.00 | 15.00 | 18.00 | 21.00 |
|                                | 2000  | 3.33                                    | 6.67  | 10.00 | 13.33 | 16.67 | 20.00 | 23.33 |
|                                | 2200  | 3.67                                    | 7.33  | 11.00 | 14.67 | 18.33 | 22.00 | 25.67 |
| 175                            | 1600  | 3.11                                    | 6.22  | 9.33  | 12.44 | 15.56 | 18.67 | 21.78 |
|                                | 1800  | 3.50                                    | 7.00  | 10.50 | 14.00 | 17.50 | 21.00 | 24.50 |
|                                | 2000  | 3.89                                    | 7.78  | 11.67 | 15.56 | 19.44 | 23.33 | 27.22 |
|                                | 2200  | 4.28                                    | 8.56  | 12.83 | 17.11 | 21.39 | 25.67 | 29.94 |
| 200                            | 1600  | 3.56                                    | 7.11  | 10.67 | 14.22 | 17.78 | 21.33 | 24.89 |
|                                | 1800  | 4.00                                    | 8.00  | 12.00 | 16.00 | 20.00 | 24.00 | 28.00 |
|                                | 2000  | 4.44                                    | 8.89  | 13.33 | 17.78 | 22.22 | 26.67 | 31.11 |
|                                | 2200  | 4.89                                    | 9.78  | 14.67 | 19.50 | 24.14 | 29.33 | 34.22 |
| 225                            | 1600  | 4.00                                    | 8.00  | 12.00 | 16.00 | 20.00 | 24.00 | 28.00 |
|                                | 1800  | 4.50                                    | 9.00  | 13.50 | 18.00 | 22.50 | 27.00 | 31.50 |
|                                | 2000  | 5.00                                    | 10.00 | 15.00 | 20.00 | 25.00 | 30.00 | 35.00 |
|                                | 2200  | 5.50                                    | 11.00 | 16.50 | 22.00 | 27.50 | 33.00 | 38.50 |
| 250                            | 1600  | 4.44                                    | 8.89  | 13.33 | 17.78 | 22.22 | 26.67 | 31.11 |
|                                | 1800  | 5.00                                    | 10.00 | 15.00 | 20.00 | 25.00 | 30.00 | 35.00 |
|                                | 2000  | 5.56                                    | 11.11 | 16.67 | 22.22 | 27.78 | 33.33 | 38.89 |
|                                | 2200  | 6.11                                    | 12.22 | 18.33 | 24.44 | 30.55 | 36.67 | 42.78 |

\* Mass of CaO as percent (1-7%) of the dry mass of the soil after compaction.

#### Notes:

- Usual maximum application rate per application is 14kg/m<sup>2</sup> for coarse particles, and 10kg/m<sup>2</sup> for fine particles of quicklime.
- The actual size of the canvas square used should be measured. If it is not exactly 1m<sup>2</sup> then proportional adjustment will be required.
- Formula used to calculate weight of cement for this table:  

$$\text{Weight (kg) of lime / m}^2 = \text{layer depth} \times \text{soil dry density} \times \frac{\text{lime application rate}}{100} \times \frac{100}{\text{grade of lime used (90 assumed)}}$$



If slaking is not completed before the lime is mixed with the soil, then the demand for water of the unhydrated portion of lime is strong enough to draw water from the soil to continue the hydration reaction. While this may be beneficial if stabilising very wet soils in winter, generally incompletely slaked lime will dry the soil too much and inhibit full compaction. If partly hydrated quicklime is to be mixed into the soil then sand-sized lime particles are preferable, as chip lime may hydrate and swell after completion of construction and create weak areas in the pavement (BR Webster, pers.comm. 1994).

Rolls (1989) recommends the addition of one tonne of water for every tonne of quicklime. This ratio will vary depending on the weather and soil water conditions at the time of slaking. In cool damp weather and/or with wet soils less water is required. In hot dry weather much water is lost by evaporation. In fact, it is difficult to add enough water, with almost no chance of over-watering (BR Webster, pers.comm. 1994; NLA 1991).

The required application of water should be completed within two to three passes of the water cart. Pressurised water jets on the water cart can assist hydration of lime chips by physically moving lime at the surface to allow the water to penetrate to the unslaked lime beneath. Plates 7 and 8 show slaking procedures in action.

If excessive steam is generated from the reaction it can indicate that insufficient water is being added. However this is not a reliable guide to under-watering because, on hot dry (low humidity) days, steam from the reaction may not be obvious although much evaporation will be occurring (BR Webster, pers.comm. 1994).

During slaking, the lime becomes very slippery and provides little traction for machinery. If excess water is added during slaking the lime remains slippery, but if just enough water is added for slaking the quicklime converts to a dry white powder of hydrated lime which gives adequate traction.

Two extreme conditions, burning and drowning, can occasionally occur when slaking lime chips:

- Burning may arise from adding insufficient water at one time, so that extremely high temperatures and coarse crystals of hydrate are generated;
- Drowning may arise from rapidly adding far too much water so that an impervious surface of hydrated lime covers each chip and prevents further slaking (Rolls 1989). Both conditions result in incomplete slaking.

#### **8.2.8 Pre-treatment (Double Application of Lime)**

For lime-stabilisation of heavy, highly plastic clay subgrades or sub-base materials, the initial pulverisation of the soil is normally not adequate.

For such materials, scarify and partially pulverise the soil, then mix in half of the calculated amount of lime (see Section 8.2.5 of this chapter). The lime-soil layer is

then sealed off by rolling and the lime is allowed to react with the clay for a period of time, ranging from a few hours to a week (NLA 1991, TRB 1987). The lime-clay reaction will make the soil more friable. Consequently the soil can be pulverised more easily and more completely when the second half of the calculated amount of lime is added.

Before mixing, water may need to be added to bring the heavy soil to at least 2% above optimum water content to aid the disintegration of the clay clods. This is not needed in cool damp weather when the excess water is unlikely to evaporate and so hinder compaction.

Although scarifying by disc harrows, giant discs, or grader tynes may be suitable for this preliminary pulverisation and mixing, high speed rotary mixers are required for the final mixing. Motor graders are, generally, unsatisfactory for mixing lime with heavy clays (NLA 1991, TRB 1987, Dunlop 1977). (See Sections 7.3, 8.2.2, 8.2.3 of this report.)

While particularly useful for heavy clays, the stabilisation of all cohesive soils benefits from the pre-treatment process of adding half the calculated amount of lime and mixing it with the soil, then adding the other half of the lime and mixing again. Pre-treatment offers little or no advantage when lime-stabilising clayey gravels, such as are encountered in road rehabilitation (BR Webster, pers.comm. 1994).

#### **8.2.9 Conditioning (Two Stage Mixing)**

To stabilise clay subgrades, the effect of lime on the material is often increased if the layer has been left for 24 to 48 hours after it has been mixed with the entire amount of lime required. Then a final mixing is carried out.

The preliminary mixing distributes the lime and allows the initial lime-soil reaction to occur. For maximum chemical action during conditioning the clay clods should be less than 50mm in diameter after this preliminary mixing.

Before this conditioning is carried out, the soil should be wetted to bring it to 2-5% above optimum water content after this preliminary mixing. This is not needed in cool damp weather, or with wet subgrades, when the excess water is unlikely to evaporate and will hinder compaction.

After the preliminary mixing the surface should be lightly sealed off, using pneumatic tyred rollers, as a precaution to avoid over-watering from heavy rain, and to minimise evaporation and lime carbonation (NLA 1991, ACI 1990, Dunlop 1977, McDonald 1991a).

Deep stabilisation is a rarely used variation on the two stage mixing approach, but may be useful for stabilising soft subgrades. The first increment of lime is mixed to a depth of 450mm to 600mm, then compacted. Mixing may be by plough, tynes or rotary

mixer. Later the second increment of lime is rotary mixed into the upper 150mm to 300mm, then compacted (TRB 1987). As the lower portion will have only a low percentage of added lime and so be frost-susceptible, this technique should not be used in areas of heavy frost (Section 2.5.7 of this report).

The comments on mixing equipment in Section 8.2.8 of this report apply also.

#### **8.2.10 Final Mixing**

Final mixing should be carried out with a rotary mixer. Plates 9 and 10 show two types of plant used. Mixing and pulverisation should continue until the material is a uniform colour and the material meets the requirements of Section 8.2.3 of this chapter.

#### **8.2.11 Compaction and Timing**

To develop the desired strength and stability, lime-soil mixtures should be compacted to a high density. The recommendation (NLA 1991) is to a density of at least:

- 102% of New Zealand Standard Compaction for bases.
- 100% of New Zealand Standard Compaction for sub-bases and subgrades.

Compaction of lime-stabilised materials while they are dry of optimum water content may not achieve the required compaction.

Inadequate compaction will reduce the strength and stability of the lime-stabilised material. It can also leave the stabilised material open and porous which will allow carbonation. Worse, it can allow rain to saturate the material and revert it to a slush (McDonald 1991a).

Granular-type materials are usually compacted as soon as possible after mixing, although delays of up to two days are not detrimental if the soil is not allowed to dry out over that time.

Clay materials can be compacted soon after mixing, although delays of up to four days are not detrimental.

When longer delays (e.g. two weeks or more) cannot be avoided, extra lime (e.g. 0.5%), may need to be added to the soil to compensate for carbonation, erosion and leaching of lime in the surface layers (NLA 1991). Some initial compaction should be made to prevent rain saturating the soil.

The desired degree of compaction may not be possible to obtain, especially if the underlying layer(s) are weak. In this case the optimum water content should be increased if a lower compaction standard is adopted, and the design (e.g. lime content, stabilised layer thickness, thickness of granular cover) of the pavement layer should be reviewed.

### **8.2.12 Joints**

Joints form between successive passes of the rotary mixer during construction. Generally the mixer works along the road and in doing this creates longitudinal joints, except at the end of mixer runs where transverse joints are created.

Unnecessary longitudinal joints should be avoided. Longitudinal joints along the wheel tracks, particularly "cold" joints which form between operations on successive days, should be avoided.

All longitudinal joints should be lapped into the previous work by at least 100mm. When forming this overlap, lime should be added on both stabilisation runs.

At the end of each day's work a straight transverse construction joint should be formed. When starting the new day's work overlap the rotary mixer a metre or so into the previous day's work. Again, for this overlap, lime should be added to the stabilisation runs on both days (Wilmot 1991; pers.comm. 1994).

### **8.2.13 Quagmires**

For very wet sites (quagmires) quicklime may be used to dry the soil. In these cases the quicklime may be partially slaked, or completely unslaked (depending on how wet the soil is) when mixed into the soil. Only fine particles of lime should be used (e.g. <5mm) (see Section 8.2.7 of this report).

In extreme cases of quagmires the lime spreading and the mixing equipment may need to be towed across the site. Even so, the lime-stabilised areas should be able to be compacted, then carry laden trucks within hours of the mixing. The quicklime draws water out of the soil to hydrate itself, but this needs to be checked on each site to ensure that complete hydration and mixing have occurred.

## **8.3 Plant-Mix Method**

### **8.3.1 Mixing**

The lime, aggregate(s) and water are all mixed together in a central plant, then trucked to the job site if the plant-mix method of lime-stabilisation construction is used.

Plant-mix may be used for projects where the material to be stabilised is imported to the project, such as stabilised wearing courses for unsealed roads, or stabilised basecourses.

Granular materials are generally used because they are easy to handle and mix. Silty and clayey soils are generally not used for plant-mixing because they are difficult to pulverise (Mueller 1991).

While the lime-soil mixture is in a stockpile, it must be kept damp at all times. It should be delivered to the job site at slightly above optimum water content.

### **8.3.2 Spreading**

Plant-mixed material is best spread with a paver, or spreader box. Compaction can then follow immediately. Plant-mixed material can also be spread by truck, then grader.

## **8.4 Effect of Rain**

Light rain is generally not detrimental during the construction of lime-stabilisation. Heavy rain can be detrimental if it raises the water content of the uncompacted lime-soil mixture too much above optimum. Compaction cannot then be achieved and the traffic will sink into the material. If heavy rain should occur during construction, the lime-soil "slush" may need to be windrowed to the side. The slush can be dried, then re-spread later and compacted, usually with no long-term detrimental effects.

Heavy rain on recently completed lime-stabilisation can leach unreacted lime from the upper 5mm to 25mm of the stabilised layer (Jack 1991, McDonald 1991a, NLA 1991). The options for restoring this leached layer then are to:

- Re-compact if the lime has had time for initial reaction with the soil, and another layer is to be laid over the top.
- Grade it off.
- Re-mix and re-compact.
- Re-pulverise, add further lime, re-mix and re-compact.

## **8.5 Curing**

Curing of lime-stabilised materials is required for maximum development of strength and stability. Generally a curing period of some 3 to 7 days is required, depending on the ground temperature.

Curing may be provided (NLA 1991, Ingles and Metcalf 1972, Dunlop 1977, Simmons 1991) by:

- Keeping the surface moist by sprinkling with water.
- Covering with wet sacks, wet sand, or plastic sheet.
- First coat chipsealing with cutback bitumen, within one day after final rolling.

- Adding a curing coat of bitumen emulsion. This may take several applications of dilute emulsion over several days. Except for unsealed roads this will require a surfacing, e.g. chipseal, before much trafficking.
- Placing the next pavement layer on top of the lime-stabilised layer, with minimum delay.

## **8.6 Surfacing**

Until the lime-stabilised base layer has cured and hardened up, traffic may distort the surface. Accordingly final surface preparation for bituminous surfacing may be left for a period, e.g. 3 to 7 days depending on the ground temperature. To limit carbonation, bituminous surfacing should be completed within about 14 days of mixing.

If any shaping is to be carried out with the grader at this stage, stabilised material is to be cut to waste. Stabilised material is not to be cut to fill low spots because any minor filling is likely to break away from the base either before or after sealing (Wilmot 1991).

For sealing directly on to a stabilised base, a clean stone mosaic surface should be exposed by power brooming, or by spreading a sparse layer of sealing chip then trafficking, or even by a light cut with the grader. First coat sealing follows, using either cutback bitumen or emulsified bitumen. If a bitumen cure coat has been used and it is still intact, a second coat seal can be applied, with any localised surface depressions filled with bituminous material.

## **8.7 Trafficking**

Traffic should be diverted from lime-stabilisation construction operations, i.e. from scarifying to compaction after mixing.

If the stabilised layer is designed as a modified material, then controlled speed traffic may be permitted on the stabilised layer immediately following compaction.

If the stabilised layer is designed as a cemented material, then its ability to carry traffic soon after stabilisation will depend on whether this traffic will crack the stabilised material. For example, stabilisation of a well-graded crushed aggregate during mid-summer may produce a cemented material that is quite suitable for immediate trafficking because the aggregate itself should have sufficient strength to resist shallow shear. Also the subgrade should be dry and hence strong enough to provide sufficient support for the stabilised material to cement without deflection and consequent cracking.

If the stabilised layer is designed as a cemented material and is constructed over a weak subgrade, traffic should be kept off for a number of days (e.g. seven, dependent on ground temperature).

Alternatively, the stabilised layer may be trafficked immediately to encourage the development of many fine cracks in the wheel paths. If these cracks are fine enough they can be covered with the second coat chipseal. Because such cracks reduce the tensile strength of the material, this loss of tensile strength should be recognised and compensated for in the design.

For rehabilitation the practical choice may be to traffic the stabilised layer immediately, or within 24 hours. Practitioners who do this claim that it results in no detrimental pavement performance (McDonald 1991b).

Immediate construction of any overlying layers will offer some protection from traffic stresses, except those from construction equipment. Therefore the weight and amplitude of rollers used to compact the overlying layer(s) should be controlled.





8. *Construction of Lime-Stabilisation*

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Plate 7. Slaking of burnt lime using a front-mounted spray bar.



Plate 8. Slaking of burnt lime using a side-arm spray bar.







Plate 9. In-place-mixing of lime using a rotary mixer.



Plate 10. In-place-mixing of lime using a modified agricultural hoe.





## **9. CONSTRUCTION OF CEMENT-STABILISATION**

### **9.1 General**

The construction procedures followed for cement-stabilisation are similar whether a modified material or a cemented material is required. They differ only in detail. Generally stabilisation to produce a cemented material requires more thorough processing and job control than that required for a modified material, particularly for control of water content and delay until compaction. The two procedures are not described separately in this Chapter 9.

As with any road construction process, care must be taken in the construction phase of stabilisation, and experienced supervisors should be available on site.

### **9.2 Mix-In-Place Method**

#### **9.2.1 General**

Mix-in-place is by far the most popular construction method. It is usually faster and less expensive than the plant-mix method, particularly for rehabilitation.

#### **9.2.2 Scarification**

Some form of scarifying (ripping), either by grader or dozer tynes, is usually carried out before pulverisation. The depth of scarifying should be slightly deeper than that required for producing the final depth of the stabilised layer. In materials that are difficult to rip (e.g. when rehabilitating old pavements with many layers of existing chipseal), the first few passes of the ripper should reach to half the depth of the final stabilised layer thickness, with subsequent ripping passes slightly offset and reaching to the full depth.

Modern powerful rotary mixers do not require pre-ripping of the existing pavement. Even with such machines however, it has advantages of reducing wear or even damage to the rotary mixer (Simmons 1991). Pre-ripping is particularly important where large rocks may be encountered, or for the rehabilitation of existing pavements in urban areas in case unexpected service covers are encountered.

After ripping, large lumps (e.g. over 100mm diam.) may be able to be broken by multiple passes of a grid roller, or windrowed, and then crushed by a heavy vibrating roller. Alternatively these larger lumps can be removed from the site.

#### **9.2.3 Pulverisation**

The degree of pulverisation achieved before compaction greatly influences the quality of the cement-stabilised layer, particularly in silts and clays.

The requirement for soil pulverisation is more stringent and more critical using cement than it is using lime. This is one of the reasons cement-stabilisation tends to be impractical for many clays which can be difficult to pulverise.

A requirement (ACI 1990) for pulverisation for cement-stabilisation is:

- All stone, old asphaltic concrete, and old chipseal are to be broken to pass a 50mm sieve.
- 100% of any sample, excluding stone, old asphaltic concrete, and old chipseal, is to pass a 25mm sieve, and at least 80% is to pass a 4.75mm sieve.

Usually the final pulverisation, and frequently all pulverisation, is done in conjunction with mixing.

#### **9.2.4 Water**

Cement does not require water for slaking, but its cementing reactions (hydration) and subsequent reactions with the soil do require water if they are to take place. Close control of the water content is very important for cement-stabilisation. To achieve the required density, the water content at the time of compaction should be near optimum. Compaction of materials that are wet of optimum increases the risk of shrinkage cracking.

For granular soils, mixing at slightly drier than optimum reduces the likelihood of cement balls forming. For fine-grained soils, water contents near optimum may be necessary for effective pulverisation. In drier conditions, some 2% additional water may be added to account for the hydration of the cement and for the evaporation that occurs during mixing (ACI 1990).

If adding water is necessary, it should be done either during the mixing process through a spray bar in the mixing chamber of the rotary mixer, or by uniformly spraying water over the road and mixing it in before spreading the cement. Some very experienced stabilisation construction crews, when working with uniform soils, will spray water on to the spread cement immediately before mixing. Such a sequence is shown in Plate 2.

The water content at the time of compaction influences long-term shrinkage, and the higher the water content the greater is the risk of shrinkage cracking. While shrinkage may be of little consequence for modified materials it can have important consequences for strongly cemented materials. Accordingly, to reduce the risk, or amount, of cracking in strongly cemented materials, the water content should be some 2% dry of optimum, although this will also increase the compactive effort needed to achieve the required density.



### **9.2.5 Spreading Cement**

Cement can be supplied and spread from bags or in bulk (Plates 11 and 12), although spreading cement in windy conditions should be avoided to reduce problems of blowing dust. The primary objective of spreading is to achieve uniform distribution of the cement in the required proportions (Table 9.1).

Bagged cement may be used with a hand-fed mechanical spreader (Plate 12), or be laid directly on the road. Bags laid on the road should be in a grid pattern that has been pre-calculated to give the required application rate (Appendix 4 in this report). The bags are cut open with a knife and emptied into piles, which can then be spread to a uniform depth by hand rake.

### **9.2.6 Final Mixing**

Final mixing should be carried out with a rotary mixer (similar to that in Plate 13). Mixing and pulverisation should continue until the material is a uniform colour and the material meets the requirements of Section 9.2.3 of this chapter.

### **9.2.7 Compaction and Timing**

To develop the desired strength and stability, cement-soil mixtures should be compacted to a high density. The recommendation (NLA 1991) is to a density of at least:

- 102% of New Zealand Standard Compaction for bases.
- 100% of New Zealand Standard Compaction for sub-bases and subgrades.

Compaction should begin as soon as possible after initial mixing of the soil and cement (Plate 14). No section should be left unworked for longer than 30 minutes during compaction operations, and compaction should be completed within two hours of the initial mixing of the cement (ACI 1990). Delayed compaction will result in lower density (Figure 9.1).

The desired degree of compaction may not be possible to obtain, especially if the underlying layers are weak. In these cases the optimum water content and the design (e.g. cement content, stabilised layer thickness, thickness of granular cover) of the pavement should be reviewed.

### **9.2.8 Retarders**

Delay between the mixing and the compaction of a cement-soil mixture will result in a final dry density that is below the maximum value able to be achieved. This lower density occurs because some of the compactive effort, after the hydration of cement begins, is required to break the cement-soil bonds (Sugiarto and Mavin 1986). At higher temperatures the cement reacts more quickly and the effects of delays will be more pronounced (Litwinowicz 1986).

Table 9.1 Weight (kg) of type GP cement after it is spread on a 1m<sup>2</sup> canvas square (after Tait 1981).

| Compacted Layer Thickness (mm) | Dry Density of Soil After Compaction (kg/m <sup>3</sup> ) | Cement Application Rate *               |       |       |       |       |       |       |
|--------------------------------|---|---|-------|-------|-------|-------|-------|-------|
|                                |   | 1%                                      | 2%    | 3%    | 4%    | 5%    | 6%    | 7%    |
|                                |   | Weight / unit area (kg/m <sup>2</sup> ) |       |       |       |       |       |       |
| 100                            | 1600  | 1.60                                    | 3.20  | 4.80  | 6.40  | 8.00  | 9.60  | 11.20 |
|                                | 1800  | 1.80                                    | 3.60  | 5.40  | 7.20  | 9.00  | 10.80 | 12.60 |
|                                | 2000  | 2.00                                    | 4.00  | 6.00  | 8.00  | 10.00 | 12.00 | 14.00 |
|                                | 2200  | 2.20                                    | 4.40  | 6.60  | 8.80  | 11.00 | 13.20 | 15.40 |
| 125                            | 1600  | 2.00                                    | 4.00  | 6.00  | 8.00  | 10.00 | 12.00 | 14.00 |
|                                | 1800  | 2.25                                    | 4.50  | 6.75  | 9.00  | 11.25 | 13.50 | 15.75 |
|                                | 2000  | 2.50                                    | 5.00  | 7.50  | 10.00 | 12.50 | 15.00 | 17.50 |
|                                | 2200  | 2.75                                    | 5.50  | 8.25  | 11.00 | 13.75 | 16.50 | 19.25 |
| 150                            | 1600  | 2.40                                    | 4.80  | 7.20  | 9.60  | 12.00 | 14.40 | 16.80 |
|                                | 1800  | 2.70                                    | 5.40  | 8.10  | 10.80 | 13.50 | 16.20 | 18.90 |
|                                | 2000  | 3.00                                    | 6.00  | 9.00  | 12.00 | 15.00 | 18.00 | 21.00 |
|                                | 2200  | 3.30                                    | 6.60  | 9.90  | 13.20 | 16.50 | 19.80 | 23.10 |
| 175                            | 1600  | 2.80                                    | 5.60  | 8.40  | 11.20 | 14.00 | 16.80 | 19.60 |
|                                | 1800  | 3.15                                    | 6.30  | 9.45  | 12.60 | 16.75 | 18.90 | 22.05 |
|                                | 2000  | 3.50                                    | 7.00  | 10.50 | 14.00 | 17.50 | 21.00 | 24.50 |
|                                | 2200  | 3.85                                    | 7.70  | 11.55 | 15.40 | 19.25 | 23.10 | 26.95 |
| 200                            | 1600  | 3.20                                    | 6.40  | 9.60  | 12.80 | 16.00 | 19.20 | 22.40 |
|                                | 1800  | 3.60                                    | 7.20  | 10.80 | 14.40 | 18.00 | 21.60 | 25.20 |
|                                | 2000  | 4.00                                    | 8.00  | 12.00 | 16.00 | 20.00 | 24.00 | 28.00 |
|                                | 2200  | 4.40                                    | 8.80  | 13.20 | 17.60 | 22.20 | 26.40 | 30.80 |
| 225                            | 1600  | 3.60                                    | 7.20  | 10.80 | 14.40 | 18.00 | 21.60 | 25.20 |
|                                | 1800  | 4.05                                    | 8.10  | 12.15 | 16.20 | 20.25 | 24.30 | 28.35 |
|                                | 2000  | 4.50                                    | 9.00  | 13.50 | 18.00 | 22.50 | 27.00 | 31.50 |
|                                | 2200  | 4.95                                    | 9.90  | 14.85 | 19.80 | 24.75 | 29.70 | 34.65 |
| 250                            | 1600  | 4.00                                    | 8.00  | 12.00 | 16.00 | 20.00 | 24.00 | 28.00 |
|                                | 1800  | 4.50                                    | 9.00  | 13.50 | 18.00 | 22.50 | 27.00 | 31.50 |
|                                | 2000  | 5.00                                    | 10.00 | 15.00 | 20.00 | 25.00 | 30.00 | 35.00 |
|                                | 2200  | 5.50                                    | 11.00 | 16.50 | 22.00 | 27.50 | 33.00 | 38.50 |

\* Mass of cement as a percentage (1-7%) of the dry mass of the soil after compaction.

**Notes:**

- The actual size of the canvas square used should be measured. If it is not exactly 1m<sup>2</sup> then proportional adjustment will be required.
- Formula used to calculate weight of cement for this table:

$$\text{Weight of cement / m}^2 = \text{layer depth} \times \text{soil dry density} \times \frac{\text{cement application rate}}{100}$$



## 9. Construction of Cement-Stabilisation

Symons and Collins (1991) found that using OPC with a range of soils, with no retarder:

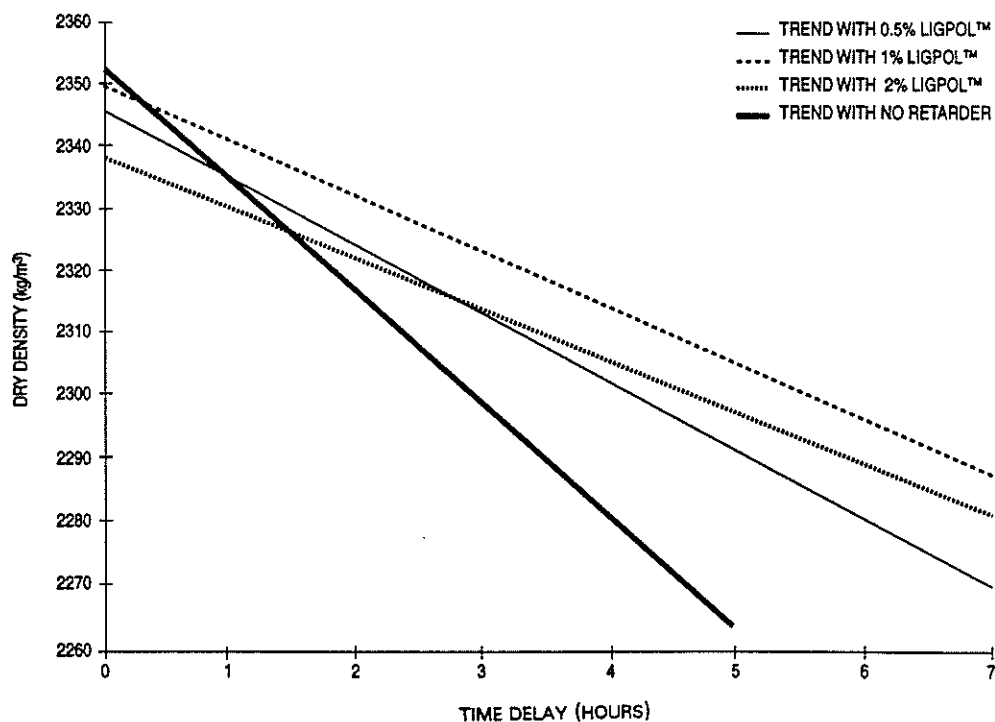
- All soils showed lower dry densities as the time between mixing and compaction increased.
- Most soils lost approximately 5% of the dry density achieved at zero delay after a 3 to 5 hour delay.

They then added each of three types of commercially available retarders separately to the range of soils. They found that:

- Even with retarders all soils showed lower densities as the time between mixing and compaction increased (Figure 9.1).
- The retarders all tended to slow the rate at which dry density decreases with delay.
- The type of soil appears to have far more effect on the effectiveness of a retarder than either the type of retarder or its dose rate.

This suggests that, even if retarders are used, the time delay between mixing and compaction should still be kept to a minimum.

Figure 9.1 Examples of effect of delay between mixing and compaction on dry density, using a retarder (Ligpol™) for cement-stabilisation using OPC (from Symons and Collins 1991).



### **9.2.9 Joints**

Joints form between successive passes of the rotary mixer during construction. Generally the mixer works along the road and in doing this creates longitudinal joints, except at the end of mixer runs where transverse joints are created.

Unnecessary longitudinal joints should be avoided. Longitudinal joints along the wheel tracks, particularly "cold" joints which form between operations on successive days, should be avoided.

All longitudinal joints should be lapped into the previous work by at least 100mm. When forming this overlap, cement should be added on both stabilisation runs.

At the end of each day's work a straight transverse construction joint should be formed. When starting the new day's work overlap the rotary mixer a metre or so into the previous day's work. Again, for this overlap, cement should be added to the stabilisation runs on both days (Wilmot 1991; pers.comm. 1994; ACI 1990).

## **9.3 Plant-Mix Method**

### **9.3.1 Mixing**

The cement, aggregate(s) and water are all mixed together in a central plant, then trucked to the job site if the plant-mix method of cement-stabilisation construction is used.

Plant-mix may be used for projects where the material to be stabilised is imported to the project, such as stabilised basecourses for new roads.

Granular materials are usually used because of their ease in handling and mixing. Silty and clayey soils are generally not plant-mixed because they are difficult to pulverise (Mueller 1991).

### **9.3.2 Spreading**

Plant-mixed material is best spread with a paver, or spreader box. Spreading and compaction should be completed as soon as practical after mixing.

## **9.4 Effect of Rain**

Light rain is generally not detrimental during the construction of cement-stabilisation. Heavy rain can be detrimental if it raises the water content of the uncompacted cement-soil mixture above optimum, as the full density cannot then be achieved. If rain is likely the job should be sealed off by rolling before the rain occurs.

Heavy rain on recently completed cement-stabilised material can leach unreacted cement from the upper 5mm to 25mm of the stabilised layer. The options for restoring this leached layer then are to:

- Re-compact if the cement has had time for its initial reaction with the soil and another layer is to be laid over the top.
- Grade it off.
- Re-mix and re-compact. In such cases greater compactive effort may be needed than for the freshly mixed material (Tait 1981).
- Re-pulverise, add a little additional cement, re-mix and re-compact.

## **9.5 Curing**

Curing of cement-stabilised materials is required for maximum development of strength and stiffness. Generally a curing period of some 3 to 7 days is required, depending on the ground temperature.

Curing may be provided (ACI 1990, Ingles and Metcalf 1972, Dunlop 1977, Simmons 1991) by:

- Keeping the surface moist by sprinkling with water,
- Covering with wet sacks, wet sand, or plastic sheet,
- First coat chipsealing with cutback bitumen, applied within one day after final rolling,
- Adding a curing coat of bitumen emulsion: this may consist of several applications of dilute emulsion made over several days, and except for unsealed roads this will require a surfacing, e.g. chipseal, before much trafficking is allowed,
- Placing the next pavement layer on top of the cement-stabilised layer, with minimum delay.

## **9.6 Surfacing**

Until the cement-stabilised base layer has cured and hardened up, traffic may distort the surface. Accordingly final surface preparation for bituminous surfacing may be left for a period, e.g. 3 to 7 days depending on the ground temperature. To limit carbonation, bituminous surfacing should be completed within about 14 days of mixing.

For stabilised bases, if any shaping is to be carried out with the grader at this stage, stabilised material is to be cut to waste. Stabilised material is not to be cut to fill low spots because any minor filling is likely to break away from the base either before or after sealing (Wilmot 1991).

For sealing directly on to a stabilised base, a clean stone mosaic surface should be exposed by power brooming, or by spreading a sparse layer of sealing chip then trafficking, or even by a light cut with the grader. First coat sealing follows, using either cutback bitumen or emulsified bitumen. If a bitumen cure coat has been used and it is still intact, a second coat seal can be applied, with any localised surface depressions filled with bituminous material.

Development of the bond between the chipseal and the stabilised layer depends on adhesion of the bitumen to sound material. Cement-stabilised fine-grained materials must be cured to prevent early drying and weakening of the surface layer. Even with good curing, it can be difficult to bond bitumen to some cement-stabilised fine-grained materials, such as pumice.

For these materials the options include:

- Armour coating (a coarse aggregate spread at about half the sealing chip application rate, about a third of the way through the compaction process, and rolled into the surface) (Jack 1991).
- Bituminous priming coat. The low flash point of a heavily cutback bitumen for priming creates safety concerns during construction using primers applied as hot cutback. Applying the kerosine-bitumen mixture as an emulsion avoids these safety concerns.

## **9.7 Trafficking**

Traffic should be diverted from cement-stabilisation construction operations, i.e. from scarifying to compaction after mixing.

If the stabilised layer is designed as a modified material, then controlled speed traffic may be permitted on the stabilised layer immediately following compaction.

If the stabilised layer is designed as a cemented material, then its ability to carry traffic soon after stabilisation will depend on whether this traffic will crack the stabilised material. For example, stabilisation of a well-graded crushed aggregate during mid-summer may produce a cemented material that is quite suitable for immediate trafficking because the aggregate itself should have sufficient strength to resist shallow shear. Also the subgrade should be dry and hence strong enough to provide sufficient support for the stabilised material to cement without deflection and resultant cracking.

## *9. Construction of Cement-Stabilisation*

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If the stabilised material is designed to produce a cemented material and is constructed over weak subgrades, traffic should be kept off for a number of days (e.g. seven days, dependent on ground temperature).

Alternatively, the stabilised layer may be trafficked immediately, or rolled approximately 24 hours after final compaction, to encourage the development of many fine cracks in the wheel paths. If these cracks are fine enough, they can be covered with the second coat chipseal. Because such cracks reduce the tensile strength of the material, this loss of tensile strength should be recognised and compensated for in the design.

For rehabilitation the practical choice may be to traffic the stabilised layer immediately, or within 24 hours. Practitioners who do this claim that it results in no detrimental pavement performance (McDonald 1991b).

Immediate construction of any overlying layers will offer some protection from traffic stresses, except those from construction equipment. Therefore the weight and amplitude of rollers used to compact the overlying layer(s) should be controlled.

Further research is necessary to properly understand the processes occurring in cemented materials under early trafficking.





Plate 11. Spreading bulk cement.



Plate 12. Hand-fed mechanical spreader for bagged cement.







Plate 13. In-place-mixing of cement for rehabilitation of an urban street.



Plate 14. Compaction following rotary mixing for cement-stabilisation.





## **10. STABILISATION IN PAVEMENT REPAIRS**

### **10.1 General**

Repairs of small areas of distressed pavement can be carried out using lime- or cement-stabilisation. These repairs can be made either by mix-in-place stabilisation of the existing material, or by dig-out and replacement with plant-mixed stabilised material. The principles of stabilisation that apply to large areas also apply to small areas.

The mix-in-place stabilisation method is gaining acceptance in some areas of New Zealand as an economic alternative to dig-outs. Typically it is used for repairs when the symptoms are shallow shear and/or alligator cracking of the seal coat. The benefits of stabilisation in these cases are likely to be its ability to improve aggregates so they are stiffer and more tolerant of moisture. This suggests that, given good workmanship, repairs using stabilisation may outperform dig-out repairs using unbound materials.

For in-place stabilisation of a few small areas, it may not be economic to use a purpose-built rotary mixer. As the material to be stabilised is likely to be a basecourse, or basecourse and subgrade mixture, satisfactory pulverisation and mixing may be able to be achieved using heavy duty rotary hoes (see Section 7.3 of this report). Repeated passes will be needed with such hoes, with more than usual attention to ensure complete and thorough mixing (Tait 1981). An alternative is to use a milling machine for pulverisation and mixing, recognising that it may reduce the size of some of the larger stones in the basecourse. Grader mixing is not well suited to small areas.

### **10.2 Depth of Stabilisation**

Guidance on the appropriate depths to which stabilisation should extend to repair small areas of distressed pavement is not readily available. Instead the following requirements are offered as guidance:

- Repair of shallow shear. Minimum depth of stabilisation 150mm. Increase this depth for repairs made in winter and/or over soft subgrades, to compensate for compaction difficulties.
- Repair of isolated small areas of cracking with loss of surface shape. Minimum depth should be at least as deep as that defined as "primary basecourse M/4 required" depth from the unbound pavement thickness design charts in the NRB (1989) Manual. Increase this depth for repairs in winter and/or over soft subgrades.

While stabilised materials are more tolerant of inadequate drainage than unbound materials, they still perform better if they are well drained. Good drainage will also increase the strength of the underlying subgrade. Thus in all cases the adequacy of the pavement edge drainage should be reviewed.

Also stabilisation should not be to a depth that will incorporate subgrade material with the granular layer, unless the mix design of the stabilised material has allowed for this.

### **10.3 Extent of Stabilisation**

Good practice allows for mix-in-place stabilisation for repairs to extend some distance (e.g. 0.5m to 1m) beyond the visible extent of distress. This allows for:

- Fine cracking not readily visible at the time of inspection.
- Water infiltration through cracks in the distressed area weakening the surrounding areas.
- Gradual change of the properties of the distressed weak pavement layers over distance. Except in trench reinstatements and old dig-outs, properties usually do not improve abruptly at the edge of the distressed area.
- Edge effects for the repair. For example less effective compaction near edges.

If the distressed areas total more than 5% of the area of that section of the road, then consideration should be given to stabilising the entire area (NRB 1989).

### **10.4 Strength**

When repairing small areas by stabilisation, the option should be kept open for future rehabilitation by mix-in-place stabilisation of that entire area of pavement. To keep the option open, repairs that produce strongly cemented materials should be avoided as they may be difficult or impossible to re-scarify or re-pulverise in the future. The exceptions to this may be when a strongly cemented material is required to bridge a particularly weak area of subgrade or a buried service, or the thickness of the existing pavement will be inadequate if lightly stabilised, and stabilising a mixed basecourse-subgrade mix is undesirable.

Specific design will be required for all cases when in-situ stabilisation will produce a strongly cemented material in the base layer.

### **10.5 Trench Reinstatement**

Stabilisation, particularly with cement-stabilised materials mixed at a central plant, has been used for many years for the backfilling of trenches. The thickness of material involved in trench backfilling means that high standards of compaction are required even with cement-stabilised materials (Tait 1981). However the cemented material tends to be tolerant of small lapses in achieving good compaction.

Cement-stabilised trench backfill that has been inadequately compacted may perform well for a number of years, then partially collapse. This creates a depression in the road surface.





## **11. STABILISATION IN UNSEALED ROADS**

### **11.1 General**

Favourable experiences using lime-stabilisation in unsealed roads in New Zealand were reported at a seminar in 1978 (Pickens 1979, Inglis 1979, Shapcott 1979). More recently very favourable results from lime-stabilisation have been reported in Cook County (Elliot 1991, McDonald 1991d). Yet in spite of these favourable experiences, lime- or cement-stabilisation of unsealed roads is still not widely accepted or practised, although there are some indications that its use is increasing (e.g. Cooper 1994). While much still has to be learnt, stabilisation of unsealed roads has a lot of potential because over 40% of the length of New Zealand's public road network is unsealed and likely to remain so.

Unsealed and sealed roads have many similarities. The principal differences of unsealed roads are the moisture conditions in the pavement, and the use of a granular and clay aggregate as the surfacing. The frequency of grading an unsealed road is usually governed by attrition of its surfacing. This grading masks deformations from subgrade rutting and shallow shear, so the pavement is able to be thinner than that of a similar sealed road, and the quality of basecourse can be lower (Ferry 1986, Ferry and Major 1991).

A gravelled unsealed road will be comprised of two or more of the following layers (Figure 1.3, Chapter 1 of this report):

- Subgrade
- Sub-base
- Basecourse
- Wearing course

Each of these layers can benefit from stabilisation (Plates 15-18). In many cases one material may function as two or more of these layers, e.g. a basecourse could function as both a base and a wearing course (Ferry and Major 1991).

Stabilisation of the subgrade, sub-base or basecourse can increase the strength of the pavement, particularly with weak subgrades or in poorly drained situations. This can reduce maintenance requirements (ARRB 1993) and reduce the fuel consumption of heavy vehicles (Cenek et al. 1996).

Conversely, stabilisation could allow thinner pavements or allow lower quality local aggregates to be used, so conserving reserves of quality aggregates and reducing trucking, and hence wear on other roads.

In spite of these advantages, the lower traffic intensities and ability to easily add aggregate or reshape by grader allows the use of thinner pavements than those used for sealed roads (ARRB 1993), even without stabilisation. Therefore the economics of stabilisation of the subgrade, sub-base or base layers requires careful consideration, and may only favour those roads with weak subgrades, expensive aggregate, or carrying numerous heavy vehicles.

Stabilisation of the wearing course for an unsealed road, or stabilisation of a basecourse that is also to function as a wearing course, can produce a material that is resistant to attrition. Stabilisation can significantly reduce the frequency of grader maintenance (Shapcott 1979, Inglis 1979, Elliot 1991, McDonald 1991d) and the frequency of re-metalling, so reducing pavement maintenance costs. In addition, such stabilised wearing courses may be used on unsealed roads (ARRB 1993, Elliot 1991) to:

- reduce potholing and corrugating,
- reduce vehicle operating costs (by providing a smoother surface),
- reduce trip times (by providing a smoother surface),
- reduce dust,
- increase skid resistance (e.g. by reducing the “greasiness” of the clays).

These advantages suggest that stabilised wearing courses could be economically attractive on many unsealed roads.

The reduced frequency of maintenance means a longer time between gradings so that shallow shear or wheel track rutting remains longer before it is smoothed by grading. Adequate pavement thickness and strength are required therefore to gain the full benefits of a stabilised wearing course, but economics may well dictate a compromise.

Stabilisation of a running course (Figure 1.3 of this report) however is not useful. A running course is an additional layer that may be thinly spread over the surface of the unsealed road to reduce wear and drying out of the wearing course. Because running course is a loose layer of, preferably, crushed quarry stone (Ferry and Major 1991), there is little point in stabilising it.

## **11.2 Edge Drainage and Crossfall**

Unsealed roads are frequently constructed of local aggregates which are of lower quality than those used in sealed roads, and which in unstabilised form may lose a significant proportion of their strength when wet (i.e. are moisture sensitive). These same materials when stabilised with lime or cement tend to be much less moisture sensitive. However, even stabilised materials have reduced strength when wet, so



good edge drainage is still desirable. Thus edge drains and culverts need to be well maintained to keep water away from the pavement.

A common defect observed on unsealed roads is potholes. To avoid development of potholes requires adequate crossfalls and superelevations. An acceptable minimum crossfall is 4%, but crossfalls of 6% are preferable (Ferry and Major 1991). While crossfalls much steeper than 6% would improve surface drainage, they are not recommended as they risk tipping loaded stock trucks onto their sides. Gentle longitudinal grades also help avoid pothole development by increasing surface slopes.

Lime- and cement-stabilised materials can resist development of potholes, particularly if these materials are cemented. Even so they can still pothole, and the same crossfalls, superelevations and longitudinal grades are recommended for stabilised unsealed roads as for non-stabilised (Elliot 1991, McDonald 1991c).

### **11.3 Frost**

In roads exposed to frost, improved edge drainage and road surface levels of 1.5m to 2m above the highest winter ground water levels are required. Lime- and cement-stabilisation producing cementation can reduce frost susceptibility of pavement materials, as it does for sealed roads (Section 2.5.7 of this report). Thus stabilisation can improve traffickability, and reduce pavement maintenance, during and after frost periods.

### **11.4 Stabilised Bases**

Lime- or cement-stabilisation of the basecourse can reduce the required thickness of pavement. It may also be used to reduce the sensitivity of the basecourse materials to excess water caused by drainage deficiencies.

Elliot (1991) and McDonald (1991d) report that, as part of the stabilisation operations, a light surface application (25m<sup>3</sup> per kilometre of road) of <20mm-sized particles of crushed limestone or crushed river shingle should be spread and compacted into the stabilised surface. This provides a friction and wearing surface. No running course is added, and thus the base layer with its armouring also acts as a wearing course (Figure 1.3 in this report).

Based on extensive experience with the lime-stabilisation of unsealed roads in Cook County (Gisborne East Cape area), Elliot (1991) and McDonald (1991d) report very good results, with substantial reductions in pavement maintenance costs using this approach.

A problem with this approach (discussed by Elliot 1991) occurs when larger stones (e.g. 63mm diameter plus) are encountered in the materials being reworked during stabilisation. If they cannot be broken down by the rotary mixer, with time and traffic they tend to become proud of the road surface. This surface creates a rough ride. Neither adding a running course nor carrying out extra compaction has been particularly effective in creating a smooth surface in this situation. A thin overlay or, in extreme cases, re-pulverisation and re-compaction have been used to rectify this roughness.

Inglis (1983) reported on the stabilisation of unsealed roads to a depth of 220mm using 2¼% lime (% quicklime by weight). An aggregate of <20mm-sized particles was applied at a rate of 100m<sup>3</sup> per kilometre of road and rolled into the stabilised surface to armour the surface and provide a running course. After 2 years of trafficking these roads, Inglis reported that the performance had been very good. The benefits included improved riding quality, and reduced maintenance with grading about once only per year. Grading was done when the pavement was wet, and was a light grading, just enough to move the windrows back into the wheel tracks. In areas of inadequate crossfall, potholes had developed which were repaired by hand patching.

### **11.5 Stabilised Wearing Courses**

An alternative approach to armouring the surface of the basecourse, and using that as the surface to be trafficked by vehicles, is to spread and compact a thickness of wearing course. The wearing course should consist of a crushed and well graded aggregate that includes a clay binder. The clay binder is for waterproofing the pavement surface and to bind the material so that it resists attrition from passing wheels. The maximum size of stone in the wearing course should be 20mm to reduce roughness (Ferry and Major 1991).

Stabilisation of wearing courses can increase their resistance to attrition, and so reduce road maintenance costs. The amount of lime or cement to use for wearing course stabilisation requires careful consideration as there are a number of conflicting requirements:

- Stabilisation with low percentages of lime or cement can increase the permeability of the wearing course. This can be compensated by waterproofing the basecourse layer below with clay, whether it is unstabilised or lime-stabilised.
- Stabilisation with low percentages of lime or cement will reduce the cohesiveness of the clays and may encourage early ravelling of the wearing course. After treatment the stabilised material should exhibit some cohesion (ARRB 1993) or binding.

- Stabilisation with higher percentages of lime or cement will produce a cemented material that afterwards cannot be reworked with graders. Such material is difficult or expensive to maintain.

Usually only stabilisation to produce a modified material is suitable for the wearing course of an unsealed road where maintenance is to be by grading. For each type of material there is an optimum lime content. Normally this will be between 2% and 4%, but for well graded clayey gravels between 1% and 3% lime may be sufficient.

A trial section, built using the proposed materials and proposed lime or cement application rate, then maintained for at least 12 months, is often the most practical way of checking the results of stabilisation on an unsealed road (ARRB 1993).

### **11.6 Construction of Stabilised Wearing Courses**

Stabilised materials for wearing courses will usually be plant-mixed (see Sections 8.3, 9.3 of this report). Spreading can be carried out by truck then grader with care to avoid segregation (Plate 17), or by paver. Either way the existing road should be wet when the wearing course is spread. Roller compaction of the wearing course, rather than just allowing the traffic to compact it, will significantly extend the life of the wearing course (Hudson and Taylor 1996). Lime-stabilised wearing courses should be slightly wet of optimum at the time of compaction.

### **11.7 Lime or Cement for Stabilisation**

Lime- or cement-stabilisation can be used for subgrades, sub-bases or basecourses. However for subgrades the low traffic volumes and periodic grader reshaping means stabilisation may only be economic for very weak subgrades. These subgrades are likely to favour lime (Figures 1.1 and 1.2 of this report).

Wearing courses that are to be maintained by grader should be stabilised to produce a material that is modified. This, combined with the usual wish to have a near-waterproof material (i.e. a well graded clayey gravel), favours lime-stabilisation. Cement is usually not an appropriate stabilising agent for wearing courses on unsealed roads (ARRB 1993), but it has been used and may be quite suitable if it is not to be maintained by grader (Section 11.10 in this Chapter).

### **11.8 Percentage of Lime or Cement**

Fine cracking of the stabilised layer is of less concern in an unsealed road than in a sealed road, except for wearing courses. For subgrades, sub-bases and basecourses, this allows the use of slightly higher percentages (e.g. by 1–2%) of lime (or cement) than for those materials used for similar layers in a sealed road.

The amount of cement still needs to be limited to control the risk of wide cracks occurring, as wide cracks may admit water which will weaken the subgrade. This also applies to lime, but to a lesser degree. (See Sections 2.3, 2.4, 2.5, 6.1 of this report.)

### **11.9 Stabilised Wearing Courses in Dry Climates**

Hastings District, on the east coast of the North Island, enjoys a sunny and dry climate. The rainfall is up to 1400mm per year in the west and 900mm per year in the east of the District, with the entire District having little rain during the summer months.

In-situ lime-stabilisation was carried out on unsealed roads throughout the District to provide a 150mm-deep stabilised base layer. Some of these stabilised sections had a running coat of sealing chip spread directly on the stabilised layer, while others had a 50mm-thick layer of unstabilised gravel wearing course compacted above the stabilised layer (Healey and Fraser 1996).

Healey and Fraser (1996) report that the stabilised sections performed well for two years, then during the summer months corrugations developed. These corrugations were expensive to repair and gave a rough riding surface. Investigations showed these corrugations were directly related to the horizontal forces applied to the pavement surface and were directly related to:

- Tight curves,
- Sections where vehicles accelerate,
- Sections where vehicles change gear.

Research is needed to gain a fuller understanding so these problems can be minimised in future.

### **11.10 Maintenance of Stabilised Unsealed Roads**

If the stabilised basecourse and/or wearing course is a lime- or cement-modified material, then maintenance can be by grading (ARRB 1993).

If the stabilised basecourse and/or wearing course is cemented, then grader maintenance is not appropriate (ARRB 1993). The routine maintenance choices would then include:

- Patch potholes, probably with a lime-stabilised material, when the road is wet,
- Add wearing course material when the road is wet,
- Add running course material,

- Re-spread the existing running course by drag broom.

Further research is needed to evaluate and refine these techniques.

Maintenance benefits from the lime-stabilisation of unsealed roads have been claimed (ARRB 1993, McDonald 1991d, Elliot 1991, Inglis 1979, Stenberg 1990) to include:

- Extended intervals between regrading,
- Reduced surface defects,
- Reduced need for adding aggregates for running and/or wearing courses,
- Reduced dust,
- Reduced maintenance costs by as much as 35%.

### **11.11 Future Sealing**

Basecourse and wearing course materials to be used for unsealed roads require significant quantities of plastic clays to attain cohesion and low permeability. Sealed pavements however require limitations on the amount of clay that can be allowed in the materials.

If the possibility exists that an unsealed road may, one day, be sealed, the clays used in the unsealed road should be lime- or cement-reactive. This reactivity then provides the option of mix-in-place stabilisation before sealing. Alternatively, stabilisation now offers the possibility of an economic unsealed pavement that may well be suitable for sealing in the future.

### **11.12 Stabilisation to Suppress Dust**

Preliminary trials in North Auckland and Bay of Plenty used bitumen emulsion, Weslig 120, Borresperse CA, and waste engine oil as stabilising agents to suppress dust generation from an unsealed road. These trials suggested (Bartley Consultants 1995):

- Each product had some success in the suppression of dust.
- The effective period of the treatments was generally not for more than a few weeks, with the exception of the waste engine oil treatment.
- Mixing the products through the aggregate resulted in dust suppression for a longer period than applying them as surface sprays.
- The waste engine oil treatment was the most effective dust suppressant. However it is thought to have detrimental effects on the environment. These effects were not evaluated in these trials.





11. *Stabilisation in Unsealed Roads*

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Plate 15. Ripping before stabilising with lime for an unsealed road.



Plate 16. Running course over a cement-stabilised wearing course for an unsealed road.







Plate 17. Spreading quarry-mixed lime-stabilised aggregate ...



Plate 18. ... to produce a lime-stabilised wearing course for an unsealed road.





## **12. QUALITY CONTROL OF CONSTRUCTION**

### **12.1 General**

All phases of the stabilisation work are to be closely supervised by experienced supervisors. It is impracticable to control the work by testing alone, but testing should be regarded as a means of confirming the adequacy or otherwise of the construction technique adapted and of adequate construction control (Dunlop 1977).

In addition to stabilisation, many other aspects may be critical to the successful completion of a job (Tan 1989), such as:

- Kerb and channel restoration,
- Drainage,
- Avoidance or relocation of services,
- Advice to affected residents,
- Temporary traffic control,
- Control of dust,
- Control of noise,
- Site clean up.

This Chapter 12 deals with the control of the construction of stabilisation itself. For construction, the following aspects of the stabilisation process must be controlled to ensure uniformity and acceptability of the stabilised layer (Dunlop 1977):

- Uniformity of the material to be stabilised,
- Levels, profiles and general pavement shape,
- Quality and distribution of the lime or cement,
- Depth of the processed layer,
- Slaking,
- Degree of pulverisation,
- Mixing,
- Water content,
- Density,
- Quality of the stabilised material,
- Surface finish,
- Curing.

Supervisory staff should check levels, profiles and general pavement shape, ensure that the slaking and mixing are adequate, check the depths of the processed layer and of the compacted pavement, judge the suitability of the water content of the material at the different stages of construction, and ensure that the compaction, surface finish and curing operations are correctly performed.

## **12.2 Uniformity of the Material to be Stabilised**

Where material is to be processed in-place, the necessity for stabilisation and the type and quality of the stabilising agent will have been determined by sampling and laboratory testing. However after the material has been prepared for stabilisation and the necessary forming is complete, it is important to check the quality and uniformity of the soil materials actually encountered in the road against those tested in the laboratory. If quantities of unexpected material are encountered during construction, the options include:

- Remove and replace the material
- Alter the designed thickness of stabilised layer
- Alter the designed application rate of lime or cement
- Mix the material by grader before stabilisation
- Overlay with other material to part depth or full depth of the proposed stabilised thickness before stabilisation
- Carry out further laboratory testing and re-design

## **12.3 Crossfalls, Shape, Levels**

Crossfalls, shape and levels should be correct and checked as being correct before beginning pulverisation and mixing. If aggregate is added to correct the shape it should be compacted before crossfalls, shape and levels are checked.

Crossfalls, shape and levels should be checked again after final compaction or trimming. The surface roughness may also need to be checked (e.g. by roughometer) after final trimming or sweeping, or after sealing.

## **12.4 Quality of Lime or Cement**

For both quicklime and hydrated lime the lime manufacturer can be asked to state the grade of the lime, in terms of TNZ M/15:1986. In addition storage conditions and storage duration should be known as a guide to hydration or carbonation that may have occurred since manufacture. If desired, testing for Available Lime Index can be used to determine the quality of the lime at the time of stabilisation.



When cement is purchased directly from the manufacturer, under NZS 3122:1995 the purchaser can request the manufacturer to supply recent test results indicating cement quality at the manufacturer's works. In addition the purchaser can specifically request certain chemical testing of the cement by the manufacturer. However, it would be unusual to request this information as the quality of cement is monitored by both the manufacturer and the ready mixed concrete industry, so giving some confidence in the quality of fresh cement to be used for stabilisation.

For cement storage, conditions and storage duration should be known as a guide to possible hydration since manufacture. If desired, the tests in NZS 3122:1995 can be used to determine the quality of the cement at the time of stabilisation, but this would be unusual particularly for bulk supply.

## **12.5 Spread Rate and Distribution of Lime or Cement**

### **12.5.1 Bagged Spreading**

Where lime or cement is supplied in bags, preparatory to spreading checks should be made to ensure that the bags are spaced correctly to the predetermined pattern. After the bags have been cut open, check that the lime or cement is spread to as uniform a thickness as practicable (Dunlop 1977). (See Sections 8.2.5, 9.2.5, and Appendix 4 of this report.)

### **12.5.2 Spreading Bulk Dry Lime**

The rate of spreading should be checked at the start of spreading, and periodically during each day's work. This can be done by placing a tray or canvas cover on the surface of the scarified/pulverised ground and weighing the quantity of dry lime deposited on it after spreading (Dunlop 1977). It is important to measure the actual area of the tray or canvas, although usually they are made with an area of 1 square metre.

As an overall check, a truck load of lime or cement of known weight is spread, and the area over which the lime or cement has been spread is measured (ACI 1990). This actual weight per actual area is then compared with the design application rate. (See Appendix 4 of this report.)

### **12.5.3 Slurry Spraying**

Lime slurries should be applied through calibrated spray bars on the water cart or from the mixing chamber of the rotary mixer. The nozzles of the spray bar should be carefully maintained and kept clear to ensure even distribution of the slurry.

The quantity of slurry used on each segment of the treated road should be measured by dipping the tank at the beginning and end of each set of spray runs. The composition of the slurry can be determined from the specific gravity of the slurry, measured either by hydrometer or by weighing a known volume. This quantity and composition, divided by the area of road sprayed, gives an overall application rate (NAASRA 1986, TRB 1987).

## **12.6 Slaking**

Usually slaking is checked visually: light pressure (e.g. foot pressure) should cause the slaked lime to crumble.

If burning (see Section 8.2.7 of this report) of the quicklime is suspected, the available Lime Index test can be used.

## **12.7 Layer Depth**

The depth of the loose layer can be determined by probing with a rule, or by hand shovelling a hole and measuring the layer depth.

The depth of the compacted stabilised layer should also be checked. The depth of the stabilised mixture can be checked by digging small holes to determine the bottom of the stabilisation.

Following compaction of a material freshly mixed with lime or cement, a check on stabilised layer depth (and mixing uniformity) can be made by squirting phenolphthalein solution down the side of a freshly cut face of lime-soil or cement-soil. The phenolphthalein on the lime-soil or cement-soil mix will turn reddish-pink while that on the untreated material will not change colour.

The compacted depth of the fresh lime-soil or cement-soil should be sufficient to allow for trimming to waste (Dunlop 1977) during final surface preparation.

For some situations with hardened lime-soil or cement-soil the depth may be able to be checked by coring (ACI 1990).

## **12.8 Degree of Pulverisation**

The degree of pulverisation will be apparent to an experienced supervisor. In addition sieving tests may be made (Tait 1981, TRB 1987).

## **12.9 Mixing**

The efficiency of mixing can be judged by the uniformity in appearance and colour of the processed layer before compaction. The uniformity in the vertical direction should be checked by digging trenches or a series of holes at regular intervals to the full depth of the treatment and then inspecting the uniformity of colour of the exposed material.

The thoroughness and efficiency of the mixing operations on the site can be quantitatively checked by sampling, then halving the sample. From one half prepare

and cure UCS or CBR specimens without further mixing. The other half is completely remixed, and from that UCS or CBR specimens are prepared and cured. The mixing efficiency is calculated (TRB 1987) as:

$$\text{Percentage of mixing efficiency} = \frac{\text{Field mixed strength}}{\text{Laboratory mixed strength}} \times 100$$

For mix-in-place operations, efficiencies normally range from 60% to 80% (TRB 1987).

### **12.10 Lime Content**

Laboratory procedures can be used to determine the lime content of uncured lime-soil mixtures. For example ASTM D3155-83 (TRB 1987).

### **12.11 Water Content**

The water content of materials should be checked regularly. Close examination of the appearance and feel of the materials will give a guide to the amount of moisture in the materials. In addition, regular testing should be undertaken by sampling and oven drying, complemented by in-place testing by nuclear meter. In calibrating the nuclear meter, note that the hydrogen ions in the hydrated lime can influence the water content readings (TRB 1987).

### **12.12 Density**

Observation of the behaviour of the lime-soil or cement-soil under heavy machinery provides some indication, but is not a reliable indicator, of compacted density. Testing for density is required.

For uniform soils, test methods such as sand replacement, balloon, or nuclear meter are frequently used to determine density. While such methods can give absolute dry densities (in kg/m<sup>3</sup>), the usual specification requirement is a percentage dry density relative to a maximum dry density determined under standard conditions. This requires the density under standard conditions to be determined for the material to be stabilised. Determining this by laboratory testing is practical for uniform soils.

An alternative for uniform soils is to determine the appropriate water content at the time of compaction, and maximum achievable density, by close measurement during construction of a series of test strips on site.

For variable cohesive soils either many maximum dry densities obtained under standard conditions are required, or control of density can be achieved by a limitation on the percentage air voids, together with achievement of a satisfactory strength. On-site trials may be needed to determine appropriate maximum air voids and minimum strength.

Air voids can be deduced from soil density (specific gravity), dry density, and water content. Strength can be obtained by vane or penetrometer tests (Tait 1981).

For cemented materials it is desirable to verify compaction as soon as practicable so that if remedial measures are required, they can be undertaken before the material becomes cemented and unworkable (NAASRA 1986).

The density-water relation for lime-soil and cement-soil mixtures changes with time as the material cures. For example, if a lime-soil layer is reworked at a later date following initial construction, the maximum dry density and optimum water content for the mixture will be different from the original mixture. In such cases it may be impossible to achieve the original density, but it is not necessary to achieve the original density because the reduction is not due to poor compaction but rather to the material being different (TRB 1987).

### **12.13 Curing**

During compaction and finishing the surface of the lime-soil, or more particularly the cement-soil, may become dry. This dryness is shown by greying of the surface. If drying occurs a very light fog-spray of water should be applied. Adequate water content for the lime or cement reactions in the compacted cement-soil or lime-soil is shown by a smooth, moist, tightly knit, compacted surface, that is free of cracks and surface dusting (i.e. no dust collects under light hand-brooming).

The pavement surface should be kept moist for the specified period (Sections 8.5 and 9.5 in this report), or until the curing coat has been applied. This surface moistness can be checked visually. The curing coat uniformity can also be checked visually, and the overall application rate for each spray run can be checked in a similar manner to that described in Section 12.5.3 of this chapter.



**PART C**  
**STABILISATION USING**  
**ADDITIVES OTHER THAN**  
**LIME OR CEMENT**

- 13. BITUMEN-STABILISATION**
- 14. OTHER STABILISERS**
- 15. THE FUTURE**



## **13. BITUMEN-STABILISATION**

### **13.1 Introduction**

Bitumen-stabilisation of a soil for a road pavement is used to:

- Introduce some cohesion into non-plastic materials; and/or
- Act as a waterproofing agent so a cohesive material will retain its strength in wet conditions.

Some of the reasons (Terrel et al. 1979) for using bitumen-stabilisation are to:

- Expedite construction,
- Upgrade marginal materials,
- Reduce moisture sensitivity of fine-grained materials,
- Reduce pavement thickness,
- Increase pavement life,
- Reduce or eliminate dust during construction,
- Provide a temporary surface suitable for carrying traffic.

Bitumen-stabilisation involves the thorough mixing of bitumen with soil. The bitumen is attracted to the soil particles on which it acts as a glue and/or waterproofing, but it does not chemically alter the soil particles (Sherwood 1994).

For bitumen-stabilisation to work, the soils to be stabilised must be able to be readily pulverised (Bowering 1988, NAASRA 1986).

The amount of binder used in bitumen-stabilisation is often considerably less than would be used in a bitumen-bound pavement. Bitumen-stabilised material may therefore require protection from abrasion by traffic by overlaying it with some form of surfacing (NAASRA 1986).

Over the last two decades a number of types of bitumen-stabilisation have been, and in some cases continue to be, used in New Zealand under a number of different names. Examples, all of which are mixed in a central plant, include:

- Open-graded emulsion mixes (OGEMs), which have been used mostly in the Dunedin and Christchurch areas.
- Bitumen macadams ("Bit Macs"), mixed in a hot-mix plant using a well-graded aggregate and hot bitumen, which were previously used widely in New Zealand but more recently have been used mainly in the North Island.

- Bitumen-stabilised pumice, mixed in a pug mill using pumice and foamed bitumen, and used mostly in the central North Island.

## 13.2 Types of Bituminous Materials

### 13.2.1 Establishing Suitability of Type

Bitumen-stabilisation may be carried out using any one of the following materials:

- Hot bitumen,
- Cutback bitumen,
- Bitumen emulsion, either anionic or cationic.

The stabilisation process must ensure thorough mixing of the bitumen throughout the stabilised soil. The material to be stabilised should be in a uniform moist condition before mixing, with the exact amount of moisture being determined by field trial.

Either 180/200 or 80/100 penetration grade bitumens may be used in bitumen-stabilisation. For soils with higher fines content (15–25% passing the 75µm sieve) 180/200 grade should be used, possibly with some flux added (NAASRA 1986). If flux is used with 180/200 grade bitumen, it should be used only in very low proportions or the stabilised material may shove, particularly on more heavily trafficked roads.

Experience in France with 13 tonne legal axle loads and high volumes of heavy traffic has resulted in use of 60/70 or even 40/50 penetration grade bitumens. This experience of shoving, even with very hard bitumens, emphasises the need to restrict or avoid flux with the softer 180/200 bitumen used in New Zealand.

The chemical bond between soil particles and the bituminous materials can be improved by the use of adhesion agents (TNZ 1991). A mineral filler, e.g. finely ground limestone (TNZ M/10:1975), may be added to improve the particle size distribution, and hence the mechanical stiffness of the bitumen-stabilised soil (NAASRA 1986).

### 13.2.2 Hot Bitumen

Stabilisation with hot bitumen involves changing the physical state of the bitumen to produce a greatly increased surface area at the time of mixing. Two methods are currently in use:

- **Foamed Bitumen Process.** The foamed bitumen process, patented by the Mobil Oil Company, injects cold water under controlled conditions and with additives into hot (e.g. 165°C) bitumen. This causes the water to boil which foams the bitumen up to 10 or 15 times its original volume. This foam is sprayed through specially designed nozzles into the cold moist soil in the mixing chamber of a pugmill or rotary mixer (Akeroyd 1989, Bowering 1988).

- **High Impact Process.** In this process patented by the BP Oil Company, hot bitumen is injected into the moist soil at high pressures to produce a foam mix in the soil (Bowering 1988).

The Foamed Bitumen and High Impact processes can be mixed either in-place or in a central mixing plant.

Straight-run bitumens made by distillation, or bitumens made by the usual blowing process, can be used. If the bitumen has been made by the propane precipitation process it will contain an anti-foaming agent which must be neutralised before the material is suitable for the production of foamed bitumen (NAASRA 1986).

The Foamed Bitumen and High Impact processes allow penetration grade bitumen to be intimately mixed into cold moist materials without adding cutback or emulsifying the bitumen.

A third method of mixing the hot bitumen with the aggregate to be stabilised is to use a conventional asphaltic concrete manufacturing plant (hot mix plant).

### 13.2.3 Cutback Bitumen

Bitumen may be mixed with a cutter oil (e.g. kerosine) to produce binders that are fluid at ambient temperatures. These heavily cutback binders may be sprayed cold or with only slight heating, and then mixed with the pre-moistened soil. Water in the soil helps the distribution and mixing of the binder through the soil mass.

The grade of bitumen will vary, as will the amount of cutter (kerosine) or flux (AGO), depending on the conditions and temperature at the time of mixing and the type of soil to be stabilised.

### 13.2.4 Bitumen Emulsion

Bitumen emulsions when used for stabilisation may be mixed with damp soil to disperse the bitumen throughout the soil. Only medium break, slow break, or very slow break (i.e. mixing grade) emulsions are suitable for stabilisation.

Stabilisation may be carried out using either anionic bitumen emulsion, or cationic bitumen emulsion. The selection of anionic or cationic relates in part to the chemical characteristics (e.g. silica content) of the soil (Terrel et al. 1979).

Other variables to consider when selecting a bitumen emulsion include its rate of break, penetration grade of the bitumen, and amount of diluent it contains. The rate of break should be related to the particle size distribution of the soil, with soils having a higher fines content (e.g. 15–25% passing the 75µm sieve) requiring slower breaking emulsions. The bitumen grade and content of diluent relate to the road temperature and traffic environments, and to the structure and stability to be obtained from the mix. Usually, but not always, the emulsion is intended to break before the stabilised material is laid on the road (JE Patrick, pers.comm. 1994).

The water content of the soil influences the efficiency of distribution of the emulsion throughout the soil. Dry soil causes the emulsion to break prematurely resulting in the formation of blobs of bitumen that do not thoroughly mix through the aggregate and coat the particles. If the water content of the soil is increased, the time available for mixing can be prolonged and this leads to a better distribution and coating of bitumen. If the water content is too high, aeration of the soil to remove excess water may be necessary before rolling commences to ensure full compaction.

Stabilisation with emulsion should be carried out only when conditions are dry and warm. After mixing, and before compaction, curing and aeration of the mixture must be allowed. Enough time must be allowed at this time to let the water released from the breaking emulsion to escape (NAASRA 1986, Bowering 1988). The exception would be open-graded emulsion mixes, where the open nature of the compacted mix allows the water from the emulsion to escape through the voids.

### **13.3 Over-Wetting**

Over-wetting occurs where too high a liquid content results from the added bitumen, plus the water in a bitumen emulsion, plus the cutter or flux in the bitumen, plus the soil water (Bowering 1988). If the liquid content is too high, the voids in the soil are filled with liquid which prevents full compaction of the stabilised soil. Over-wetting is more likely when using a bitumen emulsion or a heavily cutback bitumen. Using hot bitumen by the foamed bitumen or the high impact processes reduces the likelihood of over-wetting occurring.

### **13.4 Suitable Soils**

#### **13.4.1 Establishing Suitability**

In general soils which can be readily pulverised by the usual construction equipment are suitable for bitumen-stabilisation. The range of soils which may be suitable for bitumen-stabilisation includes pit-run gravels, sands, silty sands, sandy clays, and crushed sub-base and basecourse aggregates. The general characteristics of soils suitable for stabilisation with bitumen are set out in Table 13.1 (after NAASRA 1986) in this chapter. The requirement to be readily pulverised generally limits suitable soils to those with more than 50% of their particles greater than 75 $\mu$ m, liquid limit less than 40, and plasticity index less than 18.

The type of soil to be stabilised needs to be related to the type of bituminous material used for the stabilisation. Information linking the soil type with the different methods of bitumen-stabilisation is set out in Figure 13.1 (after NAASRA 1986).

Table 13.1 Types and characteristics of soils which can be stabilised with bitumen (after NAASRA 1986).

| Property                          | Material Type          |               |              |              |
|-----------------------------------|------------------------|---------------|--------------|--------------|
|                                   | Sand                   | Silty sand    | Sand-gravel  | Basecourse   |
| <b>Particle Size Distribution</b> | <b>% Passing Sieve</b> |               |              |              |
| <b>Sieve Size (mm)</b>            |                        |               |              |              |
| 32.5                              | —                      | —             | 100          | 100          |
| 26.5                              | 100                    | —             | —            | —            |
| 19.0                              | —                      | —             | 60-100       | 70-100       |
| 4.75                              | 50-100                 | 50-100        | 35-100       | 40-70        |
| 2.36                              | —                      | —             | —            | 30-60        |
| 0.425                             | —                      | 30-75         | 13-50        | 15-45        |
| 0.150                             | —                      | —             | 8-35         | —            |
| 0.075                             | 5-12                   | 10-35         | 0-12         | 8-20         |
| <b>Liquid Limit</b>               | —                      | Less than 30* | —            | Less than 30 |
| <b>Plasticity Index</b>           | Less than 10           | Less than 12* | Less than 10 | Less than 12 |

\* Liquid limit of silty sand may be increased to 40 and its plasticity index to 18 respectively, providing intimate mixing can be achieved.

As a general rule, if the liquid limit of a soil multiplied by its plasticity index is less than 150, the soil may be suitable for using hot bitumen. If the liquid limit multiplied by the plasticity index is less than 75, then it would be more appropriate to use cutback bitumen or emulsions to stabilise the soil (Bowering 1988):

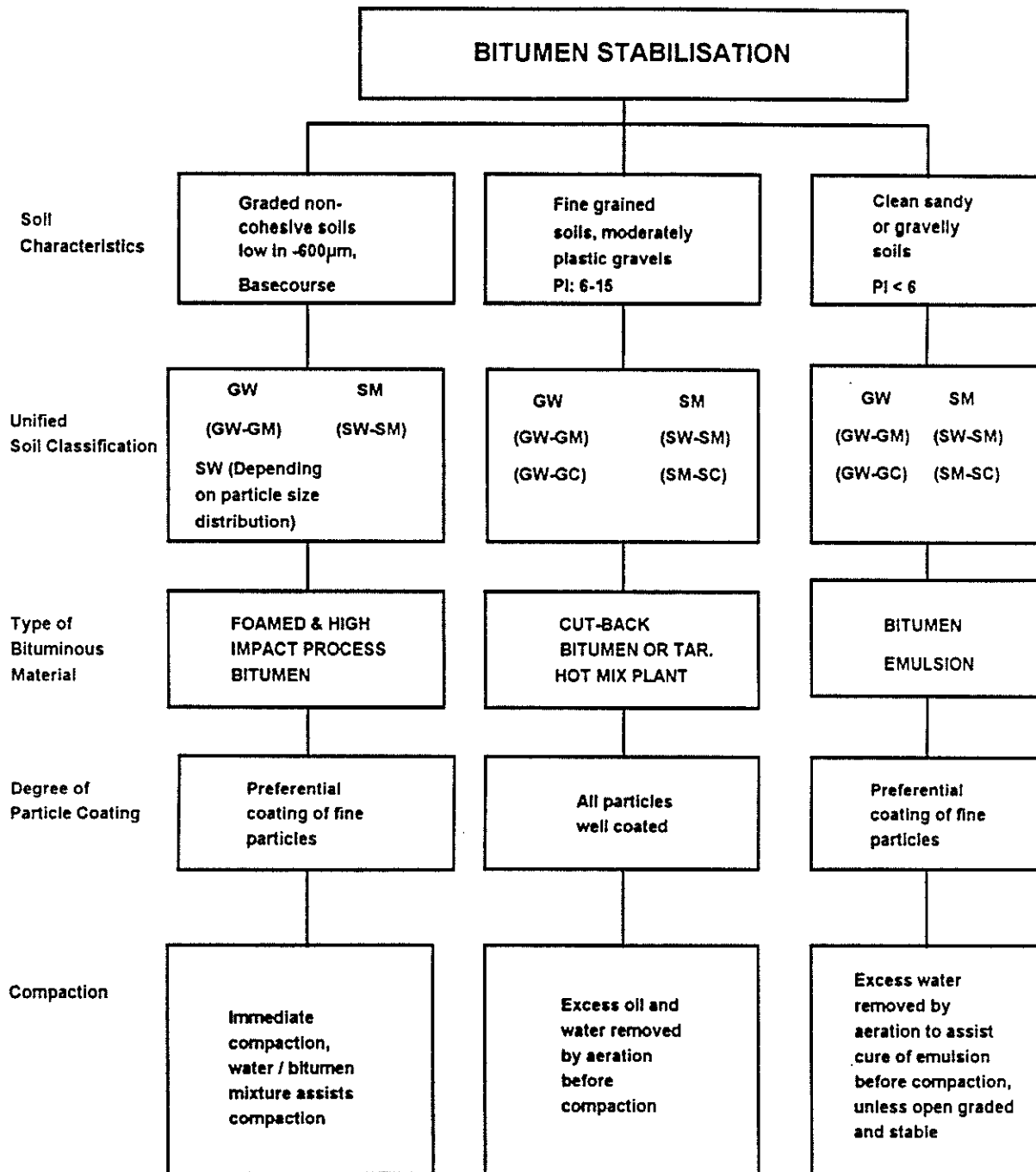
i.e. If  $LL \times PI < 150$ , use Hot Bitumen,  
but If  $LL \times PI < 75$ , use Cutback bitumen or Bitumen Emulsion.

Some soils may benefit from pre-treatment, usually with lime, to modify their properties so they become suitable for a second stabilisation treatment using bitumen.

The properties of the untreated soil can dominate the outcome of stabilisation, e.g. a coarse well-graded material with its inherently good mechanical stiffness can produce a stronger bitumen-stabilised material than that obtainable from a finer poorly graded material.

In very clean sands, the bitumen adheres only weakly to the silica surface of the sand particles. This may lead to stripping of the bitumen from the sand if water can penetrate the stabilised material (Ingles and Metcalf 1972). Use of an adhesion agent in the bitumen can prevent this stripping as the positively charged molecules of the adhesion agent are attracted to the negatively charged particles of wet sand (TNZ 1991).

Figure 13.1 Selection of appropriate bitumen material for stabilising a particular soil (after NAASRA 1986).



|             |  |   |
|-------------|--|---|
| <b>KEY:</b> | GW = Well-graded gravels, gravel sand mixtures, little or no fines | GM = Silty gravels, gravel-sand-silt mixtures |
|             | GC = Clayey gravels, gravel-sand-clay mixtures                     | SM = Silty sands, sand-silt mixtures          |
|             | SW = Well-graded sands, gravelly sands, little or no fines         | SC = Clayey sands, sand-clay mixtures         |
|             | PI = Plasticity Index  |   |
|             |  |   |



#### **13.4.2 Soil Types used with Hot Bitumen**

The soil types most suited to hot bitumen-stabilisation are graded gravels, crushed rock, and sands of low plasticity because they are clean and coarse, and are easily coated in the very short time available for mixing when using either the Foamed Bitumen or the High Impact hot bitumen processes. During this short time the particles have to be coated and mixing has to be completed while the bitumen is in its finely dispersed condition.

Foamed Bitumen and High Impact processes both require mixing to be carried out at or near optimum total liquid content, i.e. both the bitumen and the soil water are considered as parts of the total liquid content (see Section 13.3 of this chapter).

Placing may be carried out immediately, but care may be needed during the initial compaction by using light or non-vibrating rollers to prevent cracking or displacement of the mix.

#### **13.4.3 Soil Types used with Cutback Bitumen**

The soil types most suitable for stabilisation with cutback bitumen are slightly to moderately plastic sands and gravels, provided that the clays in the soil allow the soil to be readily pulverised in the moist condition and intimately mixed with the stabiliser.

Mixing should be carried out at or near optimum total liquid content (i.e. cutback plus bitumen plus soil water), taking care to avoid too high a total liquid content which will prevent compaction (see Section 13.3 of this chapter). On the other hand, too low a total liquid content also has to be avoided because once the stabilised material has been rolled, further water cannot be added to aid compaction, unless the material is completely reworked (NAASRA 1986).

#### **13.4.4 Soil Types used with Bitumen Emulsion**

The soil types most suitable for bitumen emulsion stabilisation are clean well-graded aggregates between 37.5mm and 75µm, or clean crushed open-graded aggregates. While successful bitumen emulsion stabilisation can be carried out using aggregate with a portion passing the 75µm sieve, and with aggregates containing clays with some plasticity, the amount of both fines and clay should be limited. The Asphalt Institute (1976) recommends that an aggregate for bitumen emulsion stabilisation should satisfy either:

- Less than 25% passing 75µm sieve, and a sand equivalent >30, or
- $(\text{Plasticity Index}) \times (\% \text{ of aggregate finer than } 75\mu\text{m}) < 72$ .

Water in the emulsion at the time of compaction is in addition to the soil water and to the bitumen. In a well-graded material the total liquid (water plus bitumen) content should not be too high, or compaction will be difficult or impossible (see Section 13.3 of this chapter).

### **13.5 Mechanisms in Bitumen-Stabilisation**

The purpose of bitumen stabilisation for roads is to improve the stability of the soil, i.e. increase cohesion of aggregates, and to increase the resistance of the road to weakening by water, i.e. reduce the sensitivity of the materials to water. Bitumen-stabilisation is more successful with granular materials than with cohesive soils. Bitumen-stabilised soils are, therefore, used primarily as basecourses, and to a lesser extent as sub-bases.

The bitumen provides cohesion by coating and bonding the soil particles. This cohesion can increase strength such as tensile strength, compressive strength and flexural strength, and it can increase the elastic modulus. It can also reduce ravelling, corrugating, and dust (Terrel et al. 1979) when bitumen-stabilisation is used in temporary surface courses.

The bitumen reduces sensitivity of the soil to water by providing a membrane between the soil particles. This membrane prevents or hinders the penetration of water into the bitumen-stabilised layer and down through it to underlying layers or the subgrade.

An advantage of the Foamed Bitumen and High Impact processes is that they selectively coat the finer soil particles because the bitumen is attracted to the relatively high surface areas of the finer particles. This selective coating preserves the stability that depends on the interlock of coarse particles, while creating a thicker waterproofing film on the finer moisture-sensitive particles. With these processes therefore, the resulting non-uniform dispersion of the bitumen helps maintain the internal friction and interlock, and thus maintains the stability of the stabilised material (Bowering 1988).

Hot bitumen mixed in an asphaltic concrete plant, and cutback bitumen do not selectively coat the finer particles. Instead both tend to coat all soil particles equally irrespective of size. For this reason, at low (less than 1.5%) bitumen contents the resulting strengths may equal those using Foamed Bitumen or High Impact processes, but at higher bitumen contents strengths are lower. Hot or cutback bitumen also produce thinner films of bitumen for any given bitumen content, compared to those produced by the Foamed Bitumen or High Impact processes.

Emulsions tend to equally coat all particles in a clean aggregate irrespective of size. However, emulsions selectively coat the finer soil particles in aggregates with significant proportions of fine sand-sized, or finer, particles. In such aggregates, the bitumen emulsion is attracted to the relatively high surface area of the finer particles where it breaks. The use of very slow breaking (mixing grade) emulsions allows more time for thorough mixing of the soil material and emulsion before breaking. Thus these slow breaking emulsions allow more uniform coating of all soil particles with bitumen.

### 13.6 Mix Design

The design of bitumen-stabilised mixtures does not currently follow any well established procedures. Many different criteria have been suggested, and in general those criteria that have been used are some form of strength and stability tests (both dry and soaked), together with water absorption, including capillary rise of water in compacted cylinders which have been oven-dried at 60°C (NAASRA 1986, Bowering 1988, Akeroyd and Hicks 1988).

An outline of a procedure (NAASRA 1986) for determining mix design is:

- Test and classify the soil to be stabilised.
- Select the type of stabiliser based on the soil, climate, trafficking, types of bitumen available, and specialist mixing (e.g. high impact) plant available.
- Select a suitable range of bitumen contents for the test specimens.
- Carry out laboratory compaction to determine the density–water relationships for the range of bitumen contents selected.
- Mix, aerate, compact and cure test specimens.
- Test the moulded specimens by suitable methods. Strength tests (e.g. modified Hveem Resistance, UCS, CBR, Marshall flow and stability, or modified Hubbard Field) (Asphalt Institute 1969, Bowering 1988) and a water absorption or capillary test are usually included. Voids in the mix should also be measured.
- Select the most suitable bitumen content, optimum water content, and required density for the bitumen-aggregate stabilised mixture.

As a guide to the selection of suitable binder contents for the preparation of test specimens, granular materials normally require the addition of 2–5% by mass of residual bitumen (NAASRA 1986). For foamed bitumen the bitumen contents are generally proportional to the % of soil mass passing the 75µm sieve, and as a guide (Akeroyd and Hicks 1988) may range from:

3.5% bitumen for soils with 5% <75µm,  
to 5% bitumen for soils with 20% <75µm.

The material, after it has been stabilised and compacted, must contain sufficient voids for the bitumen to fill, plus leave some air voids, e.g. 2%. Too few air voids, or too high a residual bitumen content in a soil with low voids after compaction, can result in bleeding of the bitumen through the overlying seal coat, and/or instability of the mix, under subsequent trafficking.

### **13.7 Effects of Salts or Organic Matter**

If salts or organic matter are in high concentration, and coat the soil particles, they may reduce the effectiveness of the bitumen-stabilisation. Such coatings prevent or reduce adhesion of the bitumen to the soil particles. The organic matter itself can also limit the strength of the stabilised material.

Salts on the surface (efflorescence) of the stabilised basecourse can prevent the adhesion of a seal coat to the stabilised surface (Ingles and Metcalf 1972).

### **13.8 Construction with Bitumen-Stabilisation**

Mixing at central plants is usually carried out in twin shaft continuous pug mills. Laying can be by grader or paver.

For in-place-mixing single rotor mixing machines, as used for lime- or cement-stabilisation, are suitable, and the binder is sprayed into the mixing chamber (Bowering 1988). This technique can be used for the rehabilitation of existing pavements in-place, usually with Foamed Bitumen (Akeroyd 1989, Akeroyd and Hicks 1988) or High Impact processes, to facilitate early trafficking.

Soils stabilised with hot bitumen may be placed and compacted immediately after mixing. Care during the initial compaction, by using light rollers, may be needed to prevent instability of the mix (NAASRA 1986). The material may be trafficked immediately following completion of compaction.

Open-graded mixes stabilised with hot bitumen in a hot mix plant, or with emulsion, can be compacted immediately as long as the aggregate/uncured binder mix has sufficient stability and cohesion to retain its surface shape without ravelling under traffic. The water in the emulsion then gradually evaporates through the air voids allowing the binder to cure and the mix to gain strength.

Unless they are open-graded, soils stabilised with cutback bitumens and bitumen emulsions require aeration and curing before compaction to allow unwanted volatiles and water to evaporate. Also drying out and curing after compaction may be needed for the mixture to gain strength. It is desirable to allow the material to stand for at least two weeks before sealing. Controlled trafficking may be advantageous before sealing.

Stabilisation using either cutbacks or emulsions that have not fully broken and cured, which may both require aeration before compaction, should not be undertaken if rain is likely to occur before the completion of the process (NAASRA 1986).

### **13.9 Grave Emulsion Mixes**

In Ireland and France, both cold mixed "Grave Macadams" or grave emulsion mixes are used. These are normally made using a 65% cationic emulsion binder that is medium to slow breaking. A well-graded aggregate is used and very tight control of the aggregate grading is necessary. The binder preferentially coats the fine soil particles leaving stone to stone contact between the larger particles to provide stability of the mix.

Laying can be by grader, but paver laying is preferable. Compaction is difficult as the presence of water in the emulsion can cause a build-up of pore pressure.

In Ireland and France, the grave emulsion mix is laid, compacted, then trafficked immediately and before the binder has cured as the well-graded aggregate itself has sufficient stability to withstand traffic loads. The grave mix may be overlaid with asphaltic concrete, hot rolled asphalt, or with a chipseal 2 to 3 days after laying depending on weather (Tosh 1992).

The use of grave emulsion mixes by roading contractors in New Zealand requires careful consideration of the adequate curing of the binder in a compacted and dense material. An inadequate aggregate grading combined with inadequate provision for curing could result in an unstable mix.



## **14. OTHER STABILISERS**

### **14.1 Types of Stabilisers**

Heat and a number of additives, other than lime, cement or bitumen, may be used for stabilisation. Except for mechanical stabilisation (Section 14.2), fly ash (Section 14.5.1), blastfurnace slag (Section 14.5.2), and the proprietary products (Section 14.7), none of the methods of stabilisation described in Sections 14.3 to 14.6 of this chapter have been used over the last decade in New Zealand, other than on small scale or experimental works.

These other stabilisation methods may be classified in three groups (Ingles and Metcalf 1972):

- The "mechanical" stabilisers which are used to improve the mechanical properties of the soil by the addition of other materials (e.g. fibres or graded aggregates), but do not affect the properties of the original soil itself (Sections 14.2 and 14.6 of this chapter).
- The "physical" stabilisers which are used to alter the soil properties by methods, such as heat, electricity, etc. (Section 14.3 of this chapter).
- The "chemical" stabilisers which are solid or liquid additives used to alter the chemical properties of the soil by means of a solid or liquid additive (Sections 14.4, 14.5, 14.7 of this chapter).

The examples discussed in this chapter are not exhaustive. While some of the stabilisers discussed have been used successfully, generally their effectiveness is variable, and each needs to be evaluated for any particular soil type and use (Metcalf 1977). This evaluation may well include quite intensive soil testing and monitored trials.

As a rule these other products should not be regarded as a substitute for good practice (Ingles and Metcalf 1972). Good drainage, proper aggregate selection and use of appropriate compaction plant should all be implemented before use of any of these other stabilisers is considered. Their use should complement, not substitute, good practice.

### **14.2 Mechanical Stabilisation**

Improving the soil material that is available for road construction by blending it with one or more other soils is called mechanical stabilisation or granular stabilisation. Sands, graded aggregates and/or clays, etc. can be blended with the available material as a direct means of altering the particle size distribution to obtain an improved material.

The principal properties affecting the stability of compacted basecourse or sub-base materials are internal friction, cohesion and suction. Internal friction is chiefly dependent on the characteristics of the coarser soil particles, i.e. of gravel, sand and silt sizes, and in particular the particle size distribution (or grading). The cohesion, suction, shrinkage and swelling are mainly associated with the quantity and nature of the clay fraction as indicated by the plastic properties, sand equivalent and clay index (Bartley 1980).

Preliminary mix design is based on particle size distribution and plastic properties. Strength tests (e.g. CBR, triaxial, Texas Triaxial) should be carried out in the laboratory on samples of both the existing and the stabilised materials to quantify that the required improvement has been achieved. Unconventional materials will often require more detailed investigation and testing (NAASRA 1986). Lime- or cement-stabilisation (see Part B of this report) may follow such mechanical stabilisation.

In addition to adequate investigation, testing and design, good construction and control of construction are necessary.

### **14.3 Heat Stabilisation**

The improvement of a material by heat is known as heat- or thermal-stabilisation. It was used in New Zealand last century when wood fires were used to heat-stabilise mudstones when making roads in the papa (mudstone) country.

Heat stabilisation is an age-old process that uses temperatures ranging from 60°C (for sun-dried brick) up to temperatures of 900°C or higher (for kiln-fired bricks). Heating to temperatures of around 900°C would be too expensive for soil stabilisation. While some non-durable improvements may be obtained at lower temperatures, worthwhile and durable improvements in soil properties for road construction can be obtained by heating the soil to around 400° to 500°C. Improvements include (Ingles and Metcalf 1972):

- Reduced or zero plasticity index,
- Reduced moisture absorption,
- Reduced strength loss when wet or,
- At increased heating temperatures, increased strength and stiffness.

Heat stabilisation is suitable for clays, but preferably the clays should not be too wet at the time of treatment. Driving off the water from saturated soils is very expensive as the high latent heat of evaporation takes a lot of heat energy to convert water into steam, even if the soils are at optimum water content and if cheap fuel is available. Moreover the low thermal conductivity of clay (~8 to 16 mcal/°C.cm.sec) makes any substantial volume difficult to treat effectively at the one time (Ingles and Metcalf 1972).



#### 14.4 Chemical Stabilisers

Chemical stabilisers fall into two distinct subgroups, inorganic and organic (Ingles and Metcalf 1972). Inorganic stabilisers include acids, alkalis, sodium and potassium chlorides, and the calcium salts. Examples of organic stabilisers are lignin and its derivatives.

##### 14.4.1 Acids and Alkalis

Acids and alkalis owe their effects primarily to an actual attack on the soil components (usually the clay minerals), with subsequent precipitation of new and insoluble minerals which bind the soil together. The obvious problems with acid and alkali materials are corrosion, cost and health risks (Metcalf 1977). A number of acids and alkalis have been used for soil stabilisation including:

- **Phosphoric acid.** The mechanism of chemical change seems to be controlled by the pH of the soil to be stabilised, with an optimum pH range of 3.1 to 3.5. Some soils require 2.5% or more of phosphoric acid to neutralise them before stabilisation can begin (Sameshima and Black 1982).
- **Hydrofluoric acid.** Laboratory tests have shown this to be an exceptionally rapid and effective stabiliser for all except the aluminium-rich clays (kaolins, bauxites). It attacks the silica component of either sand or clay to form insoluble silicofluorides of high strength. It cannot be applied in practice because of its extremely corrosive action, dangerous handling problems and high cost (Ingles and Metcalf 1972).
- **Sodium hydroxide (caustic soda).** Caustic soda can be easily applied with the mixing water, and acts as a powerful compaction aid giving higher densities for the same compactive effort. It reacts very effectively with soils rich in aluminium, e.g. kaolin, giving substantial strength increase after an initial curing period. The action of the caustic soda is to degrade the clay mineral lattice by alkaline attack. If the soils are aluminium-rich, the products of degradation then precipitate insoluble aluminium oxide hydrates which have considerable durability in the soil (Ingles and Metcalf 1972).

It is deleterious in low strength montmorillonite clays as it further reduces their low strength (Ingles and Metcalf 1972).

Caustic soda is extremely corrosive and will cause burns and irritation to the skin. It also carbonates very rapidly in air with consequent loss of reactivity (Ingles and Metcalf 1972).

#### **14.4.2 Sodium Chloride (Common Salt)**

Common salt (sodium chloride or NaCl) has been used as a non-durable stabilising agent and dust suppressant. The mechanism is based on the reaction of common salt with the soil to cause flocculation and cementation, and as a compaction aid to promote higher densities. These changes seem to be dependent on the soil type.

Common salt is not recommended for use in New Zealand as a soil stabiliser, or to reduce frost heave. Excess moisture from the high rainfall that is characteristic of New Zealand's climate leaches the salt from the soil, although a chipseal surfacing can slow the rate of leaching (Sameshima and Black 1982).

While common salt could be re-applied frequently, it may migrate into the subgrade where it can increase the plasticity of any swelling clays, or into the surrounding soils where it can be toxic to trees. The tolerance of trees to common salt varies depending on the species (Sameshima and Black 1982, Ingles and Metcalf 1972).

Lees et al. (1982) report on 28 day strength increases of 100–300% in lime-soil mixtures with 1% common salt added, compared to the lime-only treated soils, in a laboratory study.

#### **14.4.3 Calcium Salts (Calcium Sulphate or Gypsum, or Calcium Chloride)**

Calcium chloride ( $\text{CaCl}_2$ ) has been used overseas to protect against frost in pavement layers since the 1920s. Slate (1942) showed that addition of 0.5–2% calcium chloride effectively stabilised soils prone to frost heave by depressing the freezing point of water in the pavement.

The mechanism of calcium chloride in soil stabilisation is not fully understood, but ion exchange in clays is generally agreed to be one of the major effects, except if clays are naturally calcium saturated. Another effect occurs after compaction. Some soils show a density increase (Sameshima and Black 1982, Massa 1990), while others show a density decrease and permeability increase (Ingles and Metcalf 1972). Strengths can also increase (Shepard et al. 1991).

With both calcium sulphate and calcium chloride, efflorescence on the surface of the basecourse may create problems (Metcalf 1977) with adherence of the bitumen in the first coat seal.

Calcium chloride has an adverse effect on compaction, and it tends to increase the permeability of the soil so it is more easily leached (Ingles and Metcalf 1972, Metcalf 1977).

Calcium chloride has been used on unsealed roads for non-durable dust control. Its hygroscopic and deliquescent nature means it attracts moisture which keeps the surface damp. Compared to pure water, wet calcium chloride has a stronger moisture film, higher surface tension and lower vapour pressure. These properties and the

associated ion exchange combine to keep dust particles in place on unsealed roads (Kirchner and Gall 1991).

#### **14.4.4 Potassium Chloride**

Sameshima and Black (1982) showed in laboratory studies that potassium chloride (KCl) undergoes a cation-exchange process with some types of montmorillonite clays. This cation exchange results in lower plasticity and less swelling, and increased resistance to wetting-drying cycles and to frost damage. These changes are achieved without the risk, which sodium chloride has, of increasing the plasticity of swelling clays. Sameshima and Black considered potassium chloride showed some promise either as a stabilising agent for basecourse aggregates which contain swelling clays or as an accelerant for lime-stabilisation.

A trial section of road was stabilised using potassium chloride in Gisborne in 1985. Spraying the potassium chloride in solution onto the basecourse (some compacted, some uncompacted) was found not to be effective as the solution did not penetrate. Uniform mixing was achieved when the potassium chloride was added in granular form and mixed with the basecourse. However, Benkelman Beam results obtained over the first four years of pavement life failed to give a clear indication of the effectiveness or otherwise of the potassium chloride stabilisation, compared with an untreated control section (NRB RRU 1989).

#### **14.4.5 Ferrous Ion-Rich Water**

Some of the brown volcanic ashes occurring in New Zealand soils often exhibit high strength and stiffness in natural conditions, which are related to the structure provided by the iron oxides. This structure can be destroyed by disturbance of the soil materials (as in construction), and then the high water content retained by the allophanic clay minerals is released, causing major handling and compaction problems in the field.

Stabilisation of these volcanic ashes using lime, cement, phosphoric acid, phosphate rock, superphosphate, aluminium sulphate or iron sulphate have given only modest strength increases compared to their strength after they have been disturbed (Millar 1987).

Millar (1987) proposed accelerating the re-formation of the iron oxides so that the original strengths of these volcanic ashes are provided. He investigated using an oxidising agent together with ferrous ion-rich water. While this approach had promise, practical problems limited its potential.

#### **14.4.6 Sodium Silicate (Water Glass)**

Experimental work has shown that sodium silicate stabilisation can improve the properties of kaolin and illite clays, but can be detrimental to the properties of montmorillonite clays (Sameshima and Black 1982).

#### **14.4.7 Lignin Derivatives**

Lignin is the natural cement in wood, and lignin derivatives are a major by-product of the paper-making industry. In paper-making lignin is extracted from the wood with sulphite chemicals resulting in calcium lignosulphonic acid. This acid is treated with lime to give a solution containing about 10% calcium lignosulphonate plus wood sugars. This solution can be concentrated by evaporation and is sometimes used for road stabilisation in the concentrated form (NAASRA 1986).

Lignosulphonates are water-soluble polymers, the active constituent of which is the lignin molecule. These polymers have an affinity to silica surfaces, and form strong bonds with soil particles, particularly clays. In road materials they act primarily as compaction agents (Ingles and Metcalf 1972) but they can cement soil particles, so increasing the dry strength of the stabilised soil. The associated wood sugars tend to be hygroscopic and thus they help to reduce dust on unsealed roads (NAASRA 1986).

For soil stabilisation of unsealed roads, lignosulphonate can be either mixed with the base or wearing course, or simply sprayed onto the road surface. An application of 0.5–1% of chemical solids by dry mass of soil is usual. Repeat applications are necessary to maintain the stabilising effect.

Lignosulphonate has been used to stabilise bases under bituminous wearing surfaces using an application of 2% to 3% chemical solids by dry mass of soil. Base thickness should ordinarily be not less than 150mm (NAASRA 1986).

Another form in which lignin has been trialled for soil stabilisation is known as "chrome lignin", a mixture of potassium dichromate and the lignin-containing material. The dichromate ions strengthen the bond between the surfaces of the clay particles and the lignin molecules, so increasing the strength of the stabilised soil. It is usually used in special situations, such as the treatment of volcanic soils (Ingles and Metcalf 1972). NAASRA (1986) reported that field trials using chrome-lignin for the stabilisation of roading materials were not successful because of the difficulty of mixing and because of the large shrinkage cracks that developed.

### **14.5 Pozzolanic Stabilisers**

A pozzolan is a material that will react with lime, in the presence of water, to produce a cementitious material.

#### **14.5.1 Fly Ash**

Fly ash is the finely divided residue that results from the burning of coal, and is transported from the combustion chamber by the exhaust gases. It is reclaimed in the flue by extraction from the gases. Reclaimable fly ash in New Zealand will almost always be from coal-fired electricity generating plants (Brendel and Curry 1987).

Fly ash contains varying amounts of lime (calcium oxide). The type of coal burnt and the mode of operating the generating plant determine the chemical composition and particle size distribution of the fly ash. As it is a by-product its quality is generally not controlled. Thus not all fly ashes are suitable for soil stabilisation uses.

Some fly ashes are self-cementing, but many require the addition of lime or cement to initiate their cementing reaction. The lime reacts with the siliceous and/or aluminous fine-grained fly ashes to form cementitious products that are similar to cement. Like cement these reactions will occur whether or not the soils react to fly ash (NAASRA 1986).

The optimum ratios of lime to fly ash, and the amounts to be added to the soil, differ according to the properties of all three materials. In addition to seeking a chemical reaction to stabilise the soil, fly ash can be used as a filler to correct a particle size deficiency in the soil and so provide increased strength. (See Section 14.2 of this chapter.)

The methods of evaluation of lime-fly ash-soil mixtures are similar to those required for lime- or cement-stabilisation. However as the lime and fly ash contents are independent variables, a more comprehensive testing programme (NAASRA 1986) is required before using fly ash as a stabiliser.

A distinct optimum water content rarely exists for many fine-grained soils mixed with lime and fly ash. Even if an optimum water content does exist, generally it does not yield the maximum strength for the mixture. Because of this unusual behaviour the mixture's water-density-strength relationship must be determined before use. This is done by testing a range of water contents at compaction of the lime-fly ash-soil mixture to establish which water content will provide maximum strength after curing (Linn and Symons 1988).

The rate of reaction in lime-fly ash-soil stabilisation is usually relatively slow (see Table 14.1 of this chapter) but strength gain continues over a long period (more than one year) providing moisture is present. It is also temperature-sensitive. Cement may be used in lieu of lime to increase strengths (Boles 1987) over the first days and weeks after stabilisation.

Lime-fly ash mixtures are bulky and light, which can create dust problems during construction (NAASRA 1986).

Lime-fly ash generally works best with aggregates, sands or silty soils because the cementitious material tends to surround the particles (Boles 1987). They can also be used in clays where strength gains can range from little to useful (Linn and Symons 1988).

Table 14.1 Compressive strengths of lime- and cement-treated fly ash at different ages (Sherwood 1995).

| Fly Ash Source* | Compressive Strength (MPa) † |         |         |                       |         |
|-----------------|------------------------------|---------|---------|-----------------------|---------|
|                 | with 10% lime added          |         |         | with 10% cement added |         |
|                 | 7 days                       | 28 days | 56 days | 7 days                | 28 days |
| High Marnham    | 1.58                         | 9.95    |         | 8.35                  | 13.8    |
| Skelton Grange  | 0.55                         | 5.90    |         | 5.32                  | 13.1    |
| Cliff Quay      | 1.03                         | 3.37    |         | 4.35                  | 8.37    |
| Ferrybridge     | 3.44                         | 10.80   | 13.00   | 5.62                  | 7.33    |
| Hams Hall       | 0.83                         | 3.79    | 8.02    | 3.38                  | 7.13    |

**Notes:**

\* Names are of UK coal-fired power stations.

† Strengths are for lime with fly ash, or cement with fly ash, with no added aggregate.

**14.5.2 Blastfurnace Slag**

Slag is the general name given to the non-metallic residue formed during the production of iron and steel. Slag consists primarily of the silica, alumina and titanium from the original iron ore, together with various combinations of calcium and magnesium oxides from the fluxing materials added in the process.

Composition, properties and quality of slags vary considerably and generally their quality is not controlled. Their chemical composition and properties relate to the nature of the process which produced them (Allan 1989). The properties of specific slags should be determined before they are used in road stabilisation. Testing should include a check on their volumetric stability, as some slags swell when they hydrate (Sherwood 1995).

While iron-making slags are non-reactive and so are unsuitable for stabilisation, steel-making slags have cementing properties when mixed with lime or cement, or in some cases gypsum and soda, in the presence of water (Allan 1989). Compared to OPC these materials set slowly so are suited to road stabilisation as they allow more time for mixing, compaction and shaping (Ray 1986).

Four distinct types of slag can be produced depending on the way the molten slag is cooled (Ray 1986):

- Air cooled,
- Expanded,

- Granulated,
- Pelletised.

These slags may then be crushed to improve their cementing action.

#### 14.5.3 Cement Kiln Dust

Cement kiln dust is a waste product of cement manufacture. Levick and Dunlop (1979) reported on trials and laboratory work in New Zealand which indicated that cement kiln dust on its own, or more particularly in combination with lime or OPC, offers considerable benefits as a roading stabiliser.

### 14.6 Random Fibre Reinforcement

The engineering properties of a soil can be improved by mixing in discrete fibres. The resultant dispersed and randomly orientated fibres provide both tensile strength and shear strength to the fibre-soil mix as it undergoes deformation.

Mixing the fibres into the soil matrix can be difficult, particularly for a cohesive soil. Compaction of a fibre-soil mix is more difficult than for compacting only the soil, particularly with increasing fibre contents.

Studies to date have been experimental, using both natural and synthetic fibres (Table 14.2).

Table 14.2 Summary of fibre types reported to have been used for randomly orientated fibre reinforcing (from Bourne-Webb 1995).

| Natural Fibres        | Synthetic Fibres                  |
|-----------------------|-----------------------------------|
| Asbestos              | Glass Fibres                      |
| Cellulose             | Polypropylene Fibres              |
| Reed/Cane             | Polypropylene/Polyethylene Strips |
| Bamboo                | Nylon Fibres                      |
| Palmyra (Palm Fibres) |                                   |
| Rubber (Buna-N)       |                                   |

The performance of a fibre-soil mix is primarily influenced by:

- Density,
- Soil grading and particle shape,
- Fibre aspect ratio, modulus and surface characteristics (e.g. surface area, texture),

- Fibre content,
- Confining pressure.

Experimental studies have indicated that difficulties in achieving density with a fibre-soil mix can result in a lower strength than that of the soil alone. If a high degree of density can be obtained, then under "static" loadings a fibre-soil matrix offers:

- Increased peak strength,
- Increased strain to reach peak strength,
- Increased stiffness,
- Reduced drop-off in post-peak strength at large strains.

Reports of random fibres used in actual road pavements are limited to those of two research trials, one in France and the other in Sweden. Both report favourable initial results, although the Swedish trial showed that normal methods to shape the pavement surface could not be used during construction.

Preliminary economics suggest that material costs for the fibres may make randomly orientated fibre reinforcing uneconomic for most pavement stabilisation applications (Bourne-Webb 1995).

## **14.7 Proprietary Products**

Many proprietary products are available for soil stabilisation, and only a selection of them are discussed in this section.

### **14.7.1 Weslig 120**

Weslig 120 is a lignosulphonate-based additive marketed in New Zealand for stabilising soils and for dust control. The organic chemicals in it are composed of 74% calcium-based lignosulphonate and 26% carbohydrates (wood sugars).

Laboratory tests showed the strength gain on stabilising a domestic AP20 aggregate with Weslig 120 were small compared to stabilising the same aggregate with lime. Immersion in water indicated rapid leaching of the Weslig 120, suggesting that any strength gains may be lost in rainy periods (Orr 1986). This raises doubts about the suitability of Weslig 120 for stabilisation if the objective is to obtain long-term strength gains.

Experience with Weslig 120 on unsealed roads has been favourable. Vincent (1986) reports a reduction in the grading cycle on an unsealed portion of SH25 (in Thames Coromandel District) from 4 weeks to 15 to 20 weeks by the use of Weslig 120. Without Weslig the costs are only those of a grader, and with Weslig the costs include grader, water cart, roller and the Weslig. Nevertheless Vincent calculates that the longer grading cycle of 15 to 20 weeks still means the use of Weslig is economically justifiable, compared to the no-additive solution.



Matheson (1989) reports on trials in a low rainfall area in Australia with two 50-metre long sections, one sealed and one unsealed, when Weslig 120 was used to treat a highly plastic basecourse. Initial observation after two years indicated good performance. Matheson also reports that Weslig 120 does not work with sandy materials.

Nolan (1989) reports that Weslig 120 on an unsealed road in Australia was a good dust suppressant for three weeks, but had reduced effectiveness after that time. He also reports that the road surface became "greasy" when wet. Bartley Consultants (1995) report that 6 weeks after spray application on roads in the Bay of Plenty and Bay of Islands, New Zealand, Weslig was ineffective as a dust suppressant, but the effect was longer-lasting (e.g. more than 8 weeks) if the Weslig was mixed into the wearing course. It was suggested that the performance of Weslig as a dust suppressant relates to the properties of the soil.

Charlton (1989) claims that Weslig 120, among other benefits:

- is a compaction aid,
- controls dust on unsealed roads,
- can be used with lime and/or cement-stabilisation,
- can be applied to a pavement base before sealing with bitumen and chip,
- performs most economically with materials of which more than 20% pass the 75µm sieve.

#### **14.7.2 Reynolds Road Packer**

Dunlop (c.1980) reports on trials on an unsealed road, in a New Zealand forest, that compared the performance of clays on road sections stabilised with Reynolds Road Packer with other treatments including an untreated section and a lime-stabilised section. Benkelman Beam testing and maintenance requirements showed the lime-stabilised section to clearly out-perform the Reynolds Road Packer section. The performance of the Reynolds Road Packer section was similar to the untreated section.

Reynolds Road Packer however has been reported to be effective for controlling dust on unsealed roads.

Reynolds Technologies also market a product called RT-12. It was previously called Road Packer Premium, and is reported to have been developed and enhanced from the original Reynolds Road Packer. The manufacturer's literature claims RT-12 induces ion exchange in the soil, resulting in increased density and strength.

#### **14.7.3 Arquad 2HT**

Included in the trials incorporating the Reynolds Road Packer (Dunlop c.1980), was an unsealed road section treated with Arquad 2HT. The Arquad 2HT-treated section performed only slightly better than the untreated section, while the lime-stabilised section performed well in terms of increased Benkelman beam deflections and maintenance requirements.

#### **14.7.4 Fujibeton**

Fujibeton was marketed as a cement additive that was claimed to broaden the scope of cement-stabilisation for soils and in particular to allow the efficient treatment of clay soils. Gaerty (c.1984) reports on laboratory tests with Fujibeton on two loess soils, and concludes that no obvious advantage was indicated. The use of Fujibeton plus cement in these tests failed to show any benefits over the use of OPC.

#### **14.7.5 KOBM**

KOBM is produced at the New Zealand Steel Works at Glenbrook and it is the slag by-product of the basic oxygen-furnace process for steel manufacture (see Section 14.5.2 of this chapter).

The chemical composition of KOBM varies according to the nature of the steel-making process. KOBM contains lime (calcium oxide) but, while the total quantity ranges between 44% and 55%, only between 7% and 15% exists in the free form (Allan 1990).

Allan (1990) reports on laboratory tests and trials comparing KOBM as a stabiliser in clays, and contaminated basecourses, with the use of lime. In these soils the KOBM at 20% dosage rate generally produced similar or higher strengths than lime at 3% dosage rate. However the reactivity of the KOBM varies which must be recognised in the design of individual stabilisation jobs. Allan concluded that, within economic transport distances of Glenbrook, KOBM can offer considerable cost benefits over alternative stabilisers.

#### **14.7.6 Polyroad**

Polyroad is a polymer material which is used with a fly ash carrier. It is for use in soils containing clays. It functions by surrounding and waterproofing the clay particles to produce a non-cementitious material that will not crack. Performance can be enhanced by stabilising with 1% lime, then with Polyroad (M Wallace, pers.comm. 1994). Compatibility with the soil is a factor in the performance of Polyroad.

Some users report good performance from Polyroad in both unsealed and sealed pavements. One of its claimed advantages is the ability to grade unsealed roads that have been stabilised with Polyroad.

#### **14.7.7 Bioenzymes and Endurazyme #388**

Bioenzymes were developed as a by-product of the enzyme industry, and may be used for the stabilisation of roading soils that contain clay. The detailed mechanisms by which proprietary enzyme products stabilise soils are generally not available.

The general nature of bioenzyme stabilisers is to provide a bacterial culture in an enzyme solution. When exposed to the carbon dioxide in the air the bacteria multiply rapidly and produce large organic molecules which the enzyme attaches to the clay molecules in the soil, blanketing ion exchange points in the clay. This action prevents further absorption of moisture and results in a stable construction material. During the hydration that follows compaction, ion exchanges form linkages between closely packed soil particles, providing the cementing bond. Full strength is reached within 4 to 5 days (Scholen and Coghlan 1991).

Clay is essential for the bioenzyme to react. Thus a well-graded aggregate with some clay provides the best performance, but clean crushed rock with no fines will not react (Scholen and Coghlan 1991).

Endurazyme #388 is a bioenzyme derived from the secretions that ants use to bind soils to make ant-hills (B Jackson, pers.comm. 1996). It is a liquid which is added to water, then sprayed onto the road by water cart(s) and mixed into the soil. Mixing can be done with rotary mixers or grader windrowing. Further water is added as necessary and mixed with the material to achieve 1% to 2% dry of optimum water content. The mixture is then compacted. It is important that the Endurazyme-soil mixture is not wetter than the optimum water content of the soil mixture at the time of compaction (Bennett 1994, Scholen and Coghlan 1991, WEA undated).

Endurazyme #388 is best used in soils containing 20% or more particles passing the 75µm sieve, and with a significant clay content as evidenced by a hydrometer analysis, and by a plasticity index (PI) greater than 8 (B Jackson, pers.comm. 1996). Varying degrees of success have been achieved on soils with less than 20% passing 75µm, and for the stabilisation of such soils a high PI is desirable (WEA undated).

#### **14.7.8 Road Tech 2000**

Road Tech 2000 is claimed to be a natural biological product suitable for stabilising soils containing clays. The soils must contain 20% or more particles passing the 75µm sieve, and have a plasticity index greater than 3 or linear shrinkage greater than 2 (RTI 1996).

From use on Australian roads, Road Tech 2000 is claimed to reduce maintenance costs and dust on unsealed roads, or to strengthen subgrades on sealed roads (RTI 1996).



## **15. THE FUTURE**

Increased use of stabilisation on New Zealand roads has considerable scope, particularly using lime, cement, or bitumen. The potential includes not only use in pavement construction and rehabilitation, but also in pavement repairs and unsealed roads.

While the equipment, science and understanding of stabilisation have developed over recent decades, a certain amount of art remains. There is scope for further development of equipment. There is scope for refinements to the additives, or perhaps scope for new additives. There remains much to be learnt and refined. These are the challenges for the future.



# **APPENDICES**

- 1. BIBLIOGRAPHY**
- 2. SAFETY PRECAUTIONS WHEN USING LIME**
- 3. PRELIMINARY FIELD TESTS**
- 4. LIME & CEMENT APPLICATION RATES**
- 5. GLOSSARY**





**APPENDIX 1**  
**BIBLIOGRAPHY**



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**APPENDIX 2**  
**SAFETY PRECAUTIONS**  
**WHEN USING LIME**



## **SAFETY PRECAUTIONS WHEN USING LIME**

### **1. Introduction**

These notes are based on the safety notes given in the *NLA Lime Stabilization Construction Manual* (1991) and on first aid advice from the National Toxicology Group (A Fountain, Dunedin School of Medicine, pers.comm. 1996). The notes provide only limited information. Those working with lime should attend an appropriate First Aid course. Urgent information can be obtained by contacting telephone numbers listed under "Poisons and hazardous chemicals" in the "Emergency services" section of The Telephone Book.

Hydrated lime (calcium hydroxide), like most materials, is safe to work with provided a few simple precautions are exercised. Quicklime (calcium oxide) is considerably less safe than hydrated lime.

While both types of lime are strongly alkaline, quicklime is much more caustic and reacts immediately with moisture. Therefore it can produce severe burns quickly when in contact with moist skin. Eyes especially must be protected. A fleck of quicklime in the eye immediately takes up moisture from the eye, and so generate heat and create a burn that could do severe damage (Dunlop 1977). Mouth and nose, being moist areas, are also as vulnerable to burns.

Quicklime should not come into contact with skin. If it does, wash it off or at least brush it off immediately. Hot, humid weather conditions tend to heighten the caustic effect of quicklime on a worker's skin.

Hydrated lime ideally should not come into contact with skin. Although danger from severe burns is unlikely, prolonged contact of hydrated lime with a perspiring worker's skin, especially if the skin is chafed perhaps by tight clothing, has produced bad burns. Particularly sensitive skins may develop forms of skin irritation (dermatitis) through prolonged contact. Although normally not urgent, wash hydrated lime off skin with water as soon as convenient.

The following recommendations for working with lime will prevent the possibility of burns or skin irritation.

## **2. Clothing**

- Wear at least one long sleeved shirt or sweat shirt. Do not allow rolled up sleeves or short sleeved shirts. In cool weather, wear a second long sleeved shirt for added protection.
- Wear high top shoes or laced boots.
- Wear trouser legs tied over shoe tops. Do not permit shorts.
- Wear hat or cap to protect scalp from accumulated lime dust.
- Wear gauntlet-type gloves.
- Do not wear clothes that bind too tightly around the neck or wrists, to reduce chafing which may cause lime to be more irritating to the skin.

## **3. Protective Cream**

Apply a protective cream like "Savlon Barrier Cream" (obtainable from any chemist) to exposed parts of the body, e.g. neck, face, wrists or ankles, before a worker is to have prolonged exposure to lime dust. Properly applied, the cream provides a thin, protective film on the skin which is easily removed after work with soap and water.

## **4. Eye Protection**

Wear safety glasses with side shields, or goggles at all times while working with lime.

## **5. Mouth and Nose Protection**

When construction conditions are dusty, wear a lightweight filter mask.

## **6. After Work**

Bathe or shower at the end of the work day to clean the whole body entirely of lime and protective cream.



## **7. First Aid**

- **Skin burns:** Irrigate with copious water for 20 to 30 minutes. Use cold or lukewarm water, e.g. from a hose or shower.

Seek medical attention.

The use of ointments is not recommended as they contain the lime and heat on the skin so making the burn worse. Also, the doctor must remove the ointment to examine the burn.

- **Lime in the eyes:** Hold worker's eye open and flush out with copious water immediately. Irrigate with copious running water for 20 to 30 minutes. Too much water cannot be enough.
- Then seek medical attention from an eye specialist, e.g. an Ophthalmologist at a Hospital Accident and Emergency facility. If this is not practical, seek medical attention from a General Practitioner.

## **8. Medical Attention**

If burns do occur, the severity of the injury may be worse than indicated by the pain felt. This reduced sensation of pain is because the injury may cause loss of nerve endings. Hence it is important for all but the most trivial injuries to be seen by a doctor (A Fountain, pers.comm. 1996).

## **9. Care**

Generally the greatest hazard from lime is when workers are handling bagged lime on the roadway or are operating bulk spreaders. They are most vulnerable to lime burns, and must apply the above precautions rigorously. In general, greater care should be exercised in bag applications than bulk spreading.

Strict care should be exercised at all times with all forms of lime to prevent lime getting into the eyes. Therefore all workers emptying bags of lime must be equipped with close fitting goggles. One example why goggles are needed is when a worker in a bent-over position drops an open bag of lime on the ground. The impact can cause a dense cloud of lime dust that will rise directly onto the worker's face. If his eyes are unprotected by goggles, loss of sight may result from lime burns.

## *Appendix 2. Safety Precautions When Using Lime*

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The least hazard from lime burns is encountered in slurry applications. Only workers with unusually sensitive skins could be adversely affected by slurry (or the thick "whitewash") splashing on their bare skin. But the same rigid care should be exercised to prevent any lime slurry getting into the eyes.

Hydrated lime is not particularly hazardous, but it is a light fine dust that is prone to blow around, so care and protective clothing are necessary.

Quicklime dust is hazardous. Quicklime in chip or coarse sand sizes is less likely to blow, so reducing body contact, but care and protective clothing are always necessary.

Most contractors experienced with applying lime have not had trouble from burns. However, prevention is better than treating an injury, so all contractors should carefully brief each worker on these precautions for using lime and, most important, should check that the worker abides by these few simple safety rules.

**APPENDIX 3**  
**PRELIMINARY FIELD TESTS**



## **PRELIMINARY FIELD TESTS**

### **1. General**

Preliminary field tests are important in both the initial evaluation of projects and during construction.

During the initial evaluation of projects, tests for water content, material types and material variability can assist with preliminary judgements on the suitability of the soils for stabilisation and on the appropriate type(s) of stabilisation.

When samples are taken for formal laboratory testing, these samples will be from only a few locations. However, during construction, when the entire site is exposed, further judgements can be made on the identification of non-typical materials and on materials requiring further testing or special measures (Tait 1981).

### **2. Estimation of Soil Type**

Soil types may be broadly categorised as boulders, gravels, sands, silts, or clays, or some combination of these. Estimates may be made in the field to indicate which of these soil types are present. These estimates require experience and are based on the soil's appearance, texture (feel) and dry strength.

To estimate soil texture in the field, observations are made as follows:

*Appearance:* boulders, gravel and coarse sand particles can be distinguished with the naked eye. Fine sand particles are difficult to distinguish. Table A3.1 gives particle sizes for boulders, gravels, sands, silts and clays.

*Fine fraction textures:* mix a small amount of the soil in the hand with a little water or saliva to a firm consistency, and rub the moist soil between the fingers to judge the proportions of sand, silt and clay fractions. With experience, very good estimates of soil texture can be obtained.

Sandy soils have a gritty feel (although be aware that small proportions of sand can over-ride one's judgement), and the soil cannot be rolled into a ball. Silty soils have a slippery silky feel, and the soil will roll into a ball but cracks easily. Clay soils have a smooth greasy feel, and will roll into a ball. If the clay ball is cut with a knife, the cut surface also looks shiny or polished.

*Appendix 3. Preliminary Field Tests*

Table A3.1 Particle sizes for boulders, gravels, sands, silts and clays (after NZS 4402:1986).

| Soil Name |             | Particle Visibility                                  | Particle Size Range |
|-----------|-------------|--|---------------------|
| Boulders  |             | Individual particles distinguished by naked eye      | >200mm              |
| Gravel    | Very Coarse |  | 60-200mm            |
|           | Coarse      |  | 20-60mm             |
|           | Medium      |  | 6-20mm              |
|           | Fine        |  | 2-6mm               |
| Sand      | Coarse      | Individual particles difficult to see with naked eye | 600µm-2mm           |
|           | Medium      |  | 200-600µm           |
|           | Fine        |  | 60-200µm            |
| Silt      | Coarse      | Individual particles not distinguished by naked eye  | 20-60µm             |
|           | Medium      |  | 6-20µm              |
|           | Fine        |  | 2-6µm               |
| Clay      |             |  |                     |

*Appearance with moisture:* if the soil sample is dampened further, and water appears on its surface when it is shaken vigorously backwards and forwards in the palm of your hand, yet the water disappears when the sample is squeezed, then the soil is mainly silt.

If the dampened soil sample is allowed to dry on the fingers and it will not adhere to the fingers at all it is sandy. If the sample dusts off the fingers, it is a silty soil. If it sticks to the fingers it is a clay soil.

*Dry Strength:* roll a small lump of moist soil into a ball and air dry it (e.g. in a warm engine compartment). Test its dry strength by the resistance to crushing of the ball with the fingers. Coarse grained (sandy) soils do not form balls and show no dry strength. Silty soils form lumps which can be easily pulverised with the fingers. Clay soils form lumps which can be broken with the fingers but not pulverised. The dry strength increases with increasing clay contents (SME 1984).

### 3. Organic Matter

Preliminary field tests that indicate presence of organic matter (SME 1984) are:

- Dark colour
- Visible fibrous matter
- Rotting smell (from H<sub>2</sub>S; may have to heat the soil gently to discern it).

### 4. Preliminary Field Soil-Water Content of Clay Soils

Take a handful of the clay soil, as it is in the field (i.e. without adding water). Carry out two simple tests as follows:

*Pencil test:* Roll the sample between the hands into a "pencil". Clay soil, when it is damp, can be rolled by hand to a thickness less than that of a pencil (e.g. <5mm diameter). The fineness of the thread that can be rolled before it crumbles indicates the water content (and amount of clay) of the soil as sampled. The water content indicates if the soil as sampled is suitable for compaction (Table A3.2).

*Squeeze test:* Squeeze a handful of the soil into a ball. If it sticks together yet does not stick to the hand, it is suitable for compaction (after Tait 1981). If it is sticky it is too wet, and if it does not stick together it is too dry, for compaction (Table A3.2).

Table A3.2 Inferences obtained from preliminary field water content tests on clay soils.

| Pencil Test   | Squeeze Test                     | Suitability for Compaction |
|---|----------------------------------|----------------------------|
| Rolls to pencil thickness before crumbling                                | Sticks together                  | Correct                    |
| Rolls to very fine thickness without crumbling, or smears without rolling | Sticks to the hands, or extrudes | Too wet                    |
| Crumbles and will not roll  | Will not stick together          | Too dry                    |

## **5. Indicative Reactivity Test**

The initial evaluation of a project requires judging if stabilisation may be beneficial and if so, is the soil likely to react (e.g. with lime or cement) to give an improvement to the soil properties such as strength. See Chapters 4 and 5 of this report.

Take a 10 kilogram sample of the soil. Prepare two approximately equal-sized representative samples from it, each containing about 3 kilograms of the material that passes a 4.75mm sieve. These two samples are placed separately in plastic bags.

Take one of the two samples, mix 72 grams of cement (for cement-stabilisation) or hydrated lime (for lime-stabilisation) with one of the samples. Thoroughly mix the materials by shaking them in the plastic bag. If necessary the mixture should be passed again through a 4.75mm sieve. Compact the mixture into an available mould. After moulding, the specimen should be stripped from the mould, sealed in a plastic bag and then cured in a warm place for a few days.

The second sample (without any addition of cement or lime) is placed in a similar mould, then compacted and cured in the same way as the first sample.

After the few days, both samples are placed in water. If notable extra strength or resistance to slaking is exhibited by the first sample with added cement or lime, compared to the second sample without cement or lime, the soil should be regarded as reactive to the stabilising agent. Laboratory testing then design should follow, to more accurately evaluate the suitability and economics of stabilising the soil, and also the appropriate type and percentage of stabilising agent.



**APPENDIX 4**  
**LIME & CEMENT**  
**APPLICATION RATES**



## LIME & CEMENT APPLICATION RATES

Table A4.1 Weight (kg) of quicklime after it has been spread on a 1m<sup>2</sup> canvas square, for quicklime containing 90% available lime.

| Compacted Layer Thickness (mm) | Dry Density of Soil After Compaction (kg/m <sup>3</sup> ) | Lime Application Rate*                |       |       |       |       |       |       |
|--------------------------------|---|---------------------------------------|-------|-------|-------|-------|-------|-------|
|                                |   | 1%                                    | 2%    | 3%    | 4%    | 5%    | 6%    | 7%    |
|                                |   | Weight/Unit Area (kg/m <sup>2</sup> ) |       |       |       |       |       |       |
| 100                            | 1600  | 1.78                                  | 3.56  | 5.33  | 7.11  | 8.89  | 10.67 | 12.44 |
|                                | 1800  | 2.00                                  | 4.00  | 6.00  | 8.00  | 10.00 | 12.00 | 14.00 |
|                                | 2000  | 2.22                                  | 4.44  | 6.67  | 8.89  | 11.11 | 13.33 | 15.56 |
|                                | 2200  | 2.44                                  | 4.89  | 7.33  | 9.78  | 12.22 | 14.67 | 17.11 |
| 125                            | 1600  | 2.22                                  | 4.44  | 6.67  | 8.89  | 11.11 | 13.33 | 15.56 |
|                                | 1800  | 2.50                                  | 5.00  | 7.50  | 10.00 | 12.50 | 15.00 | 17.50 |
|                                | 2000  | 2.78                                  | 5.56  | 8.33  | 11.11 | 13.89 | 16.67 | 19.44 |
|                                | 2200  | 3.06                                  | 6.11  | 9.17  | 12.22 | 15.28 | 18.33 | 21.39 |
| 150                            | 1600  | 2.67                                  | 5.33  | 8.00  | 10.67 | 13.33 | 16.00 | 18.67 |
|                                | 1800  | 3.00                                  | 6.00  | 9.00  | 12.00 | 15.00 | 18.00 | 21.00 |
|                                | 2000  | 3.33                                  | 6.67  | 10.00 | 13.33 | 16.67 | 20.00 | 23.33 |
|                                | 2200  | 3.67                                  | 7.33  | 11.00 | 14.67 | 18.33 | 22.00 | 25.67 |
| 175                            | 1600  | 3.11                                  | 6.22  | 9.33  | 12.44 | 15.56 | 18.67 | 21.78 |
|                                | 1800  | 3.50                                  | 7.00  | 10.50 | 14.00 | 17.50 | 21.00 | 24.50 |
|                                | 2000  | 3.89                                  | 7.78  | 11.67 | 15.56 | 19.44 | 23.33 | 27.22 |
|                                | 2200  | 4.28                                  | 8.56  | 12.83 | 17.11 | 21.39 | 25.67 | 29.94 |
| 200                            | 1600  | 3.56                                  | 7.11  | 10.67 | 14.22 | 17.78 | 21.33 | 24.89 |
|                                | 1800  | 4.00                                  | 8.00  | 12.00 | 16.00 | 20.00 | 24.00 | 28.00 |
|                                | 2000  | 4.44                                  | 8.89  | 13.33 | 17.78 | 22.22 | 26.67 | 31.11 |
|                                | 2200  | 4.89                                  | 9.78  | 14.67 | 19.50 | 24.14 | 29.33 | 34.22 |
| 225                            | 1600  | 4.00                                  | 8.00  | 12.00 | 16.00 | 20.00 | 24.00 | 28.00 |
|                                | 1800  | 4.50                                  | 9.00  | 13.50 | 18.00 | 22.50 | 27.00 | 31.50 |
|                                | 2000  | 5.00                                  | 10.00 | 15.00 | 20.00 | 25.00 | 30.00 | 35.00 |
|                                | 2200  | 5.50                                  | 11.00 | 16.50 | 22.00 | 27.50 | 33.00 | 38.50 |
| 250                            | 1600  | 4.44                                  | 8.89  | 13.33 | 17.78 | 22.22 | 26.67 | 31.11 |
|                                | 1800  | 5.00                                  | 10.00 | 15.00 | 20.00 | 25.00 | 30.00 | 35.00 |
|                                | 2000  | 5.56                                  | 11.11 | 16.67 | 22.22 | 27.78 | 33.33 | 38.89 |
|                                | 2200  | 6.11                                  | 12.22 | 18.33 | 24.44 | 30.55 | 36.67 | 42.78 |

Notes on p.A4-5.

*Appendix 4. Lime & Cement Application Rates*

Table A4.2 Weight (kg) of type GP cement after it has been spread on a 1m<sup>2</sup> canvas square (after Tait 1981).

| Compacted Layer Thickness (mm) | Dry Density of Soil After Compaction (kg/m <sup>2</sup> ) | Cement Application Rate*              |       |       |       |       |       |       |
|--------------------------------|---|---------------------------------------|-------|-------|-------|-------|-------|-------|
|                                |   | 1%                                    | 2%    | 3%    | 4%    | 5%    | 6%    | 7%    |
|                                |   | Weight/Unit Area (kg/m <sup>2</sup> ) |       |       |       |       |       |       |
| 100                            | 1600  | 1.60                                  | 3.20  | 4.80  | 6.40  | 8.00  | 9.60  | 11.20 |
|                                | 1800  | 1.80                                  | 3.60  | 5.40  | 7.20  | 9.00  | 10.80 | 12.60 |
|                                | 2000  | 2.00                                  | 4.00  | 6.00  | 8.00  | 10.00 | 12.00 | 14.00 |
|                                | 2200  | 2.20                                  | 4.40  | 6.60  | 8.80  | 11.00 | 13.20 | 15.40 |
| 125                            | 1600  | 2.00                                  | 4.00  | 6.00  | 8.00  | 10.00 | 12.00 | 14.00 |
|                                | 1800  | 2.25                                  | 4.50  | 6.75  | 9.00  | 11.25 | 13.50 | 15.75 |
|                                | 2000  | 2.50                                  | 5.00  | 7.50  | 10.00 | 12.50 | 15.00 | 17.50 |
|                                | 2200  | 2.75                                  | 5.50  | 8.25  | 11.00 | 13.75 | 16.50 | 19.25 |
| 150                            | 1600  | 2.40                                  | 4.80  | 7.20  | 9.60  | 12.00 | 14.40 | 16.80 |
|                                | 1800  | 2.70                                  | 5.40  | 8.10  | 10.80 | 13.50 | 16.20 | 18.90 |
|                                | 2000  | 3.00                                  | 6.00  | 9.00  | 12.00 | 15.00 | 18.00 | 21.00 |
|                                | 2200  | 3.30                                  | 6.60  | 9.90  | 13.20 | 16.50 | 19.80 | 23.10 |
| 175                            | 1600  | 2.80                                  | 5.60  | 8.40  | 11.20 | 14.00 | 16.80 | 19.60 |
|                                | 1800  | 3.15                                  | 6.30  | 9.45  | 12.60 | 16.75 | 18.90 | 22.05 |
|                                | 2000  | 3.50                                  | 7.00  | 10.50 | 14.00 | 17.50 | 21.00 | 24.50 |
|                                | 2200  | 3.85                                  | 7.70  | 11.55 | 15.40 | 19.25 | 23.10 | 26.95 |
| 200                            | 1600  | 3.20                                  | 6.40  | 9.60  | 12.80 | 16.00 | 19.20 | 22.40 |
|                                | 1800  | 3.60                                  | 7.20  | 10.80 | 14.40 | 18.00 | 21.60 | 25.20 |
|                                | 2000  | 4.00                                  | 8.00  | 12.00 | 16.00 | 20.00 | 24.00 | 28.00 |
|                                | 2200  | 4.40                                  | 8.80  | 13.20 | 17.60 | 22.20 | 26.40 | 30.80 |
| 225                            | 1600  | 3.60                                  | 7.20  | 10.80 | 14.40 | 18.00 | 21.60 | 25.20 |
|                                | 1800  | 4.05                                  | 8.10  | 12.15 | 16.20 | 20.25 | 24.30 | 28.35 |
|                                | 2000  | 4.50                                  | 9.00  | 13.50 | 18.00 | 22.50 | 27.00 | 31.50 |
|                                | 2200  | 4.95                                  | 9.90  | 14.85 | 19.80 | 24.75 | 29.70 | 34.65 |
| 250                            | 1600  | 4.00                                  | 8.00  | 12.00 | 16.00 | 20.00 | 24.00 | 28.00 |
|                                | 1800  | 4.50                                  | 9.00  | 13.50 | 18.00 | 22.50 | 27.00 | 31.50 |
|                                | 2000  | 5.00                                  | 10.00 | 15.00 | 20.00 | 25.00 | 30.00 | 35.00 |
|                                | 2200  | 5.50                                  | 11.00 | 16.50 | 22.00 | 27.50 | 33.00 | 38.50 |

Notes on p.A4-5.

Appendix 4. Lime & Cement Application Rates

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**Notes** to Tables A4.1 and A4.2:

- \* Mass of CaO or cement as a percentage (1-7%) of the dry mass of the soil after compaction.
- Usual maximum application rate of quicklime per application is 14 kg/m<sup>2</sup> for coarse particles, and 10 kg/m<sup>2</sup> for fine particles of quicklime.
- The actual size of the canvas square used should be measured. If it is not exactly 1m<sup>2</sup> then proportional adjustment will be required.
- Formula used to calculate weight of quicklime for Table A4.1 is:

Weight (kg) of quicklime/m<sup>2</sup> =

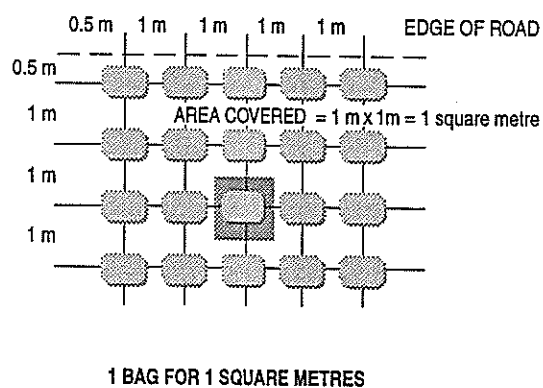
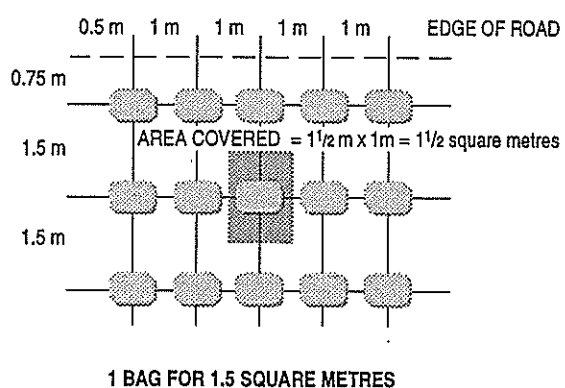
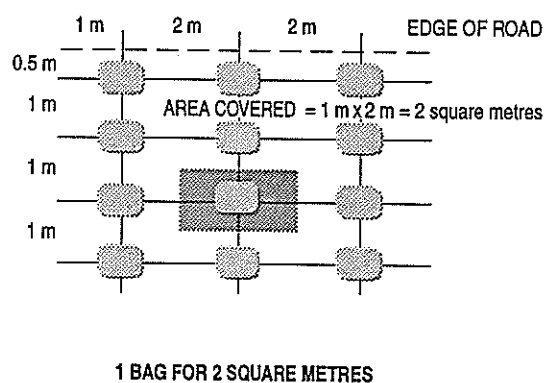
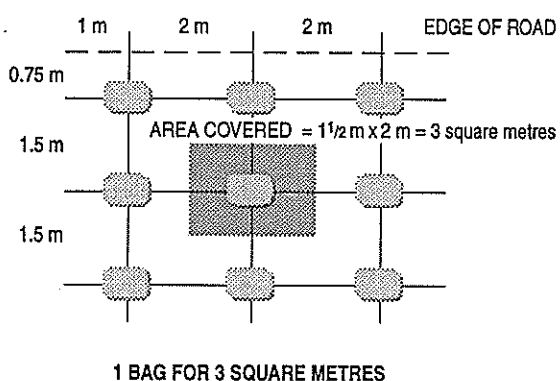
$$\text{layer depth} \times \text{soil dry density} \times \frac{\text{lime application rate}}{100} \times \frac{100}{\text{grade of lime used (90\% assumed)}}$$

- Formula used to calculate weight of cement for Table A4.2 is:

$$\text{Weight (kg) of cement/m}^2 = \text{layer depth} \times \text{soil dry density} \times \frac{\text{cement application rate}}{100}$$

#### Appendix 4. Lime & Cement Application Rates

Figure A4.1 Typical cement-bag layouts for spreading cement (after C&CA 1990).



**Note:** Required number of square metres to be covered by one bag may be read from Table A4.2.

## **APPENDIX 5**

## **GLOSSARY**





## GLOSSARY

*Many of the definitions in this Glossary are from or are based on Armitage (c.1987) and apply to terms used in this report. Terms in italics are defined separately. Alternative names for terms are in quote marks.*

|                              |  |
|------------------------------|--|
| <b>Aggregate</b>             | A general term for mineral particles such as crushed rock, slag, gravel and sand.  |
| <b>AGO</b>                   | Automotive Gas Oil, or diesel. A petroleum distillate used to flux bituminous materials (TNZ 1991).  |
| <b>Application Rate</b>      | The amount of material (usually binder) applied to a given area of road surface. Binder application rate is expressed as litres per square metre.  |
| <b>Atterberg Limits Test</b> | The test method describes the measurement of the following properties (Bartley and Cornwell 1993):<br>(a) <i>Liquid Limit</i> and <i>Plastic Limit</i> ,<br>(b) <i>Shrinkage Limit</i> ,<br>(c) <i>Plasticity Index</i> .  |
| <b>Available Lime Index</b>  | The amount of Calcium oxide in <i>Quicklime</i> or <i>Hydrated Lime</i> as determined by the Available Lime Index test of ASTM C25.  |
| <b>Base</b>                  | The layer of material constituting the uppermost structural element of a pavement, immediately beneath the <i>Wearing Course</i> or <i>Surfacing</i> .   |
| <b>Basecourse</b>            | The graded aggregate that can be used in a base layer.   |
| <b>Binder</b>                | A general term for a material used to hold aggregates in place.  |
| <b>Bitumen</b>               | A viscous liquid or solid obtained by processing the residue from the distillation of suitable crude petroleum (TNZ 1991). Bitumen may be straight-run (residue obtained after the final stage of distillation of crude petroleum); blown (hardened by blowing air or steam through it at high temperatures); or precipitated. |
| <b>Bitumen Emulsion</b>      | A liquid product in which bitumen particles are suspended in water by means of emulsifying and stabilising agents (TNZ 1991).  |
| <b>Block Cracking</b>        | A particular pattern of cracking which appears as a series of connected rectangles.  |

|                                 |  |
|---------------------------------|--|
| <b>Breaking</b>                 | Separation of particles of bituminous binder from the water in a bitumen emulsion to form a continuous film.   |
| <b>Bulk Density</b>             | The mass of material (including solid particles and any contained water) per unit volume including voids (Bartley and Cornwell 1993).  |
| <b>Burnt Lime</b>               | See <i>Quicklime</i> .   |
| <b>Carbonation</b>              | The reaction between free lime and atmospheric carbon dioxide (Dunlop 1977) to produce $\text{CaCO}_3$ .   |
| <b>CBR</b>                      | California Bearing Ratio test (NZS 4402:1986 Test 6.1.1; NZS 4407:1991 Test 3.15).   |
| <b>Cement</b>                   | The term normally given to ordinary Portland Cement (OPC), after 1995 called type General Purpose (GP) cement. See also <i>Portland Cement</i> .   |
| <b>Cement Concrete Pavement</b> | A general term for a pavement in which the <i>Wearing Course</i> is concrete. It is usually implied that the concrete layer combines the functions of both the base and of the surfacing.  |
| <b>Cement-Stabilisation</b>     | The controlled application of cement to a soil material to rectify deficiencies or to improve its properties for the intended use.   |
| <b>Cement M4</b>                | M/4 (TNZ 1985) basecourse, i.e. conforms to TNZ M/4:1995 specification, and which has been cement-stabilised.  |
| <b>Cement Percentage</b>        | <p>The mass of cement as a given percentage of the dry mass of the soil to which it is being added (based on Dunlop 1977); or</p> <p>Mass of cement as a percentage of the dry mass of the soil (dry density of the soil after compaction) to which it is being added.</p> |
| <b>Cemented</b>                 | A cement-soil or lime-soil mixture with significant tensile strength, i.e. greater than 80 kPa.  |
| <b>Cemented Soil</b>            | A soil which has reacted to lime or cement by flocculation and/or cementation (Dunlop 1977) and has a tensile strength greater than 80 kPa.  |
| <b>Clay-size Fraction</b>       | The fraction of a soil composed of particles smaller in size than 0.002mm (NZS 4402:1986).   |

|                            |   |
|----------------------------|---|
| <b>Coarse Aggregate</b>    | Aggregate with particle size greater than 5mm (Bartley and Cornwell 1993).  |
| <b>Concrete</b>            | A mixture consisting predominantly of clean graded aggregate, water and cement (after NZS 3121), usually having cement content greater than 20% by weight. See also <i>Portland Cement Concrete</i> . |
| <b>Conditioning</b>        | The process by which the lime-soil mix is left in an uncompacted state before final compaction is initiated. Also referred to as two stage mixing (Dunlop 1977).                                      |
| <b>Cure</b>                | To stiffen or harden by evaporation of volatile constituents or by chemical reaction.   |
| <b>Cure Coat</b>           | A layer of bituminous material applied to a cement-treated or lime-treated course to retain moisture and allow full hydration of the cement.  |
| <b>Cutter</b>              | A volatile liquid added to temporarily reduce the viscosity of bitumen, e.g. kerosine, turpentine, white spirits (TNZ 1991).  |
| <b>Diffuse Cementation</b> | The diffusion of lime in solution throughout the matrix and clay lumps in a lime-soil (or cement-soil) mix (from Dunlop 1977).  |
| <b>Durability</b>          | The ability of a material to continue to provide the service for which it is intended (Bartley and Cornwell 1993).  |
| <b>EDA</b>                 | Equivalent Design Axle. A measure of traffic load, related to the pavement wear caused by an 8.2 tonne single axle with dual tyres (NRB 1989).  |
| <b>Fatigue</b>             | The deterioration of a pavement or test specimen caused by the action of repeated loading.  |
| <b>Filler</b>              | A very fine aggregate, mainly passing a 75µm sieve (TNZ 1991).  |
| <b>Flocculation</b>        | Bonding together of small particles to form larger particles.   |
| <b>Flux</b>                | A relatively low volatility liquid used to permanently, or semi-permanently, reduce the viscosity of a bitumen, e.g. AGO (TNZ 1991).  |
| <b>Fluxed Bitumen</b>      | A bitumen which has had its viscosity reduced by the addition of a low volatile diluent (TNZ 1991).   |

|                               |  |
|-------------------------------|--|
| <b>Foamed Bitumen Process</b> | A process patented by the Mobil Oil Company in which hot bitumen is expanded greatly in a chamber by the introduction of cold water. Bitumen is then distributed to the soil to be stabilised via a low pressure spray bar (after Bowering 1988).  |
| <b>Grade (of lime)</b>        | <p>Quality of <i>Quicklime</i> or <i>Hydrated Lime</i>. Proportion of active calcium oxide determined in accordance with TNZ M/15:1986 (TNZ 1986).</p> <p>It is usual for the lime manufacturer to certify the grade of the quicklime. The grade is based on the calcium oxide content as measured by the <i>Available Lime Index</i> (ASTM C25-1992) then calculated from 30 consecutive tests as (TNZ M/15:1986):</p> $\text{Grade} = \text{Mean} - 1.25 \times \text{Standard Deviation}$ |
| <b>Grading, light</b>         | The surface crust is not broken, loose surface material is spread or raked back to provide a running course to protect the crust (Ferry 1986).   |
| <b>Gravel-size Fraction</b>   | The fraction of a soil composed of particles between the sizes 200mm and 2mm (NZS 4402:1986).  |
| <b>High Impact Process</b>    | A process patented by the BP Oil Company in which hot bitumen is expanded greatly by injection at high pressure into a moist soil (after Bowering 1988).   |
| <b>Hydrated Lime</b>          | Calcium hydroxide $\text{Ca}(\text{OH})_2$ , also called "slaked lime".  |
| <b>Lean Concrete</b>          | A low cement content concrete. Usually having cement content in the range 8–12% by weight. Also called "lean mix".   |
| <b>Lift</b>                   | Vertical height or depth of material placed or compacted in one stage of construction.   |
| <b>Lime Fixation Point</b>    | Point at which increased lime content produces no further marked change in <i>Plastic Limit</i> (Arabi and Wild 1989).   |
| <b>Lime Percentage</b>        | <p>The mass of lime as a percentage of the dry mass of the soil to which it is being added (based on Dunlop 1977); or</p> <p>Mass of <math>\text{CaO}</math> as a percentage of the dry mass of the soil (dry density of the soil after compaction) to which it is being added.</p>  |

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| <b>Lime-Stabilisation</b>                                       | The controlled application of lime to a soil material to rectify deficiencies or to improve its properties for the intended use.  |
| <b>Limestone</b>  | Calcium carbonate ( $\text{CaCO}_3$ ) sometimes used as filler. The material from which <i>Quicklime</i> and <i>Slaked Lime</i> are derived.                                  |
| <b>Liquid Limit (LL)</b>  | The water content of the soil at the upper boundary of the plastic state. See also <i>Atterberg Limits Test</i> .   |
| <b>Mix-In-Place</b>   | Mixing of the stabilising agent into the existing soil, while the soil is in its original position. Also referred to as "mixed in-situ".                                      |
| <b>Modified Soil</b>  | A soil which has reacted with lime or cement by flocculation and has a tensile strength of less than 80 kPa (Dunlop 1977).  |
| <b>Moisture Content</b>   | Term sometimes used instead of <i>Water Content</i> .   |
| <b>New Zealand Standard Compaction</b>                          | Compaction of a soil sample in accordance with NZS 4402:1986 Test 4.1.1.  |
| <b>OPC</b>  | Ordinary Portland cement, the traditional type of cement used in New Zealand. See also <i>Portland Cement</i> .   |
| <b>Particle-sizes</b>   | See <i>Clay-size Fraction</i> , <i>Gravel-size Fraction</i> , <i>Sand-size Fraction</i> , <i>Silt-size Fraction</i> .   |
| <b>Penetration Value</b>  | The distance (in tenths of millimetres) that a standard needle sinks into a bitumen sample under standard conditions (TNZ 1991).  |
| <b>Penetration Grade</b><br>(also called <b>Bitumen Grade</b> ) | Method of characterising bitumens based on their penetration value. Expressed by the limits of the range of penetration values for a specific penetration grade, e.g. 80/100. |
| <b>Plant-Mixing</b>   | Mixing of the stabilising agent with a soil or mixture of soils in a central plant.   |
| <b>Plasticity Index (PI)</b>                                    | The numerical difference between the Liquid and Plastic Limits. See also <i>Atterberg Limits Test</i> .   |
| <b>Plastic Limit (PL)</b>                                       | The water content of the soil at the lower boundary of the plastic state. See also <i>Atterberg Limits Test</i> .   |

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| <b>Portland Cement</b><br>(also called <b>type GP cement</b> ) | Traditionally this has been Ordinary Portland Cement ( <i>OPC</i> ) as defined in NZS 3122:1990 Specification for Portland cement. More recently this has become type General Purpose Portland Cement (type GP cement) as defined in NZS 3122:1995. Usually referred to as <i>Cement</i> . (DP Barnard, pers.comm. 1996.) |
| <b>Portland Cement Concrete</b>                                | Concrete in which the binding material is ordinary Portland cement. Usually referred to as <i>Concrete</i> .  |
| <b>Pozzolan</b>  | A naturally occurring material that has cementitious properties in the presence of water. Frequently it requires the presence of some lime or cement to initiate its reaction. Named after the Italian town of Pozzuoli where it was first used.  |
| <b>Pre-treatment</b>   | The process of initially adding a portion of the lime, thus assisting in the break-up of massive clay lumps (based on Dunlop 1977). Also referred to as "double application of lime".   |
| <b>Pulverisation</b>   | The breaking down of soil lumps to a fine grade (Dunlop 1977).  |
| <b>Quicklime</b>   | Calcium Oxide (CaO) used as an additive in soil stabilisation. Also called "burnt lime". Supplied in granular form.   |
| <b>Resilient Modulus</b>                                       | A dynamic characteristic of a pavement material (unique of the testing conditions) defined as the ratio of the repeated stress to the recoverable elastic strain (for estimating <i>Young's Modulus</i> ) (NRB 1989).   |
| <b>Ripping</b>   | See <i>Scarifying</i> .   |
| <b>Rotary Mixer</b>  | A purpose-built machine for stabilisation incorporating one or several toothed drums on transverse shaft(s) for pulverisation and mixing. Also called a "rotovator".  |
| <b>Running Course</b>  | A thin layer of loose stones which is designed to protect the <i>Wearing Course</i> or <i>Top Course</i> of an unsealed road (Ferry and Major 1991).  |
| <b>Sand-size Fraction</b>                                      | The fraction of a soil composed of particles between the sizes 2.0mm and 0.06mm (NZS 4402:1986).  |
| <b>Scarifying</b>  | The systematic disruption and loosening of the top of a pavement or of natural ground by mechanical or other means. Also called "ripping".  |

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| <b>Servicehole</b>               | A shaft with a removable cover that leads down to a sewer, drain or other underground service. May be called "manhole".   |
| <b>Shoving</b>                   | Lateral displacement of pavement structure by heavy vehicles.   |
| <b>Shrinkage Limit</b>           | Water content of a soil below the <i>Plastic Limit</i> when no further shrinkage will take place. See also <i>Atterberg Limits Test</i> .   |
| <b>Silt-size Fraction</b>        | The fraction of a soil composed of particles between the sizes 0.06mm and 0.002mm (NZS 4402:1986).  |
| <b>Slag</b>                      | General name for non-metallic residue formed during the production of iron and steel.   |
| <b>Slaked Lime</b>               | See <i>Hydrated Lime</i> .  |
| <b>Soil</b>                      | Any naturally occurring mineral matter derived from, or forming part of, the weathering upper rock layers of the earth's crust.   |
| <b>Stabilisation</b>             | A means of rectifying a deficiency, or a means of improving the properties, of a soil material for the intended use (after NAASRA 1986).  |
| <b>Stabilised Material</b>       | Any material which has been stabilised.   |
| <b>Stiffness</b>                 | The resistance of a material to permanent or recoverable deformation (Bartley and Cornwell 1993).   |
| <b>Strain</b>                    | Change of shape as a result of an applied stress (Bartley and Cornwell 1993).   |
| <b>Stress</b>                    | Force divided by the area over which it acts (Bartley and Cornwell 1993).   |
| <b>Sub-base</b>                  | A lower structural layer of the pavement, consisting of <i>Aggregate</i> or <i>Stabilised Material</i> .  |
| <b>Subgrade</b>                  | The upper layer of the pavement foundation; i.e. the top of a fill (imported and compacted material, and usually carefully selected) or bottom of cut (in-place material, and usually compacted). |
| <b>Surfacing, Surface Course</b> | The uppermost layer of the pavement (TNZ 1991).   |
| <b>Top Course</b>                | See <i>Wearing Course</i> .   |
| <b>UCS</b>                       | Unconfined Compressive Strength Test (NZS 3112, Part 2: 1986).  |

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| <b>Water Content</b>         | The mass of water which can be removed from a soil by heating to 105°C, expressed as a percentage of dry mass (NZS 4407:1991). Also referred to as "moisture content".   |
| <b>Well-graded Aggregate</b> | Aggregate having a balanced distribution of particle sizes so as to produce near-maximum density when compacted (Ferry and Major 1991).  |
| <b>Wearing Course</b>        | A pavement layer constructed on top of the <i>Basecourse</i> layer for the purpose of providing a smooth riding surface and sufficient thickness to allow for some attrition by traffic and climate. This layer may be the same material as the basecourse. Also referred to as <i>Top Course</i> or <i>Surface Course</i> (Ferry and Major 1991). |
| <b>Young's Modulus</b>       | The ratio of stress to elongation of a cylindrical element in an elastic medium (NRB 1989).  |