DESIGN OF PAVEMENTS INCORPORATING A STABILISED SUBGRADE LAYER: LITERATURE REVIEW

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AN IMPORTANT NOTE FOR THE READER

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EXECUTIVE SUMMARY

Introduction

Subgrade stabilisation, typically using lime and/or cement, is a technique for improving poor subgrade soils. It is frequently used in New Zealand and throughout the world. This report describes the results of a literature review, carried out in 1996, on the characterisation and design of pavements incorporating a lime- or cement-stabilised subgrade layer. The review has been motivated by Transit New Zealand's adoption in 1995 of the AUSTROADS pavement design procedures for use in New Zealand.

Before the adoption of the AUSTROADS pavement design procedures in New Zealand, pavements with a stabilised subgrade layer were designed so that the tensile stress occurring at the underside of the stabilised layer was less than the tensile strength of the stabilised material. While this appears to be a sound approach, a number of complexities about the behaviour of stabilised layers are not addressed in the methodology.

Soil Stabilisation

When stabilising a subgrade soil two levels of treatment can be achieved – modified or cemented. Modified materials experience a modest increase in elastic modulus, and the soils have improved handling characteristics. Cemented materials generally achieve a relatively high elastic modulus and significant tensile strength. It is this tensile strength that is relied upon to develop slab action to resist the applied traffic stresses.

Behaviour of Stabilised Subgrades

Intuitively it may appear that constructing a cemented layer with a high elastic modulus would be of great benefit. However, its disadvantage is that the layer attracts a disproportionate share of the applied stresses. The brittle nature of the material makes it vulnerable to fatigue type failure which is very difficult to model for pavement design, both in the pre-cracking phase and the post-cracking phase of the pavement life. Modelling of pavement life is further complicated by the following issues:

- cracking caused by hydration of the stabilising agent;
- cracking caused by thermal stresses;
- cracking induced by construction traffic;
- variability of material properties caused by non-uniform additive mixing;
- variability of material proporties caused by inadequate compaction, especially at the bottom of a stabilised layer;
- difficulties associated with modelling material performance in the laboratory; and
- change of material properties with time.

Conclusions

- A review of the literature revealed that not a lot of information is available. The available technical literature suggests that subgrade stabilisation should be limited to modification, i.e. where the performance of the stabilised layer is dominated by compressive stresses as opposed to tensile stresses. This results in a ductile type failure mechanism rather than a brittle type failure mechanism.

- The AUSTROADS pavement design philosophy is flexible enough to accommodate the design of a pavement with a modified subgrade layer by treating the pavement as if it has two subgrades. The pavement can be modelled using the *CIRCLY* multi-layer elastic software and the vertical compressive strains at the top of both the modified layer and the original subgrade can be ascertained. The respective strain values can then be used in the AUSTROADS subgrade performance criterion to determine the service life of each component.

- One difficulty is to ascertain the boundary between modified and cemented behaviour. One approach is to stipulate a certain tensile strength criterion, e.g. 80 kPa, but this aspect of stabilised layer behaviour requires further investigation. A preliminary proposal along these lines that draws an anology to the state of stress prevailing in a test specimen in the so-called Brazilian test, is described in the report.

- The issue of longevity of stabilised fine-grained subgrade materials should also be examined.

ABSTRACT

This report presents the results of a literature review carried out in 1996. It includes a discussion on the materials and reactions involved in lime and cement stabilisation. It describes the distinction between modified and cemented material behaviour and discusses the advantages and disadvantages of the two kinds of stabilisation. The report concludes that subgrade stabilisation should be restricted to soil modification as opposed to soil cementation.

Soil modification allows poor subgrade soils to be improved without the disadvantages and vagaries of cemented soil performance. Modified soil generally offers a modest increase in elastic modulus, and improved handling characteristics, e.g. increased resistance to water ingress and volume change. Conversely, cemented soils achieve a relatively high elastic modulus and consequently they attract a large proportion of the applied stress. This makes cemented materials vulnerable to fatigue failure and a subsequent loss of performance. Quantifying the loss of performance in a rational pavement design procedure is an extremely difficult task, and one that has not been satisfactorily addressed in the literature reviewed for this project.

1. INTRODUCTION

1.1 General

Stabilisation of subgrade soils using lime and/or cement is a practice that is commonly used for pavement construction in New Zealand and many other countries around the world. It provides a cost-effective means of dealing with poor subgrade conditions as the in situ materials are improved rather than dug out and replaced. This provides benefits which are consistent with New Zealand's Resource Management Act (1991), e.g:

- the supply of premium quality materials is conserved; and
- the need to discard material as waste is removed.

Smith (1975) conducted a survey of roading authorities throughout New Zealand and concluded that virtually all pavements constructed using a stabilised layer had performed well and provided significant economic benefits.

Lime or cement can be used to improve the properties of subgrade, sub-base and basecourse materials. However, the project recorded in this report was required to focus on subgrade improvement.

When a subgrade material is stabilised it is often referred to as a *stabilised sub-base layer* which can be confusing. Throughout this report a stabilised subgrade layer will be referred to simply as a *modified* or *cemented subgrade layer*.

In New Zealand, the design of pavements with modified or cemented subgrades, up until July 1995, has been carried out using the procedures described in the document *Transit New Zealand State Highway Design and Rehabilitation Manual* (Transit New Zealand 1989). In July 1995 Transit New Zealand adopted the mechanistic pavement design procedures of the Australian roading authority, AUSTROADS. These procedures are detailed in the document *Pavement Design: A Guide to the Structural Design of Road Pavements* (AUSTROADS 1992), herein after referred to as the AUSTROADS Guide. To facilitate the use of the AUSTROADS Guide in New Zealand, Transit New Zealand has produced a supplementary document (TNZ 1995) which addresses issues that are unique to New Zealand's conditions.

The AUSTROADS Guide (as at 1992) provides a design strategy for cemented basecourse and sub-base type materials but does not provide a detailed design procedure for pavements incorporating a stabilised subgrade layer.

1.2 Objectives

The objective of this project is to review the technical literature up to 1996 regarding the design of pavements incorporating a stabilised subgrade layer and to make appropriate recommendations for future inclusion in the New Zealand supplement to the AUSTROADS Guide.

A future stage to this project could comprise a laboratory testing programme designed to validate or extend the relevant outcomes from this literature review. Therefore, a second objective of this project is to recommend appropriate strategies and procedures to be used in future investigations.

2. SOIL STABILISATION

2.1 General

A brief description of the applications and reaction mechanisms for both lime- and Portland cement-treated soils is presented in the following paragraphs. The chemistry of the reactions is extremely complex and is not pursued in great detail. The discussion is limited to lime and cement treatment as these are the additives commonly used in New Zealand. A limited amount of stabilisation using KOBM (a derivative of the steel-making process) and bitumen is carried out in New Zealand but these materials are not discussed in further detail here.

Soil stabilisation is a technique that has been known to highway engineers for a very long time. Roman roadbuilders used lime in their pavement structures and some of their roads have survived to this day (Hudson 1996). Portland cement is a more recent soil stabilising agent, first being put to use about the 1940s (Sherwood 1993). Today, soil stabilisation is used extensively for pavement construction in many countries throughout the world, including the United States (US), France, South Africa, Australia, New Zealand, Germany and Sweden.

One reason for its increased use has been the significant increases in the price of petroleum products which inflate the costs of both the production and transportation of traditional civil engineering contruction materials, such as crushed rock aggregates. Another reason is the depletion of sources of high quality crushed rock aggregate necessitating the use of smaller quantities of high quality aggregate and/or larger quantities of lower quality materials.

Soil stabilisation is not only used in pavement construction projects. It is also used in many other civil engineering applications, e.g:

- to increase the bearing capacity of foundation soils for structures;
- to armour materials against the action of water;
- to promote slope stability;

- to produce superior backfill material for pipes and culverts;
- to construct low permeability liners; and
- to construct retaining structures such as walls and dams.

2.2 Lime Stabilisation

2.2.1 General

Lime stabilisation can be applied to a range of soil types but it is most effective when the clay content is at least 10% and the plasticity index is in the range 10% to 50% (BACMI 1986). Mateos (1964) reports that the most reactive common clay minerals are montmorillonite and kaolinite, then illite and halloysite. Highly organic soils and soils containing sulphates are not conducive to effective lime stabilisation (Hudson 1996).

Purpose-built stabilisation plant is used to mix the lime into the soil. It is important that a uniform, intimate mixture is achieved and that the soil is at its optimum water content for subsequent compaction. Introduction of the stabilising agent in a slurry form is also possible although it is generally only appropriate for large scale operations and has not been used in New Zealand to date.

2.2.2 Materials

The term *lime* is somewhat arbitrarily used to describe limestone (calcium carbonate, $CaCO_3$) and many of its associated compounds. Limestone itself has little stabilising value but it is the parent material from which the stabilising agents are produced. In New Zealand, lime used for soil stabilisation is generally available in two forms, quicklime (calcium oxide, CaO) and hydrated lime (calcium hydroxide, $Ca(OH)_2$). While calcium hydroxide is the active compound in the stabilisation reaction, the quicklime product is almost exclusively used for pavement applications. Quicklime is converted to hydrated lime in situ by adding water, a process called *slaking*. Quicklime is also easier to handle as it is produced in a granular form whereas hydrated lime is produced in a powdered form.

The process of slaking quicklime is strongly exothermic, i.e:

$$CaO + H_2O = Ca(OH)_2 + \text{heat} \qquad \text{Equation 1}$$

The exothermic nature of the slaking reaction has the benefit that the heat produced helps to dry the soil–lime mixture which is typically wet of optimum water content. In some applications, water may not need to be added to the quicklime as sufficient moisture is in the soil to complete the slaking process. However, the disadvantage of using quicklime is that if the material comes into contact with a worker's skin or eyes it can result in severe chemical burns. Hydrated lime also causes burns but to a much lesser degree.

2.2.3 Soil–Lime Reaction

The reaction of lime with soil is a two stage mechanism. The first mechanism is *flocculation*. When lime is added to a fine grained soil an immediate ion exchange reaction occurs where calcium ions from the lime replace sodium and hydrogen ions on the surfaces of the soil particles. This makes the soil particles flocculate and consequently the material becomes more friable and workable (Bell 1993). The plastic limit (PL) increases while the liquid limit (LL) remains generally constant. Consequently the plasticity index (PI, i.e. the difference between the LL and PL) decreases as shown in Figure 2.1 (Sherwood 1993).

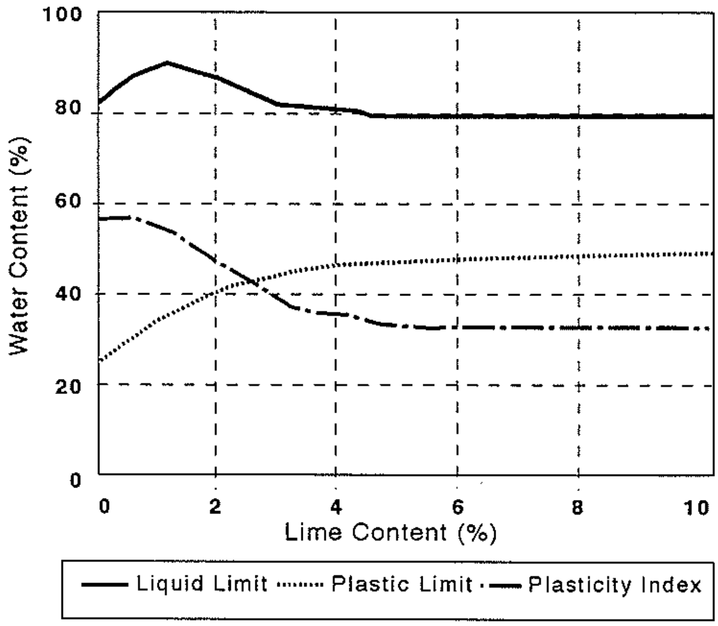


Figure 2.1 Effect of lime on the plasticity properties of London Clay (after Sherwood 1993).

The flocculation reaction is initiated immediately and continues until the soil's affinity for calcium ions is satisfied. This occurs at a lime content known as the *lime fixation point* which is typically 2 -3% by dry weight depending on the mineralogy of the soil. The flocculation reaction is not significantly dependent on temperature (Sherwood 1993).

Up to the lime fixation point the strength of the soil is only increased by the additional frictional component caused by flocculation and the reduced plasticity. Other effects of adding lime up to the lime fixation point are (BACMI 1986):

- the soil is less sensitive to water;
- the potential to shrink and swell is reduced; and
- the optimum water content is increased and the maximum dry density is decreased.

If lime is available in excess of the lime fixation point and if there are pozzolans (i.e. siliceous or siliceous aluminous materials) present, then a second mechanism is initiated. This is the so-called *pozzolanic* or *cementing* mechanism. The excess lime results in a significant increase in the pH of the soil–lime mixture and causes the clay lattice components to be dissolved from the clay structure to form crystalline calcium silicate and aluminate gels (Thompson 1976). These compounds harden with time and promote a significant increase in the strength of the soil. Elevated temperatures force the cementing reactions to proceed at an advanced rate producing stronger crystalline compounds (Ruff & Ho 1966). At low temperatures, i.e. below approximately 5 °C, the cementing reaction may be slowed significantly, or may even stop (Bell 1993). Figures 2.2 and 2.3 indicate the effects of curing time and curing temperature respectively on the strength of soil–lime mixtures.

While the primary effect of the cementing reaction is to significantly increase the soil strength, there are a number of other effects (BACMI 1986) that are beneficial to the engineering properties of the material:

- volume stability is improved;
- erodibility decreases; and
- susceptibility to frost decreases.

The constituents of a soil can have either a deleterious or beneficial affect on the soil–lime reaction (Sherwood 1993). For example, the presence of carbon dioxide can result in carbonation of the calcium ions which reduces the availablity of the calcium hydroxide for participation in the flocculation and cementing reactions. Sulphates, sulphides and some organic materials do not affect the flocculation part of the stabilisation process but they do interfere with the cementing reactions. Conversely, the presence of pozzolanic compounds provides a significant benefit to the soil–lime reaction.

2.2.4 Testing Methods

The *initial consumption of lime (ICL)* test is used to establish the quantity of lime required to accomplish the stabilisation reactions. The test is based on monitoring the pH of the soil–lime mixture in a solution of water. The lime content required to achieve a pH value of 12.4 is referred to as the ICL of the material.

The strength of soil–lime mixtures is generally determined using the California Bearing Ratio (CBR) test, the unconfined compression test, or an indirect type tension test such as the double punch test (Dunlop 1977) or the so-called Brazilian test (Peploe 1987).

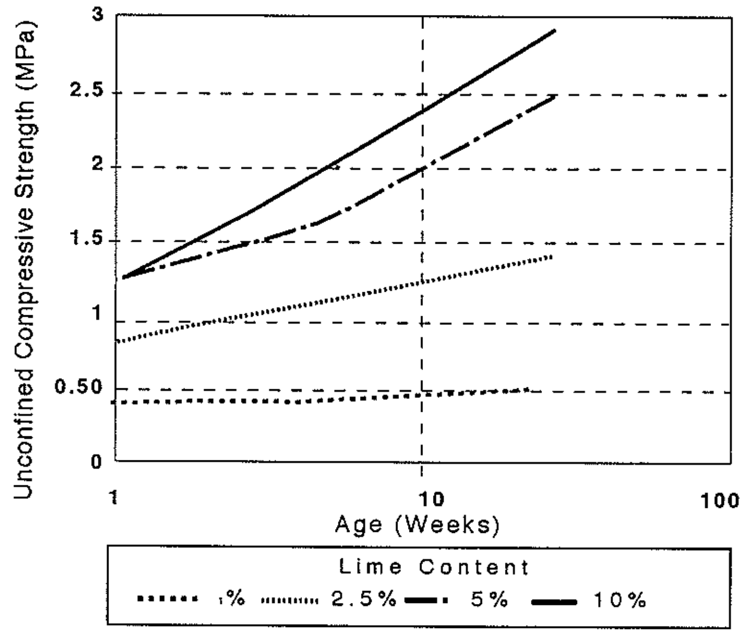


Figure 2.2 Effect of age on the strength of lime-stabilised London Clay (after BACMI 1986).

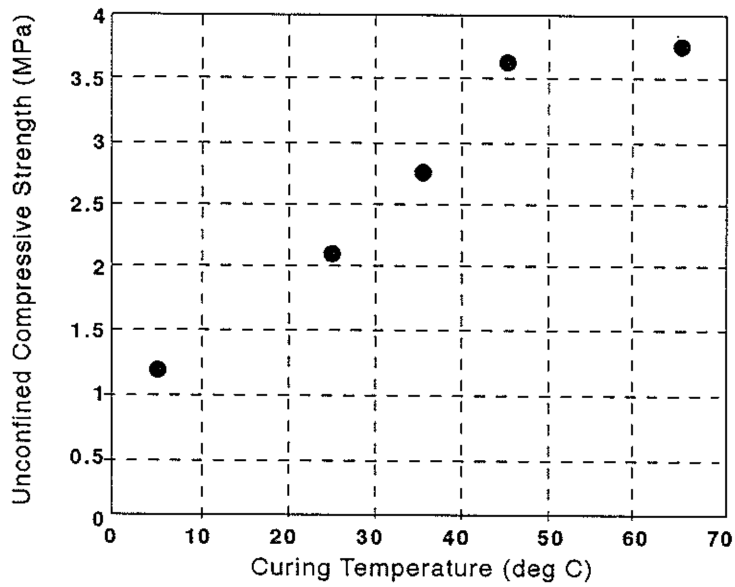


Figure 2.3 Effect of curing temperature on the strength of lime-stabilised soil specimens.

2.3 Cement Stabilisation

2.3.1 General

Soil stabilisation using Portland cement is a similar operation to that using lime, i.e. intimate mixing of the cement with the soil, addition of water to hydrate the cement and compaction of the soil–cement mixture. The main difference is that cement stabilisation is mainly used for granular materials, or for fine grained soils with relatively low plasticity. This is because the cement does not change the structure of the soil, as is the case with lime, but it simply incorporates the soil particles into a cemented matrix. High plasticity soils are not appropriate for cement stabilisation because they are not conducive to intimate cement mixing and the formation of a cemented soil matrix.

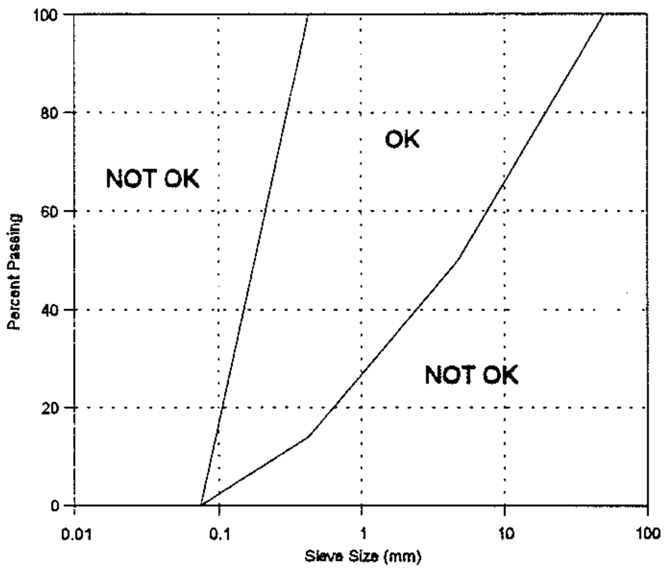
Cement stabilisation is accepted as being somewhat less forgiving than lime stabilisation as the soil–cement mixture tends to set-up more quickly. Generally, compaction should be started as soon as possible after mixing and would need to be completed within about two hours. This time may be extended by the use of retardant admixtures.

2.3.2 Materials

ACI (1990) report that cement stabilisation is appropriate for granular materials or fine grained soils having a plasticity index less than about 8. With soils of higher plasticity, clay balls tend to form on mixing, which inhibit the uniform distribution of cement. Often a light application of lime is used to reduce the plasticity of a soil before cement is used as the final stabilising agent. Soils with a high sulphate or organic content may not be suitable for cement stabilisation.

Capper & Cassie (1956) report that cement stabilisation is suitable for soils with a grading range as presented in Figure 2.4.

Figure 2.4 Grading envelope for soils deemed to be suitable for cement stabilisation (after Capper & Cassie 1956).



The quantity of cement used can range from approximately 3% by dry weight to 15% or more depending on the clay content of the soil. In general, the higher the proportion of clay, the higher is the cement content required.

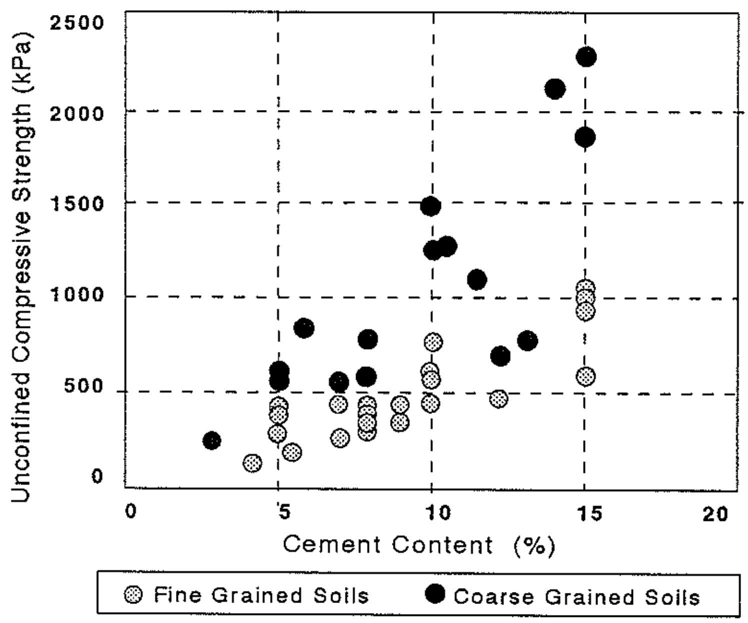
Since cement stabilisation is particularly effective for granular materials, it is often used as a means of rehabilitating existing unbound pavements. This topic is outside the scope of the current review and is not pursued further in this report.

2.3.3 Soil–Cement Reaction

The soil–cement mixture is similar to concrete except that soil replaces sand and gravel as the filler material in the mix. The chemical reaction involves the hydration of calcium silicate and calcium aluminate in the Portland cement to form a product which is less soluble than the original products, i.e. the reaction is one of solution and re-precipitation (Van Vlack 1980). Since the strength of the soil–cement mix is dependent on the hydration of the cement paste and the continuity of the bonding, a higher cement content generally results in a higher strength. Figure 2.5 shows a typical plot of unconfined compressive strength (UCS) versus cement content (%) for both fine and coarse grained soils.

The hydration of cement is a time-dependent reaction. Hence the strength of a soil–cement mix develops with time. Figure 2.6 shows a plot of unconfined compressive strength (UCS) versus curing time for both fine and coarse grained soils.

Figure 2.5 Plot of 28-day strength (UCS) versus cement content (%) for fine and coarse grained-stabilised soils (after ACI 1990).



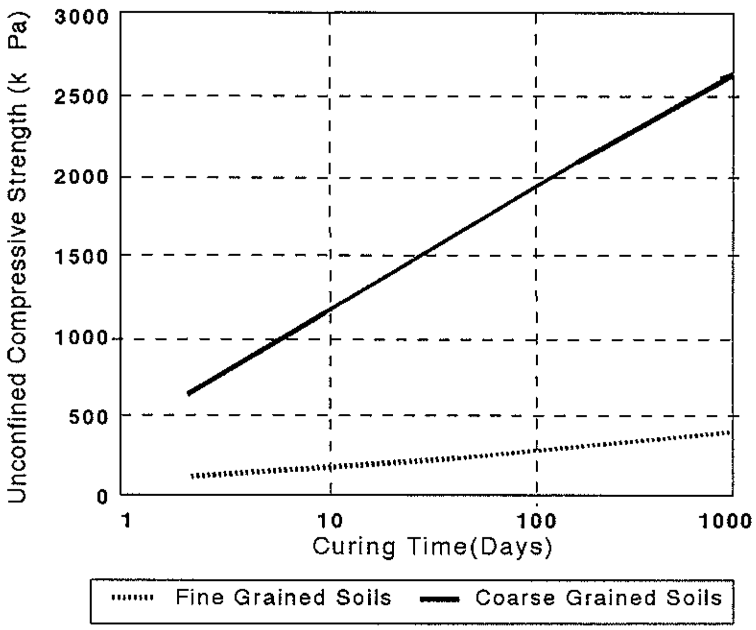


Figure 2.6 Plot of strength (UCS) versus curing time (days) for fine and coarse grained-stabilised soils (after ACI 1990).

2.3.4 Testing Methods

The strength of soil–cement soil mixes is generally expressed in terms of the unconfined compressive strength. Other common test methods are the flexural test (modulus of rupture) and the Brazilian test.

3. BEHAVIOUR OF MODIFIED & CEMENTED SUBGRADES

3.1 General

An important aspect of the design of pavements incorporating a stabilised subgrade layer is the distinction between a *modified* subgrade layer and a *cemented* subgrade layer, defined as follows.

A modified subgrade layer comprises soil which has been stabilised primarily to improve its workability and to provide a good quality construction platform.

Conversely, a cemented subgrade layer comprises soil which has been stabilised to add significant strength to the material so that it can develop slab action to resist the applied loads. Slab action is achieved through the development of horizontal tensile stresses at the underside of the layer.

Modified subgrades are generally constructed using a relatively small amount of stabilising agent and the resulting increase in strength and elastic modulus is quite modest. However, this is not to say that there is little benefit in both pavement design and construction from utilising a modified subgrade layer. Some of these benefits are as follows:

- improved bearing capacity for the top portion of the subgrade;
- provision of a stable working platform for construction plant;
- reduction in the water susceptibility of the subgrade; and
- reduction in the plasticity of the subgrade soils.

Modified materials have the distinct advantage that cracking is not a major concern. This is because the stresses attracted by a modified subgrade are relatively small. Also the shrinkage and thermal stresses are low because of the reasonably small proportion of hydraulic binder in the soil. Conversely a cemented layer has a higher quantity of hydraulic binder and consequently has a high potential for cracking caused by shrinkage and thermal stresses.

Cemented materials require the addition of binder in excess of the lime fixation point in order to develop significant tensile strength. Cemented materials generally exhibit a relatively high elastic modulus which may continue to increase with time. The high elastic modulus results in the cemented layer attracting a major proportion of the applied stress and, when coupled with its brittle behaviour, it becomes vulnerable to fatigue type cracking.

Cemented layers are also significantly influenced by variability in construction quality. This is especially pertinent at the underside of the layer where it can be difficult to achieve thorough mixing and compaction. Unfortunately, the underside of the layer corresponds to the zone of material that is subjected to high tensile stresses under traffic loading (Dunlop 1978).

3.2 Definitions of Modified and Cemented Layers

The distinction between modified and cemented materials is not clearly defined and there is little information on this topic in the technical literature. Up until the adoption of the AUSTROADS Guide in New Zealand, the boundary between modified and cemented materials was defined with respect to tensile strength. Dunlop (1978) defined a progression of material characteristics in terms of tensile strength as shown in Figure 3.1. The boundary between modified material and cemented material was defined as a tensile strength of 80 kPa when the test specimens are cured at 20 °C for 14 days for lime-treated soils, or for 7 days for cement-treated soils. The same definition is provided in Dunlop (1980).

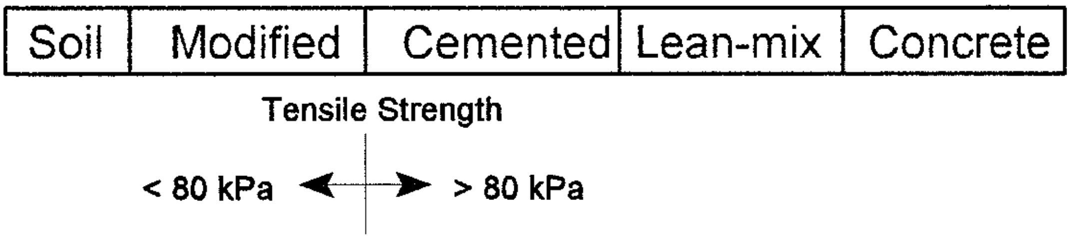


Figure 3.1 Definition of materials in terms of tensile strength (after Dunlop 1978).

Dunlop's 1978 definition was altered slightly in the Transit New Zealand *State Highway Pavement Design and Rehabilitation Manual* (TNZ 1989). This document defines a modified subgrade material as one with an initial CBR less than 10% and which exhibits less than three-fold increase in CBR after stabilisation.

McDowell (1966) suggests that a minimum unconfined compressive strength of 50 psi (345 kPa) is appropriate for a stabilised subgrade layer for it to be treated as being a cemented. This corresponds to an elastic modulus value of approximately 111 MPa (16 kips) using the empirical relationship reported by Thompson (1965), i.e.

$$E\ (\text{kips}) = 9.98 + 0.1235 q_u\ (\text{psi}) \qquad \text{Equation 2}$$

where E = elastic modulus; and
q_u = unconfined compressive strength.
(psi - pounds per square inch; kips - kilopounds per square inch)

If the ratio of the elastic modulus to the tensile strength is taken as 1400 (Dunlop 1980), then McDowell's distinction between modified and cemented behaviour occurs at a tensile strength of approximately 80 kPa, i.e. the same as that reported by Dunlop (1978).

The Australian approach is to ensure that all stabilised subgrade materials act in a modified manner. If cemented behaviour is observed it is generally unintentional and the result of inadvertently adding excess stabilising agent (B.Stacy 1996, pers.comm.).

An alternative way of considering the distinction between modified and cemented subgrade behaviour is to draw an anology between the stresses that occur in a pavement layer with the stress conditions that occur in an indirect tensile test specimen. In the Brazilian test (Figure 3.2) a cylindrical specimen is loaded diametrically, resulting in a complex distribution of stresses.

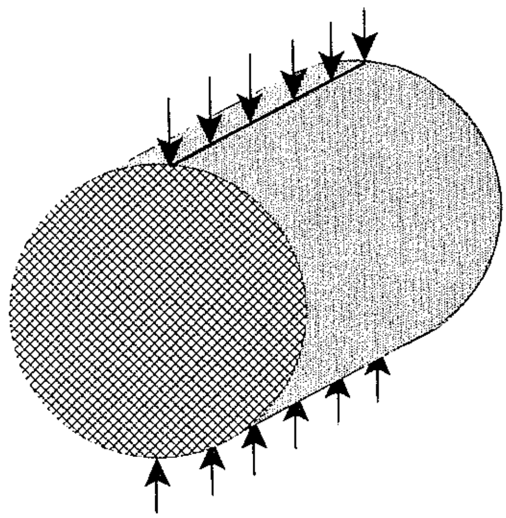


Figure 3.2 Schematic view of the Brazilian test configuration.

Frocht (1957, in Thompson 1965) used the theory of elasticity to develop the equations for stresses resulting from point loading of a circular disk and these have been used to describe the stress distribution in the Brazilian test. Figure 3.3(a) shows the distribution of vertical stresses along the horizontal axis of the specimen, i.e. the vertical stresses are compressive, ranging from zero at the edges of the specimen to a maximum at the centre. Figure 3.3(b) also shows the distribution of horizontal stresses along the vertical axis of the specimen, i.e. a constant tensile stress over virtually the complete cross-section of the specimen. The credence placed on the Brazilian test for tension testing is attributable to the uniformity of the horizontal tensile stress distribution shown in Figure 3.3.

Equations developed by Frocht (reported in Thompson 1965) show that the magnitude of the maximum vertical compressive stress (σ_y) at the centre of the Brazilian test specimen is given by:

$$\sigma_y = \frac{6P}{\pi d l} \qquad \text{Equation 3}$$

and the constant horizontal tensile (σ_x) stress along the y-axis of the specimen is given by:

Equation 4 $$\sigma_x = \frac{-2P}{\pi dl}$$

where: P = applied load;
d = specimen diameter; and
l = specimen length.

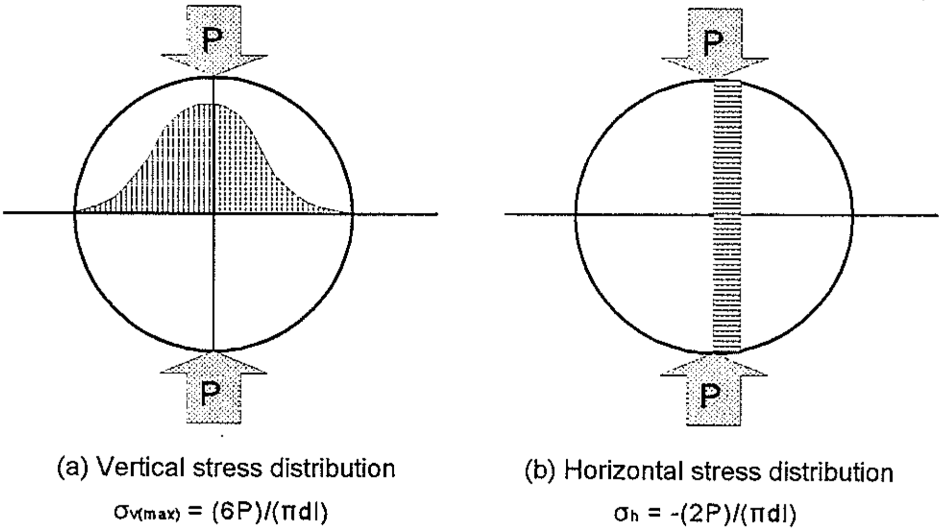


Figure 3.3 Stress distributions in the Brazilian test:
(a) Vertical stress distribution along the horizontal axis;
(b) Horizontal stress distribution along the vertical axis.

The consequence of Equations 3 and 4 is that for the Brazilian test to perform as it is intended, i.e. for the specimen to fail in tension, the compressive strength of the specimen must be at least three times the tensile strength. If this condition is not satisfied, the specimen will fail in a vertical compressive mode. This is similar to the distinction made between the behaviour of modified and cemented subgrade layers.

Hence the argument could be that, for a stabilised subgrade layer to perform in a modified (ductile) fashion, it should have a compressive strength less than three times its tensile strength, whereas for the layer to perform in a cemented (brittle) fashion it should have a compressive strength greater than three times its tensile strength. This approach could be pursued in further detail using computer modelling techniques along with laboratory verification testing.

4. PERFORMANCE CRITERIA FOR STABILISED SUBGRADES

4.1 General

In any rational pavement design procedure, performance criteria are required to relate the pavement's loading response to its expected long-term performance. This is the so-called *performance model*, i.e:

Pavement Response + Material Properties = Pavement Performance

In most mechanistic design procedures the pavement response is taken as the maximum strain occurring at critical locations within the pavement structure. These locations are as follows:

Pavement Component	Critical Strain
Subgrade	Maximum vertical compressive strain - at the top of the subgrade
Cemented Layers	Maximum horizontal tensile strain - generally at the underside of the layer
Asphaltic Concrete	Maximum horizontal tensile strain - generally at the underside of the layer

The material properties are taken as the elastic parameters E and ν, and the pavement performance is generally expressed as the allowable number of applications of an equivalent standard axle load. While most mechanistic design procedures are based on critical strains, critical stresses in the performance model can also be utilised.

In general, design procedures for pavements incorporating modified subgrade layers are much less complex than those for pavements incorporating cemented layers. This is because the function of the modified layer is to simply provide a stable platform for the construction of the overlying pavement layers. Conversely, cemented layers greatly reduce the magnitude of the vertical strain at the top of the subgrade but they must be relied upon to provide significant tensile strength by developing slab action to carry the applied loads. They achieve a relatively high elastic modulus, and consequently they attract a significant proportion of the applied stress and become susceptible to a fatigue-type failure mechanism. They also contain a reasonably high proportion of cementing agent which makes them vulnerable to cracking caused by hydration and thermally induced stresses.

4.2 Criteria for Modified Subgrades

4.2.1 NRB Road Research Unit Technical Recommendation No. 2

Dunlop (1977) presented a method for the design of pavements incorporating a modified subgrade layer. This method has also received endorsement in Australia by the Cement & Concrete Association of Australia (Finlay & Gibney 1986) but it is not included in the AUSTROADS Guide. The method is restricted to subgrade materials having a natural CBR of 5 or less before lime treatment and a modified layer thickness of 150 mm. Dunlop used the multi-layer elastic computer program *BISTRO* to evaluate the vertical compressive strain occurring at the top of both the subgrade and the modified layer. The critical strain was then substituted into the following performance criterion to obtain the service life expressed in equivalent design axles (EDA):

$$\epsilon = 0.028N^{-0.25}$$ Equation 5

where ϵ = vertical compressive strain at the top of the subgrade; and
N = subgrade service life expressed in EDA.

Dunlop recognised that the modified material possessed some level of tensile strength. *BISTRO* was used to determine the maximum tensile stress in the modified layer and that value was compared with the tensile strength of the material (S_t) using the following relationship with elastic modulus (E):

$$S_t = \frac{E}{1400}$$ Equation 6

where S_t = tensile strength of the cemented material; and
E = elastic modulus of the cemented material.

Fatigue performance of the modified layer was not considered in the design because it was argued that the stresses in the modified layer were very small and that the modified layer continued to develop strength with time and therefore became more resistent to fatigue. However, the latter argument is considered to be flawed because, as the modified material becomes stiffer, it correspondingly attracts a greater proportion of the applied stress. In addition, its resistance to fatigue would decrease rather than increase.

4.2.2 TNZ State Highway Pavement Design & Rehabilitation Manual

The *State Highway Pavement Design and Rehabilitation Manual* (TNZ 1989) retracts the design method for modified layers presented in NRB RRU TR2, stating that the method is more appropriate for cemented materials. This statement is on account of the significant tensile strength required by RRU TR2, which is inappropriate for modified soils. The Manual states that modified subgrade materials should be treated as being equivalent to the same thickness of unbound aggregate for design purposes. It provides a design chart prescribing the unbound cover required for a range of subgrade strengths (expressed in CBRs). The pavement model that the design chart is based on uses a modular ratio relationship that combines the basecourse and sub-base layers, and modified layer in this case, into one layer. This approach is deficient in that it does not allow the designer to rationally model the performance of the modified layer.

4.2.3 AUSTROADS Guide

Although the AUSTROADS Guide (1992) does not prescribe a method for designing pavements incorporating a modified layer, the philosophy of the AUSTROADS approach is sufficiently flexible that the engineer can include a modified subgrade layer in a trial pavement design without difficulty. The modified subgrade layer can be treated as a second subgrade and, as such, the maximum vertical compressive strains at the top of the subgrade and at the top of the modified layer should be evaluated separately. The critical strains are determined using the multi-layer elastic program *CIRCLY* and then substituted into the accepted subgrade performance criterion to determine the service life of the respective layers, expressed as equivalent standard axles (ESA), i.e.:

$$N = \left[\frac{8511}{\mu\epsilon}\right]^{7.14} \qquad \text{Equation 7}$$

where $\mu\epsilon$ = vertical compressive strain at the top of the subgrade;and
N = subgrade service life expressed in ESA.

The increased bearing strength achieved in a modified subgrade layer introduces two important benefits in terms of mechanistic pavement design. First, the increased elastic modulus of the modified layer provides added protection to the original subgrade. This reduces the magnitude of the strain occurring at the top of the subgrade and therefore increases its service life. The second benefit results from the improved compaction platform that a modified subgrade layer provides for the construction of the overlying pavement structure. This allows more effective compaction to be imparted to the unbound aggregate and results in higher elastic moduli being achieved in the sub-base and basecourse layers.

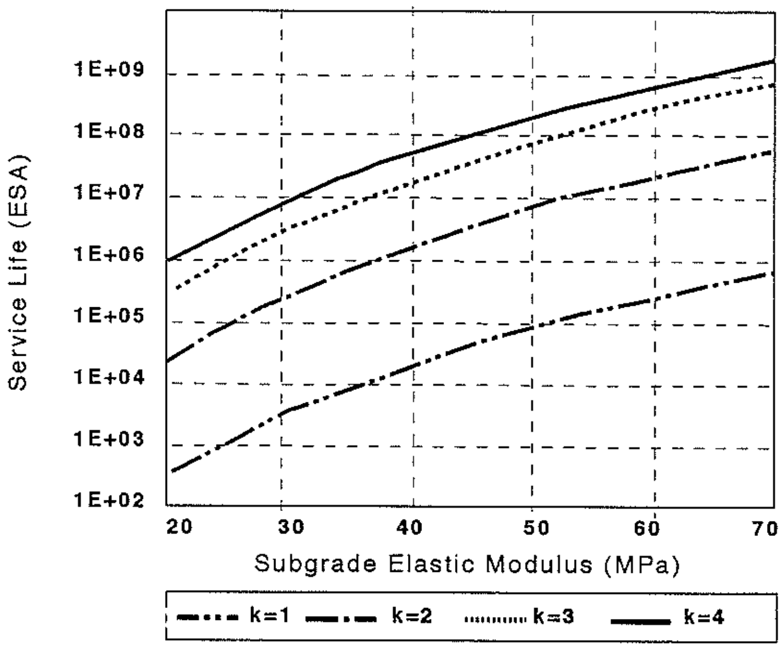


Figure 4.1 Influence of a 300 mm-thick modified subgrade layer in service life (in ESA) for four values of modular ratio (k) using the AUSTROADS mechanistic design.

The plot presented in Figure 4.1 shows the effect of a 300 mm-thick modified subgrade layer on pavement service life (in ESA) for subgrade modulus values ranging from 20 MPa to 70 MPa. The layer overlying the modified subgrade comprises 300 mm of unbound aggregate with an elastic modulus at the top of the layer of 350 MPa. The factor k is the modular ratio between the modified layer and the original subgrade. The k = 1 line represents an unmodified subgrade. The different pavement configurations are summarised in Figure 4.2. The multi-layer elastic program *CIRCLY* (Mincad Systems 1996) has been used to determine the vertical compressive strains at the top of both the subgrade and modified subgrade layers. The critical strain has then been substituted into the AUSTROADS subgrade performance criterion to determine the pavement's service life. The analysis assumes that the AUSTROADS sublayering system for the unbound material applies, and that the elevated elastic modulus of the modified layer persists throughout the service life of the pavement.

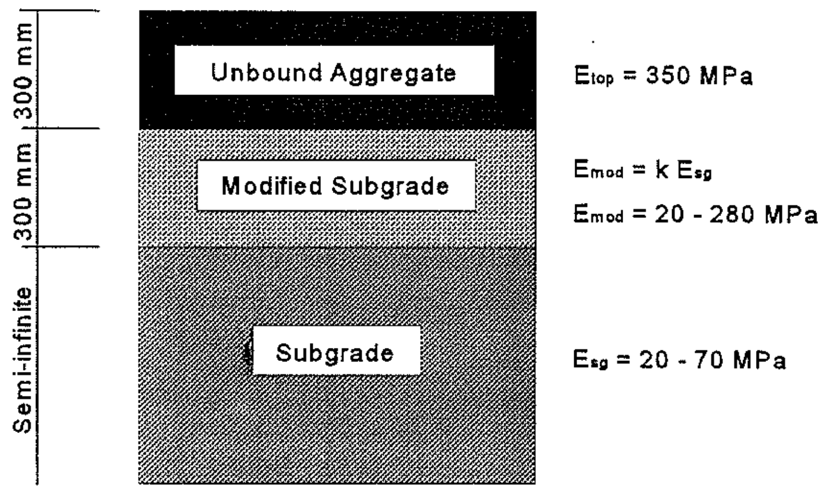


Figure 4.2 Pavement configurations used in analysis summarised in Figure 4.1.

Figure 4.1 shows that a modified subgrade layer, with an elastic modulus double that of the original subgrade, results in an increase in pavement service life of almost two orders of magnitude. A three-fold increase in subgrade elastic modulus results in an increased service life of almost three orders of magnitude. A four-fold increase in subgrade elastic modulus results in increased service life values of more than three orders of magnitude.

4.2.4 TRRL Laboratory Report 1132

The pavement design procedures used in the United Kingdom (UK) are set out in TRRL Laboratory Report LR1132 (Powell et al. 1984). This procedure assumes that the lower section of the pavement has to serve as a haul road for the construction of the upper portion of the pavement. Hence the method incorporates a capping layer placed over the top of the subgrade to provide a stable construction platform that is less susceptible both to the ingress of water and to penetration by aggregates from the overlying sub-base.

Capping layers are required for all subgrades that have CBR values less than 5%. The standard configuration uses a 350 mm-thick capping layer, but when the subgrade CBR is less than 2% the capping layer thickness is increased to 600 mm (Powell et al. 1984).

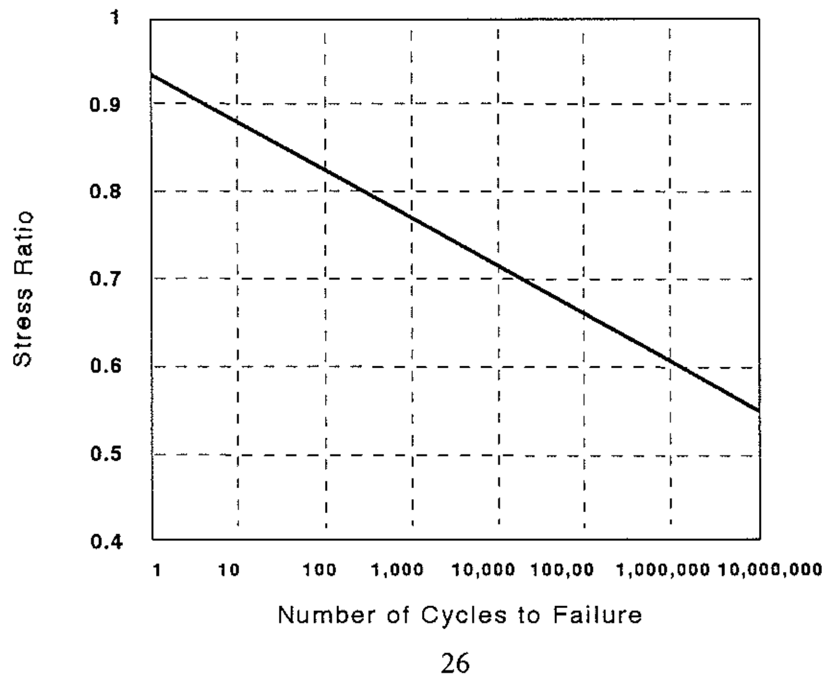
LR 1132 states that the capping layer should be constructed using locally available material and must achieve a minimum CBR of 15%. In many situations this specification can readily be acheived using lime modification (BACMI 1986).

4.3 Criteria for Cemented Subgrades

4.3.1 NRB Road Research Unit Technical Recommendation No. 2

NRB RRU TR2 (Dunlop 1977) describes a procedure for the design of pavements incorporating a cemented subgrade layer. It presents a series of design charts for a range of cemented layer thicknesses and elastic moduli. The charts were developed using a multi-layer elastic computer program (*BISAR*) to evaluate the maximum vertical compressive strain at the top of the subgrade and the maximum horizontal tensile stress at the underside of the cemented layer. The maximum vertical compressive strain at the top of the subgrade was then substituted into the subgrade performance criterion to obtain the subgrade service life value expressed as equivalent design axles. The maximum tensile stress at the underside of the cemented layer is compared with the tensile strength of the cemented material. If the maximum tensile stress exceeds the tensile strength then the layer will crack. An allowance is made for deterioration of the performance of the cemented layer caused by fatigue. The relationship between stress ratio (applied stress divided by static tensile strength) and the number of load cycles to failure is presented in Figure 4.3.

Figure 4.3 Relationship between stress ratio and number of load cycles to failure (after Dunlop 1980).



4.3.2 TNZ State Highway Pavement Design & Rehabilitation Manual

The *State Highway Pavement Design and Rehabilitation Manual* (TNZ 1989) presents design charts for pavements incorporating 150 mm- and 250 mm-thick cemented subgrade layers. An elastic modulus limit of 1,000 MPa is imposed to avoid excessive concentration of stresses in the cemented layer.

The design charts have been developed using a mechanistic approach similar to those presented in RRU TR2 (Dunlop 1977), i.e. the vertical compressive strain at the top of the subgrade and the horizontal tensile stress at the underside of the cemented layer are checked. The tensile strength of the cemented material is determined using the relationship presented in Equation 6 (for modified subgrades, in Section 4.2.1 of this report).

An allowance for fatigue in the cemented layer is made by applying a fatigue factor (f) to the tensile strength value, where f is defined as follows:

$$f = \frac{(18.25 - \log N)}{18.75} \qquad \text{Equation 8}$$

where f = fatigue factor; and
N = number of load repetitions.

The fatigue factor is taken as 1.0 when all the following conditions are satisfied:

- the subgrade soil is fine grained;
- the original subgrade CBR is less than 10%; and
- the increase in CBR is at least three-fold after stabilisation.

4.3.3 AUSTROADS Guide

The AUSTROADS Guide (1992) does not provide for the design of cemented subgrade layers. It does however provide a method of design for cemented aggregate materials with E values in the range 2,000 MPa to 10,000 MPa. The performance criteria recommended for these materials are dependent on the elastic modulus, i.e:

E = 2,000 MPa : $$N = \left[\frac{280}{\mu\epsilon}\right]^{18} \qquad \text{Equation 9}$$

E = 5,000 MPa : $$N = \left[\frac{200}{\mu\epsilon}\right]^{18} \qquad \text{Equation 10}$$

E = 10,000 MPa : $$N = \left[\frac{150}{\mu\epsilon}\right]^{18} \qquad \text{Equation 11}$$

where E = elastic modulus;
N = number of load repetitions (equivalent standard axles ESA); and
$\mu\epsilon$ = maximum horizontal tensile strain at the underside of the cemented layer.

The AUSTROADS performance criteria have been evaluated using accelerated loading trials. Dash (1994) reports that the cemented layer performance criteria used by Queensland Transport and VicRoads are superior to the AUSTROADS criteria listed above. They are as follows:

VicRoads:

$$N = \left[\frac{\frac{112{,}664}{E^{0.804}} - 1}{\mu\epsilon}\right]^{12} \qquad \text{Equation 12}$$

Queensland Transport:

$$N = \left[\frac{14{,}100}{\mu\epsilon\ E^{0.351}}\right]^{8} \qquad \text{Equation 13}$$

where E and $\mu\epsilon$ are as defined above.

While these VicRoads and Queensland Transport performance criteria predict well the behaviour of cemented gravel materials, it is not known if they are appropriate for cemented subgrade layers. It is however understood that cemented subgrade layers are not generally used in Australia. F.Bullen (1996, pers.comm.) suggests that the increase in soil strength obtained from stabilisation should not be relied upon when analysing the projected long-term performance of the pavement. The rationale behind this approach is that any subsequent ingress of water into the soil structure is likely to result in a significant strength reduction. This view is not supported by Hopkins et al. (1995) who report that cement-treated subgrade soils in a selection of US highways have maintained high CBRs over extended periods of time (up to thirty years).

4.3.4 TRRL Laboratory Report 1132

In the UK pavement design procedure, TRRL LR1132 (Powell et al. 1984) recommends that cemented materials occuring in the sub-base layer should be treated as being equivalent to an equal thickness of unbound aggregate. This is an undue simplification because the difference between the perfomance of cemented materials and unbound aggregates is significant. In addition, this type of approach introduces significant inaccuracies because the structural equivalency of a material is not fixed but is a function of the layer thickness (Otte 1978).

4.3.5 Other Literature

The AASHTO pavement design method, in the US, uses a structural equivalency approach to determine the required layer thicknesses. George (1990) reports that a layer coefficient of 0.24 is appropriate in the AASHTO method for soil–cement materials with 7-day compressive strength of at least 600 psi (4.1 MPa). George also makes mention of the Portland Cement Association (PCA) method which provides a design procedure for both cemented base layers and soil–cement layers. The PCA method uses *fatigue consumption coefficients* which are based on limiting the radius of curvature of the cemented layer. The radius of curvature approach was originally proposed by Nussbaum & Larsen (1965), i.e:

$$\frac{R_c}{R} = aN^{-b} \qquad \text{Equation 14}$$

where R_c = critical (failure) radius of curvature;
R = radius of curvature at a given load and number of load repetitions;
N = number of load repetitions; and
a,b = constants depending on soil type and specimen dimensions.

The mechanistic pavement design procedure used in South Africa applies the following fatigue performance criteria for the design of cement-treated basecourse layers (Jordaan 1992):

$$N_f = 10^{9.1(1-\epsilon_s/\epsilon_b)} \qquad \text{Equation 15}$$

where N_f = number of load repetitions at ϵ_s to cause crack initiation;
ϵ_s = tensile strain in cemented layer ($\mu\epsilon$); and
ϵ_b = tensile strain at break ($\mu\epsilon$).

Jordaan (1992) reports that the fatigue failure criterion presented above often under-estimates the life of cemented layers. He suggests that the reason for this observation is that the fatigue criterion was developed using laboratory specimens which do not simulate the field conditions adequately. To allow for this, the strain at break is factored up by a value of 4.7. This has a significant influence on the design life of cemented layers.

Jordaan also reports that the maximum tensile strain in a cemented layer is not necessarily located at the underside of the layer. The results of a large number of analyses, using a multi-layer elastic computer program, show that the location of the maximum tensile strain is dependent on the relative elastic moduli of the cemented layer and of the overlying and underlying layers.

Little further information on structural design methodology is available in the technical literature as the topic is not well accepted. A number of authors have investigated the fatigue behaviour of cemented pavement materials but most of the work has remained within the realms of research and academia.

Otte (1978) reported that failure of cemented pavement layers starts with micro-cracking and a loss of bond between the cementing agent and the filling material, i.e. soil or aggregate. The micro-cracks then propagate until failure occurs. Otte suggests that micro-cracking is not initiated until the stress ratio (applied stress divided by static strength) reaches approximately 0.35. Therefore at stress levels less than 35% of the static strength of the cemented material, the layer will sustain an unlimited number of load applications. Further, cemented materials generally are capable of sustaining approximately one million load applications at stress ratio of 0.50.

M.Thompson (1996, pers.comm.) suggests that characterising the performance of cemented materials in terms of a critical strain is preferable to using a critical stress. This view is shared by Otte (1978) who reported that strain ratio values of 0.25 and 0.33 respectively correspond to the stress ratios of 0.35 and 0.50 mentioned above.

Otte reported a relationship between strain ratio and the logarithm of the number of load repetitions to failure for cemented pavement materials, i.e:

$$\frac{\epsilon}{\epsilon_b} = 1 - 0.11 \log N \qquad \text{Equation 16}$$

where ϵ = applied strain;
ϵ_b = strain at break; and
N = number of load applications to failure.

A plot of the fatigue relationship is presented in Figure 4.4. However, Otte qualifies the fatigue performance relationship stating that the difference between the behaviour of laboratory prepared test specimens and specimens recovered from in-service pavements can be significant. The laboratory specimens show up to 30% higher resistance to loading than the field specimens, mainly because of construction variability.

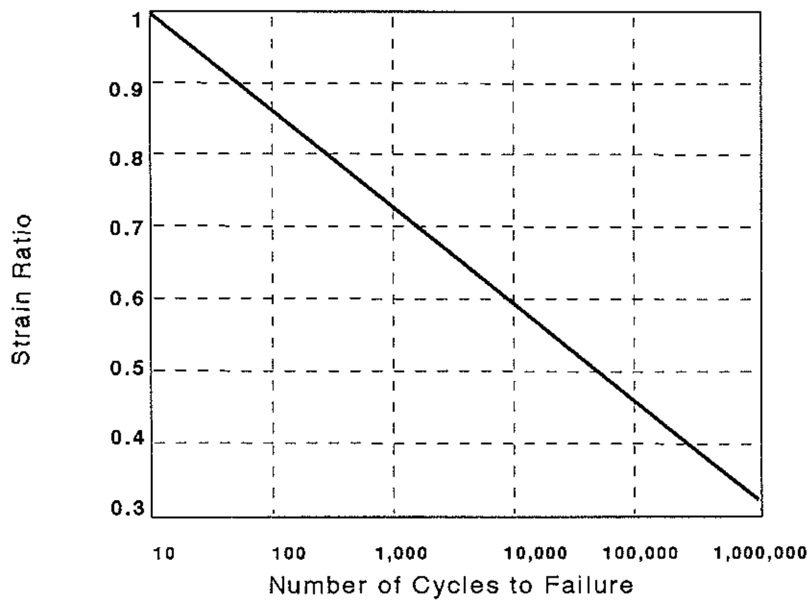


Figure 4.4 Plot of strain ratio versus number of cycles to failure for cement-treated materials (after Otte 1978).

Pretorius (1970) proposed a strain-based fatigue relationship for cemented layers. The relationship uses the absolute value of the tensile strain occurring at the underside of the cemented layer to estimate the number of load repetitions required for the material to sustain fatigue cracking. The Pretorius relationship is as follows:

$$\log N_f = 9.11 - 0.0578\, \epsilon_r \qquad \text{Equation 17}$$

where N_f = number of load repetitions before fatigue failure occurs; and
ϵ_r = initial tensile strain at the underside of the cemented layer.

Raad (1981) discussed a mechanistic design model for cemented base materials, but the concepts are also applicable to cemented subgrade layers. Raad describes the Griffith failure theory, where fracture of a material under a two-dimensional state of stress is initiated by stress concentrations at the tips of minute starter cracks. Repeated cycles of stress induced by wheel loads progressively weaken the cemented layer, and failure occurs when the applied stress is equal to the strength of the layer. M.Thompson (1996, pers.comm.) concurred that the Griffith failure theory is favoured for characterising the performance of cemented layers.

A model for the rate of decrease in tensile strength of a cemented material is presented in Figure 4.5 from Raad (1981). Using this model, the tensile strength decreases from the original value, T_i, to a value of T', at which time failure occurs. If the ratio T'/T_i and the slope of the model (a) are known, then the number of stress applications to failure can be established. Figure 4.6 shows a plot of rate of strength decrease versus stress level for soil–cement specimens subjected to repeated stress applications. Raad suggests the model described is independent of material type, cement content and frequency of applied loading. Therefore it is applicable to cemented materials in general.

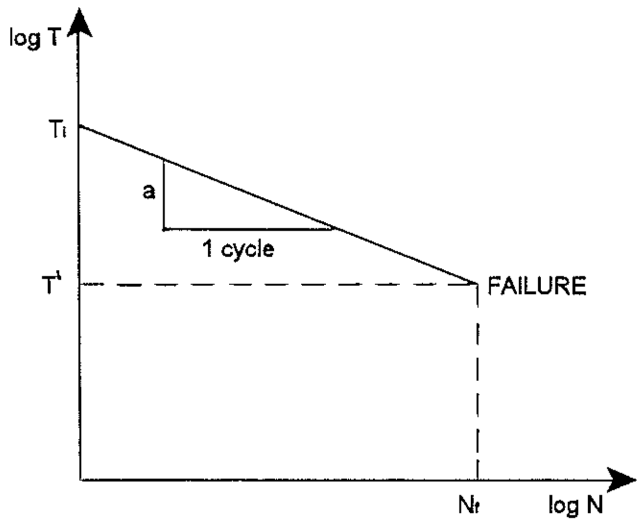
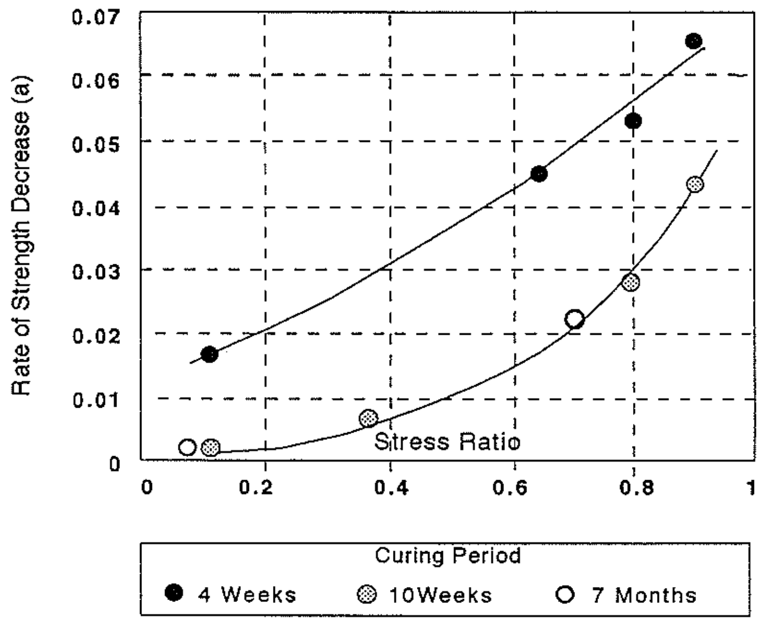


Figure 4.5 Relationship between tensile strength (T) and number of stress applications (N) (after Raad 1981).

Raad (1981) also suggests that the fatigue resistance of a cemented layer is dependent not only on the magnitude of the applied stresses but also on the sequence of application. It is shown that a cemented layer subjected to a series of stress applications of progressively increasing magnitude exhibits a greater resistance to fatigue failure than a layer that is subjected to a series of stress applications of progressively decreasing magnitude. A method of analysing this behaviour using a modified version of Minor's Theorem is presented.

Figure 4.6 Plot of rate of strength decrease versus stress ratio for cement-treated materials (after Raad 1981).



Raad (1985) presents an alternative analysis of pavements incorporating a cemented layer using the finite element method. In the analysis, granular and subgrade materials were assigned stress dependent moduli while the cemented layer was modelled as having a bimodular response, i.e. the material had different elastic moduli in tension and compression. The bimodular behaviour was confirmed by performing a number of flexural tests and split tensile tests.

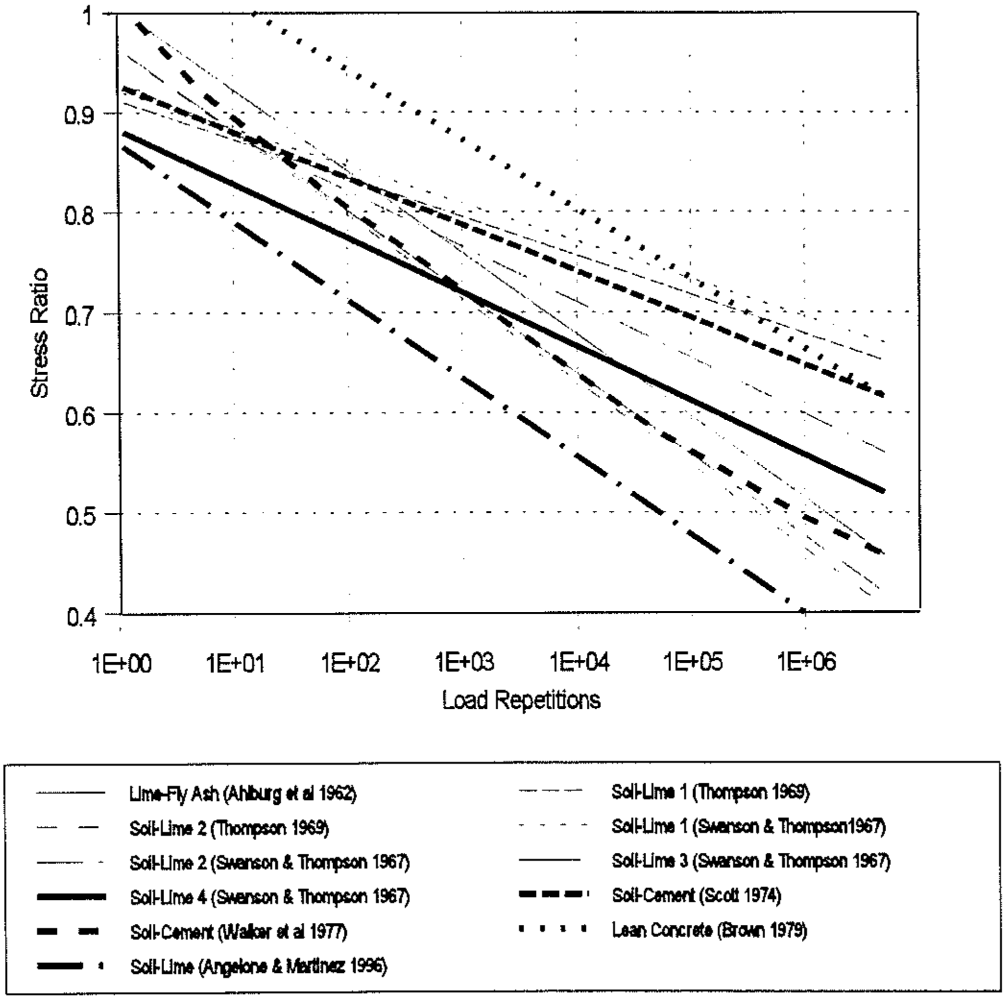
The results of the finite element modelling showed that increasing the ratio of compressive modulus to tensile modulus (E_c/E_t) from 1 to 10 caused the tensile strains at the underside of the cemented layer to increase and the tensile stresses to decrease. The results of the analysis also showed that the stress required to propagate a crack through a cemented layer can be substantially higher than the stress required to initiate a crack.

Raad (1988) discusses the concept of bimodular behaviour in greater detail, stating that the compressive modulus to tensile modulus ratio ranges from 0.5 to 5 for a cement-treated silty clay and from 0.5 to 3 for a cement-treated sand mix. Raad concludes that low values of bimodular ratio result in a relatively high resistance to both crack initiation and crack propagation in a cemented layer.

In a review of the literature on the characterisation of cement-treated pavement materials, George (1990) cites conflicting views about bimodular behaviour. Some researchers maintain that cemented layers have an E_c/E_t ratio greater than one, while others claim a value less than one. To complete the confusion Khanna and Kachroo (1966) conclude that bimodular behaviour is insignificant.

A summary of the stress-based fatigue criteria observed in the literature is presented in Figure 4.7. Although the data presented represent a number of different soil types and lime/cement contents, the fatigue responses appear to be reasonably consistent.

Figure 4.7 Summary of stress-based performance criteria for lime- and cement-treated materials.



5. EFFECT OF CRACKING ON PAVEMENT PERFORMANCE

5.1 General

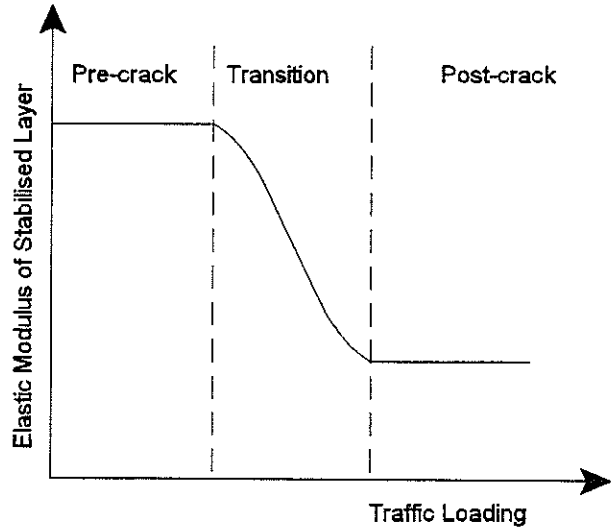
The fact is reasonably well accepted that all materials containing a hydraulic binder are prone to cracking. Cracking may be caused by shrinkage and/or thermal stresses developed as a result of the hydration of the binder. The amount of shrinkage cracking is dependent on factors such as binder content, soil type, water content, degree of compaction, and curing conditions (ACI 1990). Shrinkage cracks typically appear at the top of the stabilised layer and can occur soon after construction (George 1990).

The other source of cracking is traffic-related, where the repeated stressing of a cemented layer results in a fatigue type failure. The fatigue mechanism is initiated as a microscopic flaw which propagates up from the bottom of the cemented layer until a crack is fully developed (George 1990). This form of failure is considered to be applicable only to cemented layers where tension is the critical mode of performance.. Modified materials are not susceptible to fatigue cracking because their critical mode of performance is considered to be compression rather than tension.

5.2 Influence of Cracking in Design

Otte et al. (1982) suggest that traffic-induced cracking is generally the mechanism that will cause a significant drop in the serviceability of a cemented pavement. Hence that event should be resisted as long as possible. The pavement can however possess a significant residual life after cracking has occurred. This is sometimes referred to as a *life after death*. The pavement can be thought of as having three distinct phases, i.e. a pre-cracking phase, a transitional phase, and a post-cracking phase (Figure 5.1).

Figure 5.1 The three phases of the life of a cemented pavement layer.



During the *pre-cracking phase* the cemented layer will have a substantial elastic modulus. It will carry a significant proportion of the applied stresses and offer a high degree of protection to the subgrade.

At some point in time the *transition phase* will be entered, i.e. starter cracks will be initiated in the cemented material and the tensile strength of the layer will progressively decline. These cracks will propagate through the layer until its fatigue life has finally been consumed, i.e. the strength of the layer is equal to the imposed stresses. During the transition phase the elastic modulus of the cemented layer decreases and accordingly it loses its ability to carry the imposed stresses. Hence, the protection afforded to the subgrade is steadily diminished.

At the end of the transition phase the stresses reach an equilibrium state and the elastic modulus stabilises at a constant value. This corresponds to the *post-cracking phase*. In this phase the cemented layer is deemed to have exhausted its tensile capacity and it effectively behaves in a fashion similar to that of an unbound material.

The length of each phase in the life span of a pavement incorporating a cemented layer can be modelled using mechanistic design principles. However, for the purposes of design the first two phases are combined and treated as the pre-cracking phase. In the pre-cracking phase the elastic modulus of the cemented layer is relatively high and the pavement can be analysed to determine the fatigue life of the cemented layer, say N_{pre}. In the post-cracking phase the elastic modulus of the cemented layer is reduced considerably and the pavement is re-analysed to determine the service life of the subgrade, say N_{post}. The total life of the pavement is then determined as being the sum of N_{pre} and N_{post}. This discussion assumes that, in the pre-cracking phase, the service life of the subgrade is greater than the service life of the cemented layer.

Jordaan (1992) suggests that the life span of the pre-cracking phase can be relatively short and for all practical purposes it should be ignored. While this may appear to be somewhat pessimistic it may be a sensible approach considering that the tools available for the rational design of the pre-cracking phase are very imprecise. Also, construction variability has a major influence on the performance of the cemented layer, an aspect that cannot be predicted by the pavement designer.

Otte et al. (1982) reports that the post-cracking phase of pavement life can range from 20% to 80% of the total design life, depending on the material properties and traffic loading conditions. Otte et al. also state that modelling the pavement in the post-cracking phase is difficult because of the bimodular conditions, i.e. the elastic modulus perpendicular to the cracks is different to the modulus parallel to the cracks.

Analyses reported by Otte (1978) showed that the magnitude of the horizontal tensile strains at the underside of a cemented layer can be up to 40% higher in a cracked layer than it would be in an uncracked layer under the same conditions. This gives the designer an approximate method to rationally analyse the post-cracking phase of the cemented layer's service life.

Trying to predict the nature of the crack pattern is a significant difficulty associated with modelling cemented layers. Infrequent cracks are undesirable because they will tend to be wide and therefore unable to transfer shear. The pavement would then comprise large blocks which could rock under moving traffic loads. If water is present it can pump erodible material from the subgrade producing a significant loss of edge support along these cracks. Conversely, if the cracks are closely spaced they will be relatively narrow and able to transfer high shear stresses. Special construction techniques, such as pre-cracking using a vibrating roller, can be adopted to encourage the formation of closely spaced cracks.

6. DISCUSSION

An analysis of the information that is available suggests that subgrade stabilisation should be limited to providing a modified material rather than a cemented layer. No attempt should be made to design a stabilised subgrade layer that develops significant tensile strength to resist the applied loads. This follows the UK approach where a subgrade capping is used simply to provide a platform on which to construct the overlying pavement. This approach is also applied in Australian practice where subgrade stabilisation is generally not considered to contribute to the structural capacity of the pavement.

The factors that support the use of modified subgrade layers are:

- Modification of subgrade soils can provide significant improvement to the physical properties of the material, e.g. reduced plasticity, reduced susceptibility to water ingress, improved workability, etc.

- By definition, a modified material contains a relatively small proportion of binder. This minimises the purchase cost of the additive.

- The adoption of cemented subgrade layers is difficult to justify because of the complexities associated with characterising the material for mechanistic design in the pre-cracking, transition, and post-cracking phases of the pavement life. The simple linear elastic model that is currently used would need to be replaced with more rigorous techniques such as the finite element method.

- Laboratory characterisation of cemented materials does not accurately simulate the in situ conditions because of the complications of thermal cracking, shrinkage cracking, fatigue, construction variability and the change of material parameters with time.

- Modified layers have a low susceptibility to thermal and shrinkage cracking because they contain a relatively low proportion of hydraulic binder. Any cracks that do occur in a modified layer should be relatively narrow and therefore the material should continue to supply adequate support to the rest of the pavement.

- A very stiff cemented layer at the bottom of a pavement is considered to be illogical. The pavement structure is imbalanced with the cemented layer attracting a significant proportion of the applied load onto itself because of its high elastic modulus. But having made this point, a high stiffness layer beneath the sub-base is acknowledged to provide a good platform for the compaction of the overlying unbound aggregate layers.

- If a cemented layer is proposed, the designer must rely on achieving significant tensile strength at the underside of the layer to develop slab action. However, the underside of the layer is susceptible to poor binder mixing, it generally receives a reduced level of compaction, and its inherent variability provides numerous opportunities for cracks to start and propagate.

Many Australian designers consider that the longevity of a stabilised fine-grained subgrade layer cannot be guaranteed as the material can be susceptible to considerable strength loss on wetting. Some New Zealand and American designers may take issue with this point, but at the present time the authors of this report do not have any conclusive evidence either way.

If subgrade stabilisation is to be restricted to modification only, then the suggestion is that a material should be deemed to be modified when its critical mode of performance is compressive. Conversely, a material should be deemed to be cemented when its critical mode of performance is tensile. The modified material can be considered to perform in a ductile fashion whereas the cemented material performs in a brittle fashion.

For the purposes of design, a pavement incorporating a modified subgrade layer can be analysed by subdividing the subgrade into an upper modified subgrade and a lower natural subgrade. This is a valid extension to the mechanistic pavement design procedures adopted by both AUSTROADS and Transit New Zealand. The designer simply checks the magnitude of the vertical compressive strain at the top of both the original subgrade and the modified subgrade layers, and obtains a service life for each using the accepted subgrade performance criterion (Equation 7, in Section 4.2.3 of this report). The overall service life of the pavement will be the lesser of the two values.

The preference for modified subgrade behaviour may preclude the cement stabilisation of sands and non-plastic silts in the subgrade as these materials are generally only conducive to (chemical) stabilisation via a cementing mechanism. Sands and non-plastic silts can however be adequately improved using mechanical stabilisation techniques such as drying and/or compaction.

7. CONCLUSIONS

A review of the literature on the characterisation of stabilised subgrade soils for design reveals that not a lot of information is available. Most of the research to date (1996) has concentrated on stabilised aggregates for use in pavement base layers. Unfortunately, not much work has been carried out on stabilised soils which has been the main focus of this project. While a small number of researchers have reported on the fatigue properties of cemented materials, there appears to be a wide gulf between the research results and the rational application of the research into practical pavement design.

Subgrade stabilisation should be restricted to a modified behaviour only, as suggested by available information. The designer should not introduce a stabilised subgrade layer that attracts significant tensile stresses and consequently resists the applied stresses by the development of slab action. While a number of benefits are associated with the use of modified subgrade materials, the use of cemented subgrade materials has a greater number of associated disadvantages.

The design of a modified subgrade layer can be simply achieved using the existing AUSTROADS philosophy. The modified layer is treated as being a second (or upper) subgrade and the vertical compressive strain at the top of both the original and the modified subgrade layers is examined. The resulting strain values are then used in the existing AUSTROADS subgrade performance criterion to determine the service life of each component.

Further investigation of the boundary between modified and cemented behaviour is recommended. In particular, the analogy of the stress conditions existing in the Brazilian test (Section 3.2 of this report) appears to be worthy of a detailed study. Using this approach, a material could be expected to behave in a modified (ductile) fashion if its compressive strength is less than three times its tensile strength. Conversely a material could be expected to behave in a cemented (brittle) fashion if its compressive strength is more than three times its tensile strength. This may be achieved using both a theoretical approach and laboratory verification.

The issue of longevity of stabilised fine-grained subgrade materials should also be examined. The Australian experience suggests these materials are susceptible to significant strength loss if they are subject to water ingress during their service life. This is not necessarily consistent with New Zealand and American experience which suggests that the long-term performance of stabilised subgrade soils is generally very good.

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