MECHANISTIC DESIGN OF PAVEMENTS INCORPORATING A STABILISED SUBGRADE

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BARTLEY CONSULTANTS LIMITED
Takapuna, Auckland, New Zealand

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PO Box 2331, Lambton Quay, Wellington, New Zealand
Telephone (04) 473-0220; Facsimile (04) 499-0733

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AN IMPORTANT NOTE FOR THE READER

The research detailed in this report was commissioned by Transfund New Zealand.

Transfund New Zealand is a Crown entity established under the Transit New Zealand Act 1989. Its principal objective is to allocate resources to achieve a safe and efficient roading system. Each year, Transfund New Zealand invests a portion of its funds on research that contributes to this objective.

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Introduction

Subgrade stabilisation is a process used in road construction where a hydraulic binder such as lime or cement is added to a soil to obtain enhanced soil strength and stiffness properties. The stabilised layer provides good support for construction traffic and facilitates efficient compaction of the overlying pavement layers. It is a technique that is commonly used for roads in New Zealand.

In July 1995, Transit New Zealand adopted the pavement design procedures of AUSTROADS (Association of State Road Authorities of Australia and New Zealand). The AUSTROADS pavement design procedures are based on a mechanistic approach. This allows the designer great flexibility in deciding the materials to be used in the pavement structure. Although the AUSTROADS procedures provide means for designing cemented pavement layers, they do not allow for stabilised subgrade layers. This is because the Australian roading fraternity do not recognise the longevity of stabilised subgrade materials.

Objectives

The objectives of this project, which was carried out in 1998, are to:

- Verify (or reject) the issue of longevity of stabilised subgrade materials;
- Carry out a field investigation and a suite of laboratory tests to characterise and establish correlations between strength and stiffness parameters for lime-treated and untreated soils;
- Combine the results of the field and laboratory investigations to develop a strategy for designing pavements with a stabilised subgrade layer, for use in New Zealand.

Field Investigation

Three pavements known to include a lime-stabilised subgrade were identified in North Shore City and Rodney District, Auckland, New Zealand. The age of the test pavements ranged from eight years to approximately twenty years. They were subjected to test pitting to identify the pavement structures and to facilitate the performance of field tests on the original and the stabilised subgrade layers. The field tests showed that the enhanced strength and stiffness properties obtained by subgrade stabilisation can be relied upon for periods of twenty years or more.

Analysis

The pavement structures identified in the test pits were analysed using the AUSTROADS pavement design criterion. The analyses showed that the remaining life of the pavements was somewhat higher than expected. This suggests that either the original design parameters were somewhat conservative, or the pavements have not been subjected to the levels of loading that were envisaged at the time of their design.

Mechanical Properties of Stabilised Soils

The input to most mechanistic pavement design procedures includes the elastic moduli of the various pavement layers. A suite of laboratory tests were conducted to investigate possible correlations between the elastic modulus and other, more basic, material parameters such as bearing strength (CBR), compressive strength and tensile strength. Reasonable correlations were found to exist between each of the parameters. This means that relatively quick and inexpensive test methods can be used to obtain the elastic modulus provided the various correlations have been established in advance.

Development of Stabilised Subgrade Design Strategy

Components of the field and laboratory investigations have been considered in the development of a strategy for designing pavements including a stabilised subgrade. The procedure basically comprises the pavement being treated as if it has two subgrade layers, i.e. the original and the stabilised subgrades.

In a computer model of the pavement, the stabilised layer is divided into sublayers according to the AUSTROADS sublayering scheme. The elastic modulus of the top sublayer should be no more than three or four times the elastic modulus of the original subgrade, depending on the reactivity of the soil (i.e. response of the soil to the addition of the binder). Both the stabilised and the original subgrade layers should be treated as having anisotropic elastic parameters.

The model is then subjected to a standard loading configuration, and the vertical strains occurring at the top of both the stabilised and the original subgrade layers are used in the AUSTROADS subgrade performance criterion. The strains at the top of the intermediate sublayer(s) in the stabilised layer should also be checked but it is expected that these intermediate locations will not be critical.

The proposed design procedure has been trialed using the original subgrade, and the traffic design data applied for the three test pavements used for the field investigations. The results of the analyses show that the proposed procedure gives pavement performance predictions that are consistent with the observed pavement performance. Although this limited analysis does not represent a rigorous verification it does provide a reasonable level of confidence in the use of the design procedure.

ABSTRACT

Subgrade stabilisation is a common road construction practice in New Zealand. Stabilisation results in increased subgrade strength and stiffness, and provides a stable platform for the construction of the overlying pavement. It also provides protection of the original subgrade from stresses generated by wheel loads passing over the pavement surface.

This project carried out in 1998 has examined the performance of lime-stabilised subgrade materials using both laboratory and field investigations for three test pavements. The field investigations have shown that the enhanced strength and stiffness characteristics of stabilised subgrade materials can be relied upon for periods of twenty years or more.

Laboratory investigations have been used to examine correlations between the elastic modulus of lime-treated and untreated soils and other basic test methods. Good correlations have been established between the CBR and both unconfined compressive strength and split tensile strength, for the silty clay soil used in the investigation.

The field and laboratory data have been used to develop a design procedure for stabilised subgrade layers using a mechanistic approach. The proposed procedure provides performance predictions that are reasonably consistent with the observed pavement performance.

1. INTRODUCTION

1.1 General

In 1995, Transit New Zealand adopted the AUSTROADS pavement design procedures for use in New Zealand. These procedures are described in the document *Pavement Design: A Guide to the Structural Design of Road Pavements* (AUSTROADS 1992). The AUSTROADS procedures replaced the *State Highway Pavement Design and Rehabilitation Manual* (Transit New Zealand 1989). Transit New Zealand subsequently published a New Zealand supplement to the AUSTROADS document (Transit New Zealand 1997) to allow for aspects of pavement design and construction that are unique to New Zealand. Similar supplements are used by most of the state roading authorities in Australia.

One of the strengths of the AUSTROADS pavement design procedure is its flexibility. Designers are not constrained to a single set of material parameters, and therefore a range of materials can be considered and the sensitivity of the parameters can be examined. This gives the designer more scope for producing innovative and cost-effective solutions for pavement projects, such as subgrade stabilisation.

1.2 Subgrade Stabilisation

A cost-effective method of improving pavement subgrades is to introduce a small amount of hydraulic binder, such as lime, cement or KOBM, into the upper portion of the subgrade. This practice, generally known as *subgrade stabilisation*, is commonly used in New Zealand.

The engineering properties of the stabilised layer are dependent on a number of factors, e.g:

- mineralogy of the original soil;
- particle size distribution of the original soil;
- type of stabilising agent used;
- proportion of stabilising agent; and
- curing time and conditions.

Stabilisation can take two forms, *modification* and *cementation*. A *modified* soil is one that has had its engineering properties enhanced by the addition of a hydraulic binder, generally lime. The lime dries the soil slightly and causes the soil's plastic limit to increase and the liquid limit to decrease, thus reducing the plasticity index. The soil becomes more workable and both its shear strength and stiffness increase.

If lime is added in excess of the amount required for modification, or if Portland cement is used as the stabilising agent, then the soil can be termed *cemented*. The binder forms rigid crystalline structures that produce considerable increases in both tensile strength and stiffness. A cemented layer relies on its tensile strength to produce slab action to resist the applied wheel loads. Cemented layers tend to attract a disproportionate amount of the applied loads because their elastic modulus is generally much higher than those of the other pavement layers. Although cemented layers are not susceptible to rutting, they do have a relatively low resistance to fatigue failure.

Cemented materials are prone to cracking from tensile stresses caused by shrinkage as the hydraulic binder hydrates. The amount of shrinkage is dependent on the particle size distribution of the original material and the amount of binder used. Shrinkage cracks are generally widely spaced resulting in relatively large crack apertures. This causes the layer to act as a number of discrete blocks, thus making the post-cracking pavement performance difficult to characterise. Conversely, traffic-induced fatigue cracks are closely spaced and generally result in small crack apertures. This makes it possible to achieve effective transfer of shear stresses from one side of the crack to the other. It is generally accepted that a cracked cemented layer with small crack apertures reverts to acting as if it was an unbound material.

The AUSTROADS pavement design procedures include performance criteria for three levels of stabilised materials, viz. cemented layers with elastic moduli of 2,000 MPa, 5,000 MPa or 10,000 MPa. These elastic moduli are generally only appropriate for cement-treated gravels constructed as sub-base layers in so-called 'upside-down' pavements. No performance criteria are provided for typical stabilised subgrade materials because of the concern that the stabilised layer may lose its enhanced properties and revert to its original state if water enters the soil structure.

A review of the international technical literature on the topic of designing stabilised pavement layers has been carried out by the authors (Bartley Consultants Ltd 1998). The report of that project concludes that there is not a lot of information available and that there is no recognised uniform approach for the design of stabilised layers. It also concludes that subgrade stabilisation should be restricted to soil-modification because there is little to gain from achieving a cemented material. Furthermore, the uncertainty associated with predicting if or when a cemented layer may crack, and the resulting properties that the layer may adopt, makes the rational analysis of the design somewhat difficult to justify.

1.3 Objectives

1. Verify (or reject) the issue of longevity of stabilised subgrade materials.
2. Carry out a field investigation and a suite of laboratory tests to characterise and establish correlations between strength and stiffness parameters for lime-treated and untreated soils.
3. Combine the results of the field and laboratory investigations to develop a strategy for designing pavements with a stabilised subgrade layer, for use in New Zealand.

It is fair to say that most New Zealand practitioners would not share the view that their Australian counterparts have of the longevity of stabilised subgrade materials, and would argue that subgrade stabilisation can be relied upon in the long term. This project, carried out in 1998, examines the long-term performance of three pavements incorporating stabilised subgrades, in order to verify (or reject) this opinion.

A field investigation of pavements with lime-stabilised subgrades was made, and a suite of laboratory tests was then carried out using lime-stabilised soils to investigate suitable ways of characterising stabilised soils for pavement design and construction.

The results of the field and laboratory investigations were used to make a recommendation as to the most appropriate way of characterising and designing pavements incorporating a stabilised subgrade layer.

2. FIELD INVESTIGATION

2.1 General

A field investigation was carried out to examine the long-term efficacy of subgrade stabilisation. The investigation included the excavation of test pits in three pavements that are known to have stabilised subgrades. The pavements were selected from roads in the Auckland area and were:

- Paremoremo Road (Albany - North Shore City);
- East Coast Road (Browns Bay - North Shore City); and
- Duck Creek Road (Stillwater - Rodney District).

Three test pits were excavated in Paremoremo Road and two test pits were excavated in both East Coast Road and Duck Creek Road. The pits were approximately 500 mm square at the top and they extended through the pavement structure into the original subgrade. The seal was removed using a concrete cutting saw and a Kango hammer. The aggregate materials and stabilised layers were loosened and broken up using a vibrating pick. The material was then removed with a hand shovel.

The test pits were logged and the following field test procedures were carried out:

- Loadman portable falling weight deflectometer;
- hand-held field shear vane; and
- Scala penetrometer.

The pits were reinstated by backfilling with high quality basecourse material that was compacted using a vibrating hammer. The surface was sealed using a permanent cold patching asphalt mix.

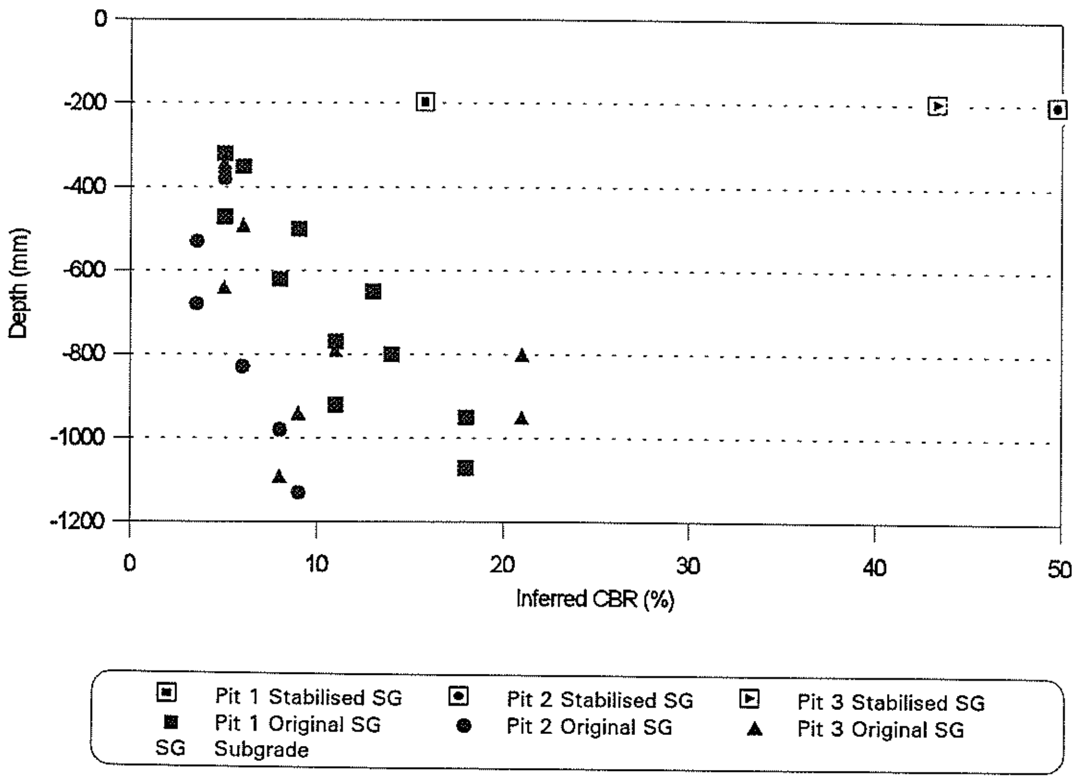
The test pits were excavated by technicians from Manukau Consultants Ltd (MCL). The MCL staff also carried out the Scala penetrometer and shear vane tests. An engineer from Bartley Consultants Ltd (BCL) oversaw the test pit excavation and performed the Loadman portable falling weight deflectometer (FWD) tests.

The Loadman portable FWD tests were carried out on the top of the basecourse layer and on the top of both the original and stabilised subgrade layers. Each Loadman test comprised up to eight test repetitions and the average of the three highest resilient modulus results was recorded. The results were recorded as effective single layer elastic moduli assuming isotropic conditions.

The Scala penetrometer tests were carried out at both the top of the stabilised subgrade layer and the top of the original subgrade.

The shear vane tests were carried out in both the stabilised and the original subgrade soils. Remoulded shear strengths were also recorded whenever possible. A characteristic shear strength for each layer was obtained by disregarding the minimum and maximum readings from five tests, and calculating the mean of the remaining three test results.

Figure 2.1 Scala penetrometer results for Paremoremo Road test pits.



2.2 Paremoremo Road Site

The test pits on Paremoremo Road, Albany, were located as follows:
Pit 1 : 85.5 m to C/L[1] Cutts Rd, offset 2.5 m from C/L of Paremoremo Rd
Pit 2 : 90 m to C/L Sanders Rd, offset 2.5 m from C/L of Paremoremo Rd
Pit 3 : 140 m to C/L Sanders Rd, offset 1.3 m from C/L of Paremoremo Rd
and the test pit logs, Loadman test results and shear vane test results are summarised in Table 2.1. The Scala blow counts converted to inferred CBR values are shown in Figure 2.1.

Table 2.1 Summary of test pit data for Paremoremo Road.

Depth (mm)	Description	Elastic Modulus (MPa) (1)	Undrained Shear Strength (kPa)		Mean CBR (%) (2)
			Undisturbed	Remoulded	
Pit 1					
0 - 20	Chipseal	n/a	n/a	n/a	n/a
20 - 200	Unbound basecourse, moderately compacted	160	n/a	n/a	n/a
200 - 320	Stabilised subgrade	90	> 179(3)	-	16
> 320	Original subgrade	30	> 179	-	11
Pit 2					
0 - 25	Chipseal	n/a	n/a	n/a	n/a
25 - 200	Unbound basecourse, moderately compacted	180	n/a	n/a	n/a
200 - 380	Stabilised subgrade	110	> 179	-	50+
> 380	Original subgrade	25	122	62	7
Pit 3					
0 - 30	Asphalt surface	n/a	n/a	n/a	n/a
0 - 200	Unbound basecourse, moderately compacted	130	n/a	n/a	n/a
200 - 340	Stabilised subgrade	150	> 179	-	44
> 340	Original subgrade	30	111	51	10

Note (1) : Elastic modulus obtained from Loadman portable FWD.
(2) : Mean CBR inferred from Scala penetrometer test results.
(3) : 179 kPa is the maximum shear strength measurable with the shear vane.

[1]C/L Centreline

2.3 East Coast Road Site

The test pits on East Coast Road were located as follows:
Pit 1 : 5 m to C/L of driveway at No. 739, 1.1 m offset from kerb
Pit 2 : 6.5 m to C/L of driveway at No. 886, 2 m offset from kerb.
Both test pits were excavated in bus bays because of the high traffic volume using the main East Coast Road carriageway.

The test pit logs, Loadman test results and shear vane test results are summarised in Table 2.2. The Scala penetration test on the original subgrade in Test Pit 1 was started at a depth of 450 mm. In Test Pit 2 the stabilised layer was too hard to be penetrated using the Scala apparatus. An inferred CBR value of 50% has been assigned to this layer. The Scala blow counts converted to inferred CBR values are shown in Figure 2.2.

Table 2.2 Summary of test pit data for East Coast Road.

Depth (mm)	Description	Elastic Modulus (MPa) (1)	Undrained Shear Strength (kPa)		Mean CBR (%) (2)
			Undisturbed	Remoulded	
Pit 1					
0 - 25	Chipseal	n/a	n/a	n/a	n/a
25 - 170	Unbound basecourse, moderately compacted	170	n/a	n/a	n/a
170 - 390	Stabilised subgrade	150	> 179 (3)	-	44
390 - 450	Original subgrade 1	55	-	-	4
> 450	Original subgrade 2	30	147	68	11
Pit 2					
0 - 25	Chipseal	n/a	n/a	n/a	n/a
25 - 230	Unbound basecourse, moderately compacted	200	n/a	n/a	n/a
230 - 450	Stabilised subgrade	300	> 179	-	50
> 450	Original subgrade	30	89	39	8

Note (1) : Elastic modulus obtained from Loadman portable FWD.
(2) : Mean CBR inferred from Scala penetrometer test results.
(3) : 179 kPa is the maximum shear strength measurable with the shear vane.

2.4 Duck Creek Road Site

The test pits on Duck Creek Road were located as follows:
Pit 1 : 250 m to C/L of Spur Rd, eastbound lane 1.9 m offset from C/L
Pit 2 : 102 m to C/L of Pinestead Ranch, eastbound lane 3 m offset from C/L.
The test pit logs, Loadman test results and shear vane test results are summarised in Table 2.3. One Scala penetrometer test was carried out in Test Pit 1, two in Test Pit 2, and the blow counts converted to inferred CBR values are shown in Figure 2.3.

Table 2.3 Summary of test pit data for Duck Creek Road.

Depth (mm)	Description	Elastic Modulus (MPa) (1)	Undrained Shear Strength (kPa)		Mean CBR (%)(2)
			Undisturbed	Remoulded	
Pit 1					
0 - 10	Chipseal	n/a	n/a	n/a	n/a
10 - 250	Stabilised base, well compacted	400	n/a	n/a	n/a
250 - 390	Unbound sub-base, loose	100	n/a	n/a	n/a
390 - 560	Stabilised subgrade	50	> 179 (3)	-	9
> 560	Original subgrade	20	85	15	4
Pit 2					
0 - 15	Chipseal	n/a	n/a	n/a	n/a
15 - 250	Stabilised base, well compacted	405	n/a	n/a	n/a
250 - 550	Stabilised subgrade	300	> 179	-	23
> 550	Original subgrade	30	91	45	13

Note (1) : Elastic modulus obtained from Loadman portable FWD.
(2) : Mean CBR inferred from Scala penetrometer test results.
(3) : 179 kPa is the maximum shear strength measurable with the shear vane.

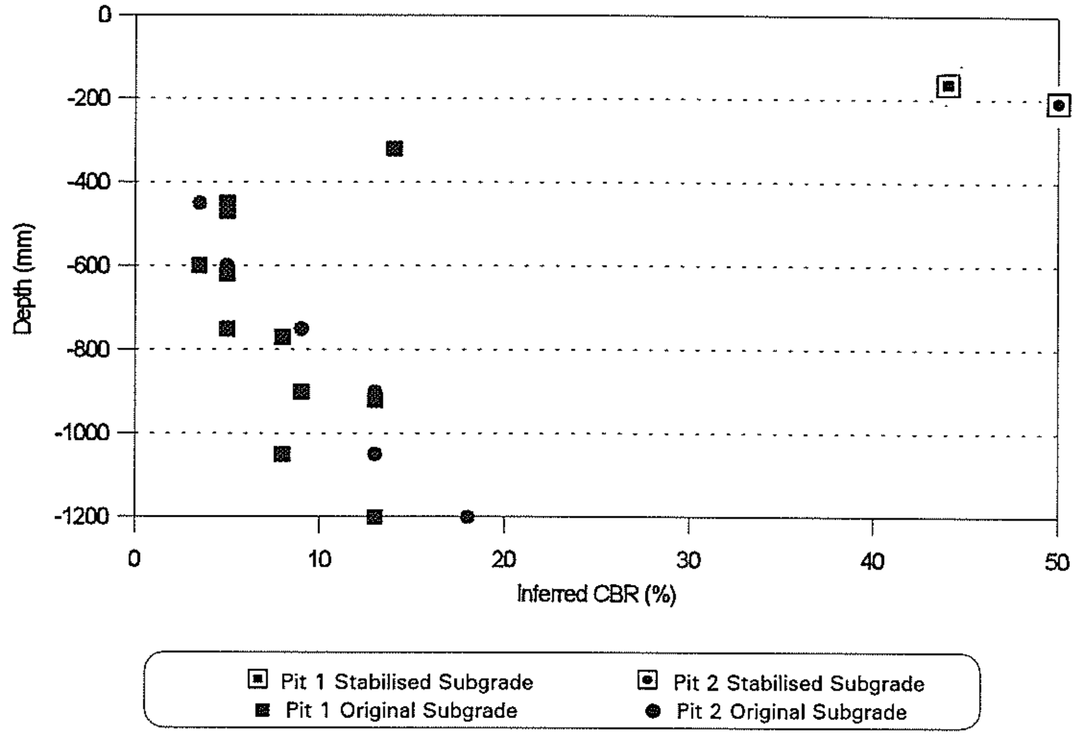


Figure 2.2 Scala penetrometer results for East Coast Road test pits.

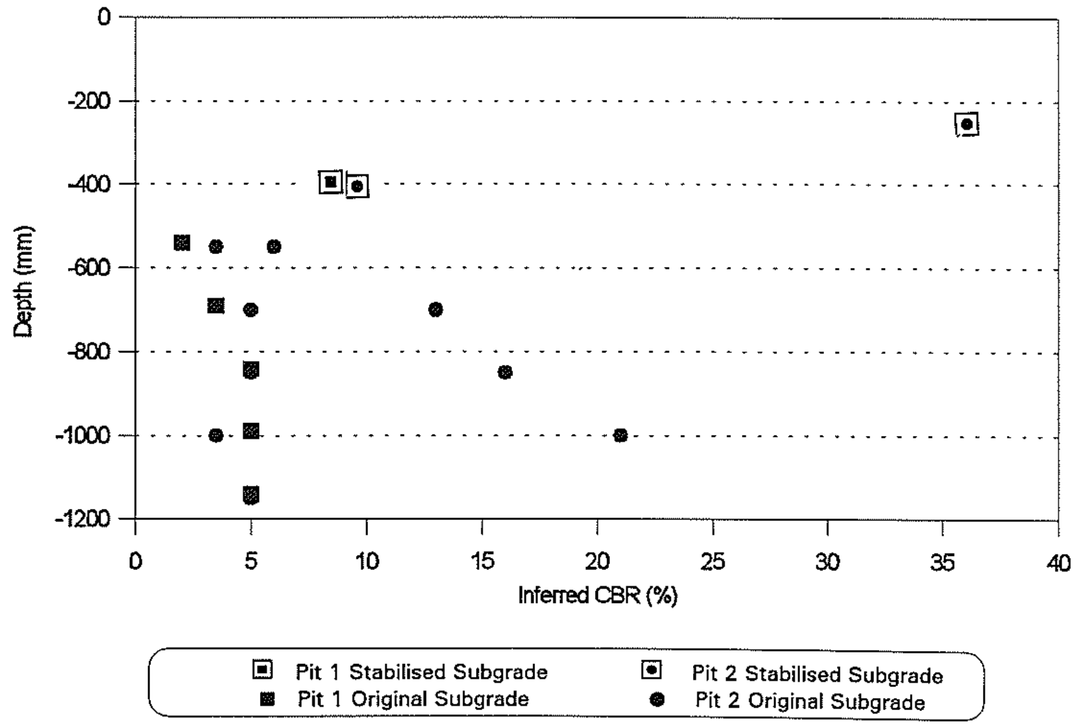


Figure 2.3 Scala penetrometer results for Duck Creek Road test pits.

3. SUBGRADE ANALYSIS

3.1 Paremoremo Road Site

Paremoremo Road was constructed in 1977 to provide improved access to Paremoremo Prison and the surrounding community. The scope of the project provided an opportunity to monitor the performance of one of the first lime-stabilised subgrades to be constructed in New Zealand. The design parameters were as follows (Malcolm 1983):

- Design traffic loading : 2.6 x 10^4 EDA (20 years)
- In situ subgrade CBR : 6 - 9%
- Laboratory subgrade CBR (sample prepared at owc[2], then soaked): 12%
- Subgrade + 4% lime CBR (unsoaked) : 40%+
- Subgrade + 4% lime CBR (soaked) : 29%
- Subgrade plasticity index : 43%
- Depth of stabilisation : 250 mm
- Depth of (M/4) basecourse : 125 mm

The 125 mm-basecourse thickness was obtained from the NRB S/4 design charts (NRB 1974) using a uniform stabilised subgrade CBR of 25%.

The stabilised subgrade was treated with a cationic emulsion cure coat prior to placing the basecourse. The pavement was surfaced with a two-coat seal using Grades 3 and 2 chip.

The performance of the pavement was closely monitored after construction. The in situ CBR of the stabilised subgrade was found to be in excess of 40% within days. After a period of 18 months the CBR had increased to 165% and the plasticity index was 0. Benkelman Beam tests showed a decrease in deflection (95 percentile) from 2.14 mm immediately before sealing to 0.85 mm 14 months after construction (Malcolm 1983).

The road has performed very well and shows no sign of significant distress some 21 years after construction.

Comparing the design parameters with the data obtained from the three test pits raises a number of issues. For example, although the pavement layer thicknesses observed in the test pits are quite uniform, they are somewhat different to the thicknesses proposed in the design. The design thickness of the stabilised subgrade layer was reported to be 250 mm, whereas the test pits indicated stabilised subgrade thicknesses of 120 mm, 180 mm and 140 mm. Similarly, the design thickness of the basecourse layer was reported to be 125 mm, whereas the test pits all indicated a basecourse layer thickness of 200 mm. The significant reduction in the observed thickness of the

[2] owc optimum water content

stabilised subgrade may be attributable to poor distribution of the lime during construction and/or a lack of compaction in the lower portion of the layer.

Apart from the possibility of an unreported alteration to the basecourse thickness, or a lack of construction control, it is unclear why the observed basecourse thickness is generally greater than the design thickness. It is unlikely that the original pavement has been overlaid because the original test pit locations are visible at the surface of the pavement.

The data presented in Table 2.1 and Figure 2.1 show that the Loadman elastic modulus values measured on the top of the stabilised subgrade layer do not correlate very well with the inferred CBR data from the Scala penetrometer tests. However, the inferred CBR values correlate reasonably well with the CBR values reported by Malcolm (1983). It is evident from the Loadman, Scala penetrometer and shear vane tests that the strength and modulus properties of the stabilised subgrade are significantly superior to those of the original subgrade and that this benefit has persisted for a period greater than 20 years.

The ratios of the (Loadman) elastic moduli for the stabilised and for the original subgrade layers in the three test pits, i.e. $E_{stabilised}/ E_{original}$, are 3.0, 4.4 and 5.0 respectively.

3.2 East Coast Road Site

East Coast Road starts at Takapuna and heads north to insect State Highway (SH) 1 just south of Silverdale. It is a busy arterial road carrying traffic to and from the many suburbs that constitute the East Coast Bays region of North Shore City. East Coast Road also shares, with SH1, the bulk of the traffic that heads north from the Auckland region before the two roads merge around Silverdale.

In the early 1980s the existing East Coast Road pavement was showing signs of severe distress from subgrade deformation and edge damage. This was primarily caused by ingress of water and subsequent softening of the subgrade. The geometrics of the existing pavement also required improvement. Therefore, a pavement reconstruction project was initiated which featured the provision of a stabilised subgrade layer.

The project was carried out in three sections. The second section of the project was located slightly to the south of the area of the test pits examined in the current project. The design parameters for that section were as follows (Malcolm 1983):

- Design traffic loading : 1.1×10^6 EDA
- In situ subgrade CBR : 4 - 12%
- Soaked subgrade CBR (owc) : 3.5%
- Subgrade + 3.5% lime CBR (soaked) : 40%+
- Depth of stabilisation : 250 mm
- Depth of (M/4) basecourse : 200 mm

The 200 mm-basecourse thickness was obtained from the NRB S/4 design charts using a uniform stabilised subgrade CBR of 30%.

The pavement was monitored closely in the months following construction. The in situ CBR of the stabilised subgrade layer ranged from 26 to 66% after one month, and 34 to 115% after nine months. The 95 percentile Benkelman Beam deflection decreased from 2.61 mm on the top of the basecourse at the time of construction to 0.96 mm nine months after construction (Malcolm 1983).

While there is some surface deformation evident, the pavement has performed well considering the volume of traffic that it has carried.

The test pits excavated in East Coast Road for the current project indicate basecourse thicknesses of 170 mm and 230 mm, i.e. close to the design value. The stabilised subgrade layer thicknesses found in the test pits were both 220 mm, again quite close to the design value.

The Loadman elastic modulus values for the stabilised subgrade layers, i.e. 150 MPa and 300 MPa (see Table 2.2) represent a high modulus material. This is reflected in the shear vane test results and the inferred CBR values obtained from the Scala penetrometer (see Figure 2.2). The inferred CBR values for both the stabilised and original subgrade layers correlate well with the design data. The ratios of the (Loadman) elastic moduli for the stabilised and original subgrade layers are approximately 5 and 10 for the two test pits examined.

It is clear that the stabilised subgrade layer has demonstrated superior stiffness and strength properties relative to the original subgrade and that the efficacy of the stabilisation has persisted for a period of at least 15 years.

3.3 Duck Creek Road Site

Duck Creek Road runs from Spur Road to Stillwater township in Rodney District. It provides the only road access to Stillwater. No design data are available for Duck Creek Road except that the 1990 AADT (two-way) was approximately 900 vpd. This would make the design life of the pavement approximately 1.6 x 10^5 EDA for a 20 year-design period with an estimated heavy commercial vehicle component of 5%. It is understood that the subgrade that was investigated in this project was treated with approximately 4% lime and the pavement is about eight years old. The subgrade along the alignment of Duck Creek Road alternates between Waitemata Group sediments and the somewhat troublesome Onerahi Chaos material.

According to local practitioners, several sections of Duck Creek Road have been stabilised over the years with mixed results. This is also reflected in the results of the two test pits (see Table 2.3). Test Pit 1 shows a Loadman elastic modulus for the stabilised subgrade of 50 MPa while Test Pit 2 shows a value of 300 MPa.

Part of this difference could be related to the influence that the thicker stabilised layer observed in Test Pit 2 has on the Loadman apparatus.

In both test pits the stabilised layer was too hard to use the hand-shear vane to measure the undrained shear strength. This, along with the difference between the elastic moduli observed in the original and stabilised layers, suggests that the stabilisation process has been effective and that it has persisted for approximately eight years. The ratios of the (Loadman) elastic moduli for the stabilised and original subgrade layers are approximately 2.5 and 10 for the two test pits examined.

4. PAVEMENT MODELLING

4.1 General

The multi-layer elastic computer program *CIRCLY* (Wardle 1996) has been used to model generalised pavement structures corresponding to the three pavements included in the field investigation. *CIRCLY* requires the thickness, elastic modulus and Poisson's ratio of the various pavement layers to be provided as input. The program calculates the resulting stresses, strains and displacements under standard loading conditions. The critical strains have been used with the AUSTROADS subgrade performance criterion to determine the remaining life of both the stabilised and original subgrade layers.

One of the key aspects of pavement modelling is determining appropriate elastic moduli for the pavement layers. This is complicated by the non-linear stress/strain response of most pavement materials. In the following models an assessment of appropriate elastic moduli has been made using the Loadman test results presented in Tables 2.1, 2.2 and 2.3 as a starting point. The Loadman data have been adjusted to reflect the fact that the device generally produces elevated stress conditions compared to a standard wheel load.

In all models the Equivalent Design Axle (EDA), as defined in the (now superseded) Transit New Zealand (1989) *State Highway Pavement Design and Rehabilitation Manual* has been used. This is because the pavements were originally designed using the EDA as the standard traffic loading parameter.

For each test pavement modelled in this study, the actual traffic loading and pavement performance data are unknown. Its authors have used estimated traffic loadings based on the original design data or, in the case of Duck Creek Road, a single traffic count value. The pavement performance has been characterised by visual inspection. Therefore, the models described in this study are subject to inaccuracies associated with the assumptions adopted.

Note that the models all use anisotropic elastic modulus parameters as according to the AUSTROADS pavement design procedures. The elastic moduli presented in Tables 4.1 to 4.4 all correspond to the vertical direction.

4.2 Paremoremo Road Site

The generalised pavement model assessed for Paremoremo Road is shown in Table 4.1, and has been analysed using *CIRCLY*. The analysis assumes that the stabilised subgrade acts as a second subgrade layer.

Table 4.1 Generalised pavement model for analysis of Paremoremo Road site.

Layer	Material	Thickness (mm)	Elastic Modulus (MPa)	Sublayered?
1	Unbound aggregate	200	150	✔
2	Stabilised subgrade	120 - 180	120	✔
3	Original subgrade	n/a	50	✖

As the test pits were excavated in a wheel track and the pavement is approximately 20 years old, the pavement would be expected to have consumed its service life and would have little or no remaining life. However, the analysis shows that this is not the case. Using the layer details shown in Table 4.1, the Paremoremo Road pavement model has a remaining life in the order of 5.0×10^4 EDA to 1.7×10^5 EDA.The comparatively long remaining life is verified to some extent by the appearance of the pavement, as it shows little or no significant distress.

4.3 East Coast Road Site

The generalised pavement model assessed for East Coast Road is shown in Table 4.2, and has been analysed using *CIRCLY*. The analysis assumes that the stabilised subgrade acts as a second subgrade layer.

Table 4.2 Generalised pavement model for analysis of East Coast Road site.

Layer	Material	Thickness (mm)	Elastic Modulus (MPa)	Sublayered?
1	Unbound aggregate	170 - 230	170	✔
2	Stabilised subgrade	220	150 - 300	✔
3	Original subgrade	n/a	50	✖

The East Coast Road pavement is approximately 15 years old. Therefore, it could be assumed that most of the pavement's service life has been consumed. However, the test pits were excavated in bus bays because accessing the main carriageway was difficult. Therefore, the pavement structure observed in the study would not have received the same level of loading as the design traffic lane.

The *CIRCLY* analysis indicates that the remaining life of the pavement structure ranges from approximately 2.5 x 10^5 EDA to 2.0 x 10^6 EDA. This range of remaining life values appears to be of the correct order considering the age of the pavement and the reduced loading associated with the bus bay location. The remaining life prediction also appears to be consistent with the appearance of the pavement, which shows little or no significant distress.

4.4 Duck Creek Road Site

A generalisation is not feasible for the pavement for Duck Creek Road because the structures observed in the two test pits were quite different. Therefore, *CIRCLY* analyses have been carried out for each structure. The *CIRCLY* model used to represent the Duck Creek Road pavement at Test Pit 1 is presented in Table 4.3, and that at Test Pit 2 is presented in Table 4.4.

Table 4.3 Pavement model for analysis of Duck Creek Road Test Pit 1.

Layer	Material	Thickness (mm)	Elastic Modulus (MPa)	Sublayered?
1	Stabilised base	250	350	✔
2	Unbound sub-base	140	80	✖
3	Stabilised subgrade	170	70	✔
4	Original subgrade	n/a	40	✖

Table 4.4 Pavement model for analysis of Duck Creek Road Test Pit 2.

Layer	Material	Thickness (mm)	Elastic Modulus (MPa)	Sublayered?
1	Unbound aggregate	250	350	✔
2	Stabilised subgrade	200	250	✔
3	Original subgrade	n/a	40	✖

The generalised pavement models for Duck Creek Road have been analysed using *CIRCLY*. The analysis assumes that the stabilised subgrade acts as a second subgrade layer.

The Duck Creek Road pavement is approximately eight years old and it had a design life of approximately 1.6 x 10^5 EDA. Therefore, it could be assumed that the pavement has most of its service life still ahead of it.

The *CIRCLY* analyses indicate that the remaining life of the pavement structure ranges from approximately 1.8 x 10^6 EDA to 3.0 x 10^6 EDA. This range of remaining life values is higher than expected and actually exceeds the original design life of the pavement.

4.5 Discussion

The analyses of the three generalised pavement structures described in Sections 4.2 to 4.4 give an indication of the performance of the pavements with respect to their original design lives. The pavements were originally designed using the earlier Transit New Zealand design procedures but the remaining life has been estimated using the approach of the AUSTROADS pavement design procedures which are currently (1998) used in New Zealand. Although this mixing of design procedures is somewhat inconsistent, it is considered to be reasonable for the purposes of this study.

In each case the pavements appear to have a higher than expected remaining life. This may be due to the fact that typically, in design, lower 10 percentile material parameters are adopted to provide a high level of reliability. While the extent of the data used in this analysis has been limited, it generally corresponds to the mean conditions.

Other factors include:

- the pavements may not have received the level of loading that was foreseen at the time of design;
- the material parameters used in the original design were somewhat conservative; and
- the performance criteria used in the original design were somewhat conservative.

5. MECHANICAL PROPERTIES OF STABILISED SOIL

5.1 General

In mechanistic pavement design, such as the AUSTROADS procedures, each pavement layer must be described by three parameters for use in a computer model. They are layer thickness, elastic modulus and Poisson's ratio. Layer thickness is a variable that the designer controls while Poisson's ratio is generally estimated. The elastic modulus can also be estimated but to achieve an accurate design it should be established by field or laboratory testing.

Repeated load triaxial (RLT) testing is recognised as one of the best methods for determining elastic parameters, However it is expensive and time consuming. Thus establishing a correlation between the elastic modulus and other, more basic, mechanical properties would be desirable to provide data for design purposes and for quality control of construction.

The suite of tests that were carried out were as follows:

- Atterberg Limits
- Compaction;
- CBR;
- Unconfined compression;
- Spilt tension (Brazilian test); and
- Repeated load triaxial tests.

The tests were carried out using a sample of Waitemata Group sediment, i.e. a clayey silt or silty clay of moderate plasticity. The stabilising agent used was hydrated lime.

The strength and stiffness of lime-stabilised soil generally increases with time. However, the rate of increase diminishes until effectively an equilibrium state is reached. For this reason the lime-treated specimens were allowed to cure for 7 days and 28 days before testing. All tests were carried out at the Geotechnical Laboratory of the University of Auckland.

5.2 Atterberg Limits

Atterberg Limit tests showed Plastic Limit and Liquid Limit values of 35% and 73% respectively for the untreated soil. The Plasticity Index was therefore 38%.

Plotting Plasticity Index and Liquid Limit data on the Casagrande Plasticity Chart shows that the soil falls on the boundary of the CH[3] and MH[3] classifications. According to the AUSTROADS Pavement Design Guide, the CH soil has a typical presumptive CBR value of 6 to 7% in well drained situations and 4 to 5% in poorly drained situations.

[3] CH clay, high plasticity; MH silt, high plasticity

5.3 Compaction Tests

Standard compaction tests were carried out to establish the optimum water content (owc) and maximum dry density for the untreated soil and for specimens with lime contents of 3% and 5%. The results are presented in Figure 5.1.

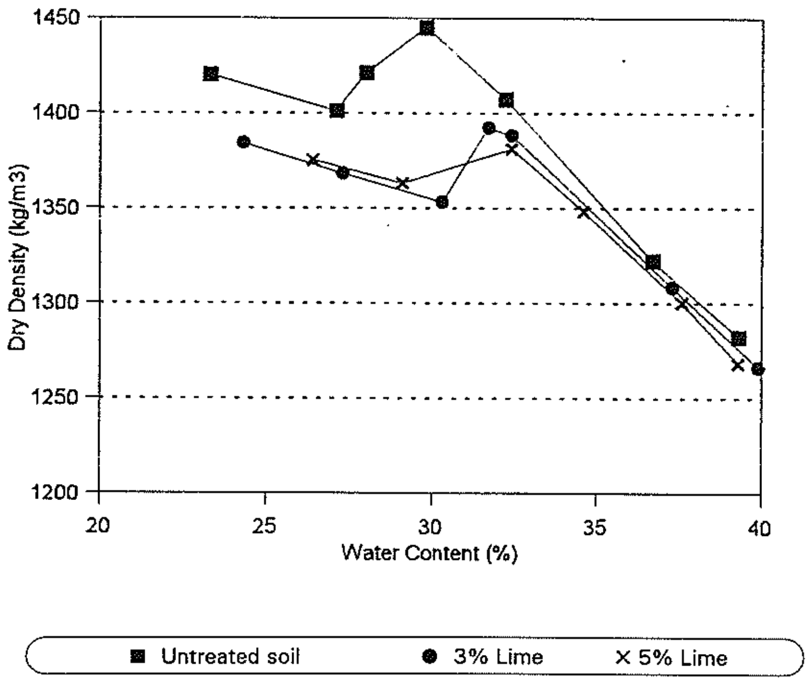


Figure 5.1 Compaction test results.

Figure 5.1 shows that the compaction test results were somewhat inconsistent. Maximum dry density and optimum water contents have been assessed as shown in Table 5.1. As expected, the maximum dry density tends to decrease and the optimum water content tends to increase as the percentage of lime increases.

Table 5.1 Summary of maximum dry density and optimum water content results.

Specimen Configuration	Maximum Dry Density (kg/m^3)	Optimum Water Content (%)
Untreated	1420	31.0
3% Lime	1390	32.0
5% Lime	1380	32.5

5.4 CBR Tests

California Bearing Ratio (CBR) tests were carried out on soil specimens prepared at optimum water content with lime contents of 0%, 3% and 5%. In addition, tests were carried out after curing times of 0, 7 and 28 days. The tests were performed with the specimens in both soaked and unsoaked states, except for the zero curing time specimens, which were tested only in the unsoaked state. The CBR test results are presented in Figure 5.2 and Table 5.2.

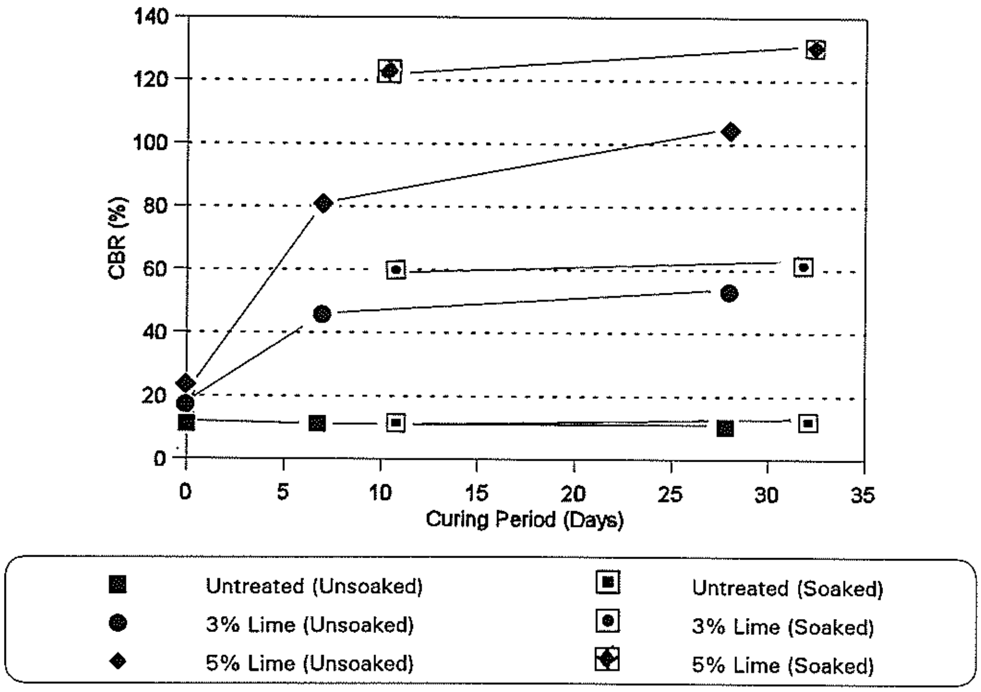


Figure 5.2 CBR test results.

Table 5.2 CBR test results.

Specimen Configuration	Soaked / Unsoaked	Mean CBR (%)		
		0 Days Curing	7 Days Curing	28 Days Curing
Untreated	Unsoaked	12	11	11
	Soaked	n/a	11	13
3% Lime	Unsoaked	18	46	54
	Soaked	n/a	59	64
5% Lime	Unsoaked	21	81	105
	Soaked	n/a	112	131

The results of the CBR tests show that the addition of lime to the test soil has a considerable effect on the soil's bearing capacity, as reflected in the CBR parameter. In addition, the curing period has a significant influence in the short term but that influence reduces with time. A measurable increase in CBR was observed almost instantaneously with the addition of lime, as shown in the *zero curing time* results. Considering the results of the lime-treated specimens only, the ratio of the CBR after 7 days of curing to the CBR after 28 days of curing was within the range 0.77 to 0.92, with a mean value of 0.85.

The data in Figure 5.2 and Table 5.2 show that the soaked CBR values were generally higher than the unsoaked values. This is contrary to what would generally be expected. However, examination of the test specimen details showed that the dry densities of the soaked test specimens at the time of preparation were slightly higher than the corresponding values for the unsoaked test specimens. This was mainly because the soil was initially slightly drier for the soaked specimens. In addition, the soaking period, i.e. four days, provided extra curing time compared to that allowed for the unsoaked specimens. These factors may have been responsible for the elevated soaked CBR test results. Irrespective of these complications, the test results showed that soaking the lime-treated specimens before testing did not cause the CBR parameter to reduce.

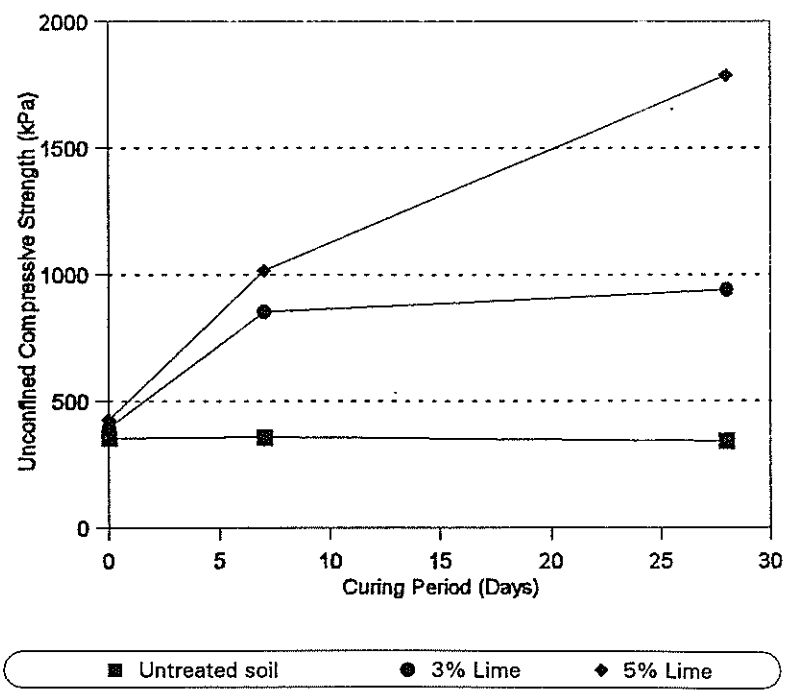


Figure 5.3 Unconfined compression strength (UCS) test results.

5.5 Unconfined Compression Strength Tests

Unconfined compression strength (UCS) tests were carried out on soil specimens prepared at optimum water content with lime contents of 0%, 3% and 5%. In addition, tests were carried out after curing times of 0, 7 and 28 days. The UCS test results are presented in Figure 5.3 and Table 5.3.

As expected, the UCS of untreated soil specimens was independent of the curing time. Clearly, in this case there is no binder to require curing, but these tests did verify that the test procedure and equipment provided consistent results. The specimens treated with lime showed a slight immediate increase in UCS but more significant increases were observed after 7 and 28 days of curing.

The 3% lime specimen showed a dramatic increase in strength after 7 days of curing, but the rate of strength increase slowed significantly between the 7-day and 28-day periods. The ratio of the 7-day UCS to the 28-day UCS for the 3% lime specimen was 0.91. The 5% lime specimen also showed a dramatic increase in strength after 7 days of curing. However the rate of strength increase from 7 to 28 days did not drop off as quickly as for the 3% lime specimen. The ratio of the 7-day UCS to the 28-day UCS for the 5% lime specimen was 0.57.

Table 5.3 Unconfined compression strength (UCS) test results.

Specimen Configuration	Mean Unconfined Compressive Strength (kPa)		
	0 Days Curing	7 Days Curing	28 Days Curing
Untreated	352	357	340
3% Lime	393	852	938
5% Lime	424	1014	1785

5.6 Split Tensile Tests

Split tensile (Brazilian) tests were carried out on soil specimens prepared at optimum water content with lime contents of 3% and 5%. In addition, tests were carried out after curing times of 0, 7 and 28 days. The split tensile test results are presented in Figure 5.4 and Table 5.4.

The split tensile test is only valid for materials having a compressive strength at least three times greater than their tensile strength. If this condition is not met, the specimens simply fail in compression because of the high stresses around the loading strips. This makes it an inappropriate procedure for the untreated soil specimens. For this reason the test results presented for *zero days curing* should be treated as being indicative only.

The split tensile strength data presented in Figure 5.4 and Table 5.4 show that a significant increase in tensile strength occurred after 7 days of curing. Little or no further increase occurred in tensile strength after 28 days of curing.

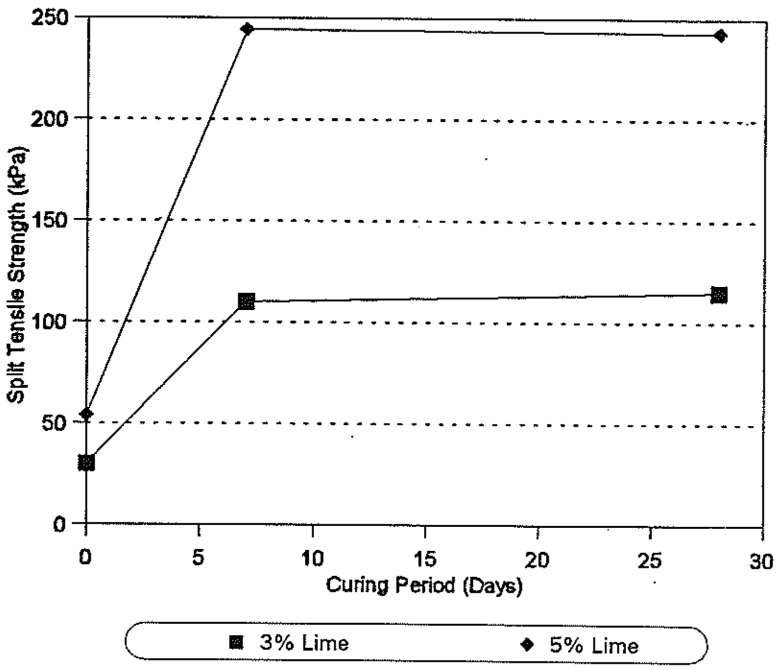


Figure 5.4 Split tensile test results.

Table 5.4 Split tensile test results.

Specimen Configuration	Mean Split Tensile Strength (kPa)		
	0 Days Curing	7 Days Curing	28 Days Curing
3% Lime	30	110	115
5% Lime	54	244	243

5.7 Repeated Load Triaxial Tests

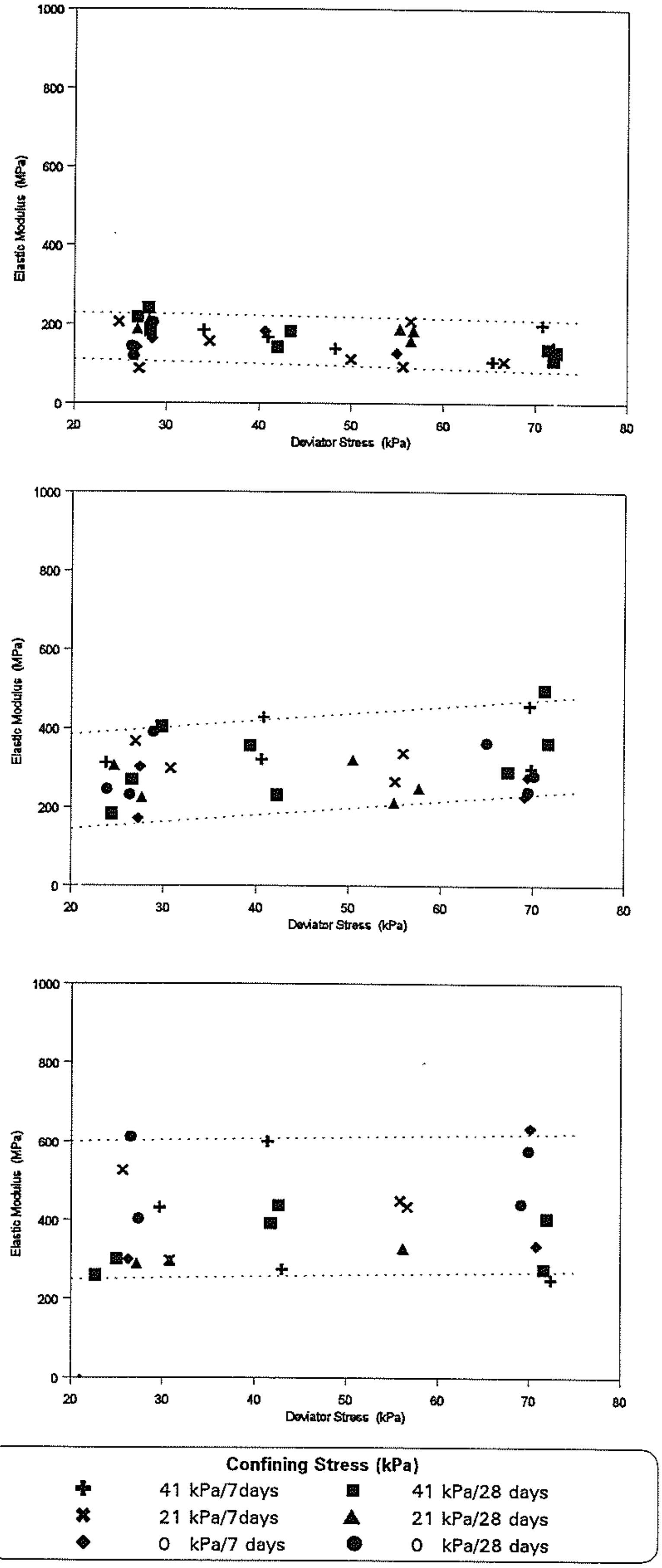
Repeated load triaxial (RLT) tests were carried out on soil specimens prepared at optimum water content with lime contents of 0%, 3% and 5%. In addition, tests were carried out after curing times of 7 and 28 days. The tests were carried out using specimens (nominally)100 mm in diameter by 200 mm long. All specimens were tested in an undrained condition. The AS 1289:1995 (SAA 1995) procedure was basically followed, though some amendments were necessary because of practical problems that arose during the testing programme.

Figure 5.5 Elastic modulus versus deviator stress.

(a) 0% lime

(b) 3% lime

(c) 5% lime



Test measurements were taken using both internal and external force and displacement transducers. Measurements taken internally, i.e. within the triaxial cell, are considered to be superior because they are independent of any compliance in the test equipment. This compliance can represent a significant effect, especially for relatively high stiffness materials.

Elastic modulus test results for cohesive soils are presented with respect to the prevailing deviator stress. This is because the elastic modulus parameter is generally dependent on the state of stress experienced by the specimen.

The results of the RLT tests are presented in Figures 5.5(a) to (c). Figure 5.5(a) shows a plot of elastic modulus versus deviator stress for the untreated soil while Figures 5.5(b) and (c) show the corresponding data for specimens with 3% and 5% lime respectively. On all three plots the data are somewhat scattered and therefore lines depicting representative bands of data have been included.

Graphs of elastic modulus versus deviator stress for pavement materials are generally plotted on log scales, but Figures 5.5 (a) to (c) have been plotted on arithmetic scales to show the magnitude and variation of the test data clearly.

Figure 5.5(a) shows that there is only a slight decrease in elastic modulus with increasing deviator stress for the untreated soil. Figures 5.5(b) and (c) show a very slight trend for the elastic modulus to increase with increasing deviator stress for the 3% and 5% lime-treated specimens. The plots also show that there is no discernible difference between the elastic modulus results for the specimens cured for 7 and 28 days. Lines of best fit have been drawn on the elastic modulus plots to characterise the response of the specimens.

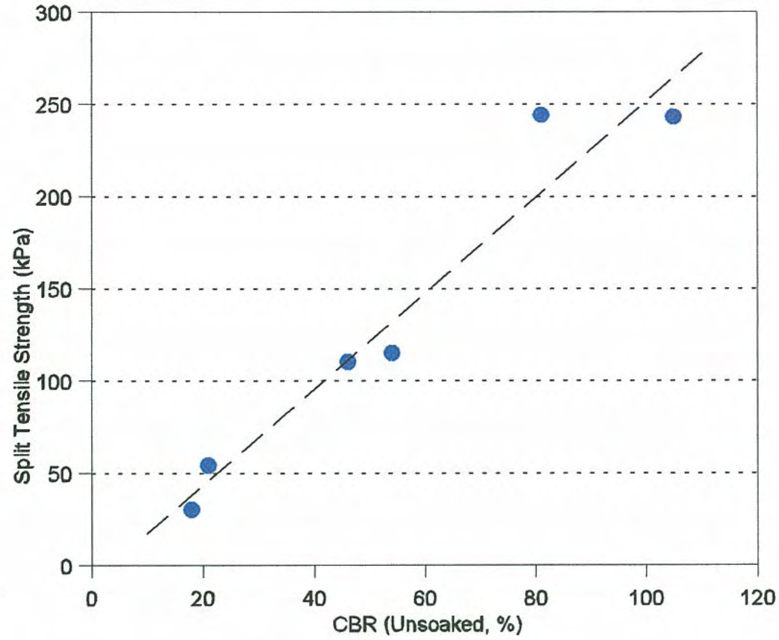
The deviator stresses that occur in-service are clearly dependent upon the elastic modulus and thicknesses of the various layers of material that make up the pavement structure. However, a typical flexible pavement structure has been analysed using *CIRCLY* and the results show that the deviator stresses at the levels of the modified subgrade and original subgrade layers are of the order of 60 kPa and 20 kPa respectively. Therefore, the resilient modulus test results can be generalised by considering the data at a deviator stress of 20 kPa for the untreated soil (original subgrade layer) and of 60 kPa for the treated soils (stabilised subgrade layer).

Correspondingly, from Figure 5.5(a), the untreated soil can be characterised as having an elastic modulus in the range approximately 110 to 230 MPa (deviator stress = 20 kPa). The central elastic modulus value from the band of data for the untreated soil is approximately 170 MPa.

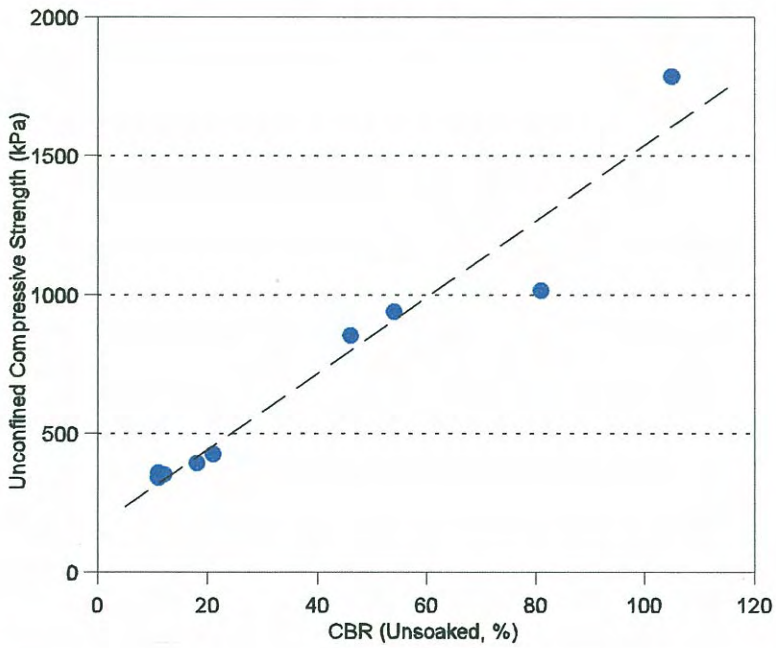
Figures 5.5(b) and (c) show characteristic elastic modulus values for the lime-treated soil in the ranges of 200 to 450 MPa and 300 to 600 MPa for the 3% lime and 5% lime specimens respectively (deviator stress = 60 kPa). The central elastic modulus values from the bands of data for the 3% lime and 5% lime specimens are approximately 325 MPa and 450 MPa respectively.

Figure 5.6
Unsoaked CBR (%) versus other mechanical properties of subgrades

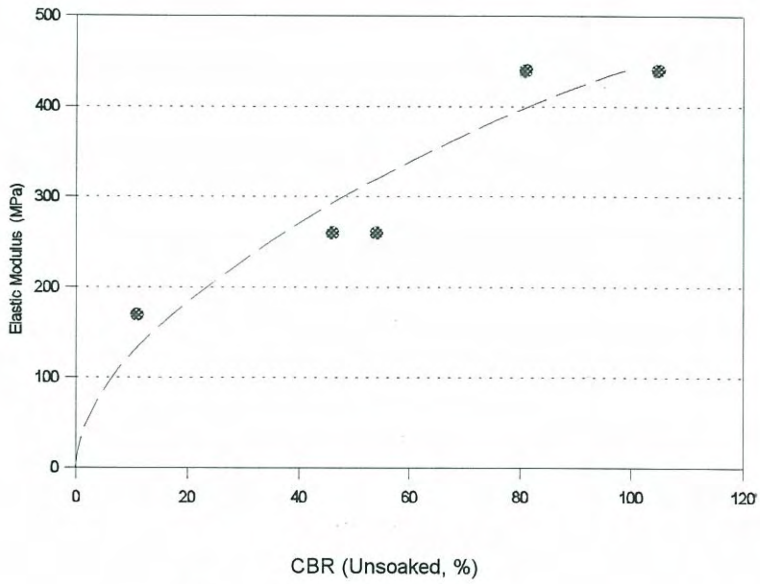
(a) v Split tensile strength (kPa) for lime-treated specimens



(b) v UCS (kPa) for all specimens



(c) v Elastic modulus (MPa) for typical subgrade stress conditions



5.8 Discussion

The laboratory testing programme carried out in this investigation has brought to light a number of issues regarding the mechanical properties of lime-treated soils. The main issues are as follows:

- A reasonably good correlation was established between the CBR and the split tensile strength of lime-treated specimens. Figure 5.6(a) shows a plot of split tensile strength versus (unsoaked) CBR. The Pearson correlation coefficient for the data in Figure 5.6(a) is 0.97.

- A reasonably good correlation was established between the CBR and the UCS of lime-treated specimens. Figure 5.6(b) shows a plot of UCS versus (unsoaked) CBR. The Pearson correlation coefficient for the data in Figure 5.6(b) is 0.97.

- The CBR, split tensile and elastic modulus data all show either an indiscernible, or very modest, change in strength or stiffness for curing periods of 7 and 28 days. This is consistent with the elastic modulus test data.

- Although the UCS test results show a good correlation with the CBR data, the UCS data also shows quite significant increases in specimen strength from 7 to 28 days, particularly for the 5% lime-treated soil. This is not consistent with the elastic modulus test data.

- A relationship between elastic modulus and CBR has been established from the data presented in Sections 5.4 and 5.7. Figure 5.6(c) shows a plot of elastic modulus versus (unsoaked) CBR for typical pavement stress conditions.

- On average, the ratio of the elastic moduli for the 5% lime-treated specimens and the untreated specimens was approximately 2.8.

Considering that the elastic modulus of the soil used in this investigation was not overly sensitive to the prevailing deviator stress, adopting a quick and inexpensive testing procedure, e.g. unconfined compression, is reasonable to verify elastic modulus parameters. However, the correlations must first be established in the laboratory using RLT testing or some other reliable test procedure.

6. DEVELOPMENT OF STABILISED SUBGRADE DESIGN STRATEGY

6.1 General

The main objective of this project has been to develop a strategy for the design of stabilised subgrade layers. This has been achieved by analysing the performance of stabilised soil using both field and laboratory investigations.

One of the most important observations of the field investigation is that subgrade stabilisation can be relied upon in the long term. Each of the three pavements that were subjected to test pitting showed that the enhanced properties of the stabilised subgrade layer persisted for extended periods of time. This is contrary to the Australian practice which is to not rely on stabilised subgrade layers in the belief that the layer may revert to its original properties if water can get into the pavement.

6.2 Design Strategy

The proposal is that subgrade stabilisation should generally be limited to improving the properties of the original subgrade soil, rather than attempting to form a highly cemented slab-type structure, i.e. the subgrade should be considered to be *modified* and not *cemented*. The reasons for adopting this strategy include:

- Achieving a modest increase in the bearing capacity of the top of the subgrade, which results in a stable construction platform that allows construction plant to function efficiently. It also makes the construction platform less susceptible to poor weather conditions.

- Limiting the stabilisation process to produce only a modified or lightly cemented soil ensures that brittle, slab-type layers are not formed. The behaviour of highly cemented layers can be very difficult to characterise, particularly given the variability of soil-cement. It is extremely demanding to predict the life of a cemented soil layer prior to cracking, and then to predict the residual mechanical properties after cracking. These issues are complicated by the presence of shrinkage and thermal cracks that may form as the binder hydrates.

- The performance of highly cemented materials is further complicated by the fact that load-induced tension cracks are generally initiated at the underside of the layer. This area can be susceptible to insufficient or variable compaction which provides opportunities for stress concentrations to occur, and these may result in cracking.

- Achieving a steady progression of the stiffness of the pavement layers from the subgrade up to the surface is logical because the stresses imposed by wheel loads tend to increase with increasing elevation in the pavement. This gives a balanced pavement structure without the complication of isolated layers attracting disproportionately large amounts of stress and being susceptible to fatigue type failure.

Considering a stabilised subgrade layer as being only modified allows the design to be simply analysed, as if the pavement has two subgrade layers - one being the original subgrade and the other being the modified layer. The AUSTROADS subgrade performance criterion, i.e.

$$N = \left[\frac{8511}{\mu\varepsilon}\right]^{7.14}$$

should therefore be applied at the top of the original subgrade and the top of the stabilised subgrade layer.

6.3 Characterising the Modified Layer

The construction of a modified subgrade layer involves mixing and hydration of the stabilising agent, followed by compaction of the layer. The quality of the end product is closely related to the efficacy of both the mixing and the compaction processes.

It is widely accepted that the ability to compact a layer of pavement material is a function of the elastic modulus of the underlying layer. If the underlying layer has a low elastic modulus it does not facilitate the effective transfer of compactive effort to the layer being compacted. Therefore, when characterising the elastic modulus of a modified subgrade layer for design, the value adopted should be a function of the elastic modulus of the original subgrade.

Dormon & Metcalf (1965) defined the modular ratio parameter, i.e. the ratio of the elastic modulus of an aggregate layer relative to the elastic modulus of the subgrade as follows:

$$k = 0.2h^{0.45}$$

where k = modular ratio ($2 < k < 4$);
h = thickness of aggregate layer (mm); and
the thickness of typical modified subgrade layers generally lies in the range 150 mm to 300 mm. Using the above relationship, this corresponds to modular ratio values of 2.0 to 2.6.

A drawback of Dormon & Metcalf's modular ratio concept is that the material being compacted is treated as a single, uniform layer. As the compactive effort tends to be less efficient in the lower part of a layer it makes good sense to allow the elastic modulus to decrease progressively with depth in the layer. This is the approach that is used in the sublayering of aggregate layers in the AUSTROADS pavement design procedures.

The AUSTROADS sublayering scheme dictates that sublayers should have a thickness of no less than 50 mm and no more than 150 mm, and that the maximum modular ratio between successive sublayers should not exceed 2.0. The number of sublayers that are assigned to a layer is dependent on the thickness of the layer, and on the ratio of the elastic modulus at the top of the layer and the elastic modulus of the underlying layer.

With the thickness of a typical modified subgrade layer generally being in the range 150 mm to 300 mm, the AUSTROADS procedure indicates that two or three sublayers are appropriate. Therefore, the maximum modular ratios between the upper sublayer of a stabilised subgrade layer and the original subgrade are 4.0 or 6.0 depending on the pavement configuration.

The Loadman FWD tests carried out in the eight test pits presented and described in Sections 2 and 3 of this report indicate that the ratio of elastic moduli between the stabilised and the untreated subgrade layers ranged from 2.5 to 10. These tests are not ideal for establishing top sublayer elastic modulus values because they only indicate what is effectively a single layer elastic modulus. However, the values obtained are expected to be of the correct order.

In addition, the laboratory testing programme described in Section 5 of this report showed that the average increase in elastic modulus for lime-treated specimens was approximately three-fold. On these bases it is proposed that, for practical design purposes, the elastic modulus at the top of a modified subgrade layer be taken as no greater than three or four times the elastic modulus of the original subgrade, depending on the response of the soil to lime treatment.

If reliable testing procedures indicate that the elastic modulus of the modified soil is less than three times the elastic modulus of the original subgrade, then that value should be adopted and the modified layer should be sublayered accordingly. Conversely, if tests indicate that the elastic modulus value of the modified soil is greater than four times that of the original subgrade, then the modular ratio of four should be maintained.

In addition, designers should be wary of adopting stabilised subgrade moduli in excess of 150 MPa. Note that laboratory tests on stabilised specimens can produce higher strength and stiffness values than the corresponding in-service values because of the superior mixing and compaction that can be achieved in the laboratory.

It is also beneficial to characterise a modified layer as being sublayered according to the AUSTROADS sublayering scheme. This reduces the elastic modulus of the lower sublayers to reflect the progressive decrease in the efficacy of compaction with depth in the layer. The AUSTROADS subgrade strain criterion should be checked at the top of each sublayer, although it is generally expected that the critical strain will be located either at the top of the stabilised subgrade layer or at the top of the original subgrade.

6.4 Application of the Design Procedure

The proposed stabilised subgrade pavement design procedure was tested on the Paremoremo Road, East Coast Road and Duck Creek Road pavements by analysing the original subgrade data and the modelling technique described in Section 6.3 of this report.

In the following analyses all materials are treated as having anisotropic elastic parameters. Unbound aggregate and stabilised subgrade layers have been sublayered using the AUSTROADS sublayering scheme. All elastic moduli are described in terms of the vertical component, and the horizontal modulus is half of the vertical value.

6.4.1 Paremoremo Road Site

The original design data for Paremoremo Road indicated a subgrade CBR of 6%, a depth of stabilisation of 250 mm and an aggregate depth of 125 mm. The original design life of 20 years comprised 2.6×10^4 EDA.

Applying the AUSTROADS relationship for subgrade elastic modulus (E_{sg}) results in an E_{sg} value of 60 MPa. Assuming that the ratio of elastic moduli between the top of the stabilised layer (E_{stab}) and the subgrade is 3.0, results in the E_{stab} value being 180 MPa. However, a maximum stabilised subgrade elastic modulus value of 150 MPa is considered to be appropriate for design. To complete the model the authors have assumed an elastic modulus for the top basecourse sublayer of 350 MPa.

CIRCLY analyses show that the design life of the pavement structure described above is approximately 4.5×10^4 EDA. This design life is almost double the original design life of the pavement and, considering that after 20 years service the pavement is performing very well, this suggests that the proposed design procedure gives a reasonable result.

6.4.2 East Coast Road Site

The original design data for East Coast Road indicated a subgrade CBR of 4%, a depth of stabilisation of 250 mm and an aggregate depth of 200 mm. The original design life of 20 years comprised 1.1×10^6 EDA.

Applying the AUSTROADS relationship for subgrade elastic modulus (E_{sg}) results in an E_{sg} value of 40 MPa. Assuming that the ratio of elastic moduli between the top of the stabilised layer (E_{stab}) and the subgrade is 3.0, results in the E_{stab} value being 120 MPa. With a modular ratio of 4.0 the E_{stab} value is 160 MPa, however a maximum modulus of 150 MPa is considered to be appropriate. To complete the model the authors have assumed an elastic modulus for the top basecourse sublayer of 350 MPa.

CIRCLY analyses show that the design life of the pavement structure described above ranges from approximately 4.5×10^5 EDA to 8.5×10^5 EDA for the 120 MPa and 150 MPa stabilised subgrade modulus values respectively. These design life values are somewhat less than the original design life of the pavement considering that the pavement is approximately 15 years old. However, the result is believed to be reasonable considering the assumptions that have been made in the analysis.

6.4.3 Duck Creek Road Site

The original design data for Duck Creek Road are not known with any degree of precision. However, the data obtained from the investigation indicated a subgrade CBR of 4%, a depth of stabilisation of 200 mm and an aggregate depth of 250 mm. The original design life of 20 years comprised an estimated 1.6 x 10^5 EDA.

Applying the AUSTROADS relationship for subgrade elastic modulus (E_{sg}) results in an E_{sg} value of 40 MPa. Assuming that the ratio of elastic moduli between the top of the stabilised layer (E_{stab}) and the subgrade is 3.0, results in the E_{stab} value being 120 MPa. With a modular ratio of 4.0 the E_{stab} value is 160 MPa however, a maximum modulus of 150 MPa is considered to be appropriate. To complete the model the authors have assumed an elastic modulus for the top basecourse sublayer of 350 MPa.

CIRCLY analyses show that the design life of the pavement structure described above ranges from approximately 8.5 x 10^5 EDA to 1.1 x 10^6 EDA for the 120 MPa and 150 MPa stabilised subgrade modulus values respectively. These design lives are somewhat higher than the original design life and it is difficult to judge their suitability as the pavement is only eight years old.

6.4.4 Discussion

The analyses described in Sections 6.4.1 to 6.4.3 show that the proposed design procedure provides results that are reasonable with respect to the original design data and the performance of pavements at Paremoremo Road, East Coast Road and Duck Creek Road. The results for Paremoremo Road and East Coast Road are particularly encouraging while the Duck Creek Road result is somewhat difficult to resolve because of a lack of original design data and the relatively short life of the pavement to date.

6.5 Materials Testing Requirements

For all pavement projects the quality of the design is directly related to the quality of characterisation data of the materials. Another undisputed fact is that the quality of the materials characterisation data is proportional to the investigation and testing budget.

The scope and extent of the materials characterisation testing should be dictated by the value and importance of the project. For example, a motorway or major arterial road would warrant a thorough materials testing programme that includes RLT tests on samples of subgrade, stabilised subgrade, sub-base and basecourse. Conversely, a minor access road may only warrant a few CBR tests to confirm the validity of field tests and to confirm that a soil is reactive to a given stabilising agent.

Clearly, if the funds are available, RLT testing should be utilised as much as possible, although this report has shown that, for the soils used in this project, good correlations have been established between CBR and UCS, and between CBR and split tensile strength (Figures 5.6(a) and (b)). Also a reasonable relationship can be established between CBR and elastic modulus, although the effects of stress dependence must always be taken into consideration (Figure 5.6(c)).

It must be noted that correlations between parameters such as CBR, UCS, split tensile strength and elastic modulus, should be established for all materials that are encountered along the pavement alignment. Initially this may be time consuming and expensive but the information can be held in a database for future use. A benefit of utilising correlations between parameters such as CBR and UCS is that the latter can be measured in a quick and inexpensive fashion using portable equipment. Therefore, quality assurance strategies can be developed utilising on-site testing programmes that provide timely and appropriate data.

7. CONCLUSIONS

This report describes the development of a mechanistic design procedure for pavements incorporating a stabilised subgrade layer. The objectives of the project were as follows:

- to verify that stabilised subgrade layers can be relied upon in the long term;
- to carry out a field investigation and a suite of basic laboratory tests to characterise and establish correlations between strength and stiffness parameters for lime-stabilised and untreated soils;
- to combine the results of the field and laboratory investigations to develop a strategy for designing pavements with a stabilised subgrade layer, for use in New Zealand.

7.1 Long-term Performance of Stabilised Subgrade Layers

The first objective was achieved by performing test pitting in three pavements that are known to have a stabilised subgrade layer. Field tests such as Loadman portable FWDs, Scala penetrometers and field shear vane tests were carried out on both the stabilised and original subgrade layers that were exposed in the test pits. The investigation showed that the stabilised layers maintained a high level of strength and stiffness in the long term. This is contrary to the approach used by Australian roading practitioners with respect to stabilised subgrade layers. The ratio of elastic moduli for the stabilised layers and the original subgrade ranged from 2.5 to 10.

Modelling of the pavements observed in the test pits was also carried out using the *CIRCLY* program. The analyses showed that adopting the AUSTROADS subgrade strain performance criterion generally showed that the pavements had a greater remaining life than may have been expected. This may indicate that the original design procedure was somewhat conservative. However, the modelling was subject to a number of assumptions because of the lack of substantial traffic loading and pavement performance data. It is important that road authorities maintain accurate pavement management databases so that investigations such as the one described in this report can be used to verify and fine-tune pavement design and characterisation processes.

7.2 Laboratory Test Procedures

A laboratory testing programme was carried out to investigate the characteristics of lime-treated soils with respect to basic testing parameters. The soil used in the investigation was Waitemata Group silty clay which has a Plasticity Index of approximately 38 and it plots on the boundary between CH and MH on the Casagrande Plasticity Chart. Tests were carried out on specimens with hydrated lime contents of 0%, 3% and 5% with curing periods of 7 and 28 days.

The test results showed a good correlation between the CBR and both the split tensile strength and the UCS of the specimens. Also the correlation between the CBR and the elastic modulus parameters was reasonably consistent, although the relationship was not linear. This correlation was most likely benefited by the fact that the elastic modulus was not highly sensitive to the magnitude of the prevailing deviator stress.

For the specimens used in the current investigation, an inexpensive and portable test procedure, such as the UCS test, would be feasible to use to verify field values of elastic modulus.

The RLT tests showed that, on average, the specimens with 5% lime added achieved approximately a three-fold increase in elastic modulus over the untreated specimens. The tests also showed that the difference between the elastic moduli of the lime-treated specimens after 7 days and 28 days curing were indiscernible. Therefore testing after 7 days appears to allow an appropriate curing period.

7.3 Proposed Design Procedure

Based on the findings of this investigation the proposal is that pavements incorporating a stabilised subgrade layer should be designed as if they have two subgrade layers, i.e. the stabilised layer and the original subgrade. This assumes that sufficient hydraulic binder (e.g. lime) is added to the stabilised layer to improve the properties of the soil, but not enough to produce a highly cemented material that is susceptible to fatigue cracking.

In addition, the stabilised layer should be analysed as being sublayered, as in to the AUSTROADS sublayering scheme.

The elastic modulus of the top sublayer should be taken as a maximum of three or four times the elastic modulus of the original subgrade, depending on the response of the soil to the addition of the binder.

The layer should be treated as having anisotropic elastic parameters as required for the AUSTROADS criterion for normal subgrade layers.

This procedure has been used to analyse the Paremoremo Road, East Coast Road and Duck Creek Road pavements using the original subgrade investigation and design traffic data. The proposed design procedure produced results that were consistent with the observed performance of these pavements. While the analyses of them do not represent a rigorous verification of the proposed design procedure, they did provide a reasonable level of confidence in its legitimacy.

8. REFERENCES

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