CHARACTERISATION OF SOME NEW ZEALAND SUBGRADES

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Characterisation of some New Zealand Subgrades

Opus International Consultants Ltd
Opus Central Laboratories
PO Box 30 845
LOWER HUTT

D. Alabaster
G.J. Fairless
R. Bailey
J.E. Patrick

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PO Box 2331, Lambton Quay, Wellington, New Zealand

Telephone (04) 473-0220; Facsimile (04) 499-0733

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EXECUTIVE SUMMARY

Pavements constructed in New Zealand on volcanic soils, when back-analysed using results from the Falling Weight Deflectometer (FWD) Test and the AUSTROADS (1992) design procedure, should not have been able to withstand the imposed traffic loading. These pavements have much higher deflections and subgrade strains than would be expected.

The economic consequences of direct application of the AUSTROADS (1992) criteria for pavement rehabilitation is that the thickness of the basecourse layer would need to be increased by 300-400mm over the thickness that has traditionally been used.

In addition, to continue the progress towards performance based specifications it is necessary, for all types of subgrade materials encountered in New Zealand, to quantify the relationship between elastic strains generated by loading and pavement life. Without this quantification non- destructive deflection testing of the finished pavement cannot provide meaningful answers.

The project was carried out in two stages.

Stage 1:

- identified a number of models in the literature for relating traffic loading to plastic strain accumulation;
- reviewed literature on mathematical models that have been developed to enable the modelling of elastic/plastic properties under dynamic loading imposed by traffic;
- used the computer programme CIRCLY to determine the range of stresses occurring in typical New Zealand pavements and conducted a sensitivity analysis on values used for Poisson's ratio and whether the pavement model is considered isotropic or anisotropic; and
- developed an experimental design for Stage Two of the project and identified sites for sampling.

Stage 2:

- the five sites had tube samples taken and FWD , *In-situ* California Bearing Ratio (CBR), and Spectral Analysis of Surface Wave (SASW) tests were conducted;

- repeat load triaxial (RLT) tests were carried out on three samples from each site, as well as further classification tests; and

- the results were compared to the criteria given in AUSTROADS, mathematical models in the literature and the moduli obtained from the site tests.

It was found that the AUSTROADS criterion for predicting a modulus for the subgrade from the CBR was not a useful indicator of the build up of plastic strain in the volcanic soils. Materials with similar CBRs had end-of-test permanent strains differing by up to a factor of about ten. Similar problems were found with the FWD and SASW moduli. One particular problem with these two moduli was that the soil with the second highest FWD and SASW modulus had the greatest plastic strain at the end of the tests. In the RLT tests, this soil had the smallest modulus in each test.

In the RLT tests, it was found that the resilient modulus (the load divided by the elastic deflection it caused) decreased as the load increased. These moduli were considerably less than the SASW and CBR-predicted moduli and similar to the FWD moduli.

Multivariate polynomial regression was used to fit the mathematical models to the RLT test data. The model of Bayomy and Al-Sanad (1993) fitted the five tested materials very well, but the four fitted coefficients in the model were significantly different between the materials.

It is recommended that further work be carried out on determining the relationship between the accumulation of permanent strain in the RLT test and rutting in service conditions. The highly variable strain accumulation properties of the volcanic soils need further investigation.

ABSTRACT

The current AUSTROADS (1992) pavement design procedure used in New Zealand uses a basic assumption that suggests that a vertical compressive strain applied to any subgrade will cause the same amount of plastic strain to occur irrespective of the type of subgrade. From the work carried out it seems that this assumption appears to be incorrect for volcanic soils.

As Stage 1 of this project a literature review was undertaken which indicated that there are a number of models for relating elastic and plastic strain for different materials. A sensitivity analysis was also conducted on the parts of the elastic model used in design and an experimental design developed for testing volcanic soils in Stage 2 of this project.

In Stage 2, five sites were sampled and tested using *in-situ* CBR, FWD and SASW to determine the in-situ moduli of the pavement. RLT tests were carried out on four of the volcanic soils and one non-volcanic soil obtained from the pavements. Each soil was subjected to one test at each of three nominal axial loads. The test results were fitted to the mathematical models found in the literature.

From the results in Stage 2 it was found that the CBR was not a useful indicator of the build up of plastic strain in volcanic soils. The resilient modulus obtained from the RLT tests was found to decrease as the load increased. These moduli were also considerably less than the SASW and CBR predicted moduli but similar to the FWD moduli.

The soil model reviewed in the literature, which produced the 'best-fit' comparison with the RLT data, was found from the model developed by Bayomy and AL-Sanad (1993).

Recommendations for further work on determining the relationship between the accumulation of permanent strain in the RLT test and rutting conditions, together with the analysis and testing of different volcanic soils to develop specific failure criteria for each soil are suggested.

1. Introduction

1.1 Background

The AUSTROADS (1992) pavement design procedure adopted for use in New Zealand uses a mechanistic approach based on the elastic characteristics of the pavement layers. The basis of the analysis is to limit the vertical compressive strain applied to the subgrade to a value dependent on the design traffic. The basic assumption is that there is a direct relationship between the elastic strain generated by the load and the effective life of the pavement. The assumed relationship used in AUSTROADS (1992) may be appropriate for a range of subgrade types, but it does not appear to be the correct relationship for the volcanic soils that are prevalent in the North Island.

There are a large number of pavements constructed in New Zealand that, when back-analysed using results from the FWD test and the AUSTROADS (1992) design procedure, should not have been able to withstand the imposed traffic loading. These pavements have much higher deflections and subgrade strains than would be expected using AUSTROADS (1992). This suggests that the current fatigue strain criteria for pavement design used by AUSTROADS (1992) is too conservative for some subgrades.

To continue the progress towards performance based specifications it is desirable, for common subgrade materials encountered in New Zealand, to quantify the relationship between elastic strains generated by the load and the plastic strain which remains after loading. Without this quantification non-destructive deflection testing of the finished pavement cannot provide meaningful answers. The current AUSTROADS fatigue relationship suggests that the relationship between elastic and plastic strains is constant for all materials.

In order to develop appropriate subgrade fatigue strain relationships for use with New Zealand subgrades in pavement design, the relationship between the elastic and plastic characteristics of typical subgrades must be determined. This will enable the classification of the soil types for which the AUSTROADS (1992) criteria are appropriate and identify those that require more investigation.

The economic consequences of direct application of the AUSTROADS (1992) criteria for pavement rehabilitation on volcanic soils is that the thickness of the basecourse layer would need to be increased by 300-400mm (Section Ten, AUSTROADS (1992)) over the thickness that has traditionally been used.

The refinement of the subgrade strain criteria should result in a higher degree of confidence in the application of mechanistic design. It will also allow more confidence to be obtained in calculating the remaining life of a pavement. This is crucial in determining the effect of changes of pavement use, e.g. the consequences of logging traffic.

A clearer understanding of the effect of stress on the accumulation of permanent strain will also assist in determining the effect of heavy vehicles on pavement performance, and the consequences of increasing axle loads and tyre pressures.

1.2 Objectives

The objective of this project is to compare the accumulation rate of plastic strain of a range of (New Zealand) volcanic subgrade soils with that of non-volcanic soils. Through this comparison, a more rational system of determining the allowable compressive strain that should be imposed on a subgrade to ensure the design life of a pavement is achieved will be obtained.

1.3 Benefits

The benefits of this project cover a wide range of Transfund's objectives including:

- minimising life cycle costs - through a pavement design system that reduces the risk of premature failure with some materials, and results in reduced pavement thicknesses with others;
- improving the efficiency of the expenditure decision through a better understanding of pavement performance and in being able to better quantify the remaining life of a pavement;
- optimising the use of materials: without an understanding of how low modulus subgrades can withstand heavy traffic, the tendency will be to stabilise the existing subgrade with lime or to use pavement layer thickness significantly greater than required;
- reducing the impact of traffic on pavements an improved: understanding of subgrade behaviour will allow greater confidence in predicting the effects of heavy traffic, specifically the different effects of axle weight and spacing, and tyre pressures, which have a major impact on the costing for road user charges; and
- improving load carrying capacity of pavements through the ability to predict pavement performance more realistically.

2. Literature Review

2.1 Introduction

A review of the literature into the effect of dynamic strain on the rate of accumulation of permanent deformation in soils under traffic loading was carried out.

The search also reviewed literature on mathematical models that have been developed to enable the modelling of elastic/plastic properties under dynamic loading imposed by traffic.

2.2 Permanent Deformation Models

Two fundamentally different procedures for estimating the amount of rutting from repeated traffic loading were identified by Monismith *et al* (1975). The first procedure involved the use of an elastic layered system to represent the pavement and either RLT tests or creep tests (for the asphalt-bound layers) to characterise the materials used. The second method involved the use of a visco-elastic layered system to represent the pavement structure and materials characterisation by creep tests. Monismith *et al* (1975) considered the first method to be more reasonable but does not provide reasons. For New Zealand conditions the visco-elastic system would seem inappropriate as it was primarily developed to investigate asphaltic concrete pavement layers. The constitutive equation in Equation 1 was derived to fit observations of the performance of a silty clay under a constant repeated load condition. The term permanent strain is interchangeable with the term plastic strain used earlier.

$$\overline{\in^{p}} = A\,N^{b}$$

where

$\overline{\in^{p}}$ = *permanent strain*

N = *number of stress applications*

A, b = *experimentally determined coefficients*

Equation 1. (Monismith et al, 1975)

Most of the models derived since the work done by Monismith *et al* (1975) have tended to stay with the general form of this model with refinements being made to the calculation of the coefficients A and b. It was shown that the exponent b was relatively independent of the stress state of the soils tested, i.e. it didn't change with increasing deviator stress. The coefficient A is defined by the above equation as being the permanent strain after the first cycle.

The difficulty of obtaining an all-encompassing constitutive equation coupling stress and strain in a particular soil is best described by Shackel (1973) in a review of repeated loadings of soils. He noted that the variables in such an equation include the geometry of the problem, x, the density ,γ, the moisture content, w, the stress history (from time s=0 to time s=t) and the temperature T. The equation can be written symbolically, with ~ denoting matrices as:

$$\underset{\sim}{\sigma}(\underset{\sim}{x},t) = \underset{s=0}{\overset{s=t}{\underset{\sim}{F}}}\left[\, \underset{\sim}{\varepsilon}(\underset{\sim}{x},s), T(\underset{\sim}{x}), \gamma(\underset{\sim}{x},s), w(\underset{\sim}{x},s), \ldots\ldots\ldots\ldots ; \underset{\sim}{x},t \,\right]$$

Equation 2. Shackel (1973)

Majidzadeh *et al* (1978) confirmed and extended the work by Monismith *et al* (1975) by testing sand, silt and clay samples in both saturated and unsaturated conditions. The extension used E^*, the dynamic modulus of resilience, to reflect material properties such as dry density, moisture content and soil structure. However, this assumption is only valid where the E^* remains constant with changing applied stress. For the soils in the study this was only true when the applied stress was above 55.2 kPa. Changes in applied stress are then dealt with by $exp(\sigma_{apl}/\sigma_{ult})$, as in Equation 3.

Equation 3. Majidzadeh *et al* (1978)

$$\frac{\varepsilon_p}{N} = A\,N^m$$

where

ε_p = *permanent strain*
N = *number of stress applications*
$A = R\,E^{*-c}\exp(\sigma_{apl}/\sigma_{ult})$
R, m, c = *experimentally determined material constants*
σ_{apl} = *applied stress*
σ_{ult} = *unconfined ultimate compressive strength*
E^* = *dynamic modulus of resilience*

Bayomy and Al-Sanad (1993) used Equation 4 for sand subgrades in Kuwait. The model follows the general form of the work by Majidzadeh *et al* (1978) without the inclusion of applied stress via the ultimate stress ratio.

$$\varepsilon_p = R(M_r)^s \exp(c\ \sigma_d) N^b$$

where

ε_p = *permanent strain*
N = *number of stress applications*
R, s, c *and* b = *experimentally determined constants*
σ_d = *deviatoric stress*
M_r = *modulus of resilience*

Equation 4. Bayomy and Al-Sanad (1993)

Statistical analysis showed that the parameter *b* was independent of the stress and moisture conditions and therefore considered constant for the material. The applied deviatoric stress and the resilient modulus are also shown to be statistically significant in the correlation. The material constants are thus considered to be independent of stress and compaction conditions.

Ullidtz (1987) noted the need for a model able to predict road failures at higher stress levels and higher numbers of load repetitions. The problem with models based on the Monismith *et al* (1975) equation (Equation 1) is that the solution tends toward some fixed value of permanent strain at high load repetitions, whereas field observation shows measured rutting keeps increasing in a linear fashion (Ullidtz, 1987).

The approach suggested by Ullidtz (1987) comes from the work of Vyalow and Maksimyak. They suggested that the deformation of a clay material as a function of time can be separated into three phases: Phase 1, decreasing strain rate (corresponding to the work of Monismith *et al*, 1975); Phase 2, constant strain rate; and Phase 3, accelerated strain rate. Equation 5 covers Phases 1 and 2 with the transition between the phases assumed to take place at a critical level of plastic strain (ε_0 is defined as the strain at N_0).

Phase 1 :

$$\varepsilon_p = A\, N^B \left(\frac{\sigma_1}{\sigma'}\right)^C \qquad \text{for } \varepsilon_p < \varepsilon_0$$

Phase 2 :

$$\varepsilon_p = \varepsilon_0 + (\,N - N_0\,)\, A^{(1/B)}\, B\ \ \varepsilon_0^{(1-1/B)} \left(\frac{\sigma_1}{\sigma'}\right)^{(C/B)} \qquad \text{for } \varepsilon_p > \varepsilon_0$$

where

$$N_0 = \varepsilon_0^{(1/B)}\, A^{(-1/B)} \left(\frac{\sigma_1}{\sigma'}\right)^{(-C/B)}$$

ε_p = *permanent strain*
N = *number of stress applications*
A, B, C = *experimentally determined coefficients*
σ_1 = *applied stress*
σ' = *reference stress*

Equation 5. Ullidtz (1987)

Reasons for Phase 3 development are discussed by Li (1994): the accelerated strain rate can be due to the build-up of pore pressure leading to failure. Wolff and Visser (1994) note that Phase 3 observed in real pavements tends to be associated with cracking of the surfacing and the entry of water into the pavement structure, so Phase 3 could also become a problem associated with maintenance of the surfacing as opposed to subgrade design. Changes in dry density and moisture content are not allowed for by this model, so tests need to be conducted for each new physical state.

Ullidtz quotes Danish work, which found that the coefficient A is approximated by A=(5/CBR)×0.07, which corresponds to a mean subgrade CBR of 5%. Ullidtz makes no comment about the value of the reference stress, σ' other than to say it is set to arbitrary 100 kPa.

A similar approach is given by Li and Selig (1996) for silts and clays (Equation 6). The stress state of the soil is taken into account with the deviatoric stress. The problems of soil moisture content and dry density are dealt with using the unconfined compressive strength of the soil. Li and Selig's model is the basically the same as Ullidtz's, except for the use of the unconfined compressive stress for the reference stress rather than perhaps an arbitrary value (0.1 MPa in Ullidtz, 1987).

Equation 6. Li and Selig (1996)

$$\varepsilon_p = a\left(\frac{\sigma_d}{\sigma_s}\right)^m N^b$$

where
ε_p = *permanent strain*
N = *number of stress applications*
a,b,m = *experimently determined coefficients*
σ_d = *deviator stress*
σ_s = *unconfined compressive strength*

Raad and Zeid (1989) proposed a two-part model which is interesting for its concentration on the stability of the accumulation of permanent strain and the prediction of permanent strain accumulation in the unstable region (Equation 7). The stable accumulation of permanent strain is simply where the amount of strain accumulated per loading cycle decreases with the increasing number of loading cycles. It was found that for the silty clay under investigation, the threshold stress value between stability and instability was between 0.8 and $0.9\times q_r/q_{rl}$. Below the threshold, strain hardening or shakedown is observed. Subgrade modulus, defined as the ratio of repeated stress to total strain per load repetition, rises and reaches a constant value at high numbers of load repetition. Above the threshold stress level, strain softening occurs and the modulus decreases.

Phase 1:

$$q_r = \frac{\varepsilon_a}{a_L + S_L \log N} \qquad \textit{for } q_r < q_{rl}$$

Where
ε_a= *permanent strain*
N = *number of stress applications*
a_L, S_L = *experimentally determined coefficients*
q_r = *ratio of repeated deviator stress to the strength obtained from a standard triaxial test at a strain rate of 0.5 percent/min*
q_{rl} = *threshold stress level which corresponds to the stress level below which the accumulation of axial strain will eventually cease and lead to a stable response*

Phase 2:

$$q_r = \frac{\varepsilon_a}{a_h + b_h \varepsilon_a} \qquad \textit{for } q_r > q_{rl}$$

where
ε_a= *permanent strain*
N = *number of stress applications*
$B_h = B_h + S_h \log N$
a_h, S_h, B_h = *experimentally determined coefficients*

Equation 7. Raad and Zeid (1989)

Shakedown theory can be used as a framework for applying the range of models found in the literature. Collins *et al* (1993) discusses the four following long-term responses to loading:

(i) **Purely Elastic.** In this instance, the loads are sufficiently small so that no element of the structure is ever plastically stressed. The response will be purely elastic and the deformation recoverable throughout the complete loading history.

(ii) **Elastic shakedown.** The structure's response is partially elastic and partially plastic for a finite number of load applications, but the build-up of residual stresses returns further response to a purely elastic form. When this occurs the structure is said to have 'shaken down'. The maximum load level at which this occurs is known as the shakedown limit.

(iii) **Plastic shakedown.** In this regime, the final steady-state response of the structure is a repeated closed cycle of elastic/plastic deformation, where the stress cycle just touches the yield surface and there is no continual increase in the increment of plastic strain accumulated with each loading cycle.

(iv) **Ratchetting, or incremental collapse.** At still higher load levels, the plastic part of the response grows with each load application and results in an increasing increment of plastic strain accumulated in each load cycle.

Brown and Dawson (1992) provide an example of a model in the purely elastic category. It is not reviewed here. Models based on the Monismith *et al* (1975) model fall into the elastic shakedown category. The two-phase models from Ullidtz (1987) and Raad and Zeid (1989) attempt to describe elastic shakedown in Phase 1 and plastic shakedown in Phase 2.

Shakedown theory suggests it is possible to determine which form of model should be used by a simple stress limit. For example, above the elastic shakedown limit but below the plastic limit use Ullidtz's model. The lower bound under which elastic shakedown will occur is described by Melan's theorem:

> "If any residual stress distribution can be found which, together with the stress field produced by the passage of a load, does not exceed the yield condition at any time, then elastic shakedown will occur."

Kinematic or Koiter's theorem can define the upper bound:

> "If any kinematically possible plastic collapse mechanism can be found in which the rate of work done by the elastic stresses due to the load exceeds the rate of plastic energy dissipation, then incremental collapse will occur."

However, it is not as easy as it seems to apply shakedown theory for granular materials to fine grained materials. Li (1994) discusses the problem of changing stress conditions when pore water pressure builds up in fine-grained soils under repetitive loading in undrained conditions. The increasing pore water pressure leads to a reduction in effective stress and eventual failure. In drained conditions, repetitive loading on normally consolidated soils can have a beneficial effect on the shear strength of the soil. Dissipation of excess pore water pressure leads to reconsolidation and an increased resistance to cyclic shear deformation. With over-consolidated soil in drained conditions quite the reverse can occur. Excess pore water pressure increases with successive cyclic loading as does shear strain. Current shakedown design ignores the problem of changing stress conditions and focuses on the strength conditions occurring early in the loading history of the soil, assuming the strength and stress conditions remain constant. However, the theory does elegantly describe the phases of soil behaviour as an overall concept.

The key point to note in all the models is that they all have material constants. Each expects that different materials accumulate plastic strain at different rates. Li and Selig (1996) have provided material constants for low plasticity silts, high plasticity silts, low plasticity clays and high plasticity clays. At 10,000 cycles of the same stress a high plasticity clay could accumulate 2.5 times the plastic strain of a low plasticity clay with the same static strength.

In summary, the models that include a parameter to allow for changing stress conditions would appear to be the most practical. They allow two or three RLT tests to be performed and the performance of the material can then be calculated under different stress conditions. Models with some limit to the amount of stress that can be applied are the most logical. The models by Li and Selig (1996), Ullidtz (1987), Majidzadeh (1978) and Raad and Zeid (1989) cover the problems of differing stress conditions and have some limit on the stress that can be applied. Ullidtz (1987) and Raad and Zeid (1989), however, run into problems in that quite a number of tests are required to establish the change in phase of the models, which makes them impractical for normal design use.

The difficulty of obtaining the shakedown limit is illustrated by Zhang and Pidwerbesky (1998). They suggest it is a function of loading frequency and confining pressure. The models by Li and Selig (1996) and Majidzadeh (1978) have the advantage of being simple and that their general shape is what is observed in pavement rutting studies. These two models are recommended for use with cohesive soils. Bayomy and Al-Sanad (1993) and the Phase 1 of Ullidtz (1987) are recommended for use with non-cohesive soils. While none of these models have been specifically developed for use with volcanic soils, unpublished data from RLT tests conducted at Opus Central Laboratories suggests that they can be usefully applied to such soils.

2.3 Mathematical Models of Pavement Stresses

The basic requirements of any mathematical model relating distress to performance are given in Pidwerbesky (1996):

- the pavement models must be able to predict both the type and the degree of distress that will occur under any given set of conditions;

- the model must be able to predict the component effect of any particular form of distress on the primary output of the pavement (its serviceability-age history); and,

- the effect of various maintenance strategies on the serviceability-age history of the pavement must also be modelled.

The approaches to the problem of mathematical modelling can be separated into two fundamental groups, the first being analytical methods and the second being numerical techniques.

2.3.1 Analytical methods

Analytical methods use linear elastic theory. Based on Boussinesq's one-layer theory, the main assumptions are that the materials are homogeneous, isotropic and linearly elastic. The boundary conditions for the problem are: infinite lateral dimensions; infinite thickness (finite for the surface layers); that the upper layers are weightless; that the layers are in continuous contact; that the surface layer is free of shearing; and there is full continuity at interfaces between layers. Soil models have also been developed however, which incorporate and solve multi-layer problems.

Computer programmes such as CIRCLY, BISAR, ELSYM5, EFROMD2, DAMA and PAS use linear elastic theory to calculate the stresses and strains generated within a pavement. Each programme calculates the stress and strain slightly differently. For example, CIRCLY allows anisotropic conditions to be entered. The more advanced programmes could be considered as simple finite element programmes retaining the concepts of linear elastic theory; DAMA, for example, allows non-linear characterisation of granular materials (Chen *et al*, 1995).

The disadvantage in using packages based on linear elastic theory is spelt out by Pidwerbesky (1996) and Haas (1994): real conditions in real pavements contradict assumptions made in using linear elastic theory. Pidwerbesky (1996) measured the actual strains in a test track pavement and concluded that actual vertical compressive strains measured in the unbound granular layers and subgrade were substantially greater than the levels predicted by multi-layer linear elastic models on which AUSTROADS (1992) and the National Roads Board flexible pavement design procedures are based, for the same number of loading repetitions to failure. This suggests that linear elastic theory does not calculate the real stresses and strains occurring in the pavement. However, AUSTROADS design method can be altered by sub-layering unbound granular layers so that modulus dimishes with depth and also the horizontal modulus can be assumed to be much less than the vertical modulus. By making these changes, linear elastic analysis can make an attempt to model non-linear stress distributions more closely.

In terms of the basic requirements set out by Pidwerbesky (1996) and given above, it seems that only the first ("the pavement model must be able to predict both the type and degree of distress that will occur under any given set of conditions") is satisfied by linear elastic theory, and even then only partially. By definition elastic design requires an empirical link with distress. That link can only be guaranteed to hold true for the conditions it was created for. In the case of current elastic design, the link is only true for particular soil types and only predicts one level of distress.

2.3.2 Numerical techniques

Numerical techniques are based on finite element methods. Using finite element techniques it is possible to model assumptions of anisotropic and non-linear conditions. Even non-homogenous conditions and alternative boundary conditions can be modelled if desired. Finite element methods have been incorporated into computer packages such as ILL-PAVE and MICH-PAVE, methods for dealing with boundary conditions being the difference between these two programmes. Finite element analysis can also be used in more specialised design techniques such as viscoelastic analysis and mechano-lattice analysis.

Finite element analysis replaces a continuum with an assemblage of discrete elements connected at node points. The number of elements required to represent the problem depends in principle on the severity of the stress gradient and the extent of the continuum (Kumar, 1986). Standard computer packages such as MICH-PAVE account for material non-linearity in the basecourse and subgrade layers, the unbound nature of granular materials and the locked lateral stresses arising from compaction (Harichandran *et al*, 1990). These packages could be used to model the stresses required for input into the constitutive equations outlined in the literature review.

Viscoelastic analysis is primarily used for the design of asphalt pavement structures. The finite element programme PACE has been developed for pavement design specifically using viscoelastic material characteristics defined by testing mix specimens with repeat load creep tests or conducting frequency sweep tests. It allows for strain hardening and heat flow which cannot be dealt with by linear elastic solutions (Rowe *et al*, 1995). Viscoelastic techniques can be extended to model the whole pavement. Hyde and Brown (1976) investigated the link between creep loading tests and RLT tests for a silty clay. They found that creep loading curves can adequately predict RLT test curves.

Smith and Yandell (1986) discuss the use of the mechano-lattice which directly allows for the elasto-plastic behaviour observed in real pavements. The model uses an axial load versus deflection hysteresis loop during cyclic loading. Behzadi and Yandell (1996) suggest the RLT tests can be used to provide the necessary inputs for the technique.

While numerical techniques provide more realistic calculations of the strains actually occurring in the soil, it must be remembered that the design charts and fatigue criteria that currently exist have been back calculated using linear elastic methods. This means that it may not be wise to use more sophisticated methods with current fatigue criteria developed with elastic theory.

When modelling a soil it must be remembered that the model is an idealisation of what is really happening. In the case of linear elastic models the assumptions made can be vastly different to what is actually happening. Because of the highly stress-dependent nature of volcanic soil behaviour (Jacquet, 1987), the assumptions made in linear elastic theory may be inaccurate. Numerical techniques provide the best way to model the naturally occurring conditions found in pavements founded on volcanic soils because of their ability to model non-linear, anisotropic conditions with varying boundary conditions. Standard computer packages such as ILL-PAVE and MICH-PAVE are numerical techniques that have been developed specifically for pavement design. The ability to predict the actual stresses in the subgrade means that it is possible to apply the deformation equations found in the literature once a link has been confirmed between observed behaviour from the RLT tests and field studies.

3. Determination of Stress Levels

3.1 Stress Ranges in Typical New Zealand Pavements

The second part of the research programme involved determining the ranges of stresses occurring in typical New Zealand pavements. The stresses were calculated to provide input for the design of Stage 2 of the programme which involves laboratory testing of various subgrade soils. The widely-used linear elastic analysis programme CIRCLY was used to calculate the stresses.

Nine pavements were analysed with CIRCLY and stresses determined at the top of the subgrade and at every 200 mm of depth for the first 1 m of subgrade. Three subgrades with different Young's Modulus were used in combination with three loading levels. The Young's Modulus values used were 30, 100 and 200 MPa. The loading levels were 10^5 ESA, 10^6 ESA and 10^7 ESA. The minor and major principal stress are shown in Figures 3.1 and 3.2 respectively. These stresses give an indication of the confining stress and deviator stress to apply in the Stage 2 triaxial tests.

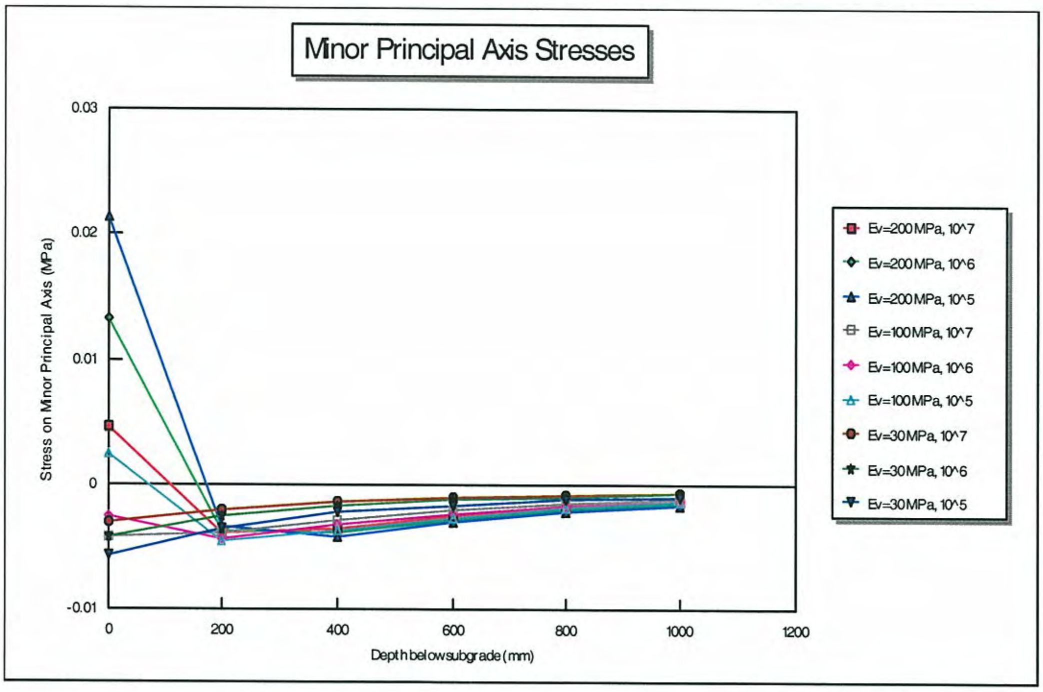


Figure 3.1 Minor Principal Stresses Calculated From Circly Analyses.

Difficulties arise in that the minor stress plot suggests a tensile stress in the soil which can't be modelled with the triaxial test apparatus. However, the results of the CIRCLY analysis neglect the *in-situ* stress state of the soil, which would provide a confining stress greater than the change calculated by CIRCLY.

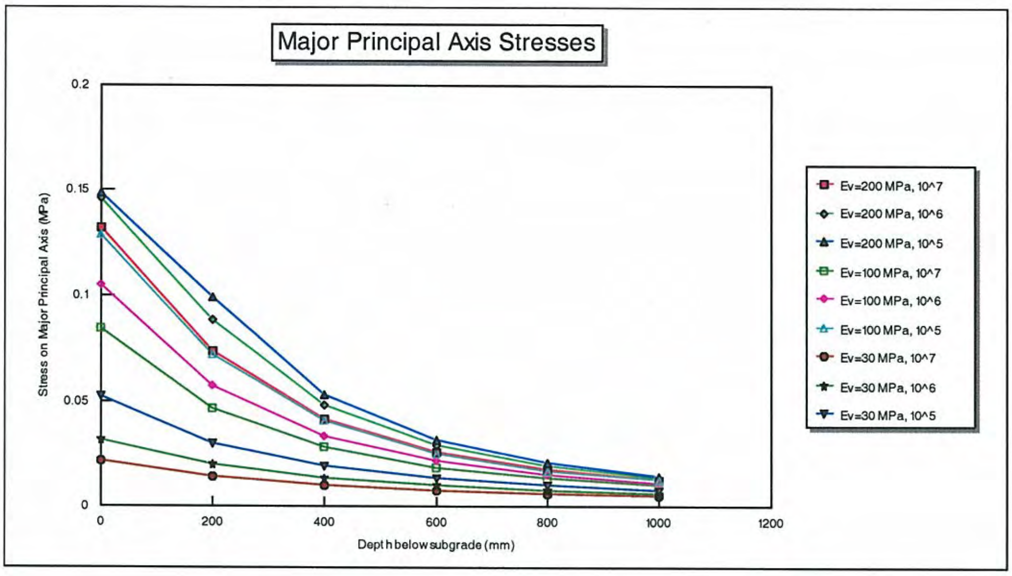


Figure 3.2 Major Principal Stresses Calculated From CIRCLY Analyses.

The allowable vertical compressive strains are small compared to the strains observed in a real pavement according to Pidwerbesky (1996). On this basis it must be considered that the major principal stresses determined above would also be too low. Thus it would be prudent to test the soil at higher stress levels than indicated by the CIRCLY results.

Applying a factor of 2 would be appropriate considering the actual and CIRCLY-calculated strain values determined by Pidwerbesky.

3.2 Sensitivity Analysis

3.2.1 Assumptions Tested

As part of the CIRCLY determination of stresses occurring in typical New Zealand pavements a sensitivity analysis was conducted into the assumption of isotropic or anisotropic strength and the effect of varying Poisson's ratio.

3.2.2 The Effect of Anisotropic *versus* Isotropic Strength Modelling

Anisotropic conditions were modelled as a 2 to 1 ratio of vertical to horizontal resilient modulus as used in AUSTROADS (1992). Isotropic conditions, by definition, were modelled as a 1 to 1 ratio. Pavements were modelled using Young's modulus of either Ev = 30 MPa or Ev = 100 MPa, combined with three separate design traffic values (10^5 ESA, 10^6 ESA and 10^7 ESA) to provide six separate pavement depths. Both anisotropic and isotropic models were run for each. The initial pavement depths used were obtained from Figure 8.4 in AUSTROADS (1992).

The effect on the stress at the subgrade for a particular ESA is shown in Figure 3.3. It can be seen that the assumption of isotropic or anisotropic behaviour has a significant effect on the stress level at the subgrade and ranges from a 20% change in low-modulus soils to a 10% change in high-modulus soils. However, this is to be expected as by changing the degree of anisotropy and Poisson's ratio the soil has also effectively been altered therefore producing different stresses. The effects above are small compared to variations in the soil and moisture conditions in the real world. It should also be noted that the degree of anisotropy in natural soils is not limited to a vertical to horizontal ratio of 2.

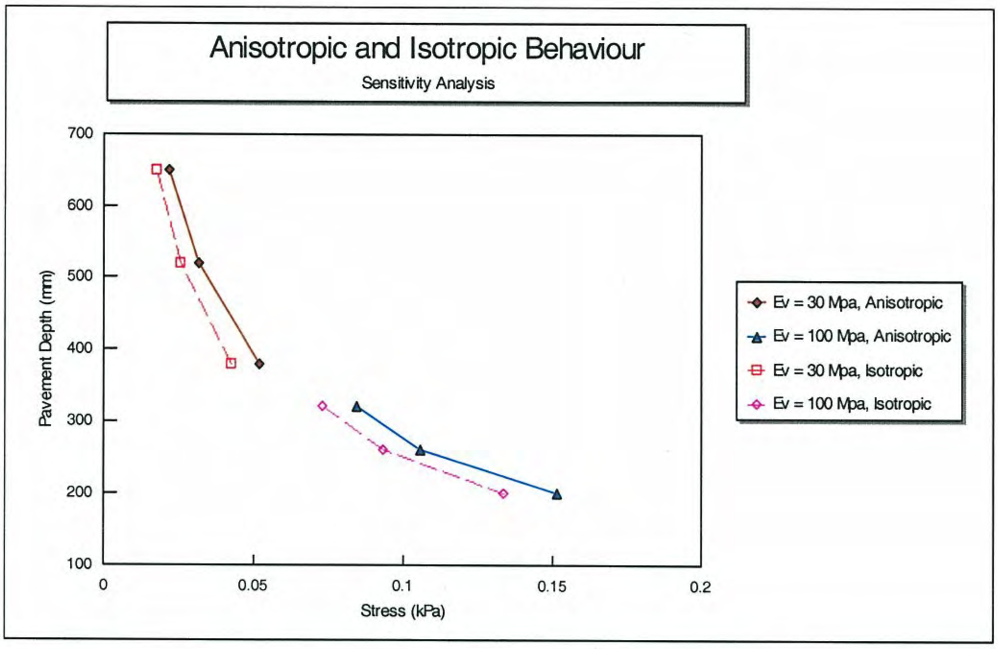


Figure 3.3 The Effect of Isotropic and Anisotropic Soil Models in CIRCLY Analyses.

Transfunds Research Report No. 177 suggests that it may be more accurate to predict the anisotropic and isotropic behaviour of a pavement using the relationship:

- vertical modulus (where an anisotropy of 2 is assumed) = 1.5×isotropic modulus,
- isotropic modulus = 6.7×CBR
- and the vertical modulus (where an anisotropy of 2 is assumed) = 10 CBR

3.2.3 The Effect of Poisson's Ratio.

The effects of variation in Poisson's ratio were investigated in the same manner as the anisotropic/isotropic problem. Poisson's ratios varying from 0.2 to 0.5 were used, covering the broadest possible range of ratios encountered in AUSTROADS (1992). The ratio was the same for all layers in the pavement and the stress determined at the top of the subgrade. In terms of stress at the subgrade surface, the maximum change observed between a Poisson's ratio of 0.2 and 0.5 was 14% (Figure 3.4). It was found that for a given pavement depth, varying Poisson's ratio does not significantly affect the calculated stress at the top of the subgrade.

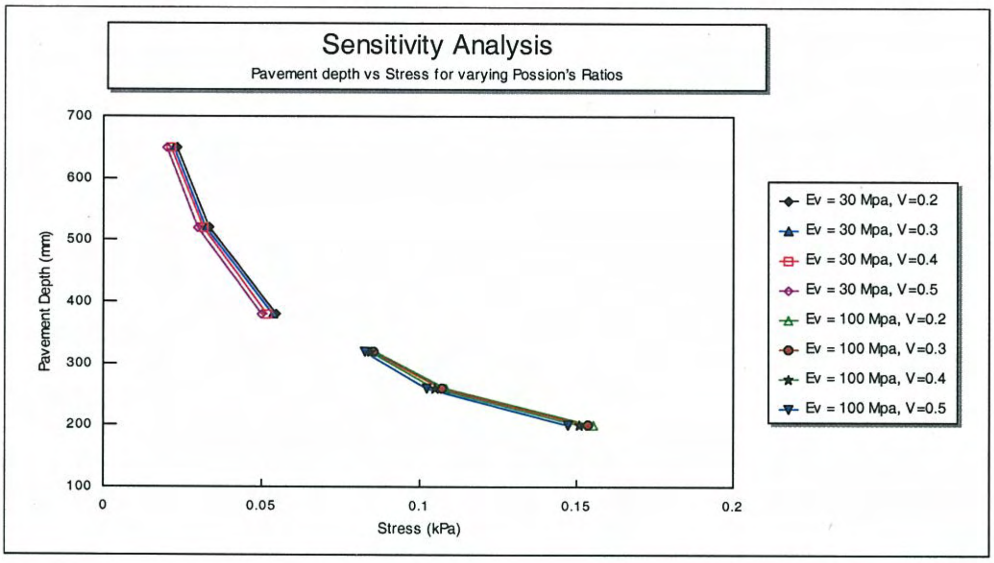


Figure 3.4 **The Effect on Stress at the Subgrade Surface of Varying Pavement Poisson's Ratios.**

4. Experimental Design For Stage 2

4.1 Overview

Based on the foregoing discussion, five subgrades will be sampled using a piston sampler to obtain undisturbed samples of the subgrade material from existing pavements. At each site, FWD and SASW testing will be performed to obtain an estimate of the subgrade modulus, as well as *In-situ* CBR tests.

Each material will be classified in terms of the standard classification tests of particle size distribution, moisture content and Atterberg limits.

From each site, RLT tests will be performed at three levels of stress to determine modulus and rates of permanent deformation up to 10^5 cycles of load. The stress levels and rate of loading will be those recommended below. With samples from five sites, a total of 15 RLT tests will be performed.

4.2 Soil Samples

The following subgrade types will be considered:

- Taranaki brown ash (Stratford ash) (soil group: brown granular loam; composition: andesitic tephra),
- Taupo ashes (soil group: yellow brown pumice soil; composition: rhyolite lapilli and ash),
- Egmont ashes (soil group: yellow brown loam; composition: hornblende andesite ash),
- Mairoa ash (Soil group: yellow brown loam; composition: augite andesite ash, and
- silty clay.

4.3 RLT Test Stress Levels

The confining stress applied will be 50 kPa, as suggested by O'Reilly and Brown (1991). This takes the middle ground in the literature where confining stresses range from 0 to 100 kPa. The CIRCLY analysis suggests there is little confining stress applied in a subgrade due to the wheel loading at the point of maximum deviator stress.

The CIRCLY analysis suggests that the deviator stresses applied should range from 25 kPa to 150 kPa. Combined with the information from Pidwerbesky (1996) the deviator stresses used will have a factor of safety of 2 and are proposed as follows:

- 50 kPa,
- 175 kPa, and
- 300 kPa.

The deviator stresses listed above tie in well with the range of deviator stresses observed in the literature for fine-grained materials. However, the final decision on the stresses used in testing will be based on the initial triaxial test results and where the material would be placed in a pavement the stress conditions applied must relate to the material being tested. For example, Fullarton (1978) indicates that the proposed deviator stresses may exceed the static shear strength of the Taranaki brown ash.

4.4 Loading Rates

A triangular load pulse with a rate of loading of 0.1 seconds will be attempted with a rest period of 2.9 seconds. To a certain extent the loading rate is governed by the limitations of the testing equipment. Barksdale (1971) suggests that a load pulse like this would simulate the traffic flowing at between 32.2 and 64.4 km/hr, depending on the depth of the pavement. This loading regime was also used by Monismith *et al* (1975).

4.5 Loading Conditions

Drained and unsaturated loading conditions will be adopted to best simulate real conditions. The confining pressure will be applied in drained conditions and allowed to reach equilibrium. Majidzadeh *et al* (1978) tested both saturated and unsaturated conditions and concluded that in either state the material followed the general rutting model, but with different values for the material parameters. This indicates that in terms of validating the models the loading conditions should be irrelevant if the stress conditions are carefully chosen.

5. Site Selection and Sampling

5.1 Soil Sampling Sites

Five sites were selected, four from the different ash deposits specified in the experimental design stage. The sites were selected using Figure 5.1.

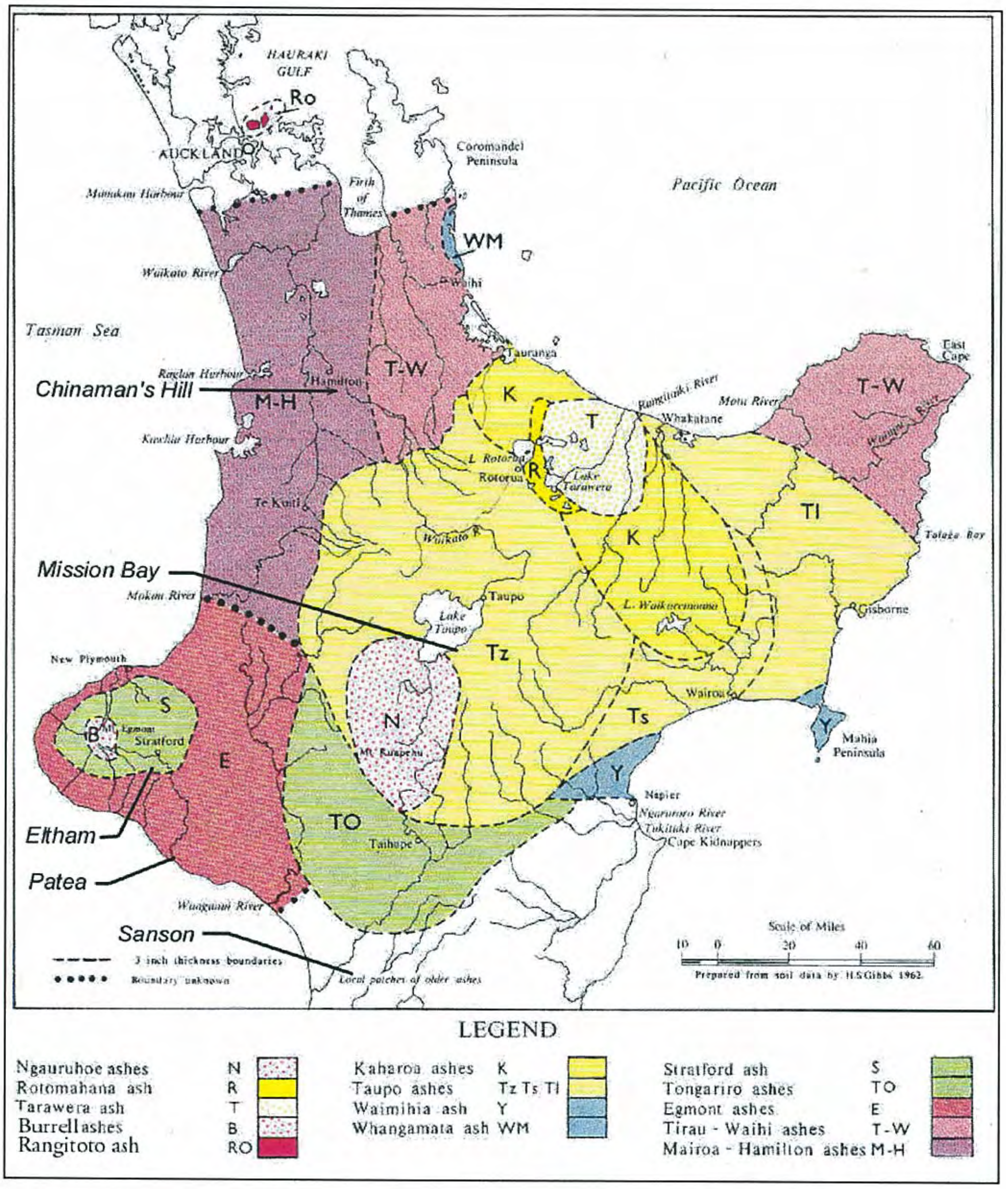


Figure 5.1 Surface pattern of soil-forming volcanic ash, North Island (from Gibbs, 1968).

5.2 Material Descriptions

The materials and their depths at each site are listed in the Tables 5.1 to 5.5.

Table 5.1 Eltham: SH3 RP 279 / 1.54, Stratford Ash.

Layer	Thickness (mm)	Description
Seal	120	
Basecourse	240	Volcanic, silty, sandy GRAVEL
Subbase	60	Gravelly, light-brown plastic SILT
Subgrade		Soft brown ASH (*In-situ* CBR 13.2)

Table 5.2 Mission Bay: SH1 RP 639 / 9.0, Taupo Ash.

Layer	Thickness (mm)	Description
Seal	20	
Basecourse	140	AP40 (Cement Treated). Fresh Angular DACITE, dense, moist, non-plastic. Some reconstituted chip seal mixed in.
Subbase	110	Cream pumice, silty SAND, fine to medium grained, some 30 mm pumice gravels, firm, moist, non-plastic. (*In-situ* CBR 70)
Subgrade		Very dark grey SAND, some 60 mm Pumice, some 40 mm rounded hard gravels, loose, moist, non-plastic. (*In-situ* CBR 35)

Table 5.3 Patea Township: SH3 RP 321 / 16.85, Egmont Ash.

Layer	Thickness (mm)	Description
Seal	70	
Basecourse	140	Volcanic, grey, silty, sandy GRAVEL
Subbase	70	Gravelly, light-brown SILT
Subbase	70	Black SAND
Subgrade		Light-brown SILT (*In-situ* CBR 33)

Table 5.4 Chinaman's Hill: SH3 RP 0 / 3.20, Mairoa Ash.

Layer	Thickness (mm)	Description
Seal	30	
Basecourse	270	AP40. Fresh angular to sub-angular GREYWACKE, excess fines, dense, moist, non plastic. Some reconstruction chipseal mixed in.
Subgrade		Orange clay SILT, very stiff, moist, plastic. (*In-situ* CBR 15)

Table 5.5 Sanson: SH3 RP 450 / 1.80, Clay SILT.

Layer	Thickness (mm)	Description
Seal	100	
Basecourse	210	Not recorded
Subgrade	70	Clay SILT. dark-brown, mottled soft (*In-situ* CBR 5)
Subgrade		As above lighter in colour, similar strength

5.3 *In-situ* Testing

At each site *In-situ* CBR, FWD and SASW tests were conducted to characterise the site conditions. The results and analysis are given in Table 5.6.

Table 5.6 Results of *In-situ* Tests on Subgrade.

Site	*In-situ* CBR	FWD Modulus (MPa)	Ratio FWD E to CBR	SASW Modulus (MPa)	Ratio SASW E to CBR
Eltham	13.2	70	5.3	171	12.6
Patea Township	33	64	1.9	126	3.8
Sanson	5	48	9.6	109	21.8
Chinaman's Hill	15	65	4.3	108	7.2
Mission Bay	35 (70)	95	2.7	257/479	7.3(6.8)

5.4 Laboratory Testing

Following the RLT, the water content, Atterberg limits and particle size gradation were establish for each sample. The results are given in Table 5.7 and Figure 5.2. A triaxial compression test was also carried out on each specimen after the RLT testing was completed, to obtain a compressive strength. A uniform compression rate of 2.03 mm per minute was used for the compression tests for all samples except Chinaman's Hill 100 kPa (4.06 mm/min) and 50 kPa (1.02 mm/min). The compressive strengths determined are given in Table 5.7.

Table 5.7 Laboratory Testing Results.

Sample name	**Egmont ash**	**Taranaki brown ash**	**Mairoa ash**	**Silty clay**	**Taupo ash**
Sample source	Patea SH3 RP 321/16.85	Eltham SH3 RP 279/1.54	Chinaman's Hill SH3 RP 0/3.2	Sanson SH3 RP 450/1.8	Mission Bay SH1 RP 639/9
Sample description	CLAY-SILT: medium plasticity	Silty SAND: high plasticity	Silty CLAY: high plasticity	Silty CLAY: medium placticity	Gravelly SAND: fine to coarse
As rec'd w/c (whole soil)	29.5	68.5	70.6	21.3	8.6
Sample history	Natural	natural	natural	natural	natural
Soil fraction tested	-0.425mm	-0.425mm	-0.425mm	-0.425mm	-
Liquid limit	-	-	-	45	-
Cone penetration limit	50	116	105	-	-
Plastic limit	24	72	51	19	-
Plasticity index	26	44	54	26	-
Linear shrinkage	-	-	-	-	-
Solid specific gravity	2.74	2.86	2.72	2.67	-
Compressive strengths	(kPa)				
50 kPa sample	254	199	191	266	346
100 kPa sample	277	160	192	287	424
150 kPa sample	250	207	220	319	393
Dry density (t/m^3)	1.36,1.38,1.34	.87,.82,.86	.91,.90,.93	1.59,1.65,1.58	1.95,1.95,1.94

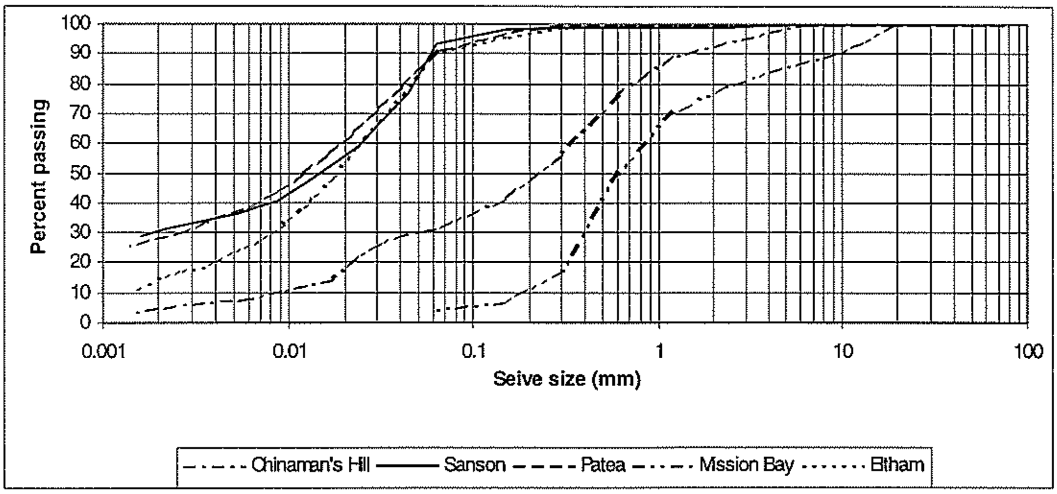


Figure 5.2 Particle Size Distributions for the Five Sites.

5.5 Stress Conditions at Subgrade Based on FWD Data

For each site a CIRCLY analysis was completed to determine the stress conditions imposed on the subgrade by a 550 kPa tyre load. The FWD elastic modulus was used and the pavement model configured with the site specific pavement layers. Isotropic conditions were assumed because the FWD back-calculated elastic moduli are based on isotropic conditions. An approximate estimate of the static stress conditions was also made based on the test pit log and assuming that confining pressure is developed under wheel loading by the passive earth pressure.

5.6 Revised Test Procedures

As a result of the analysis of the FWD data, it was decided to revise experimental conditions for the RLT testing. The concern was that the originally-proposed (section 4.3) stress levels were vastly greater than the levels suggested in the analysis with the FWD data and could have caused (instant) failures in some of the materials. Table 5.8 below shows the calculated stresses using the FWD modulus.

Table 5.8 Stress Conditions Calculated by CIRCLY Using FWD Modulus and Observed Pavement Layers.

Site	Dynamic 1st Principal Stress (kPa)	Dynamic 3rd Principal Stress (kPa)	Static 1st Principal Stress (kPa)	Static 3rd Principal Stress (kPa)
Eltham	39	3	10	58
Mission Bay	79	29	6.6	38
Patea Township	106	16	8.6	50
Chinaman's Hill	37	8	7.4	42
Sanson	48	1	7.6	44

Hence the target deviatoric stress levels to be used in the testing were:

- 50 kPa,
- 100 kPa, and
- 150 kPa.

These stresses are within the range of expected stresses predicted by CIRCLY calculations, as discussed in section 4.3. Thus the original plan of using laboratory stress about twice those predicted by the CIRCLY calculations has been set aside.

No change to the proposed drained conditions was considered necessary for the RLT tests. During preparation, construction and early traffic loading the subgrade will undergo some consolidation, so all samples were consolidated prior to RLT testing.

6. Test Equipment

The RLT machine consists of a double acting Bellofram pneumatic ram in a standard triaxial frame acting on a standard 100 mm cell. The pressure supplied to the cell is controlled by an electronic solenoid valve, which regulates the pilot pressure supplied to a volume booster. The air from the ram is exhausted via a quick exhaust valve, which also allows a seating pressure to be maintained. The double acting ram also allows an upward pressure to be applied to the ram, thus elevating the datum for the exhaust pressure.

The electronic solenoid valve is controlled by a National Instruments data acquisition card. The card also logs amplified signals from DCDT, load cell and, optionally, a pore-pressure transducer. All items were calibrated before use. The data from the data acquisition card is logged by a PC Direct 560 Ti personal computer utilising a purpose-written programme in Labview. The program allows the control of the pulse timing, while the loads are manually set. The data is logged 400 times a second. This allows both the permanent strain and the resilient modulus to be determined. The programme also provides real time displays of the load pulse and the load *versus* deflection curve.

Examples of the load pulse and load *versus* deflection curves are shown in Figures 6.1 and 6.2.

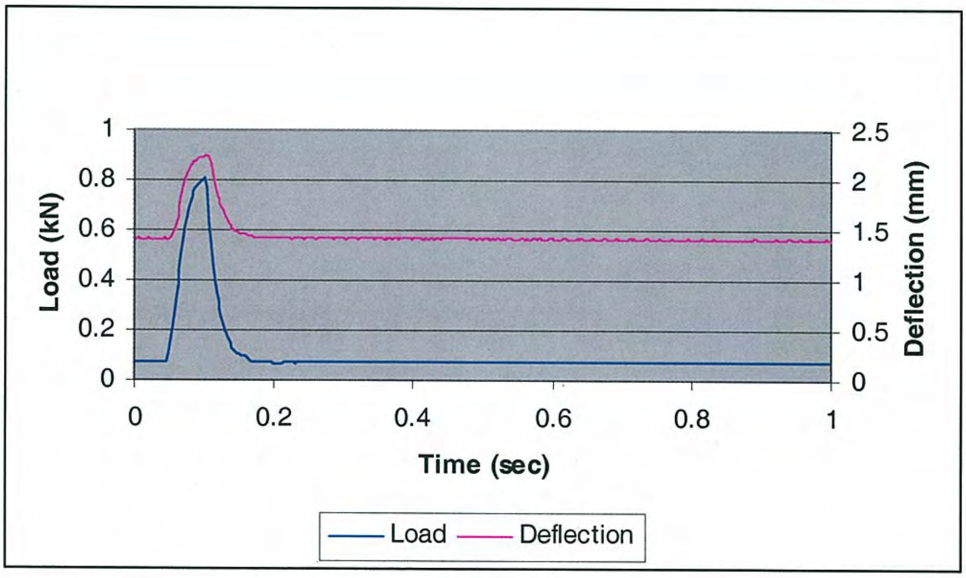


Figure 6.1 Load and Deflection Pulse Diagram.

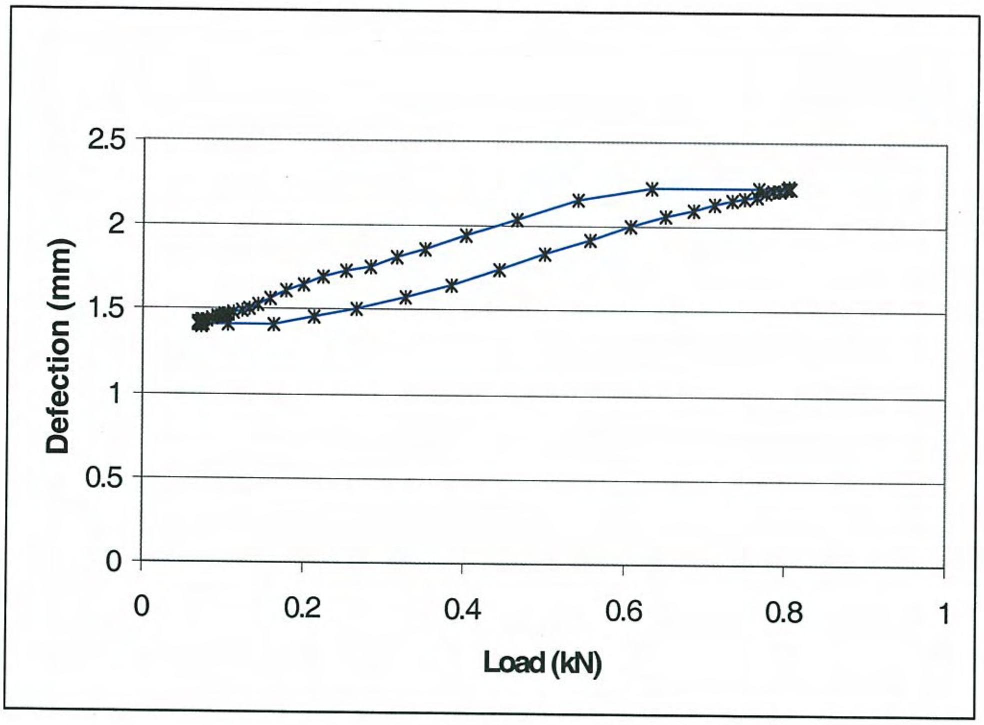


Figure 6.2 Load versus Deflection Diagram.

7. Repeat Load Triaxial Test Results

As planned, three RLT tests were carried out on samples from five sites. All samples were nominally 200 mm long and 102 mm diameter. The actual deviatoric stresses used are shown in Figure 7.1. In the Sanson tests, the early load cycles often had a deviatoric stress of about zero; these were the first tests carried out and there were teething problems.

Chinaman's Hill

Mission Bay

Patea

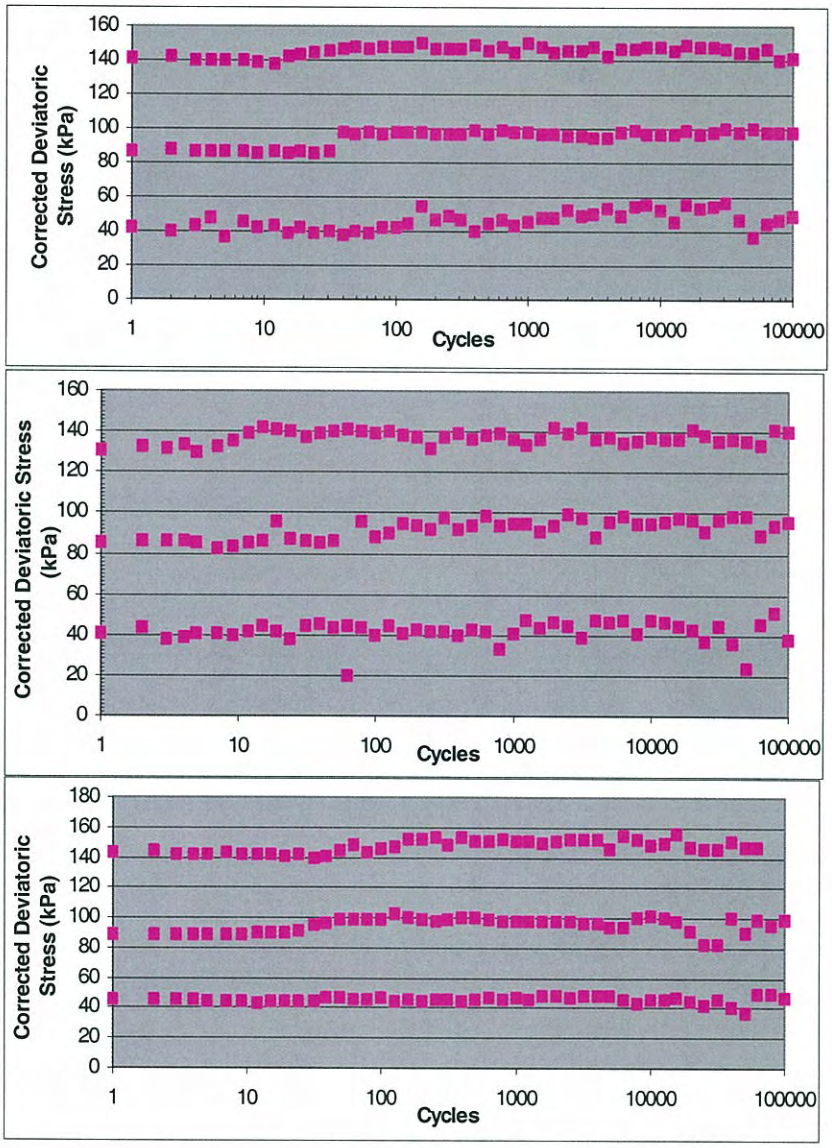
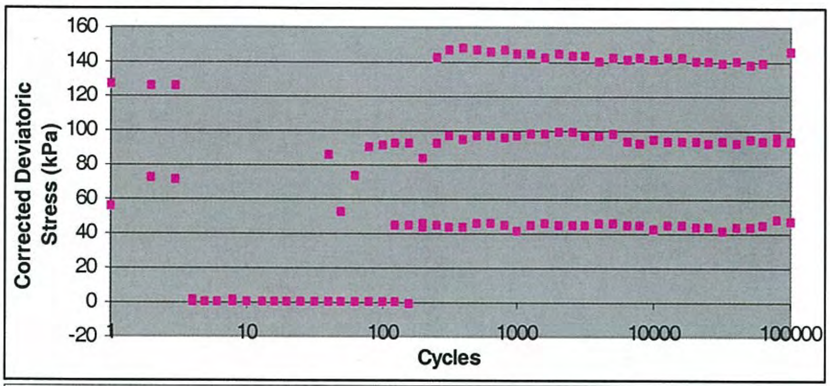


Figure 7.1 Observed Deviatoric Stresses.

Sanson



Eltham

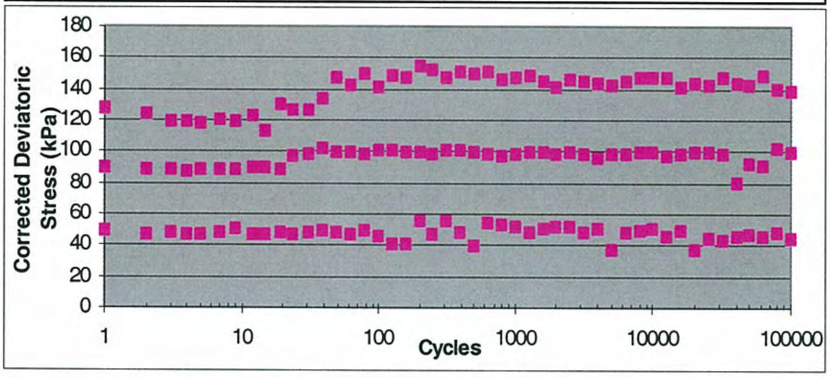
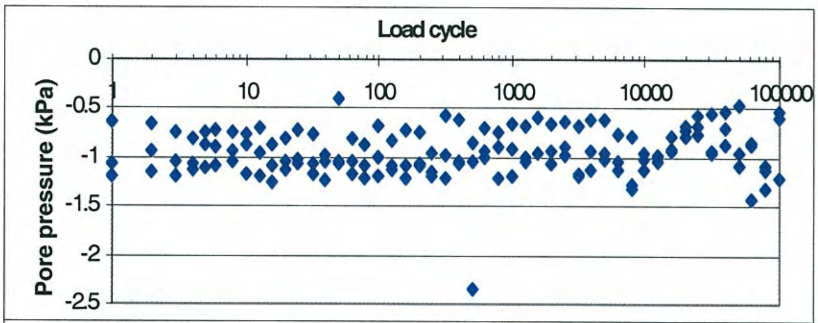


Figure 7.1 (cont.) Observed Deviatoric Stresses

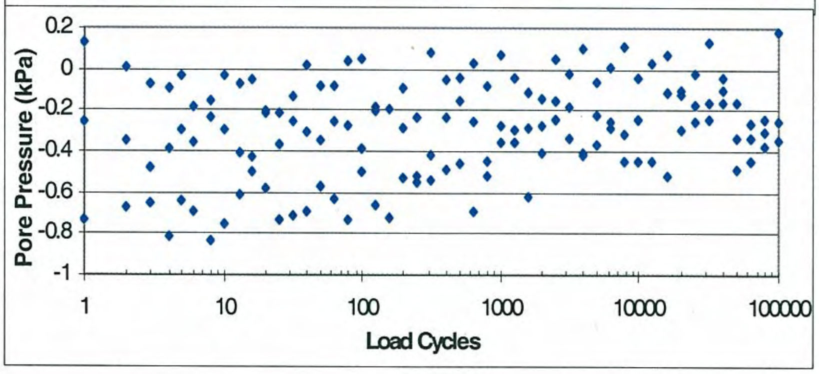
The pore pressures are shown in Figure 7.2. The very low pore pressures confirm that the tests were carried out under essentially drained conditions.

Figure 7.2. Observed Minimum Pore Pressures.

Chinaman's Hill



Mission Bay



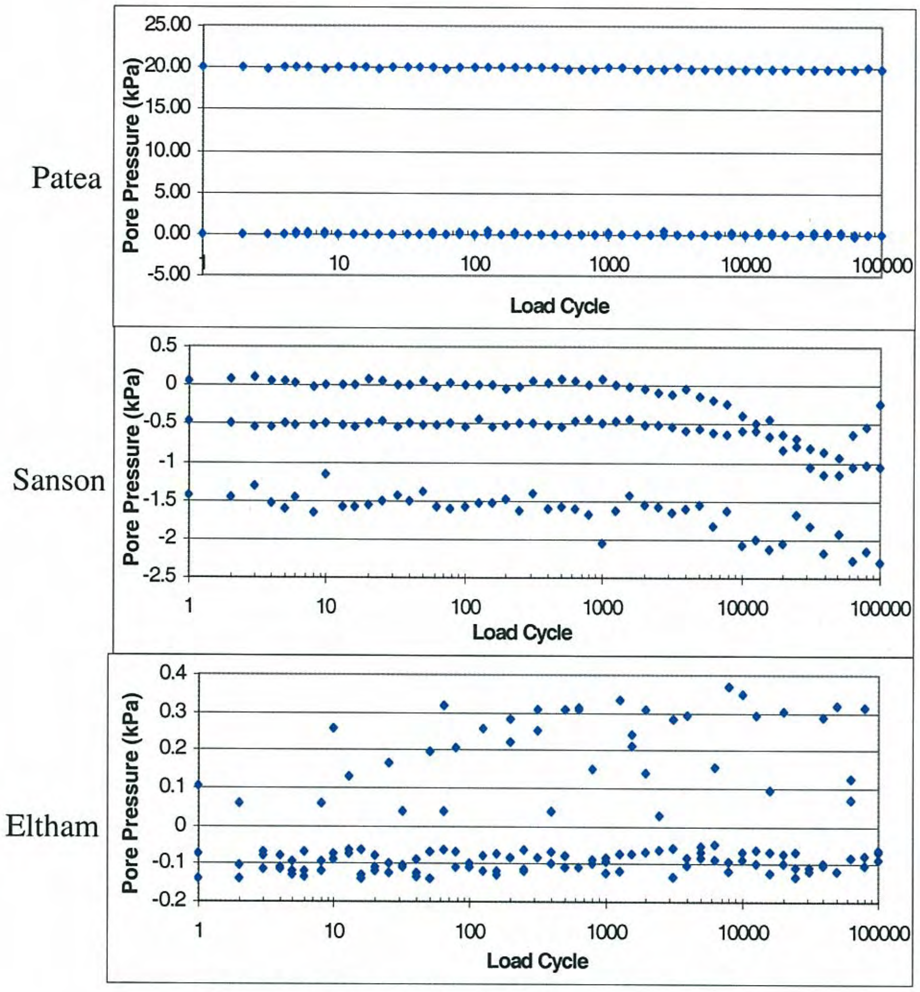


Figure 7.2 (cont). Observed pore pressures.

The resilient moduli for each test are shown in Figure 7.3. Resilient modulus is defined as the ratio of the corrected deviatoric stress to the resilient axial strain, and the resilient axial strain is calculated from the difference between the maximum and minimum deformation for each load cycle. Poisson's ratio was set at 0.35 for all samples.

Manual dial gauge readings of sample axial deformation were made at the end of a number of cycles during each test. These provide verification of the electronic readings logged during the test. Good agreement was found in all tests between the manual and electronic readings, except for the Eltham 50kPa sample. For this test it appears there was a setup problem of some kind and the first cycle resulted in an apparent expansion of the sample, followed by axial compression for all subsequent cycles. The manual and electronic measurements disagree, although the slopes of the graphs of permanent axial strain *versus* the logarithm of the cycle number are very close for both the manual and electronic readings. Thus we believe it was an initialisation problem and not a serious malfunction or material property that caused the apparent initial expansion in this test.

Chinaman's

Mission Bay

Patea

Sanson

Eltham

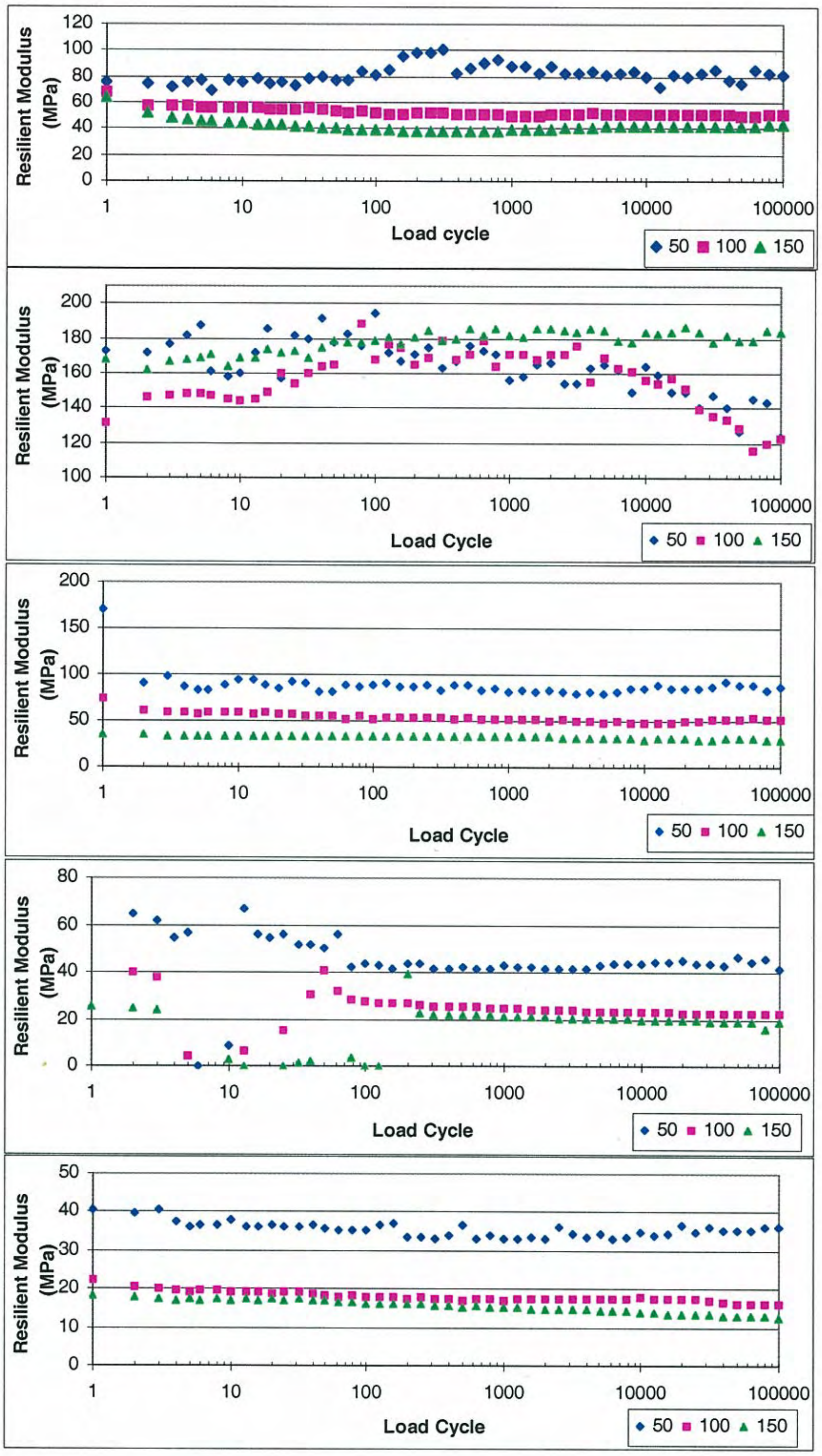


Figure 7.3 Resilient Moduli for each test.

For each nominal deviatoric stress level, the permanent axial strains are shown in Figure 7.4. In all cases the Eltham material strains are the greatest and the Mission Bay material strains the least.

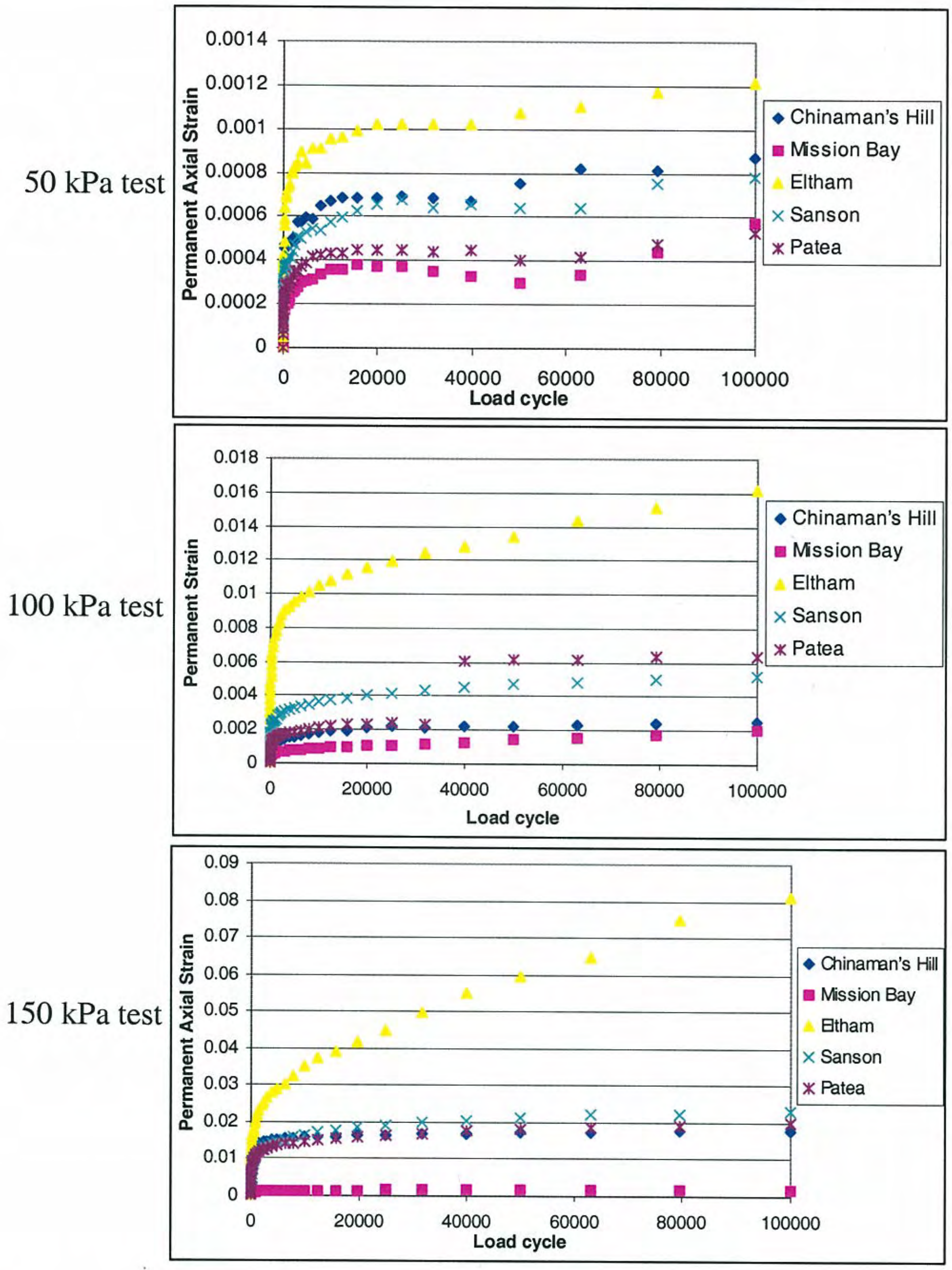


Figure 7.4 Observed Permanent Axial Strains (Note the scales vary).

It can be seen in the 100 kPa test for Patea that the permanent strain stepped up before load cycle 40,000. This is thought to be due to the operator noticing that the deviatoric stress was reducing and attempting to adjust it, but instead accidentally adjusting the cell pressure momentarily, resulting in rapid consolidation of the specimen. While the slope of the strain-load cycle plot for load cycles greater than 35,000 is similar to that immediately before the problem occurred, these cycles have been excluded from consideration in the analysis.

In the 50 kPa results in Figure 7.4 there seems to be a distinct plateau where accumulation of permanent strain ceases in all samples between approximately load cycles 20,000 and 40,000. This plateau is followed by renewed increasing strain in the Chinaman's Hill and Eltham samples. The plateau continues past 60,000 load cycles for the non-volcanic sample from Sanson and also for the Patea material. The Mission Bay material actually contracted for about 30,000 cycles, before strain began to increase again, at a rate greater than in any of the other samples. This plateau is not evident in either the 100 or 150 kPa test data. The 50 kPa load may be near the limit at which elastic shakedown occurs.

From the graph shown in Figure 7.5 it can be seen that the resilient modulus reduces for all samples, apart from the Mission Bay material at 150kPa, as the deviator stress is increased. The permanent strain, shown in Figure 7.6 increases for all samples as the deviator stress increases. It therefore seems that the permanent strain is, in general, directly related to the resilient modulus at 10000 cycles (inversely proportional).

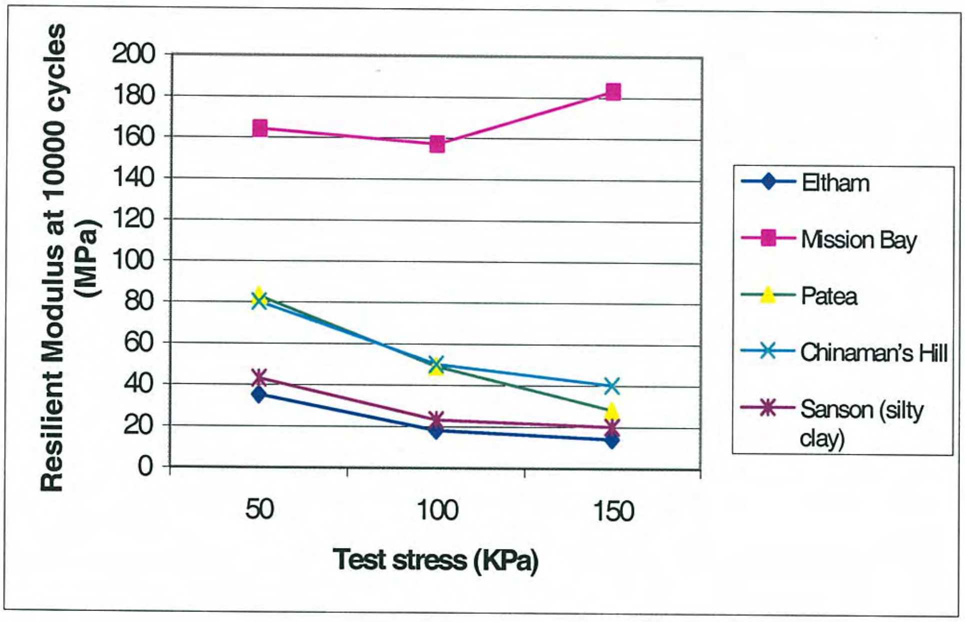


Figure 7.5 Resilient Modulus at 10000 Cycles versus Test Stress.

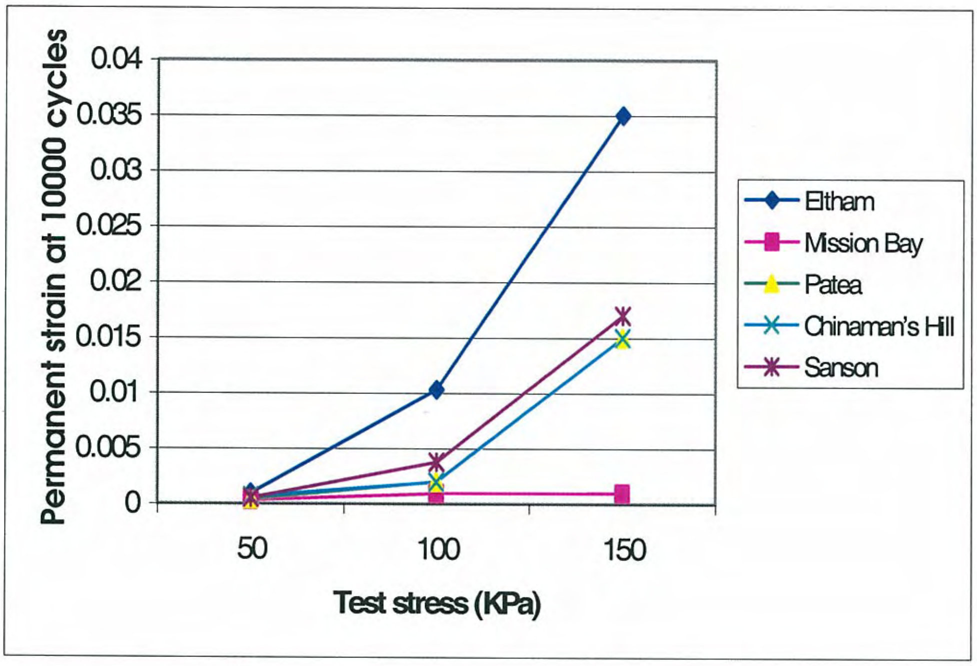


Figure 7.6 Permanent Strain at 10000 Cycles versus Test Stress.

The soil obtained from Sanson (the non-volcanic material) can be seen to follow a similar pattern to the volcanic soil from Chinaman's Hill and Patea, which were both classified as clay SILT and silty CLAY respectively similar classifications to the Sanson soil. The soil with the highest resilient modulus and lowest permanent strain was the Mission Bay material, which is to be expected as the soil is a gravelly SAND due to its generally high bearing capacity compared to silty clay. The Eltham material produces the lowest resilient modulus and consequently the highest deformation. Apart from the Eltham material, the Sanson material had, the highest permanent strain compared to the volcanic soils of Mission Bay, Patea and Chinaman's Hill.

8. Analysis

8.1 Comparison With Mathematical Models

To remove any initial bedding, seating and gauge initialisation problems from the numerical comparison, only data for load cycles 100 to 100,000 were used, with two exceptions. As stated in section 7.0; data for load cycles 35,000 and on were rejected from the Patea 100 kPa test. The other exception is the Sanson 150 kPa test, where the deviatoric stress was variable and usually near zero for up to about 250 cycles, so that data was also rejected from the model fitting. The permanent strain was set to zero at these nominal start cycle numbers. Multivariate polynomial regression using data from all three tests for each material, was used to determine the best-fit coefficients for each mathematical model and to determine the correlation coefficient R^2. The results are shown in Table 8.1. When each test is fitted individually, the fits are much better for most models, but this is of little value because the deviatoric stress varies in service conditions.

Table 8.1 Correlation coefficients R^2 for the mathematical models reviewed.

Sample	**Chinaman's Hill**	**Eltham**	**Mission Bay**	**Patea**	**Sanson (non-volcanic)**
Description / Soil Model	Silty CLAY: high plasticity	Silty SAND: high plasticity	Gravelly SAND: fine to coarse	CLAY-SILT: medium plasticity	Silty CLAY: medium placticity
Li and Selig (1996) (Eqn 6)	0.773	0.94	0.94	0.98	0.94
Ullidtz (1987) (Eqn 5)	0.84	0.98	0.96	0.96	0.95
Bayomy and Al-Sanad (1993) (Eqn 4)	0.96	0.99	0.98	0.99	0.98
Majidzadeh et al (1978) (Eqn 3)	0.77	0.97	0.35	0.99	0.92
Raad & Zeid (1989) (Eqn 7)	0.05	0.18	0.53	0.11	.17

Clearly, the Raad and Zeid (1989) model does not fit any of the test data very well and the Bayomy and Al-Sanad (1993) model has the largest correlation coefficient for all materials, although for the Patea material Majidzadeh et al (1978) appears to be equally as good as Bayomy and Al-Sanad (1993). Li and Selig, and Ullidtz give similar fits as expected, being essentially the same model apart from the selection of reference stress.

The coefficients fitted to each set of data by multivariate polynomial regression on the Bayomy and Al-Sanad model are shown in Table 8.2 and data fitted using this model is shown in Figure 8.1. The fitted data uses the observed load cycle number (set to zero at cycle 100), the observed deviatoric stress and the observed resilient modulus for that cycle. The fitted data shown in Figure 8.1 compares well with the test data, with the exception of the Eltham and the Sanson soils after a load cycle of 6×10^4. It seems that the fitted values for the Eltham soil after this point are under-predicted compared to the actual performance of the soil, which may be due to the model not being able to predict the soils high plasticity. The Sanson soils fitted data, however, is over-predicted by the model at cycles higher than 6×10^4. The model seems to have some difficulty in modelling the behaviour of the Sanson material, which is perhaps to be expected as it tends to produce better fits with the volcanic-type soils. From the coefficients in Table 8.2 it would appear that the Chinaman's Hill material is different from the others (for example, s is positive) and that the Eltham and Mission Bay materials appear to be quite similar. But in Figure 7.3 and Table 8.2 we see that the Mission Bay volcanics have resilient moduli some 4 to 7 times those of the Eltham material. Also, we see in Figure 7.4 that the permanent strain for the Eltham material is at least 10 times greater at 100,000 load cycles in the 150 kPa test.

Table 8.2 Bayomy and Al-Sanad (1993) Coefficients fitted to each material by multivariate polynomial regression.

Sample	Chinaman's Hill	Eltham	Mission Bay	Patea	Sanson (non-volcanic)
Description / Coefficient	Silty CLAY: high plasticity	Silty SAND: high plasticity	Gravelly SAND: fine to coarse	CLAY-SILT: medium plasticity	Silty CLAY: medium placticity
R	$3.234x10^{-10}$	0.727	0.532	$9.624x10^{-3}$	$1.187x10^{-4}$
S	2.397	-2.492	-1.797	-1.232	-1.038
C	0.051	0.014	0.019	0.023	0.03
B	0.139	0.163	0.087	0.132	0.358

Thus we conclude that while the differences between coefficients appear to be small, the model seems to be quite sensitive to these values.

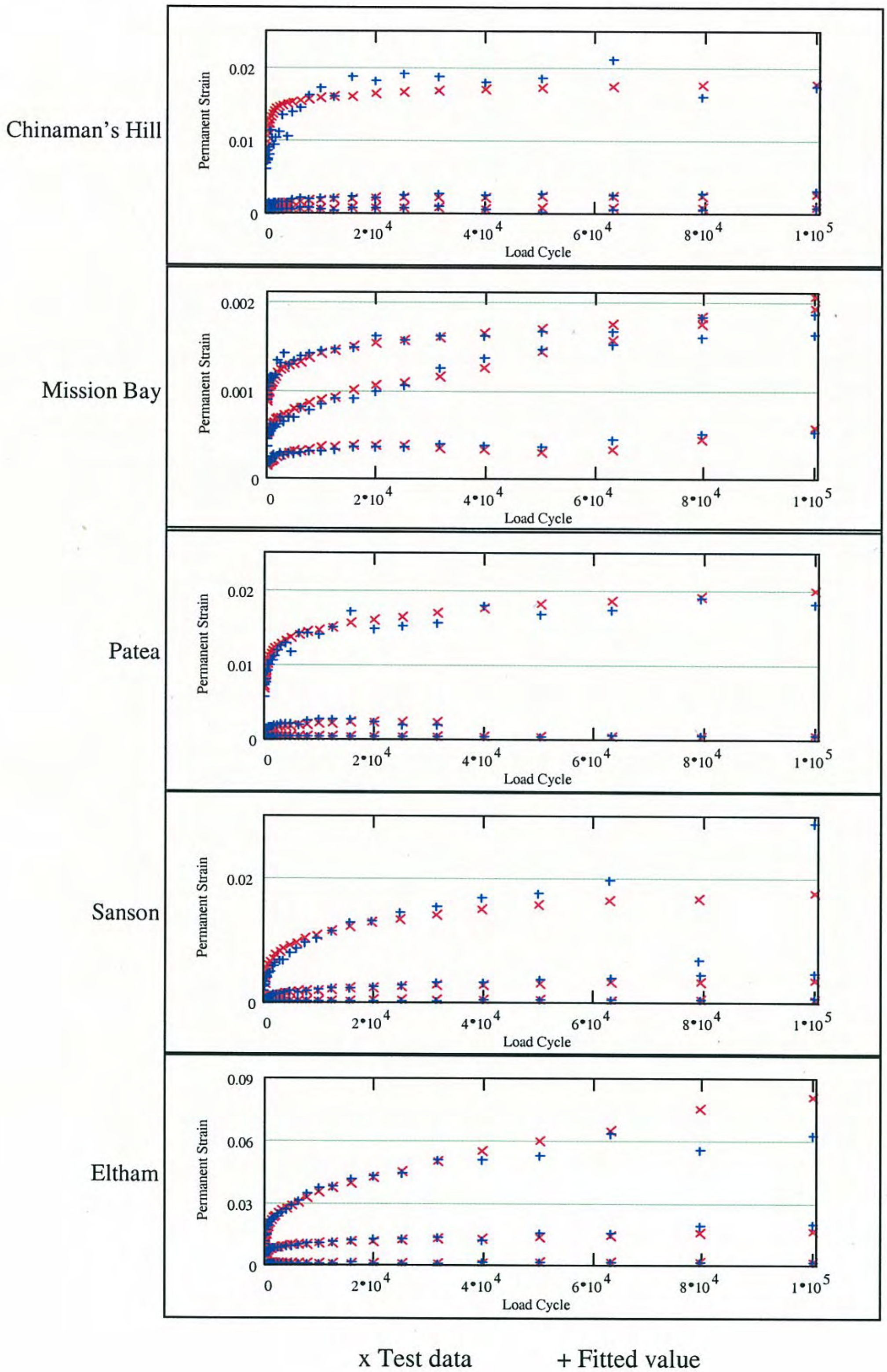


Figure 8.1 The Bayomy and Al-Sanad (1993) Model fitted to each material. (Note that the permanent strain scales vary.)

8.2 Comparison with Resilient Moduli Obtained from site tests

The resilient moduli obtained from the RLT tests, the FWD tests and the SASW tests are given in Table 8.3. The equivalent AUSTROADS isotropic modulus obtained from the CBR tests is 6.7×CBR, see section 3.2.2, the isotropic modulus was used, because the SASW values were based on the equivalent isotropic modulus. The RLT moduli are also shown in Figure 7.3 and in Figure 7.5 at 10000 cycles. The RLT tests show that the resilient modulus decreases as the repeated stress inducing it increases. With this in mind, we would then expect the SASW moduli to be the greatest because the deflections from which they are determined are the least, and this is indeed the case.

Table 8.3 Comparison of Resilient Moduli Derived from Various Tests. The RLT Values are Means for Each Test.

Site	RLT Modulus (MPa) at 50 kPa	RLT Modulus (MPa) at 100 kPa	RLT Modulus (MPa) at 150 kPa	FWD Modulus (MPa) (insitu)	SASW Modulus (MPa) (insitu)	6.7× CBR (MPa) (insitu)
Chinaman's Hill	85	51	40	65	108	101
Eltham	35	17	14	70	171	88
Patea	85	51	32	64	126	221
Sanson	43	24	19	48	109	34
Mission Bay	157*	158*	182	95	257/479	235

* Values fell away as the test progressed (see Figure 7.3).

The FWD values are more similar to the RLT values when compared to the CBR or the SASW calculated values, although there is some variability in the relationship. It is notable also that the RLT moduli for the non-plastic Mission Bay gravelly SAND are considerably higher than for the other volcanic soils, which are either medium or highly plastic. When examining the CBR calculated values, it appears that the AUSTROADS criterion (Modulus (MPa) = 6.7×CBR) for obtaining an isotropic resilient modulus from the CBR is less appropriate for volcanic soils than for silty CLAY's like the Sanson soil. It predicts greater modulus values for the volcanic soils than the Sanson (non-volcanic) soil when compared to the RLT values. However, the 'pattern', of the CBR values are similar to the test soils. The difference in values when compared to the volcanic soils may be due to the volcanic soils being sensitive to disturbance, which may have contributed to the resilient modulus in the laboratory being less than the *in-situ* tests. The SASW results are also a lot greater than the resilient modulus values possibly for the same reason. Therefore, in the field the FWD tests may under-predict the *in-situ* modulus, they are a good indication of the possible resilient modulus.

The 6.7×CBR values are similar for the Patea and Mission Bay materials. However, the Patea-materials permanent strain at 150 kPa deviatoric stress is ten times that for the Mission Bay material at 100,000 load cycles. It can also be observed that 6.7×CBR for the Eltham material is more than 2.5 times that for the Sanson material, but the permanent strain for the Eltham material accumulates most rapidly of all the materials tested. Similarly, the Eltham and Chinaman's Hill material 6.7×CBR s are similar, but at 100,000 load cycles the permanent strain in the Eltham material was more than four times that for the Chinaman's Hill material. Thus the CBR is not a good indicator of RLT test performance in volcanic materials, and is therefore not an appropriate property to characterise the repeat load performance in design of pavements on volcanic materials.

The RLT moduli seem not to be a reliable indication of field performance. The Mission Bay soil has the highest SASW modulus and the smallest permanent strain buildup, while the Eltham soil has the second highest SASW modulus and the fastest/greatest permanent strain buildup. The same relationship is also true for the FWD moduli for these two soils. The Chinaman's Hill and Patea SASW and FWD moduli are similar, as are their permanent displacements in the 100 and 150 kPa tests, but not in the 50 kPa test.

Within the RLT test itself, the resilient modulus is similar for the Eltham and Sanson materials, but permanent strain accumulates considerably quicker for the Eltham material. The resilient moduli for the Chinaman's Hill and Patea materials are the same for the 50 kPa test, but the Chinaman's Hill sample accumulated nearly 80% more strain in 99,900 cycles. In the 100 and 150 kPa tests, however, the permanent strains in these two materials are quite similar; they are also 10 to 20% less than for the Sanson material. (This statement is made assuming the slope of the Patea 100 kPa test strain/cycle number graph above 35,000 cycles represents the true situation at the correct confining pressure.) We note also that the Bayomy and Al-Sanad (1993) model contains the resilient modulus, and that its exponent s ranges from 2.4 to –2.5 for the four volcanic materials (see Table 8.2).

9. Summary And Conclusions

Back-analysis from FWD deflections of pavements on volcanic soil subgrades has suggested that using the AUSTROADS (1992) pavement design procedure leads to excessive pavement thicknesses over these soils (Sutherland et al (1997)). The basic AUSTROADS (1992) assumption is that there is a direct relationship between the elastic strain generated by the load and the pavement design life. Most importantly, it is assumed that this relationship is the same for all soil types. The review of the literature and an examination of the models for predicting plastic strain accumulation suggest that this assumption is incorrect.

Performance based specifications relying on deflection testing will also require quantification of the relationship between total strains generated by loading and plastic strain development. Without this quantification non-destructive deflection testing of the finished pavement cannot provide answers as to the long-term performance of a pavement.

There are a wide range of models available in the literature which relate dynamic strain to the rate of accumulation of permanent deformation in soils under traffic loading. The models are introduced in the form of accumulated plastic strain versus number of loading repetitions. There are underlying basic forms in each model. Changes in this basic form can be explained by shakedown theory which outlines four basic types of soil behaviour:

(i) purely elastic;

(ii) elastic shakedown;

(iii) plastic shakedown; and

(iv) ratchetting, or incremental collapse.

The changes between each type of behaviour are deemed to occur at various levels of stress. While describing the initial behaviour of the soil the theory neglects more complex issues such the build-up of pore water pressure under repetitive loading and the associated gain or loss of soil strength.

Numerical techniques using finite element methods which are capable of producing more accurate values of stress and strain in the soil are available. These methods have been incorporated into commercially available computer packages such as ILL-PAVE and MICH-PAVE. Because of the accuracy of such programs it is possible to use them in combination with the equations determined in the literature review. While producing better values of stress and strain than analyses using linear elastic methods, it is not possible to use numerical techniques with traditional design charts. The limiting vertical compressive strain values in traditional design charts are mainly based on the linear elastic back calculation of strains in failed pavements from assumed linear elastic models and are therefore not the actual strains occurring in the soil, but approximations based on linear elastic theory.

Further investigation into relationships between pavement performance and elastic strain is well worthwhile as it will lead to better performance prediction models and consequently less risk. Better performance models will allow:

- the prediction of the type and degree of distress expected under all conditions;
- the serviceability with age-history to be determined; and
- the effects of various maintenance strategies to be evaluated.

The current linear elastic model only allows a prediction to be made of the type of distress which may occur.

A sensitivity analysis using CIRCLY, which is part of the AUSTROADS (1992) design system, shows that care needs to be taken when assigning a degree of anisotropy to a model. Poisson's ratio, on the other hand, is less important to the outcome.

Four mathematical models have been identified as having particular promise: Li and Selig (1996), Majidzadeh (1978), Bayomy and Al-Sanad (1993) and Ullidtz (1987). There are two main reasons for their potential: firstly, they are simple, and secondly, they mirror what is observed in pavement rutting studies. While not specifically developed for volcanic soils it can be said that pavements on volcanic soils accumulate rut in the same manner as other more normal pavements.

As with non-volcanics, the resilient modulus of volcanic soils is load-dependent which maybe difficult to model. The Bayomy and Al-Sanad (1993) model better fits the relationship between the number of load cycles, the resilient strain (or modulus) and the permanent strain, it also includes a load term, which takes care of the preceding problem. Unpublished data from RLT tests conducted at Opus Central Laboratories also suggests that these models can be applied to volcanic soils.

RLT tests have been conducted on five soils, four of them from different types of volcanic soil in the North Island. During sampling, FWD, SASW and *in-situ* CBR tests were conducted and estimates of resilient modulus produced from the results. There is a large range, with the largest values derived from the SASW tests, which also have the smallest soil deflection during the test.

The data from the RLT tests was fitted using multivariate polynomial regression to the mathematical models identified earlier. The Bayomy and Al-Sanad (1989) model was found to best fit all materials, including the non-volcanic soil tested. However, the volcanic soils seem to vary somewhat, in that the coefficients derived for each for the Bayomy and Al-Sanad model have significant ranges. The Majidzadeh (1978) model fitted one of the volcanic soils equally as well, but was a very poor fit to one of the other volcanic soils. As expected, because they are very similar models, Li and Selig (1996) and Ullidtz (1987) had similar fits, with Ullidtz being slightly better, though still not as good as the Bayomy and Al-Sanad model. The correlation coefficients for the Bayomy and Al-Sanad model ranged from 0.96 to 0.99.

It was found that the resilient modulus (defined as the deviator stress divided by the elastic, resilient or recovered, strain) decreased as the load increased. Estimates of the resilient modulus from the 6.7×CBR determinations were greater, significantly in the case of the SASW results, than was observed during the RLT tests at the smallest load. Therefore it may be concluded that the *in-situ* CBR does not predict the resilient modulus very well. The FWD moduli were closer however, generally either larger than the smallest from the RLT (50 kPa) tests or between the 50 and 100 kPa load values.

When comparing similar resilient moduli obtained from 6.7×CBR the end-of-test permanent strain found in the RLTs differed by up to a factor of about ten. Therefore the *in-situ* CBR may not be a useful indicator of the build-up of plastic strain. Similar problems were found with the FWD and SASW moduli. One particular problem with these two moduli was that the soil with the second highest FWD and SASW modulus had the greatest plastic strain at the end of the tests. In the RLT tests, this soil had the smallest modulus in each test. It, therefore, seems that the RLT provides a good indication of the likely permanent strain, although the strain results will need to be compared to the performance of a 'real' pavement.

A number of factors may also need to be investigated when analysing or comparing RLT test results with those obtained *in-situ*, these are; the uncertain relationship between plastic strains in the lab and field performance of pavements; the relationship between the RLT test stress conditions and the subgrade stresses in service is ill-defined, and leads to uncertainty in the applicability of the test to pavement design; the possibility of sample disturbance from the extrusion on the road and the laboratory testing; the RLT test equipment is not widely available at the moment; and a RLT test takes considerably more time than other types of *in-situ* and laboratory tests.

10. RECOMMENDATIONS

The relationship between RLT tests and *in situ* pavement performance should be further researched in the literature and/or verified with experiments with accelerated loading facilities such as CAPTIF or ALF.

The literature review highlights the inaccuracies of using linear elastic analysis. Additional research should look at the possibility of moving away from elastic design and towards the finite element methods that have become significantly easier to use with the development of new computer software and faster hardware. However, additional work will need to be carried out to determine the appropriate design parameters for the finite element programmes.

The sensitivity analysis indicates that further work needs to be carried out to identify what effect the degree of anisotropy has on the stresses and strains calculated from computer programmes. Testing the degree of (if any) anisotropy in the field, possibly using instruments such as the Pressuremeter, would also be useful. This would lead to the need for modelling anisotropic conditions in the laboratory testing.

Testing using the RLT apparatus to determine the failure criteria for various volcanic soil types so that these can be developed and used in conjunction with AUSTROADS (1992) design is also desirable. This should allow for better understanding and prediction of pavement life. Intuitively, the RLT test would appear to be suitable because it emulates, in the laboratory in a comparatively short time, the longer term build-up of permanent strain from repeated loading. However, comparisons between the test results and observation, of the permanent strain of the pavements *in situ* would also need to be conducted.

Further investigation (possibly included in the above tests) of the use of the RLT test and the Bayomy and Al-Sanad (1993) strain accumulation model in pavement design should be carried out. Together with investigating other methods of determining the relationship between the resilient modulus and other relatively easily obtainable material properties. Particular problems to be resolved include the relationship between permanent strain build-up in the test, rutting under service conditions and the apparently highly variable strain accumulation properties of the volcanic soils, as evidenced by the Bayomy and Al-Sanad coefficients. Other volcanic soils should be tested as well.

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