SASW – A Method to Determine Pavement Composition & Strength

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An Important Note for the Reader

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Contents

Contents (continued)

List of Figures

List of Tables

Glossary

AC	Asphaltic cement
ARRB	Australian Road Research Bureau
CBR	California Bearing Ratio
FWD	Falling Weight Deflectometer
GPR	Ground Penetrating Radar
HDM-4	Highway Design & Maintenance Standards Model, version 4 (World Bank)
SASW	Seismic Analysis of Surface Waves
SN	Structural Number
SNC	Modified Structural Number
SNP	Adjusted Structural Number
UCS	Unconfined Compressive Strength
dTIMS	Deighton's Total Infrastructure Management System (software)
ρ	Mass Density
ν	Poisson's Ratio
λ	Wavelength
E	Young's Modulus
G	Stiffness
K	Bulk Modulus
V_s	S-wave Velocity
V_p	P-wave Velocity
μ	Shear Modulus

Executive Summary

Introduction

This report documents the investigation into the application of SASW (Seismic Analysis of Surface Waves) to measure the layer thickness and material moduli of pavements in order to assess pavement strength. It was carried out on roads in New Zealand in 2003.

The following work was carried out in the study:

- Comparison of FWD (Falling Weight Deflectometer) and SASW results;
- Comparison of test pit information and SASW results;
- Investigation of applying the technique within the New Zealand roading industry;
- Assessment of the economic feasibility of collecting pavement strength data;
- Preparation of initial guidelines for the use of SASW on New Zealand road pavements.

The use of sophisticated asset management tools (such as HDM-4 and dTIMS) requires comprehensive information about the existing road networks. Using existing "pavement and condition" data and deterioration models to predict road performance supplements the experienced engineer's judgement, and provides indications of long-term maintenance and budget requirements at a network level of management. In addition, non-destructive methods of evaluating the properties of existing pavements for design at project level are useful where destructive testing is not possible.

Overview of SASW

SASW is a well established, non-destructive technique for the characterisation of subsurface geotechnical properties. Its potential to describe the subsurface properties of road pavements was investigated in this study by testing it on 13 sites on state highways, in Auckland City and Thames-Coromandel District, where test pit and FWD data were available.

Using SASW the following information was obtained:

- shear moduli at different depths;
- the average shear modulus of the basecourse;
- layer thickness information;
- a continuous longitudinal profile of the moduli showing the variability within a pavement structure.

Results of Investigation

SASW showed reasonable correlation with the layer information from the test pit data and some qualitative agreement with FWD.

The shear modulus measured in the basecourse showed a trend between bound, unbound and improved base layers.

The results showed the potential of the system to assist in the understanding of pavement performance at both project and network levels. They also demonstrated that the system could be combined with FWD surveys to make SASW cost effective and to contribute towards using it to make more robust estimates of pavement properties.

Economic Evaluation of SASW

An economic evaluation of SASW and other techniques used in New Zealand for pavement testing and design was carried out to determine the potential benefits of improved pavement strength knowledge. The analysis was carried out at both network and project levels.

Network Level – The dTIMS asset management tool was used to simulate the actual budget requirements for a road network over 20 years, for overestimated and underestimated pavement strengths on that network. The network was assumed to have an optimum design SNP (Modified Structural Number) that had been assigned based on the traffic volume on each road.

If the estimated SNP was 30% lower than the actual, then the road controlling authority cost (including routine and periodic maintenance type treatments) was overestimated by $365 (approximately 3.1% of the total agency costs) per year per km. Overestimating the SNP in this scenario did not result in significant budget differences. However, if the network was generally composed of "weak" pavements, overestimating the SNP would have been significant.

Project Level – Various material and design parameters were tested for sensitivity in the design process for a chipseal with a flexible, granular basecourse, at project level. These parameters were subgrade CBR, existing pavement layer thickness, basecourse modulus, and design traffic loading.

Overestimating the subgrade CBR and existing pavement thickness was found to dramatically reduce the theoretical life of the pavement (by <50%) and hence the funding agency costs increased substantially. Overestimating the base modulus had less severe consequences and traffic volume was the least sensitive design parameter.

These two examples demonstrated the effects of using material or pavement strength parameters that are not representative of a network or existing road. Consequently, some benefit is to be gained by improving our knowledge of pavement strength at both network and project levels.

Conclusions & Recommendations

SASW will only be an economically viable option at the network level if it is developed into a measurement that can be made while on the move. At present, SASW is static, requiring setting up of equipment, which is a major time-consuming limitation.

Further developments required for SASW in order to use it in pavement design include:

- developing rolling type measurements for network level surveys possibly in combination with FWD surveys;
- improving measurement resolution;
- improving data interpretation;
- using SASW in conjunction with existing methods, to supplement FWD data, to improve pavement strength information;
- developing SASW as a completely different system for measuring pavement strength.

An improvement in the analysis procedure is also possible. For instance, the development of an algorithm to estimate pavement layers from the raw data would be beneficial in trying to model the pavement structure.

In conclusion, this study has shown that SASW (Seismic Analysis of Surface Waves) has the potential to be of significant benefit in the estimation of pavement strength, subject to the development of procedures that will enable more rapid collection and analysis of data.

Abstract

An investigation into the application of SASW (Seismic Analysis of Surface Waves), to measure the layer thickness and material moduli of pavements to assess pavement strength, was carried out in 2003 on roads in New Zealand. SASW is a well established, non-destructive technique for the characterisation of subsurface geotechnical properties.

Its potential to describe the properties of road pavements was investigated by testing it on 13 sites on state highways where test pit and FWD data were already available. Using SASW the following information was obtained:

- shear moduli at different depths;
- the average shear modulus of the basecourse;
- layer thickness information;
- a continuous longitudinal profile of the moduli showing the variability within a pavement structure.

An economic analysis of SASW and other techniques used in New Zealand for pavement testing and design was also carried out to determine the potential benefits of improved pavement strength knowledge, at both network and project levels.

1. Introduction

1.1 Background

With much of the New Zealand road network now well developed, the emphasis has shifted from the construction of new roads to the maintenance and improvement of existing networks. The maintenance of existing road networks involves both network level planning (i.e. the maintenance requirements of the road network for an acceptable level of service), and project level design (i.e. the detailed design of a rejuvenation treatment to address specific failures on a piece of road).

A common thread to both levels of planning is that network managers and project designers both need adequate information or data about the existing roads to make the correct decisions as to the best maintenance requirements or rejuvenation techniques.

Collection of road condition data has become a sophisticated process that utilises a number of innovative processes to measure the various parameters used to describe a network's condition and to assess its capacity to carry future traffic. As expected, when deciding on the measurement and testing techniques to be employed, a cost is associated with this data collection and testing process, and the relative benefits must also be taken into consideration. Consequently, the tendency is to steer developments into making these techniques more available and affordable.

This report documents the investigation carried out on New Zealand roads in 2003 into an alternative method to measure the pavement layer thickness and material moduli in order to assess pavement strength. The method is SASW or Seismic Analysis of Surface Waves, which is a well established, non-destructive technique used for the characterisation of subsurface geotechnical properties.

1.2 Explanation of the Problem

Currently, Falling Weight Deflectometer (FWD) testing is supplemented with test pit information for detailed project design purposes only. On a network level the FWD tests are mostly used without layer thickness information available. This decreases the ability to reliably estimate pavement strength from the FWD data since it is necessary to make assumptions on the total layer thickness and in-situ CBR. Tonkin & Taylor (2001) recommended that the additional accuracy obtained from supplementing FWD with accurate test pit information does not warrant destructive test pit testing of the network.

Furthermore, there are some limitations to the interpretation of FWD information on certain material types in New Zealand. For example, high deflections on volcanic soils suggest poor performance of these materials (i.e. low SNP), but in practice these materials have been found to perform adequately as a subgrade material because they recover well from loading and accumulate irrecoverable strain slowly.

The SASW system has the potential to overcome most of the limitations at comparable costs. However, the system has not previously been used in New Zealand to estimate pavement strength. Consequently there is a need to determine the applicability of this technology to New Zealand conditions.

1.3 Purpose of Study

The main objective of the study was to investigate the applicability of the SASW technique to New Zealand conditions for estimating pavement strength. The work that was carried out for the study included:

- Comparison of FWD and SASW results;
- Comparison of test pit information and SASW results;
- Investigation of applying the technique within the New Zealand roading industry;
- Assessment of the economic feasibility of collecting pavement strength data;
- Preparation of initial guidelines for the use of SASW on New Zealand pavements.

1.4 Study Methodology

The methodology for this study is summarised in the following sections.

1.4.1 Task 1 – Test Sites with SASW system

SASW surveys were undertaken on 19 test sections located on roads around the North Island of New Zealand (Appendix A, Table A1) between January and April 2003. The networks used for this study included:

- State Highways,
- Auckland City, and
- Thames-Coromandel District.

Test pit and FWD investigations were done on these sections by geotechnical laboratories, to allow comparison with the SASW measurements.

SASW measurements were performed using an 8 x geophone spread with a falling weight (ball peen hammer) used as a source of the seismic waves. Measurements were optimised to ensure that wavelengths of the order of the pavement layer thickness were generated and measured (Section 4.3).

1.4.2 Task 2 – Analyse SASW Results

Data was analysed using standard SASW techniques, to yield surface wave velocity (which closely approximates shear wave velocity) as a function of wavelength. The known pavement layer thickness at some sites from test pits has been used to provide a wavelength/depth calibration factor. Shear velocity was converted to shear modulus using the standard equation (Equation 2, Section 3.2).

Shear modulus as a function of depth was directly compared with modulus of subgrade reaction (as inferred from FWD measurements) on a site-by-site basis.

Key issues addressed were:

- Applicability and limitations of SASW in determining pavement layer thickness;
- Applicability and limitations of SASW in determining shear modulus as a function of depth; and
- Range of applicability of SASW as compared to FWD measurements.

1.4.3 Task 3 – Compiling Existing Data on Sites

The existing data on the sites included:

- FWD tests measured at 50 m intervals;
- One to two test pits per test site that included the measurement of layer thickness, material type, and in-situ subgrade CBR;
- Laboratory tests on test pit samples from the sections of state highways.

1.4.4 Task 4 – Comparative Analyses

A comparison was made between the test pit information, FWD results and the SASW results. The intention was to establish the robustness of the SASW measurements and the potential use of the SASW at network and project level pavement management. The comparative analysis also included a cost comparison between the different testing methods.

2. Measuring & Modelling Pavement Strength

2.1 Introduction

The basic principle in pavement construction is the protection of the natural ground (referred to as the subgrade) from the loads imposed on it by traffic. The subgrade will quickly deform if it has no cover. This cover comes in the form of high strength materials, such as gravel, placed in layers on top of the subgrade, to distribute the traffic load and limit deformation of the subgrade.

Consequently, the most important considerations in pavement strength are:
- the strength of the existing subgrade;
- the strength and thickness of the materials above the subgrade.

Knowledge of the existing subgrade strength is important because a weaker subgrade requires a greater strength and/or thickness of overlying material to protect it from deformation.

Determining the pavement structure of an existing road gives an indication of the ability of a road to withstand future traffic loading.

2.2 Pavement Strength

The strength of the pavements on an existing road network is an integral part of determining the future maintenance needs and budget requirements in the management of that network. On a project level, this information will tell us what rejuvenation work needs to be done to an existing road to ensure it performs well during its future design life.

Indicators of pavement performance include:
- Roughness,
- Cracking (initiation and propagation),
- Rutting,
- Potholing, and
- Shoving.

The following two sections describe the use of pavement strength as a parameter in network management and project level design.

2.2.1 Network Level Requirements

In the World Bank's Highway Design and Maintenance Standards Model (HDM-4), pavement strength is modelled as SNP (Adjusted Structural Number). This is an advancement of the SNC (Modified Structural Number), which was found to overestimate the contribution of subgrade and sub-base layers to the pavement strength in deep pavements (over 700 mm) (Salt & Stevens website). SNP is a single number, usually between 1 and 6, and is a measure of how the different layers in the pavement contribute to the stiffness of the pavement.

This number is primarily dependent on the strength of the subgrade, and the thickness and material properties of the pavement layers above it.

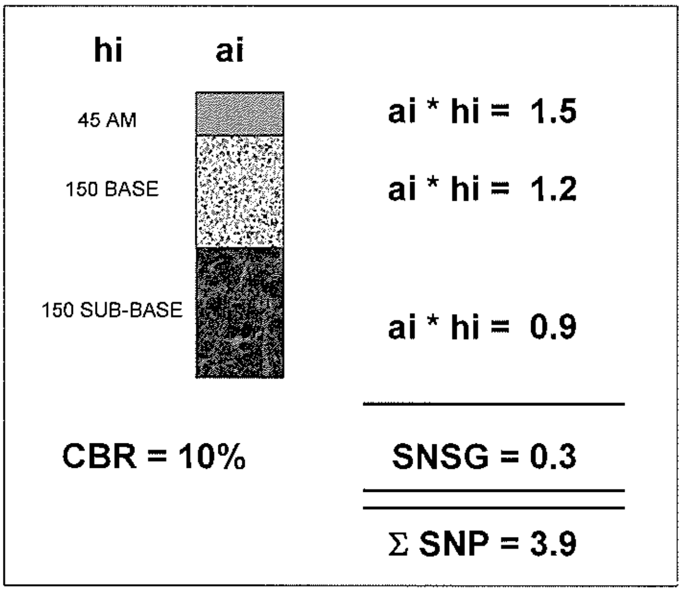

Figure 2.1 Calculating pavement strength.

$$SNP = \Sigma^{n}_{1}\, a_i\, h_i + SNSG \qquad \text{Equation 1}$$

where: a_i is the strength coefficient of the i^{th} layer as defined by Watanada et al. (1987)

h_i is the thickness in millimetres of the i^{th} layer provided that the sum of h_i is less than 700 mm

n is the number of pavement layers

SNSG is the modified structural number contribution of the subgrade, given by:

$$SNSG = 3.5\ \log_{10}CBR - 0.85(\log_{10}CBR)^2 - 1.43 \text{ (Tonkin \& Taylor 2001)}$$

Figure 2.1 demonstrates how the various layers are calculated to contribute to the pavement strength in the calculation of SNP. Equation 1 summarises the calculation.

In New Zealand, SNP is currently firmly entrenched in pavement deterioration modelling as the primary indicator for modelling pavement strength in network management. Analysis of the SNP values of the roads on a network gives a broad indication of the condition of a network and the rate at which pavement decay will take place.

However, a single number cannot fully describe the strength of a pavement, or the complex nature of its constituent materials and their interactions. It follows therefore that SNP is not always sufficient to describe the ability of a pavement to withstand repeated rolling wheel deflections (i.e. traffic), or its likely mode of failure.

The example in Section 6.2 of this report demonstrates the financial implications of underestimating or overestimating the SNP number.

2.2.2 Project Level Requirements

At a project level, testing is carried out on an existing road in order to design a suitable pavement for future traffic loading. Pavements are usually rejuvenated because they are showing signs of distress (e.g. high roughness, rutting). Knowledge of the existing pavement structure helps to determine what treatment should be applied to it in order to build a pavement that has sufficient structural capacity. This is an important economic consideration. Pavements that are under-designed will show signs of early failure and require high maintenance or, at worst, complete reconstruction. Pavements that are over-designed are a waste of economic resources.

Generally, more detailed testing is carried out at project level because the testing is relatively inexpensive compared to the capital cost of the project, and the benefits gained from it are widely acknowledged.

The example in Section 6.1 (Table 6.2) of this report demonstrates the financial implications of inadequate pavement testing and investigation on an existing unbound granular pavement that is due for rejuvenation.

2.3 Common Modes of Failure in Pavements

In terms of structural composition pavements can be categorised as *unbound* or *bound* pavements.

2.3.1 Unbound Pavements

These pavements are made up of a granular base and a sub-base overlying the subgrade. The primary mode of failure is in the subgrade material and can be regarded as an accumulation of the irrecoverable strain in the subgrade after repeated traffic loading. Rutting is a sign that a pavement is experiencing this mode of failure.

Granular pavements can also fail in the top basecourse layer by *shallow shear* if the basecourse material is not suitable, i.e. low shear strength of basecourse material, low hardness. When this is the case shoving, comprising lateral movement with edge heave, may be visible.

Lime-stabilised pavements are regarded as unbound pavements. The lime stabilisation is modelled as an improvement, not as a binding material. This is because during the initial stages of lime stabilisation the clay particles will react with the lime to improve the material qualities such as the workability and bearing strength. Cementation only occurs if sufficient lime is added to complete the initial physio-chemical reactions in the clay fraction and will continue to take place over several years (Ballantine & Rossouw 1999).

2.3.2 Bound Pavements

Concrete and structural Asphaltic Concrete pavements are the most common types of bound pavements. An Asphaltic Concrete pavement is considered to have failed when the damage, as a result of traffic loading, exceeds the design limit for either subgrade strain (rutting) or fatigue (asphalt distress) (Transit New Zealand 2000). Concrete pavements are usually deemed to have reached failure when they have cracked beyond their useable service state.

2.4 Standard Pavement Testing Techniques

Standard pavement testing techniques can be categorised as *destructive* or *non-destructive*. Destructive techniques include test pits (with laboratory measurements of material properties and Scala Penetrometer testing in the field). Non-destructive techniques include Falling Weight Deflectometer (FWD), Benkelman Beam and Deflectograph.

2.4.1 Destructive Techniques

2.4.1.1 Test pits and laboratory measurements

Test pits and laboratory testing of in-situ materials provide the best indicators of pavement structure and material properties. The results provide a direct measurement of layer thickness and visual record of material type at a point in the road. Samples taken from test pits can be laboratory tested for a range of moduli, which include (de Beer et al. 1997):

- Elastic Moduli (both uniaxial and triaxial),
- Resilient Moduli (both uniaxial and triaxial), and
- Strength (compressive and shear).

Testing is also done in the laboratory to find other material properties that affect pavement performance such as:

- Plasticity Index (a measure of the moisture content range where a material exhibits "plastic" behaviour),
- Hardness (resistance to crushing),
- Grading (distribution of particle sizes),
- California Bearing Ratio (CBR, resistance to deformation under loading),
- Weathering Resistance (ability to withstand environmental effects),
- Optimum Moisture Content (OMC, determines the water content at which maximum compaction can occur), and
- Unconfined Compressive Strength (UCS, load in kPa required to crush a sample).

The wide range of tests that are used to assess the suitability of a material for road construction purposes gives an indication of the complexity of the interaction between the materials within the pavement layers and their performance subsequent to loading and exposure to the environment.

Subgrade CBR and depth to subgrade are very important parameters in determining pavement strength. Currently these can only be measured accurately with destructive testing techniques.

The localised nature of a test pit means that the results may be skewed by inappropriate or unlucky siting of a test pit. Laboratory tests also test materials that have been disturbed and are not always an indication of how the material is performing in the pavement structure. For example, moisture content will vary seasonally in the field and inter-layer friction is not measured when considering individual material types. Both these parameters have a significant impact on pavement performance. It is, therefore, of great benefit to carry out testing in the field where the materials are in their "working" state.

2.4.1.2 Scala Penetrometer (In-situ CBR measurement)

Scala Penetrometer testing is carried out in the field on the subgrade to measure the resistance to penetration of a material, from which the California Bearing Ratio (CBR) is inferred. The CBR value is used as a direct input into the Adjusted Structural Number (SNP) calculation used in pavement modelling and for detailed design purposes.

The CBR value is a measure of the bearing capacity of a material. This should not be confused with the modulus, which is a measure of the elastic behaviour of a material under loading. Direct comparison with modulus-based methods (e.g. FWD) therefore relies on the establishment of an empirical relationship between elastic and plastic deformation.

Scala measurements can show significant variation on a specific site, implying that several measurements may be required to attain sufficient statistical significance. In addition, significant systematic error due to material type may be encountered, resulting in over- or underestimation of CBR (Patrick & Dongal 2001).

2.4.2 Non-Destructive Techniques

2.4.2.1 Falling Weight Deflectometer

The FWD (Figure 2.2) uses a number of geophones to determine the radial deflection arising from the impact of a falling weight. This weight is designed to apply a surface stress of around 450-700 kPa (depending on requirements). The shape of the deflection bowl measured by the sensors (located up to 2000 mm from the load point) and the magnitude of the deflections can be used to determine the stiffness of the underlying pavement.

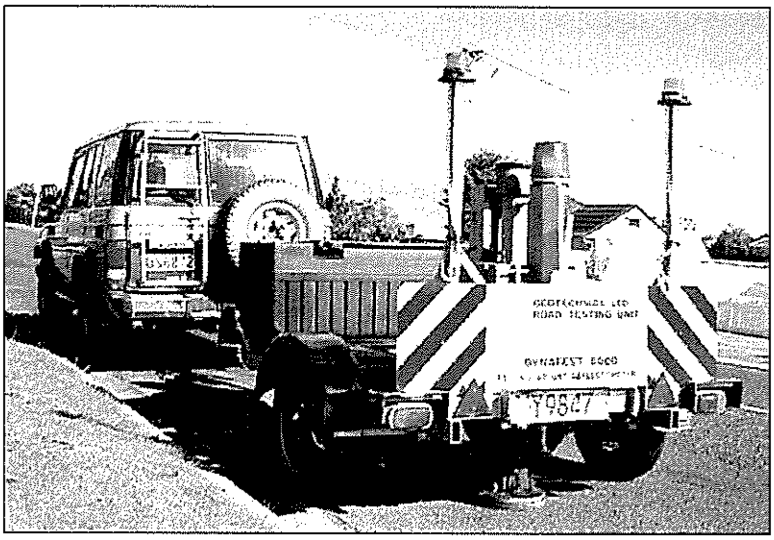

Figure 2.2 An FWD testing vehicle (photo supplied by Tonkin & Taylor).

For example, a broad bowl with little curvature indicates that the upper layers are stiff in relation to the subgrade (Figure 2.3). A bowl with the same maximum deflection but high curvature around the loading plate indicates that the upper layers are weak in relation to the subgrade (Tonkin & Taylor website).

Figure 2.3 shows some typical deflection values from actual FWD testing and demonstrates the deflection bowl shape.

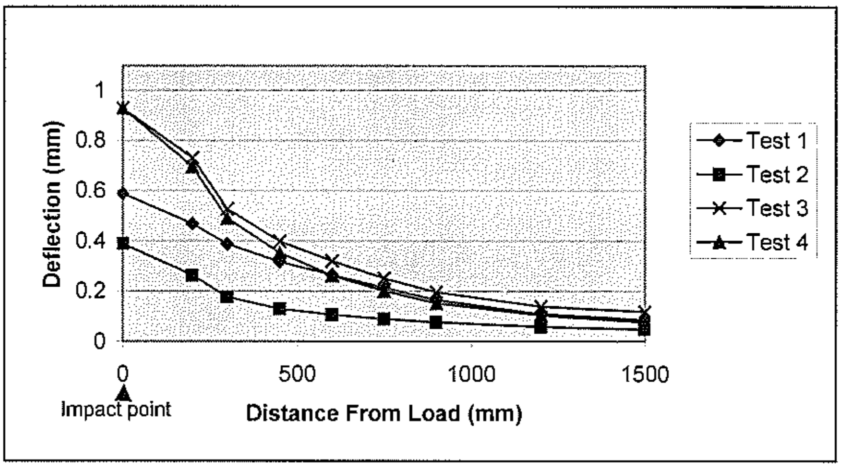

Figure 2.3 Graph of typical deflections measured by FWD testing.

Because the geophones are at varying distances from the load, the resulting deflection bowl arising from the impact provides a range of measurements at different applied strains. Consequently the FWD (in principle) can be used to determine the strain-dependent stiffness of a pavement. This is important because the materials in a pavement are not linearly elastic. For example, the modulus of an unbound granular basecourse layer will increase under loading, whereas the modulus of a clay subgrade will generally decrease during loading (Austroads 2000).

Back-calculation of the layer moduli from the deflection bowl yields stiffness as a function of depth and layer type. However FWD does not provide direct information on layer thickness (Martin & Crank 2001).

Correlation of FWD modulus measurements with CBR measurements can be attempted empirically, and is material- and site-dependent (Patrick & Dongal 2001).

2.4.2.2 Benkelman Beam

The Benkelman Beam test procedure involves the measurement of a pavement surface rebound with a cantilevered beam when a loaded axle moves from rest. Measurements are made between the dual tyres at specified intervals (Addo 1997). The Benkelman Beam has been largely superseded by Deflectograph surveys and FWD measurements (Martin & Crank 2001).

2.4.2.3 Deflectograph

Various Deflectograph type measurement systems are in existence. All rely on moving measurements of the deflections induced on a pavement by a loaded axle. Measurements can be done from a moving platform (Martin & Crank 2001), and the Deflectograph has a daily survey capacity of about 20 km.

Figure 2.4 A Deflectograph testing vehicle *(from www.wdm.co.uk).*

2.4.2.4 SASW

Seismic Analysis of Surface Waves (SASW) is a widely used technique in the general characterisation of subsurface geotechnical properties. Applications include soil improvement and bedrock delineation (Foti 2000 gives a general description of the technique in non-pavement applications).

In principle, SASW should be suitable for measuring layered media such as those encountered in pavements. Also, SASW provides both a shear modulus and a layer thickness. Shear moduli determined by SASW have been compared to those calculated by other seismic methods, including cross-hole seismic cone penetration tests, and found to be reliable (Addo & Robertson 1992).

The application to pavement layer measurement is, however, complicated by the fact that pavements disperse inversely (i.e. the stiffer layer is closest to the surface). Nevertheless, significant work has been done on the application of SASW to pavement analysis, which has resulted in the development of the Spectral Pavement Analyzer (Addo 1997), and the Seismic Pavement Analyzer (Nazarian et al. 1993, Yuan et al. 1999). Figure 2.5 shows the Spectral Pavement Analyzer, which has been developed to fit on a small trailer for towing by a passenger vehicle.

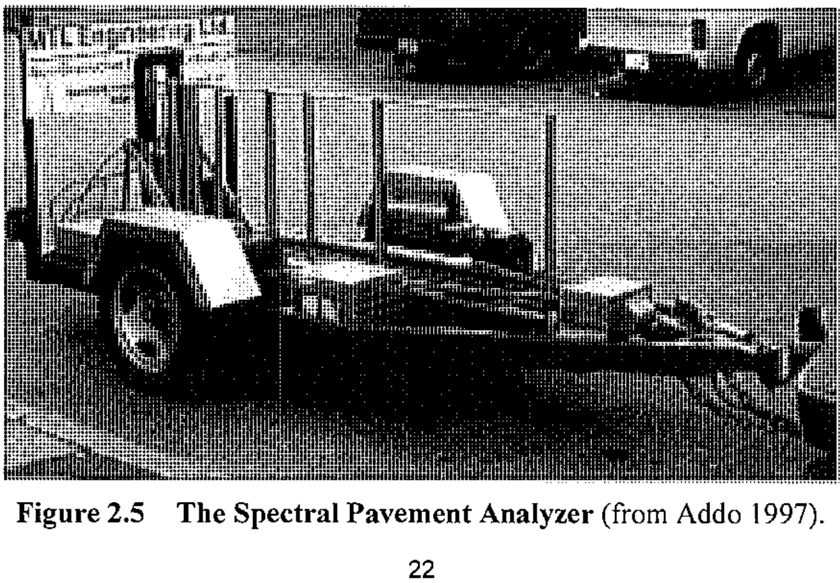

Figure 2.5 The Spectral Pavement Analyzer (from Addo 1997).

Figure 2.6 is a schematic diagram showing the set-up of the Seismic Pavement Analyzer.

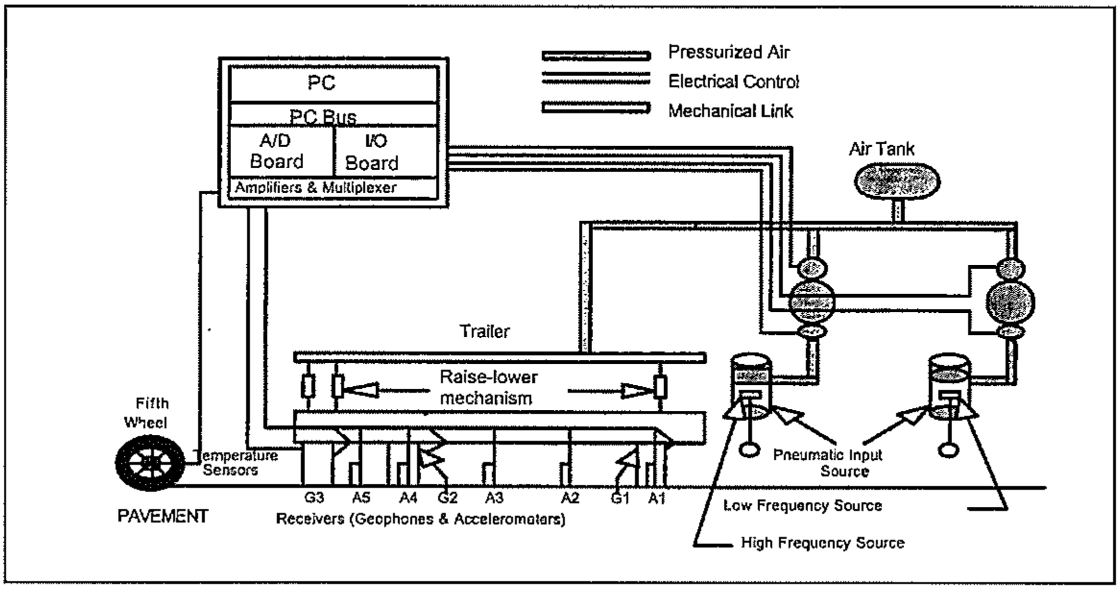

Figure 2.6 Diagram of the Seismic Pavement Analyzer (from Yuan et al. 1999).

Note the use of two different sources (at high and low frequencies) for inducing the seismic waves and that the receivers use both geophones (lower frequency sensitivity) and accelerometers (higher frequency sensitivity).

2.4.2.5 Ground Penetrating Radar

Ground Penetrating Radar (GPR) operates by transmitting short pulses of electromagnetic energy into the pavement. The waveform of the reflected energy contains a record of the properties and thickness of the layers within the pavement. GPR has been found to be accurate in estimating the thickness of the asphalt layer in pavements (Maser 1994). The readings are heavily influenced by the moisture content of the subsurface material.

2.4.3 Benefits and Limitations of these Measuring Methods

Table 2.1 summarises the benefits and limitations of SASW, FWD, test pits, laboratory testing and GPR.

Table 2.1 Summary of benefits and limitations of the testing methods investigated for this report.

Testing Method	Testing method benefits	Testing method limitations
SASW	Tests pavement in its working environment Non-destructive Direct measure of material property	Not well established; Still under development; Slow
FWD	Test loading approximates traffic Tests pavement in its working environment Non-destructive Cost effective Well established Quick	Requires knowledge of subgrade material; Moduli calculation requires layer depths to be known or reasonably inferred
Test Pits	No sophisticated equipment required	Destructive; Expensive; Slow
Lab.Testing	Testing occurs in controlled environment	Expensive; Destructive; Requires skilled persons; samples are disturbed; Slow
GPR	Very quick	Readings heavily influenced by in-situ moisture content; No layer moduli inferred; Reflections within overlaid asphalt cause error

2.4.4 Properties Measured by these Measuring Methods

Table 2.2 provides a summary of the pavement and material properties derived from various testing methods.

Table 2.2 Measured parameters and calculated properties for the testing methods.

Testing Method	Measured parameter	Calculated pavement or material property
SASW	Reflected surface wave velocity	Material shear moduli Pavement layering
FWD	Load pulse acceleration	SNP – Network level Elastic moduli
Test Pits	Layer depths Penetration under loading (Scala Penetrometer)	Pavement layering Subgrade CBR
Lab. Testing	Moisture content Force, mass Moisture content Force Mass	Optimum moisture content Crushing resistance Plasticity Index CBR, UCS Grading
GPR	Dielectric Constant	Layer thickness

Table 2.2 illustrates how material properties are inferred by the measurement of different parameters. It also illustrates how the various material properties may not be measured directly, but are inferred from other more easily measured parameters with which they have relationships.

Understanding these relationships is the foundation to developing successful and meaningful testing methods. The complex physical nature of naturally occurring sub-surface materials make this a challenging task and it is unlikely that a single method will provide all the information that may be needed to accurately predict pavement performance. The continual development of new techniques and the refinement of existing ones may enable the pavement analyst to get closer to the "truth" and continually improve pavement performance predictions.

3. Technical Overview of SASW

3.1 Introduction

The elastic moduli of the materials encountered in pavement construction are affected by various properties of the subsurface, including matrix and structure, lithology, porosity, interstitial fluid properties, temperature, degree of compaction and density. Empirical relationships between the elastic moduli and these properties have been examined for specific cases in the literature (for a general review, see Guegen & Palciauskas (1994) for example). Generalisations are difficult, and usually these empirical relationships are valid only over a limited range of variation within any one property. Nevertheless, the measurement of the elastic moduli of a material provides a useful method of characterisation of the material. In particular, it is reasonable to assume (for a given material) that strength increases as modulus increases. For example in Auckland soils it has been shown that shear strength is approximately proportional to shear modulus (G. Williams & H. White, unpublished note, 2002).

Measurement of seismic velocity provides a means to determine the elastic moduli. In particular, P-wave velocity depends on bulk modulus and Poisson's ratio, while S-wave velocity depends on shear modulus.

3.2 Theory of SASW

3.2.1 Seismic Wave Types

Several types of *seismic waves* are propagated outward from a seismic source. In a uniform infinite medium only P and S waves are present. If the medium is bounded or non-uniform, as is the case with the immediate subsurface of a road, other types of wave are present, of which Surface waves are most relevant here. S waves, P waves and Surface waves are described below.

P waves: also called Primary, pressure or "push" waves. These are longitudinal waves in which the directions of motion of the particles of the medium are in the direction of propagation. The velocity of the wave is denoted V_p.

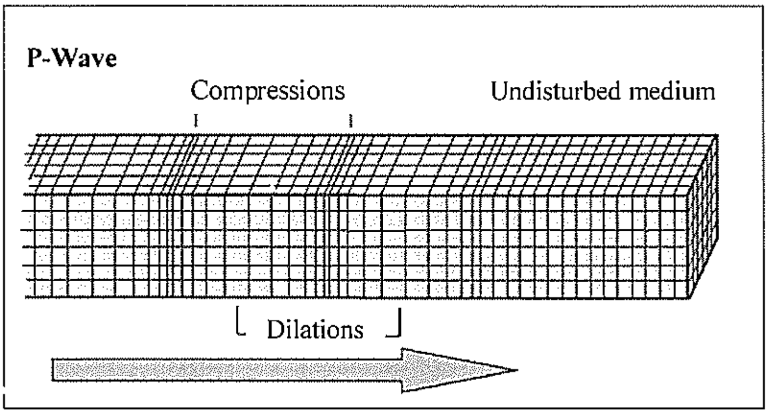

Figure 3.1 P-wave representation (from Menzies 1993).

S waves: Also called Secondary or shear waves. These are transverse waves in which the direction of motion of the particles of the medium is perpendicular to the direction of propagation. The velocity of the wave is denoted V_s. Since $V_p > V_s$, the first waves to arrive from any disturbance will be P waves.

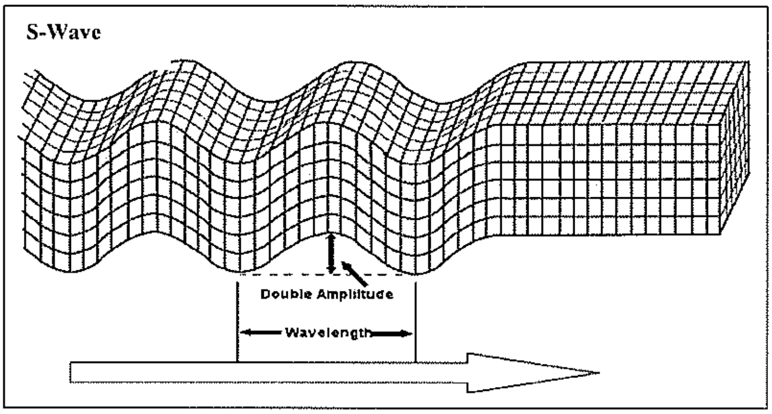

Figure 3.2 S-wave representation (from Menzies 1993).

Surface waves: These waves travel in the near-surface of the solid medium. In non-uniform media, surface waves travel at a velocity dependent on their frequency. Surface waves are divided into Rayleigh waves and Love waves.

Rayleigh waves are waves beneath a free surface, the amplitudes of which decay exponentially with depth (Figure 3.3). The particles of the medium move in vertical planes, so their motion describes vertical ellipses in which the vertical axes are about 1.5 times the horizontal. At the highest points of the ellipses the particle motion is opposite to the direction of wave advance. If a solid is layered, i.e. it consists of layers of different materials, a particular type of Rayleigh wave appears, known as Love waves. They are horizontally polarised.

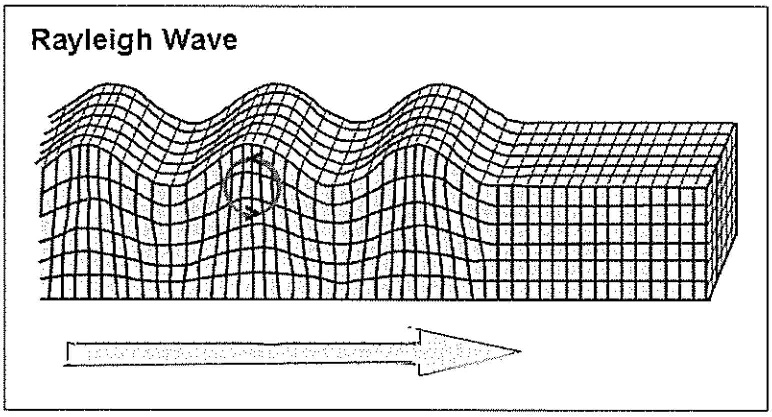

Figure 3.3 Rayleigh wave representation (from Menzies 1993).
Arrowed circle and arrows indicate motion of the particles in the medium.
The amplitude of the wave decreases from source of disturbance.

3.2.2 Definitions of Moduli used in SASW

Bulk Modulus (K): The ratio of the change in volume of a solid substance to the change in applied pressure.

Young's Modulus (E): The ratio of stress to strain on the loading plane in the loading direction.

Shear Modulus (μ) – The ratio of shear stress to shear strain on the loading plane. From elastic theory the Shear Modulus, μ, can be derived exactly as:

$$\mu = E/2(1+\nu)$$

where: ν is Poisson's ratio
E is the Young's Modulus.

The Stiffness G is the Shear Modulus at zero shear strain, i.e.:

$$G = \mu(\varepsilon = 0)$$

where: ε is the shear strain.

3.2.3 Theoretical Relationships

The relationship between P-wave velocity and the elastic moduli is given by:

$$V_p = \sqrt{\frac{K + 4\mu}{3\rho}} \qquad \text{Equation 2}$$

where: V_p is P-wave velocity
K is Bulk Modulus
μ is Shear Modulus
ρ is Mass Density

Unfortunately V_p depends both on the Shear and Bulk Moduli, and therefore, in the absence of independent measurement of one of these, it is not particularly useful for extracting material properties.

S-wave velocity depends only on Shear Modulus:

$$V_s = \sqrt{\frac{\mu}{\rho}} \qquad \text{Equation 3}$$

where: V_s is S-wave velocity

Consequently, the S-wave velocity is far more useful for directly characterising the subsurface. Shear and pressure velocities are related through Poisson's ratio by:

$$\frac{V_s}{V_p} = \sqrt{\frac{0.5 - \nu}{1 - \nu}} \qquad \text{Equation 4}$$

where: ν is Poisson's ratio

Therefore, having a measure of both P- and S-wave velocities allows the full suite of geotechnical moduli (Shear Modulus, Bulk Modulus and Poisson's ratio) to be determined.

Note also that seismic methods for investigating surface layers rely on seismic waves having a finite wavelength. Consequently the moduli are averages over finite wavelength-dependent volumes. Furthermore, because of the very small strains induced by seismic waves, all moduli are effectively zero strain moduli. The implications of this are important in the consideration of a property of a material. The types of materials found in pavements are not linear elastic and will exhibit a different resistance to deformation under different levels of applied loading.

For example, a pavement with a granular base that may deflect 1 mm under an 80 kN load, may only deflect 1.5 mm under a 160 kN load. This is in contrast to uniformly elastic materials, such as steel, in which the elastic deformation is expected to be directly proportional to the applied load (providing no plastic deformation has occurred).

It follows from this that SASW is the measure of the material property at a particular point (i.e. zero strain).

3.3 Seismic Waves and Subsurface Characterisation

Stiffness (i.e. Shear Modulus at zero strain), G, depends on the shear wave velocity as follows:

$$G = V_s^2 \rho \qquad \text{Equation 5}$$

where: ρ is mass density

In general, shear wave velocity is approximately equal to surface wave velocity (Foti 2000), and the two can be used interchangeably. Surface wave velocity at different depths therefore gives an indication of the varying stiffness of the subsurface, since the wavelength λ at different frequencies is given by:

$$\lambda(\omega) = \frac{V_s(\omega)}{f} \qquad \text{Equation 6}$$

where: $\omega = 2\pi f$

$\lambda(\varpi) \approx 3d$

d is the depth below surface

f is the frequency

In non-uniform media, surface waves travel at velocities that are dependent on their frequencies. This phenomenon is known as dispersion. This is analogous to light, where a glass prism disperses or separates white light according to wavelength. This is demonstrated in Figures 3.4 and 3.5 using actual data collected during this project.

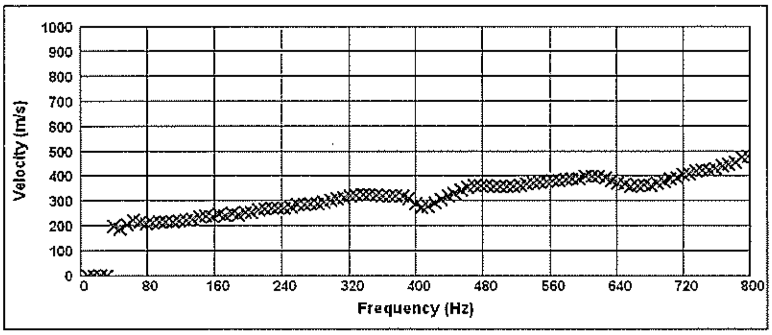

Figure 3.4 Typical graph of phase velocity as a function of frequency.

Note that the **phase velocity** of a wave is the rate at which the phase of the wave propagates in space. Any particular phase of the wave (for example the crest) would appear to travel at the phase velocity.

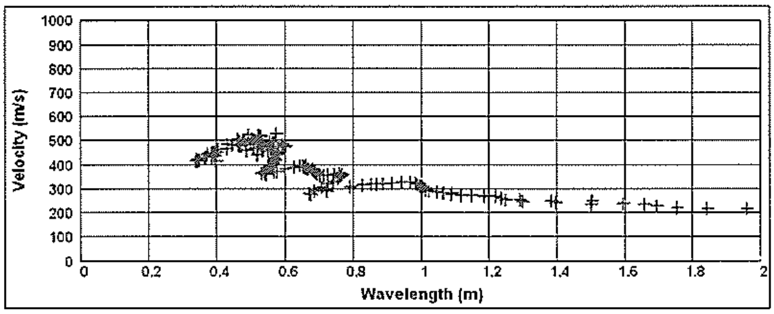

Figure 3.5 Velocity as a function of wavelength or depth, $d \approx \lambda/3$.

Note the layered nature of the graph, and decreasing stiffness with increasing wavelength (or depth equivalent), typical of a pavement.

As discussed before, Rayleigh (or Surface) waves have depths of penetration which depend on their wavelengths (depth = $\lambda/3$). Combining this fact with knowledge of the dispersion (Figure 3.5) enables one to find a relationship between the phase velocity and depth, and therefore the shear modulus and depth using Equation 3.

$$V_s = \sqrt{\frac{G}{\rho}} \qquad \text{Equation 3}$$

A typical set-up for SASW measurements is similar to that shown below, using receivers (G1-G3) at regular distances (D) from source (S):

With the seismic source, S, in line with the receivers (G1, G2, and G3), the phase velocity of the surface waves between receivers G1 and G2, say, is given by:

$$V_s = \frac{D}{t_{12}(\omega)} \qquad \text{Equation 7}$$

where: $t_{12}(\omega) = \frac{\Theta_{12}(\omega)}{\omega}$ is the time delay between the receivers

$\Theta_{12}(\omega)$ is the phase of the cross power spectrum

Knowing that:

$$\lambda(\omega) = \frac{V_s(\omega)}{f} \qquad \text{Equation 8}$$

where: f is frequency, and
assuming $\lambda(\omega) \approx 3d$,

then equation 8 can be used to determine the corresponding depth for each value of velocity.

4. Resources & Methods Required for SASW Data Collection

4.1 Introduction

In the following discussion, the SASW measuring cycle is discussed under the categories of:

- Resource requirements, and
- Data collection methodology.

The current data collection methodology is static. Sensors are laid out and remain in position until testing on that section is complete.

4.2 Equipment and Human Resource Requirements

In the current configuration, data collection can be performed by one suitably trained person, although two are preferable as they enable the process to be completed about 60% faster. The collection method follows a set procedure and is not a physically demanding activity.

Table 4.1 illustrates the resources utilised for collecting data on a 30 m profile, in 6 m spreads of a series of geophones:

Table 4.1 Resource requirements for collecting SASW data.

Item	Quantity and comments
Time (including set-up)	2.5 hours
Operators	1
Sensors	7 x geophones
Seismometer	1 x 8-channel 24-bit data acquisition unit
Data collection computer	Standard notebook
Seismic source	Ball peen hammer
Power	1 x standard car battery – sufficient for a day's work
Traffic management	Required: as a profile is performed in a wheel track, a 'contra flow' of traffic usually can be set up

4.3 Data Collection Methodology

4.3.1 Deployment of Equipment

The basic deployment of equipment for multi-station SASW data collection is depicted in Figures 4.1 and 4.2.

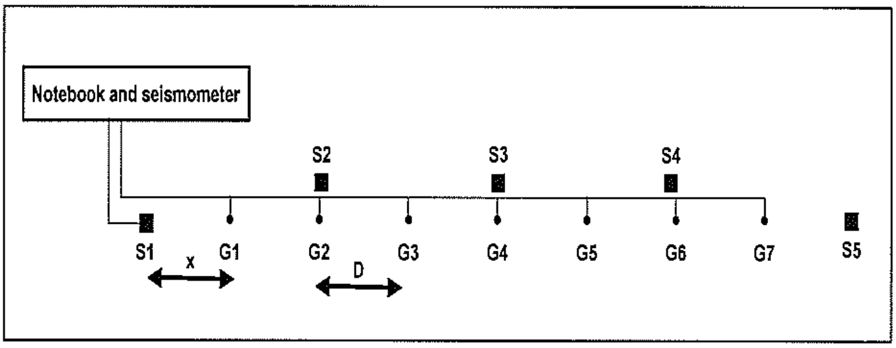

Figure 4.1 Basic deployment of equipment for SASW data collection.
S1–S5 are source positions, G1–G7 are geophone positions.
Geophone spacing is D, X is distance between source and nearest geophone.

- Geophones (velocity transducers) G1-7 are fixed to the surface in line in a 'string' at regular intervals, D, that are determined by the depth of investigation required. Effectively, the upper limit of the depth of investigation is equal to half a wavelength of the Rayleigh waves propagating in the subsurface.
- The string of linked geophones is connected to a seismometer, which is usually powered by a motor vehicle battery and situated in the trunk of a vehicle, at the side of the road.
- A notebook computer is connected to the seismometer to control acquisition parameters and to store the data. The seismometer and notebook fits into a small case.
- An impulse source (Figure 4.3) is applied at, say, positions S1 – S5. At these locations, a rugged trigger geophone is fixed to the surface to trigger the data acquisition in the logger.
- The basic configuration is repeated until the full profile has been surveyed.

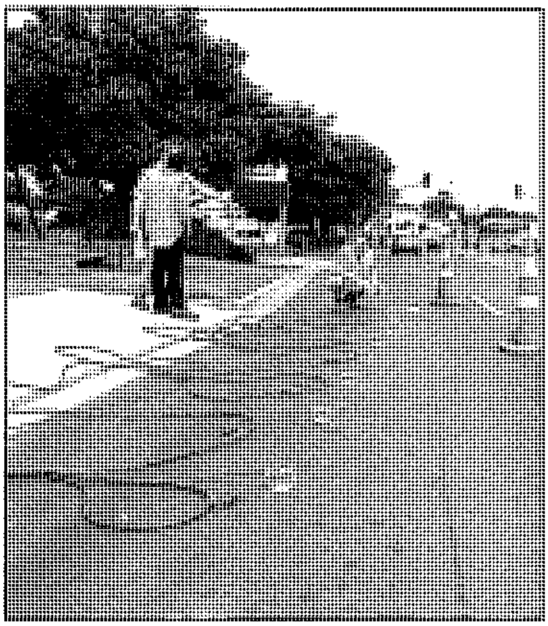

Figure 4.2 Geophone string fixed to road surface and connected up ready for use.

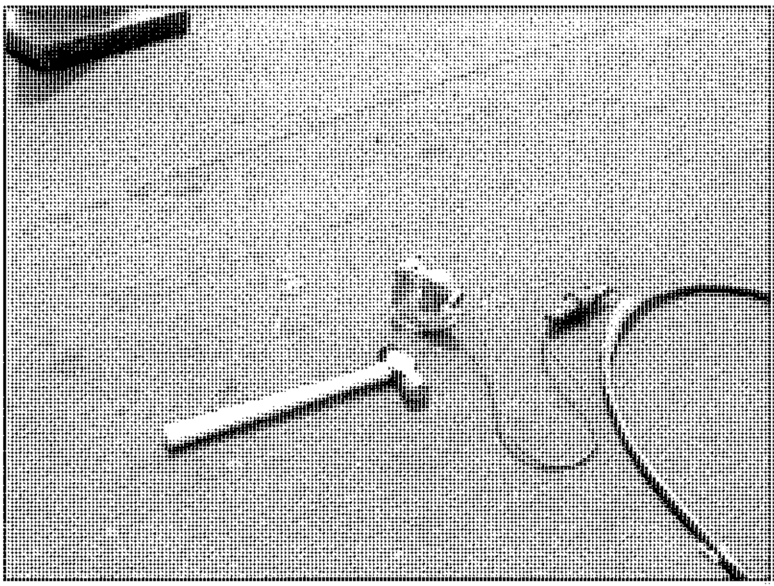

Figure 4.3 Ball peen hammer impulse source and geophone.

4.3.2 Operating the Equipment

The operator arms the system and hits the surface of the pavement with the hammer (i.e. the impulse source). The system then records the resulting wave as it passes each geophone. The following considerations have to be borne in mind when collecting data:

- External noise, and vibrations from passing traffic and construction activities, can also be picked up and recorded. Although these are usually low frequency vibrations which can be filtered out in the processing phase, recording data only in quiet periods is preferable. This does not pose a major limitation, since the system records a time period of only about a second.
- Quality control should be performed on at least two shots in a spread. Recording software is usually provided with a rudimentary viewing utility. Alternatively, a spreadsheet program can be used to plot and visually inspect seismograms (Figure 4.4 is an example of a good-quality seismogram). The purpose of the inspection is to identify loose connections to geophones and verify data quality.

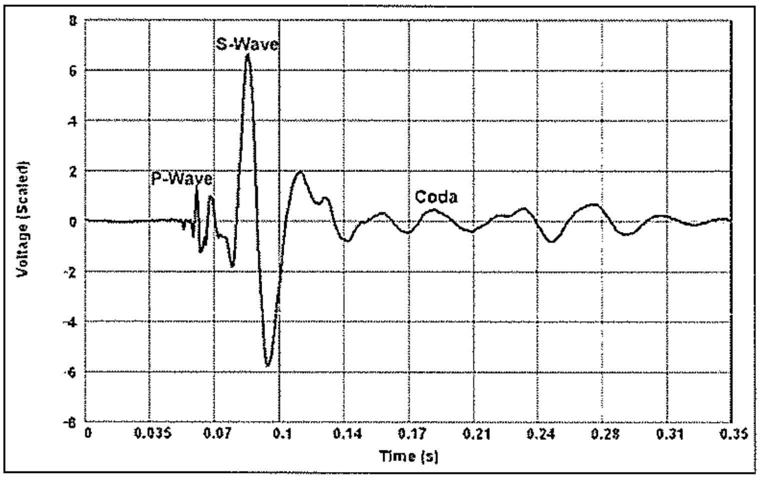

Figure 4.4 A typical good quality seismogram.

The seismogram in Figure 4.4 exhibits a pattern with clear P-wave, S-wave and Coda (i.e. a long wave-train that consists of multiple scattered waves).

Seismograms are recorded as voltage v time graphs, and a system-specific conversion factor is used to produce a velocity v time graph for the geophones.

4.4 Possible Future Improvements to SASW Method

Routine application of the technique will be enhanced in terms of time and cost if measurements can be taken without the operator having to venture onto the road, and with the vehicle-mounted system only stopping briefly for each measurement (within the limits defined for mobile traffic management). Experimental measurements (performed on 11 May 2003) on an area of asphalt surface at Ports of Auckland have shown that geophone coupling similar to that employed when using the FWD is acceptable. This means that the time-consuming exercise of fixing the transducers to the road with an adhesive can be avoided. Also there is the potential to carry out SASW testing in conjunction with FWD testing using the same vehicle.

Most of the time spent at a site is taken up by repeating the basic configuration of sensors until the complete profile has been covered. If the number of sensors connected to the digital seismometer can be extended, then this would correspondingly reduce the number of times the operator is required to set the system up.

4.5 Limitations and Constraints to SASW Method

4.5.1 Comparison of SASW with FWD Method

The relationship between the low and high strain regimes of the pavement moduli is not linear (Menzies 1993). For example, measurements of Young's modulus (E), measured in compression in triaxial tests, showed that low strain (about 0.01% to 0.1%) moduli are much larger than high strain (about 1%) moduli values.

Finite element back-analysis from observed movements around real structures result in larger moduli from smaller (typically less than 0.1%) strains. Dynamic tests using the Resonant Column Apparatus apply very small strains (typically less than 0.001%) and give stiffness values greater than the back-analysed values.

The generic variation of moduli (such as Young's Modulus) with strain is presented in Figure 4.5. It is important to note that the modulus of a material is dependent upon its state of loading. Decreasing modulus with increasing strain is expected in cohesive soils, e.g. the subgrade of a pavement. Most granular materials are thought to behave elastically at very small strains (less than 0.001%), giving rise to a constant modulus. The strain induced by the propagation of seismic waves is within this range and is therefore taken to provide a measure of the upper bound for stiffness (i.e. the maximum shear modulus that occurs at very near zero strains). In order to predict the performance of a pavement it is therefore preferable to have knowledge about that pavement under loading.

FWD imposes a load during testing that resembles the rolling wheel load imposed by traffic on a pavement, and thus FWD gives an indication of the stiffness of the pavement in its "working state". Benkelman Beam testing also applies a loading during testing meant to represent a standard axle, but differs from FWD in that it is a static load.

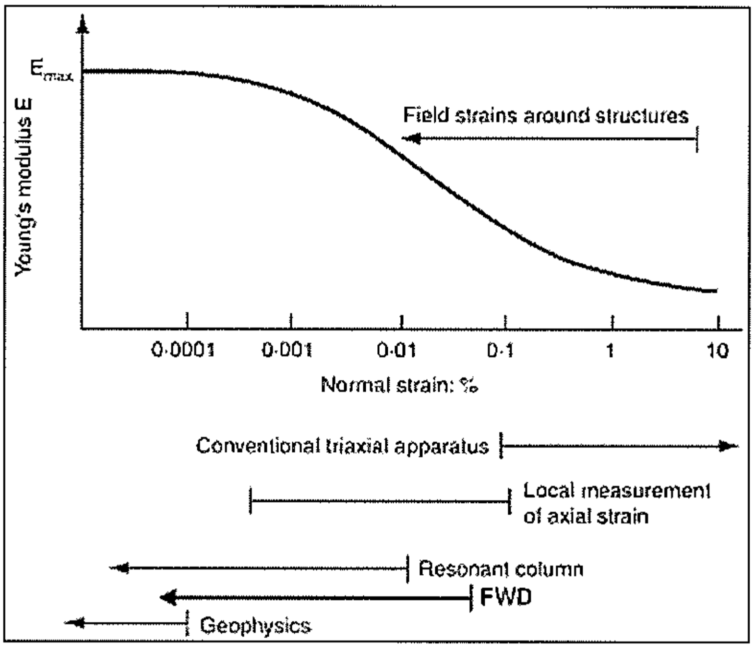

Figure 4.5 Typical variation of Young's modulus (E) with strain (%) (from Menzies 1993).

4.5.2 Spacing of Sensors

From Equation 6 in Section 3, it can be seen that the higher the frequency content of the wave, the shorter the wavelength and therefore the shallower the depth that can be probed. However, the high frequency components of a wave are attenuated very quickly, so that higher frequencies require smaller intervals between sensors (i.e. geophones or accelerometers).

4.5.3 Sampling Frequency for Data Acquisition

The sampling frequency controls the highest frequencies present in the digitised waveforms. The maximum frequency required (high frequencies imply shallow penetration) in a digitised waveform should never exceed 50% of the sampling rate (to prevent aliasing).

Physical limits are placed on the maximum sampling rate by the type of sensors employed. Geophones lose sensitivity above certain frequencies, while accelerometers have extremely high upper limits.

4.5.4 Sources of Seismic Wave Energy

The seismic source determines the frequency content of the input wave, as well as its energy content. The energy content determines how quickly the wave dissipates. Generally, a physically smaller (i.e. lower energy) source results in higher frequencies, at the cost of useable propagation distances.

5. Results of SASW Investigation

Three analyses on the data are presented here:

- A determination of first arrival P-waves, to determine the Shear Modulus in the top layer of a pavement;
- An SASW dispersion analysis, to determine the Shear Modulus at different depths in the pavement;
- A direct comparison between D_0 (the maximum FWD value at zero displacement) and the maximum value of shear modulus obtained on each site tested during the dispersion analysis.

Appendices A and B provide summaries of the test pit data, associated with the 20 test sites.

5.1 Comparison between SASW and Test Pit Methods

5.1.1 Method of Analysis

To extract a comparison between basecourse characteristics as determined by test pits, Subgrade CBR data and SASW, the Stiffness *G* was determined for each section using Equation 5:

$$G = V_s^2 \rho$$

Since the basecourse is considerably stiffer than the subgrade, the faster P-waves are assumed to propagate mainly in the basecourse. With the P-wave velocities (Figure 5.1), S-wave velocities, required for the calculation of the Shear Moduli, are determined through Poisson's ratio by:

$$\frac{V_s}{V_p} = \sqrt{\frac{0.5 - \nu}{1 - \nu}}$$

where Poisson's ratio, ν, is taken to be 0.33.

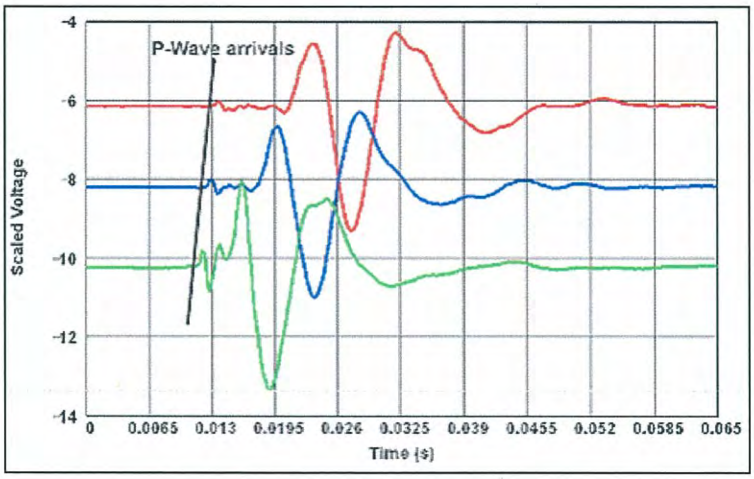

Figure 5.1: Arrival times of P-waves at geophones 1 to 3.
Green = closest geophone to source; Blue = mid distance from source; Red = furthest from source

The seismograms were obtained from geophones spaced 1 metre apart.

Table 5.1 A summary of Test Pit information for 10 sites of the total 13 sites listed in Appendix B, and the base layer shear modulus determined from P-wave velocities.

Site	Site No.	SASW Shear Modulus base layer (MPa)	Basecourse Description	Basecourse Depth (mm)	Sub-base Description	Sub-base Depth (mm)	Subgrade CBR (in-situ) (%)	Model
BM3	3	740	AP40	170	AP65	220	20	Unbound
BM12	4	590	AP40 – possible Cement Stab.	90	Sandy 65mm/40mm agg.	390	45	Unbound/ Improved
Atkinson Ave	19	505	AC	100	AP40, AP65	400	3	Bound
BM22	8	450	AP40	90	Cement Stab. AP40 + AP65	190+180	20	Unbound
BM24	10	450	AP40 – possible Lime Stab.	100	AP40, AP65	430	25	Unbound/ Improved
BM21	7	415	Cement or Lime Stab. AP40	160	Sandy gravel	240	14	Unbound/ Improved
BM14	6	345	AP40	260	AP65	250	8	Unbound
SH030	15	330	No layer information available	–	–	–	–	–
Jellicoe Road	20	310	Several AC layers	90	AP40, AP65	700	unknown	Unbound
Pauanui Road	17	305	Well graded AP40	80	Sand + stiff silt layer	140	20	Unbound
BM25	11	270	Cement Stab. AP40	110	Cement Stab. AP65	130+300	6	Bound
Beach Road	16	270	Well graded AP40 – Greywacke	80	AP65	200	20	Unbound

Key: AP40 – all passing 40 sieve; AP65 – AP 65 sieve;
AC – Asphalt concrete/cement; Stab. – stabilisation; agg. – aggregate

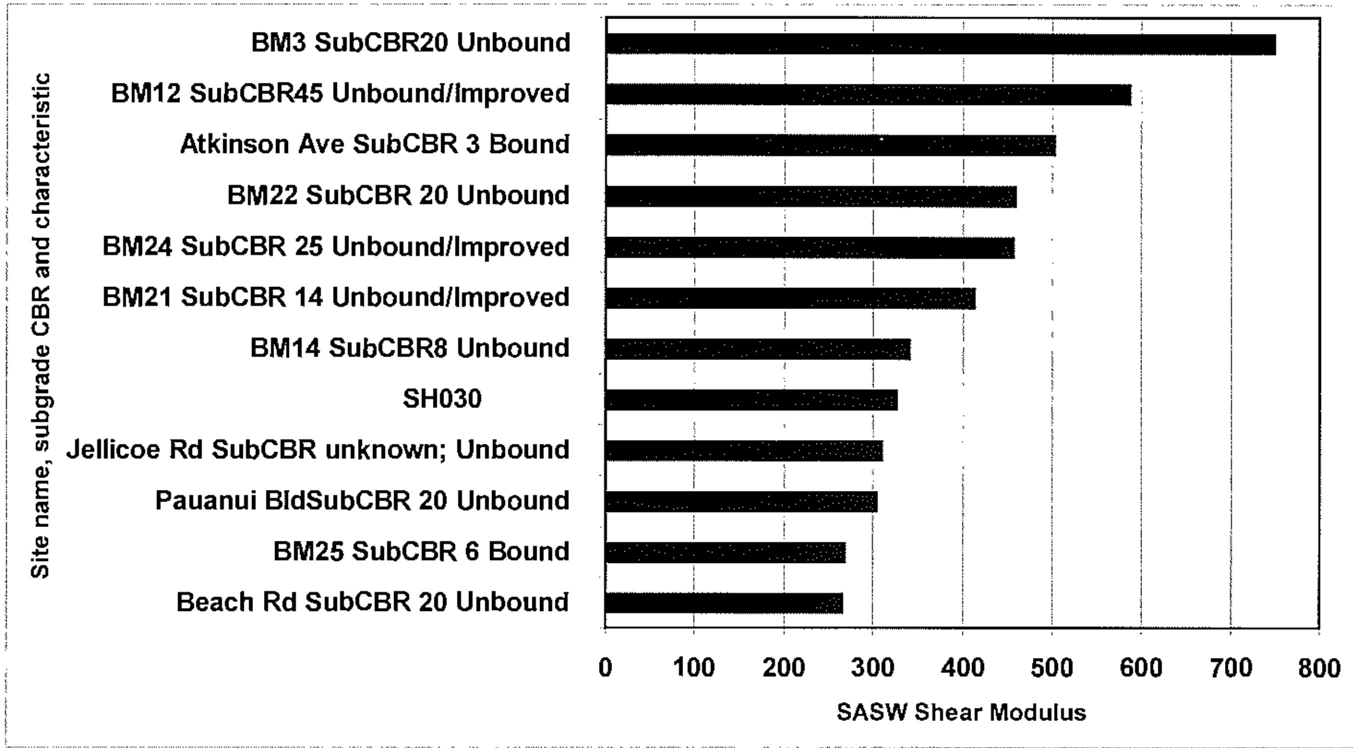

Figure 5.2 Comparison between Test Pit data, Subgrade CBR, and shear modulus (MPa) measured by SASW.

Table 5.1 presents a summary of the test pit data and Figure 5.2 graphically illustrates the same.

5.1.2 Discussion of Results

From the results no easily defined boundary between bound, improved, and unbound base layers is obvious. It is well known that the stiffness of the base layer will also depend on the underlying structure (though this is less so in the case of bound materials).

The base layers have been described as unbound or lime/cement-stabilised by a visual examination of the test pits. It is difficult to accurately model the extent of modification of the material comprising the basecourse because the degree of binding is not known.

Apart from BM25, however, a general trend is seen whereby the "improved" or "bound" sections have higher shear moduli.

Both the Jellicoe Road and Atkinson Avenue sections have AC surfaces. The stiffness of AC is highly temperature dependent, and perhaps temperature should be included as a parameter in future analyses.

5.2 Shear Modulus at Different Depths

5.2.1 Method of Analysis

The shear moduli at different depths was calculated using dispersion analysis, and the full series of results is presented in Appendix B. Figure 5.1 for site 3 on SH001 represents a case that gives good layer information and qualitative agreement with FWD results. Figure 5.4 for Atkinson Avenue, Auckland, represents the converse. Note that Test Pit results are also summarised on Figures 5.3 and 5.4, and the moduli graphs are presented for the seven sites in Appendix D.

For the reasons discussed in Section 4.5.1, direct comparison with moduli calculated from FWD testing is difficult. However, a relative comparison was attempted whereby the moduli from the SASW testing were transposed by a factor to bring them to the same order of magnitude as that inferred by FWD. The values were then graphed so that the change in modulus with depth could be compared between SASW and FWD results. These graphs are presented on the outputs in Appendix D.

5.2.2 Discussion of Results

Qualitative comments on the results of each section are presented in tabular form in Appendix B, which also exhibits the data acquisition parameters employed at each site.

Figure 5.3 (for Site No. 3, site BM3 on SH001) is a good example of consistent layer information which agrees well with the test pit data. The grey area across the top of the graph represents material that cannot be recorded because data acquisition in the upper layer of the pavement, in which waves have very short wavelengths, is technically difficult with the equipment used in this study. However, the P-wave velocity can be used to obtain a modulus for this layer (see Section 5.1).

Figure 5.3 iGEO Site No. 3, Site BM03, SH001, RS144, Whangarei, Northland. Sampling rate 4000Hz. Source 1.5kg hammer. An example of well-defined, continuous layer information and qualitative agreement with trends in FWD data.

iGeo Ltd
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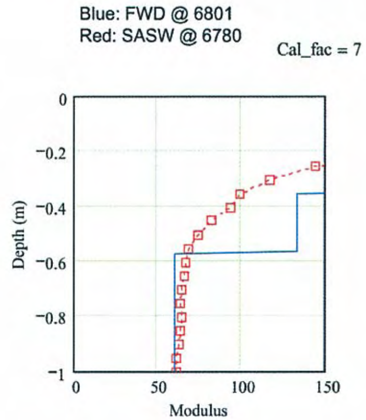
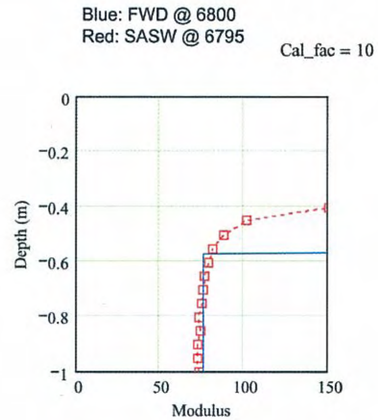
web: www.igeo.co.nz





Test Pit 2, Ch 6650

0 to -0.035	Bitumen bound Chipseal
-0.035 to -0.200	AP40 aggregate with Silty Sandy fines
-0.200 to -0.340	AP65 aggregate with Silty Sandy fines
-0.340 to -0.560	AP65 weathered aggregate
-0.560 to -0.660	Orange Silty Clay
-0.660 to -0.760	Dark brown Silty Clay

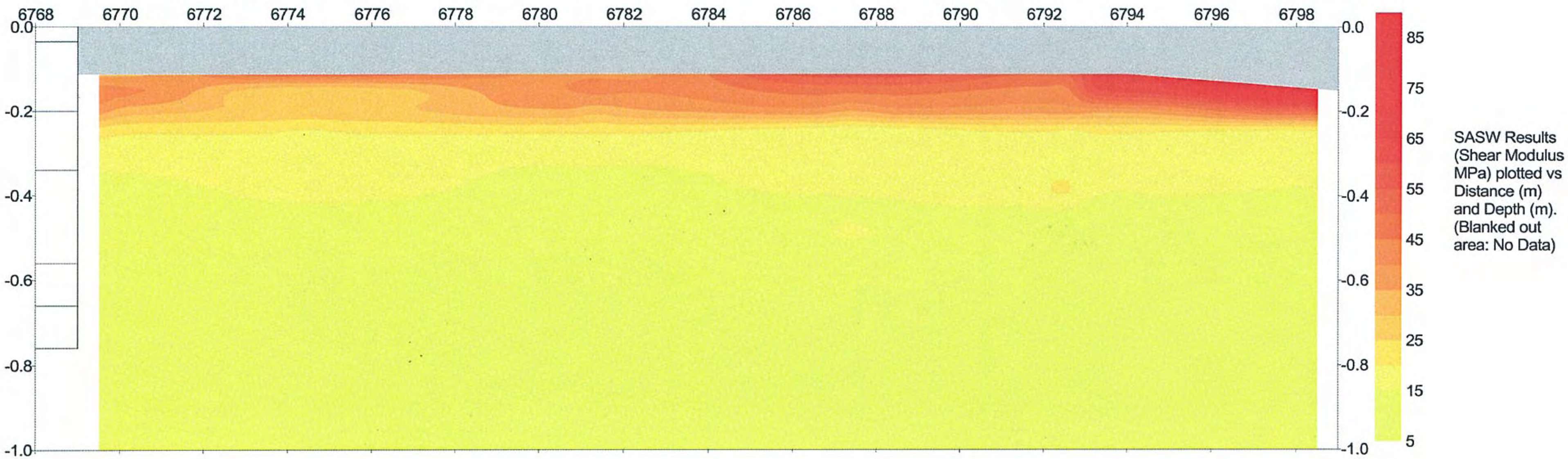


Figure 5.4 iGEO Site No. 19, Atkinson Avenue, Auckland City. Sampling rate 1000Hz. Source small ball peen hammer. An example of little or no continuous layer information and qualitative disagreement with FWD results.

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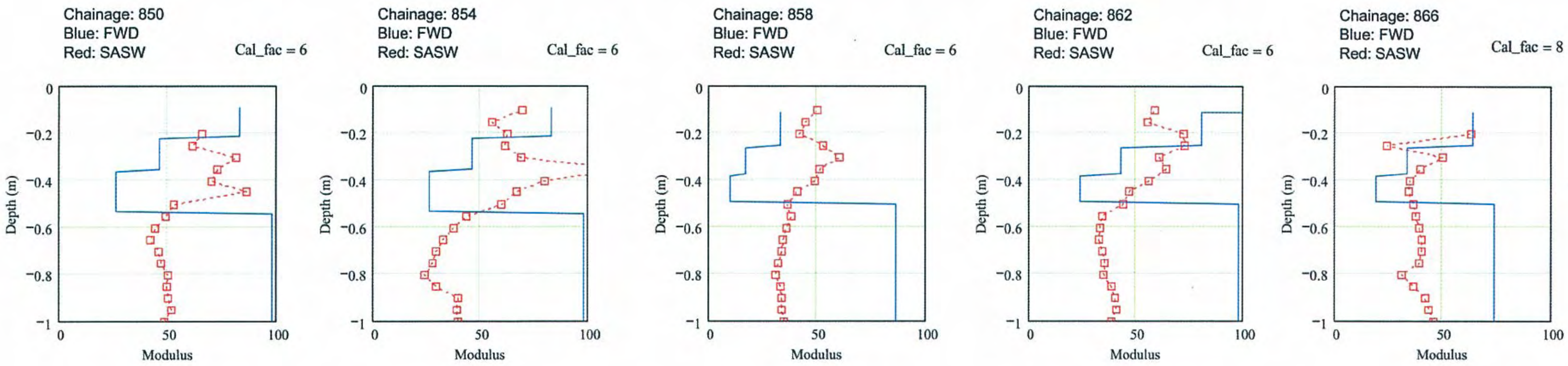

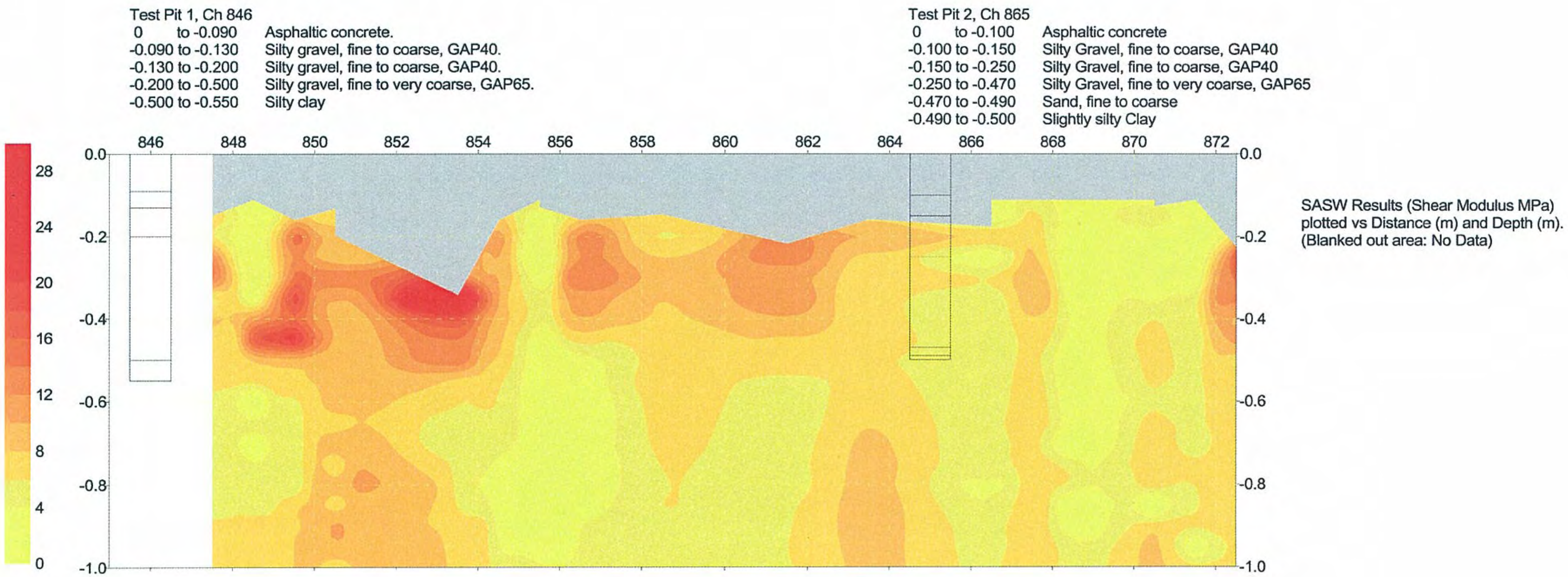

Figures 5.3 and 5.4 demonstrate the ability of SASW to:

- provide layer thickness information;
- measure shear moduli at different depths;
- illustrate the variability of a pavement structure.

5.3 Comparison between Pavement Stiffness & Dispersion Analysis

5.3.1 Method of Analysis

The possibility of a relationship between the maximum base moduli, extracted from the dispersion analysis, to the pavement stiffness (D_0) as a whole was also considered. This possible relationship exists because the layering presented in the dispersion analysis is analogous to the manner in which the stiffness in a pavement is dependent on the underlying layers. FWD measurements are also representative of the pavement stiffness as a whole. A simple comparison was therefore carried out and the results are shown in Figure 5.5.

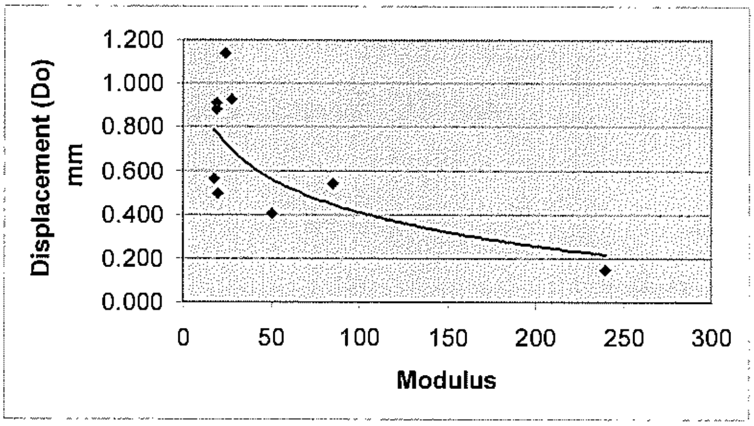

Figure 5.5 Displacement (mm) of pavement obtained by FWD D_0 compared to maximum shear modulus obtained by Dispersion analysis.

5.3.2 Discussion of Results

Figure 5.5 shows that a potential relationship exists between FWD readings and the moduli extracted from SASW. If SASW measurements could be carried out using the same vehicle at the same time as FWD, a comparative analysis between the results obtained from the two measurement techniques would be possible.

6. Economic Evaluation of Pavement Testing for Network & Project Management

The economic aspects surrounding pavement testing and design in New Zealand are discussed in this Chapter 6. The potential usefulness of improved knowledge about the strength of existing pavements at both network and project levels using worked examples is demonstrated. A summary of indicative costs for different testing methods, such as SASW, Test Pits, FWD, is also presented.

6.1 Evaluation at Network Level

6.1.1 Scenario for Designing a Pavement

The following example (Table 6.1) uses the dTIMS network management tool to demonstrate the sensitivity of the predicted maintenance budgets to changes in the strength model parameter (SNP).

The example is designed to demonstrate how improved confidence in pavement strength evaluation on a network level can improve planning for future budget requirements.

Table 6.1 Predicted maintenance budgets (NZ$) for different SNP scenarios.

	Budget Requirements						
SNP	**–30%**	**–20%**	**–10%**	**BM**	**+10%**	**+20%**	**+30%**
Agency Cost ($000's)	18,366	18,006	17,871	17,797	17,796	17,796	17,795
VOC ($000's)	10,778	10,722	10,764	10,713	10,678	10,674	10,658
TTC ($000's)	29,144	28,728	28,635	28,510	28,474	28,470	28,453
	Difference in cost to SNP BM						
Agency Cost ($000's)	569	209	74	–	–1	–2	–2
VOC ($000's)	65	9	51	–	–35	–39	–56
TTC ($000's)	633	218	125	–	–36	–41	–58
	Difference per km						
Agency Cost ($)	2,915	1,071	379	–	–6	–8	–11
VOC ($)	332	46	260	–	–181	–200	–285
TTC ($)	3,248	1,117	639	–	–187	–208	–296
	Yearly budget difference per kilometre						
Agency Cost ($)	145.8	53.6	19.0	–	–0.3	–0.4	–0.6
VOC ($)	16.6	2.3	13.0	–	–9.1	–10.0	–14.2
TTC ($)	162.4	55.9	32.0	–	–9.3	–10.4	–14.8

SNP – Modified Structural Number
BM – Benchmark
VOC – Vehicle operating costs
TTC – Total Transport cost (VOC + Agency Cost)

Actual data from an existing network was used. The analysis period used was 20 years and Table 6.1 summarises the results. The network was analysed with the strength value assigned according to traffic for each analysis section, i.e. an optimum pavement for a 25-year design life (designated as the Benchmark SNP). The strength values were then decreased and increased by 10%, 20% and 30%, and the resulting maintenance budget requirements were compared for each scenario.

Further details and explanations of the analysis are included in Appendix C (Tables C2, C3), including a description of some typical SNP values in terms of pavement structure.

6.1.2 Potential of SASW to Improve Network Management

The most obvious observation of the results is that the decreased maintenance costs associated with stronger pavements is far outweighed by the increased costs required to maintain weak pavements.

However, if the network was generally composed of weak pavements, overestimating the SNP would have been significant.

This means that the maintenance budget is particularly sensitive to the strength parameter for weaker pavements. Many New Zealand networks have a significant portion of weak pavements. The need to properly model the network deterioration and hence obtain useful budget predictions therefore depends on reliable pavement strength data being available for analysis.

For Transit New Zealand, the evaluation of pavement strength on a network level is predominantly done by FWD. There is a limit to the confidence that can be placed on this method and it is almost always carried out without the benefit of existing pavement layer knowledge. Test pits are not used for network level surveys because of their high cost.

The potential exists therefore for SASW to be used in the following ways in network management:

- As a new system of strength determination with an "SASW SNP" developed for use in the modelling process;
- As a supplement to existing methods.

6.1.3 Practical Issues for SASW Measurement

The testing procedure used in this study involved fixing the geophones to the road with a commercial adhesive. Some trial testing was carried out subsequent to the analysis for the benchmark sections, and it showed that fixing to the road is not necessary and that good coupling could be achieved without it. This would allow SASW to become a rolling measurement system and, as a result, costs would be dramatically reduced.

The Seismic Pavement Analyzer (described in Section 2.4.2.4), which utilises similar principles and equipment for pavement measurement, has been developed to do rolling measurements.

Because the two testing procedures use similar equipment, both SASW and FWD could conceivably be employed on the same vehicle at the same time. This would allow direct comparison between methods and also provide layer information for the FWD analysis. In this case, the costs would be expected to increase only marginally, compared to carrying out either FWD or SASW testing only.

6.2 Evaluation at Project Level

6.2.1 Scenario for Designing a Pavement

The following scenario is presented for an existing pavement requiring a rejuvenation treatment. In this case the pavement needs an overlay to meet the required structural capacity to carry future traffic. The pavement structure, costs and traffic load used in this example are typical of a rural road in the North Island of New Zealand.

The example demonstrates effects of overestimating and underestimating the following design parameters:

- Subgrade CBR;
- Existing pavement layer thickness;
- Existing basecourse modulus; and
- Design traffic loading.

Traffic loading has been included in this assessment to show the relative sensitivity to required pavement depth, when compared with the other design parameters, using mechanistic design procedures.

Consider an existing 2 km length of pavement with the following structure:

100mm basecourse
150mm Sub-base
Subgrade CBR 4

25-year traffic design = 500 000 ESA

Tables 6.2 and 6.3 summarise the costs and pavement life implications of making the incorrect assumptions during the design process. (Appendix C, Table C1 has details.)

Summaries of the four scenarios are compared, but are considered independently, i.e. each example has one parameter deviated from the structure or traffic shown above. The Circly mechanistic analysis design program was used for the analysis.

Table 6.2 Example for designing a pavement overlay at project level, to provide design assumptions and actual values.

	Parameter	Actual value	Value used in design		Testing & Investigation Costs ($NZ)
			Under-designed	Over-designed	
1	Subgrade CBR %	4%	5%	3%	$1,500
2	Existing pavement thickness (mm)	250mm	300mm	200mm	$2,500
3	Basecourse modulus (MPa)	300MPa	400MPa	200MPa	$600
4	Traffic Load (ESA)	500 000 ESA	400 000 ESA	600 000 ESA	$400

Table 6.3: Example for designing a pavement overlay at project level, to provide design and actual budget requirements, and residual life.

Parameter	1. Subgrade CBR (%)			2. Existing pavement thickness (mm)			3. Basecourse Modulus (MPa)			4. Traffic Load (ESA)		
Optimum (actual) or Design	Opt.	U-D	O-D	Opt.	U-D	O-D	Opt.	U-D	O-D	Opt.	U-D	O-D
Construction overlay (mm)	170	120	235	170	120	220	185	170	215	170	160	175
Construction Cost Estimate ($000's)	400	360	480	400	360	460	415	400	455	400	390	405
Theoretical remaining life of pavement (years) - subgrade strain criterion	25	9	30	25	9	30	25	20	30	25	24	28
PV Agency Costs incl. routine and periodic maintenance ($000's)	439	480	512	439	480	493	454	454	488	439	454	440

Key: pt. – Optimum; U-D – Under-designed; O-D – Over-designed; PV – Present Value; PV Agency – funding Road Controlling Authority
Notes on the analysis methodology and calculations are in Appendix C (Table C1).

Optimum design is based on actual values: e.g.
CBR Actual = 4%; Pavement thickness = 250 mm;
Basecourse modulus = 300 MPa; Traffic loading = 500,000 ESA

6.2.2 Potential of SASW to Improve Pavement Design

Table 6.3 shows how making the wrong assumptions can severely impact on the life of a pavement and consequently increase the costs to funding agencies. The sensitivity of parameters like subgrade CBR and existing pavement depth demonstrate the need to make informed decisions during the design process. The potential exists for SASW to be used to improve the confidence of pavement designers when evaluating the structure of an existing pavement.

6.2.2.1 Subgrade CBR

Subgrade CBR is currently only measured by destructive means, i.e. laboratory testing or Scala Penetrometer testing. The testing methodology used on this project did show that SASW has the potential to differentiate between weak and strong subgrades. This would probably require further investigation of the testing methodology about using more closely spaced sensors with high frequency sources to improve the resolution at those depths in the pavement.

6.2.2.2 Existing pavement thickness

The existing pavement thickness is currently only measured by test pitting in New Zealand. SASW provides a visual tool that gives a continuous illustration of the differential in Shear Modulus (Figures 5.3 and 5.4) which, in this study, has shown reasonable correlation with test pit data.

Given the expense of coring (and subsequent repair to test holes) and the often limited accessibility to highly trafficked roads with AC and/or Concrete bases, SASW can be a useful non-destructive tool. The existing thickness of these bound layers is the most important parameter for determining a maintenance treatment. In a study in the Canadian city of Surrey to evaluate the use of SASW on urban roads, the asphalt thickness was estimated to a high accuracy with an error of less than 6% (Addo 1997).

Pavements which have highly variable structures, in terms of layer thickness and depth, are usually a cause for concern because sections of such pavements are likely to be inadequate for the traffic loading. It may be possible to make the continuous picture of the pavement structure that is provided by SASW into one of the design considerations.

FWD data can be used for project level design when destructive testing is not possible. If no layer data is available the back-analysis uses assumed pavement layer thickness to calculate moduli and stiffness, and subsequently the required rejuvenation technique to be applied. The layer moduli are calculated with greater confidence when the pavement structure is available, and SASW could be used to provide that layer information.

6.2.2.3 Base modulus

Granular bases – The basecourse modulus can be a difficult parameter to assess in-situ. The assumption often made is that the existing layer has adequate structural capacity when applying an overlay treatment. However, if the basecourse layer is weak and the design overlay (calculated by subgrade strain criteria) is not adequate to compensate for this, then the pavement may still fail early. This failure may be caused by shallow shear or subgrade strain failure.

The example in Tables 6.2 and 6.3 was demonstrated for failure by subgrade strain, with an early rejuvenation treatment required after 20 years. Therefore, in that scenario the pavement does not have sufficient stiffness (because of the low basecourse modulus) to protect the subgrade for the duration of the design life.

Although the process of shallow shear failure is common and well known in New Zealand, there appears, as yet, to be no obvious correlation between basecourse modulus and the residual pavement life for pavements failing in this manner. This lack of correlation is partly because of the practical difficulty in assessing a failure as being only due to shallow shear in the field. At best, there is a broad trend indicating that gravels with a crushing resistance below a certain value will be susceptible to shallow shear.

Bound bases – Pavement managers have indicated that knowledge of the modulus of the top layer in structural AC pavements in New Zealand would be useful information, particularly if it could be related directly to performance. The ability of SASW to measure the shear modulus in the upper base layer may present this opportunity.

6.2.2.4 Traffic Loading

Traffic has been included in the example in Tables 6.2 and 6.3 simply to show its relative sensitivity, compared to the other parameters, in the design process. It shows that underestimating the traffic loading does not to have the same impact as underestimating the other design parameters.

6.3 Costs of Different Testing Methods

Table 6.4 presents indicative costs associated with different tests, and they vary from region to region in New Zealand. For FWD testing, this is largely due to the differing costs related to particular traffic control requirements. The cost of laboratory testing can also vary considerably by region.

At a network level, the cost for SASW testing has not been included but, as discussed in Section 2.4, the techniques to produce a rolling measurement machine for this purpose do exist and some effort needs to be applied to develop them. The costs are expected to be similar to those for FWD testing.

Table 6.4: Costs of various testing methods.

Level	Testing Method	Approx. Cost (NZ$)	Notes
Network Level	FWD	$50 per lane/km	Includes traffic management and testing at 100 metre intervals
Project Level	Test Pits	$250 – $300 each	Varies greatly depending on traffic management requirements, sampling rate and reinstatement method
	Laboratory Testing		Does not include sampling costs
	1. CBR	$125	
	2. Grading	$190	
	3. Crushing Resistance	$170	
	4. Clay Index	$165	
	5. Compaction curve	$600	
	6. Weathering Resistance	$525	
	7. Sand Equivalent	$110	
	FWD	$400 per site	Based on 3 sites per day with traffic control
	SASW	$400 per site	Based on 3 sites per day with traffic control

7. Conclusions & Recommendations

7.1 Conclusions

SASW demonstrated the ability to:

- measure the shear modulus of pavement materials;
- determine layer thickness;
- measure the shear modulus in the base layer; and
- illustrate the longitudinal variability within a pavement.

SASW will only be an economically viable option at the network level if it is developed into a rolling type measurement.

7.2 Recommendations for Future Research

Measurements were done in this research project to determine the applicability of SASW to the New Zealand pavement management environment. Results have been positive, and the next major step would involve the following considerations.

7.2.1 Converting SASW into a Rolling Type Measurement

Converting the set-up to allow moving measurements to be made rather than static measurements is the next step in the practical development of SASW testing. The set-up may also be incorporated onto the FWD testing vehicle, so that exactly the same sections are tested at the same time. The benefits of this would be:

- reduced costs compared to separate testing;
- SASW layer information can be used to supplement FWD data for use in the data analysis (i.e. layer depth at the FWD test point);
- direct comparison between FWD and SASW can be made.

Some practical aspects to consider would be the practical consideration of mounting the system on a trailer, and ensuring that the controls and on-site quality assessment are easy to carry out.

7.2.2 Improved Measurement Resolution

Incorporating both geophones and accelerometers to allow improved resolution within the shallowest layers would be a significant enhancement. This may also require the incorporation of at least two different sources for producing sound waves (i.e. at higher and lower frequencies). Accelerometers tend to be slightly less robust than geophones, so packaging requirements would need to be investigated.

7.2.3 Data Interpretation

This study demonstrated how the information obtained from the testing could be graphically displayed to provide a continuous pavement profile. There still exists potential for further development to make the information extracted more useful to network managers and pavement designers.

For instance, an algorithm to extract layer thickness from the data would be a useful tool, especially where the material moduli are showing a high degree of variability on the graphs and the layering is not apparent. This would aid considerably in modelling the pavement structure in processes like mechanistic design.

7.2.4 Utilisation of SASW Information

The information about the pavement extracted from the SASW testing may be utilised:

- As a completely separate system for the measurement of pavement strength. For instance, on a network level the data could be used to develop an "SASW SNP" for use in the deterioration modelling process.
- In conjunction with existing methods. For example, layer information provided by SASW may be used to supplement FWD data.

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Appendix A

Table A1: Details of test sites used for comparing SASW, Test Pit and FWD methods.

iGeo site	MWH Database Number	Area	Road SH / RS	RP	FWD	Test Pit	SASW	Comments
1	BM1	Northland / Auckland (Whangarei)	SH001 RS188	Start	0.1	0.3	11.001	SASW out of range
				End	1.101	0.9	11.041	
2	BM2	Northland / Auckland (Whangarei)	SH001 RS220	Start	13	13.4	13.111	
				End	14	13.8	13.141	
3	BM3	Northland / Auckland (Whangarei)	SH001 RS144	Start	6.1	6.35	6.77	
				End	6.8	6.65	6.801	
4	BM12	Hamilton / Central Waikato (Taupo)	SH005 RS169	Start	6.82	7.221	7.47	
				End	7.821	8.37	7.5	
5	cancelled			Start				Cancelled by MWH
				End				
6	BM14	Hamilton / East Waikato (Tauranga)	SH029 RS50	Start	0.02	0.22	0.170	
				End	1.71	0.77	0.200	
7	BM21	Napier / Gisborne (Gisborne)	SH035 RS250	Start	9.57	9.92	10.27	
				End	10.47	10.07	10.294	
8	BM22	Napier / Gisborne (Gisborne)	SH002 RS375	Start	0.68	0.881	1.019	
				End	1.561	1.43	1.049	
9	BM23	Napier / Gisborne (Gisborne)	SH002 RS474					Left out due to time constraints
10	BM24	Napier / Gisborne (Napier)	SH002 RS544	Start	12.47	12.52	13.001	
				End	13.2	12.72	13.03	

iGeo site	MWH Database Number	Area	Road SH / RS	RP	FWD	Test Pit	SASW	Comments
11	BM25	Napier / Napier (Napier)	SH002 RS729	Start	8.6	9	9	
				End	9.6	9.401	9.03	
12	BM26	Napier / Napier (Napier)	SH002 RS204	Start	13.63	14	14.22	
				End	14.78	14.58	14.25	
13	BM27	Napier / Napier (Napier)	SH005 RS233	Start	10	10.2	13	SASW out of range
				End	11	10.8	13.03	
14	cancelled	Hamilton / Rotorua (Rotorua)	SH005 RS77	Start				No FWD No test pit
				End				
15	ROTORUA SH30_RS170	Hamilton / Rotorua	SH030 RS170	Start	9.26		9.270	
				End	9.561		9.300	
16	TCDC_BEACH RD_(WHA)	Thames - Coromandel	Beach Road	Start	200	340	328	
				End	500		346	
17	TCDC_OCEAN RD	Thames - Coromandel	Ocean RD	Start	0.05	180		Left out due to time constraints
				End	0.85			
18	TCDC_PAUANUI BLVD	Thames - Coromandel	Pauanui Boulevard	Start	1550	1730	1715	
				End	1750		1745	
19	AKL_Atkinson (Avenue) NB_DRP	Auckland	From Criterion	Start	834	846	846	
				End	906	865	874	
20	AKL_Jellicoe (Road) RD_WB_DRP	Auckland	From Panmure roundabout	Start	5	55	105.5	
				End	145	115	135.5	

SH – State Highway; RS – Reference Site;
TCDC – Thames–Coromandel District Council; AKL – Auckland

Appendix B

Table B1: Results of seismic tests at the test sites.

* Site names and numbers used in Table 5.1, and Appendix A

Sites*	Area	Geophone spacing	Data acquisition sampling rate	Source and shots	Comments
2, 3 (BM2, BM3) Figures A1, A2	Northland	1 m	4000	1.5 kg hammer at G1 – 1m, G7 + 1 m	BM2: The top layer is not present, while the next three almost appear to appear to form a unit. The SASW analysis appears to resolve some finer structure in these layers. Some non-uniformity along the section appears in the last layer. BM3: Layers below 0.1 m are delineated well. Note the qualitative agreement with FWD and Test Pit data. Layering is visible in both the sections. The top layer is absent.
4 (BM12), Figure A3	Taupo	1 m	1000	Small ball peen hammer at G1 – 1 m, G2, G4, G6 and G7 + 1m	The macrostructure is well-delineated, with a hard patch at chainage 7942. Substantial variation along the length of the section. Note less agreement with the FWD data than BM2 and 3, and some agreement with Test Pit data. The top layer is not present

Sites*	Area	Geophone spacing	Data acquisition sampling rate	Source and shots	Comments
7, 8, 10 (BM21, 22 and 24) Figures A4 to A6	Napier/ Gisborne	1 m	1000	Small ball peen hammer at G1–1m, G2, G4, G6 and G7 + 1m	Significant variation along the length of all three sections. BM21: Some layering can be discerned, disrupted by relatively harder patches. BM22: A significant, abrupt change in the Shear Modulus occurs along the section. BM24: Note the change in conditions about half way through the section. Not so much agreement with the FWD data, with some agreement with Test Pit data. The top layer is not present.
11 (BM25) Figure A7	Napier	1 m	1000	Small ball peen hammer at G1–1m, G2, G4, G6 and G7 + 1m	Large variations in Shear Modulus along the length of the section. Little agreement with FWD results. The top layer is not present.
6 (BM14). Figure A8	Hamilton/ East Waikato	1 m	4000	Small ball peen hammer at G1–1m, G2, G4, G6 and G7 + 1m	Large-scale layering is present, up to chainage 190, where a sudden change of conditions occur. Two of the basecourse layers, although not separated, are visible. Qualitative agreements with FWD and Test Pit data. A change of surface was noted at chainage 192, and appears to be reflected in the data. The top layer is not present.

Sites*	Area	Geophone spacing	Data acquisition sampling rate	Source and shots	Comments
16 (Beach Road (WHA); 17 (Pauanui Blvd) Figures A10, A11	TCDC	1 m	4000	Small ball peen hammer at G1–1m, G2, G4, G6 and G7 + 1m	Beach Road: Layers apparent. Qualitative agreements with FWD and Test Pit data. Pauanui Blvd: Qualitative agreements with FWD and Test Pit data. Qualitative agreements with FWD and Test Pit data. In both cases, the top layer is not present.
19 (Atkinson Ave), 20 (Jellicoe Road) Figures A12, A13	Auckland	1 m	1000	Small ball peen hammer at G1–1m, G1, G2, G3, G4, G5, G6, G7 and G7 + 1m	Atkinson Avenue: Broken layering visible, with signs of extensive reworking. Little agreement with FWD results. Jellicoe Road: Some layering discernible up to chainage 122 to 126, where conditions change significantly. In both cases, the top layer is not present.

Appendix C

C1: Network Level Example

Table C1: Analysis parameters used for Network example.

Length of Pavement Network	195 km
Average SNP value	2.7
Average traffic volume	1340
Number of Treatment Lengths	445
Analysis mode	Performance, i.e. no budget constraint, treatment selection driven by intervention criteria such as roughness
Optimisation	Minimise total transport costs (i.e. agency cost + vehicle operating costs)
Analysis Period	20 years

To help understand the SNP concept the following typical SNP values for a chipseal, on granular basecourse pavement are described in Table C2.

Table C2: Approximate SNP v pavement structure.

Inc/Dec	SNP	Subgrade CBR	Base thickness(mm)	Sub-Base
+30%	3.5	5	300	300
+20%	3.2	5	250	300
+10%	3.0	5	200	300
–	2.7	5	200	250
-10%	2.4	5	150	200
-20%	2.1	5	100	150
-30%	1.9	5	100	100

(*dTIMS Technical Reference Manual – Volume1 (1999)*)

Explanation of the output

The VOC is not always increased when the pavement strength is decreased. This is because of the treatments that are selected by dTIMS. For instance, if the SNP value is only slightly (i.e. 10%) below the ARRB value, then the model might apply a smoothing treatment as opposed to a complete reconstruction on a particular section. This results in a certain amount of decreased VOC. If the SNP is even lower (i.e. 20%) then the model might apply a reconstruction because of the rapid deterioration of the pavement. This results in greater savings in the VOC because of the significantly reduced roughness of the road surface. The agency costs will, however, increase as the treatment is more costly.

C2: Project Level Example

Notes on the analysis methodology

The testing costs are based on sampling rates and type of tests and are only a rough indication of the expected amount for comprehensive testing. Generally they will vary considerably depending on the region and sampling rate used. These estimates are adequate for the purposes of this study because the comparison with construction costs is obvious. See Tables 6.2 and 6.3 in Chapter 6 of the main report.

The theoretical remaining pavement life according to the subgrade strain criteria was much greater than 30 years in some cases, but this upper limit has been applied using engineering judgement and experience. This is because pavements will reach a state of failure for a variety of reasons, not just the accumulation of subgrade strain.

Pavements which had remaining life at the end of the 25-year analysis period were also accorded a residual life value which was subtracted from the agency costs, i.e. if a pavement still had a useable life of 5 years, then the residual value was calculated as:

(Initial Construction Cost / 25) x 5

This amount was then discounted to a present value.

The under-designed examples for *subgrade CBR* and *existing pavement thickness* were deemed to have been rehabilitated at year 10 for a 25-year design life. Therefore these pavements were given a 10-year residual value at year 25.

Routine maintenance costs were allocated at $600 per year for all scenarios.

Pre-reseal repairs were done at years 9 and 19. Reseals were carried out at years 10 and 20.

The following spreadsheet is a presentation of the calculations.

Table C3: Detailed calculations of example at Project Level.

Routine and periodic costs $

Year	Treatment	Subgrade CBR			Existing Pavement			Basecourse Modulus			Traffic		
		Opt	U-D	O-D	Opt	U-D	O-D	Opt	U-D	O-D	Opt	U-D	O-D
0		400 000	360 000	480000	400000	360000	460000	415000	400000	455000	400000	390000	405000
9	Prereseal	3000	0	3000	3000	0	3000	3000	3000	3000	3000	3000	3000
10	Reseal	60 000	300 000	60000	60000	300000	60000	60000	60000	60000	60000	60000	60000
19	Prereseal	3000	3000	3000	3000	3000	3000	3000	0	3000	3000	3000	3000
20	Reseal	60 000	60 000	60000	60000	60000	60000	60000	300000	60000	60000	60000	60000
25	Routine	600	600	600	600	600	600	600	600	600	600	600	600
	Residual -ve	0	120 000	80000	0	120000	76667	0	0	75833	0	300000	43393

Present Value of Costs $

Year	Treatment	Subgrade CBR			Existing Pavement			Basecourse Modulus			Traffic		
		Opt	U-D	O-D	Opt	U-D	O-D	Opt	U-D	O-D	Opt	U-D	O-D
0		400000	360000	480000	400000	360000	460000	415000	400000	455000	400000	390000	405000
9	Prereseal	1272	0	1272	1272	0	1272	1272	1272	1272	1272	1272	1272
10	Reseal	23133	115663	23133	23133	115663	23133	23133	23133	23133	23133	23133	23133
19	Prereseal	491	491	491	491	491	491	491	0	491	491	491	491
20	Reseal	8919	8919	8919	8919	8919	8919	8919	44593	8919	8919	8919	8919
25	Routine	5142	5142	5142	5142	5142	5142	5142	5142	5142	5142	5142	5142
	Residual -ve	0	10069	6712	0	10069	6433	0	20137	6363	0	25172	3641
	PV TOTAL	438 956	480 145	512 244	438 956	480 145	492 523	453 956	454 003	487 593	438 956	454 128	440 315

Appendix D

Figure D1 iGEO Site No. 2, Site BM02, SH001, RS220, Whangarei, Northland. Sampling rate 4000Hz, Source 1.5 kg hammer.

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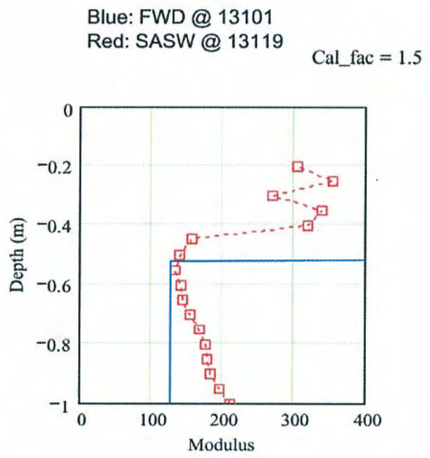
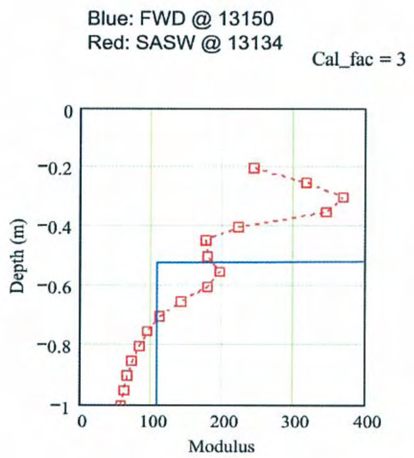




Test Pit 1, Ch 13400

0 to -0.030	Bitumen bound Chipseal
-0.030 to -0.100	Dark AP40 agregate with Silty plastic fines
-0.100 to -0.320	AP40 Lime and cement with Sandy Silty fines
-0.320 to -0.510	Dark brown AP65 aggregate
-0.510 to -0.800	Grey orange Silty Clay

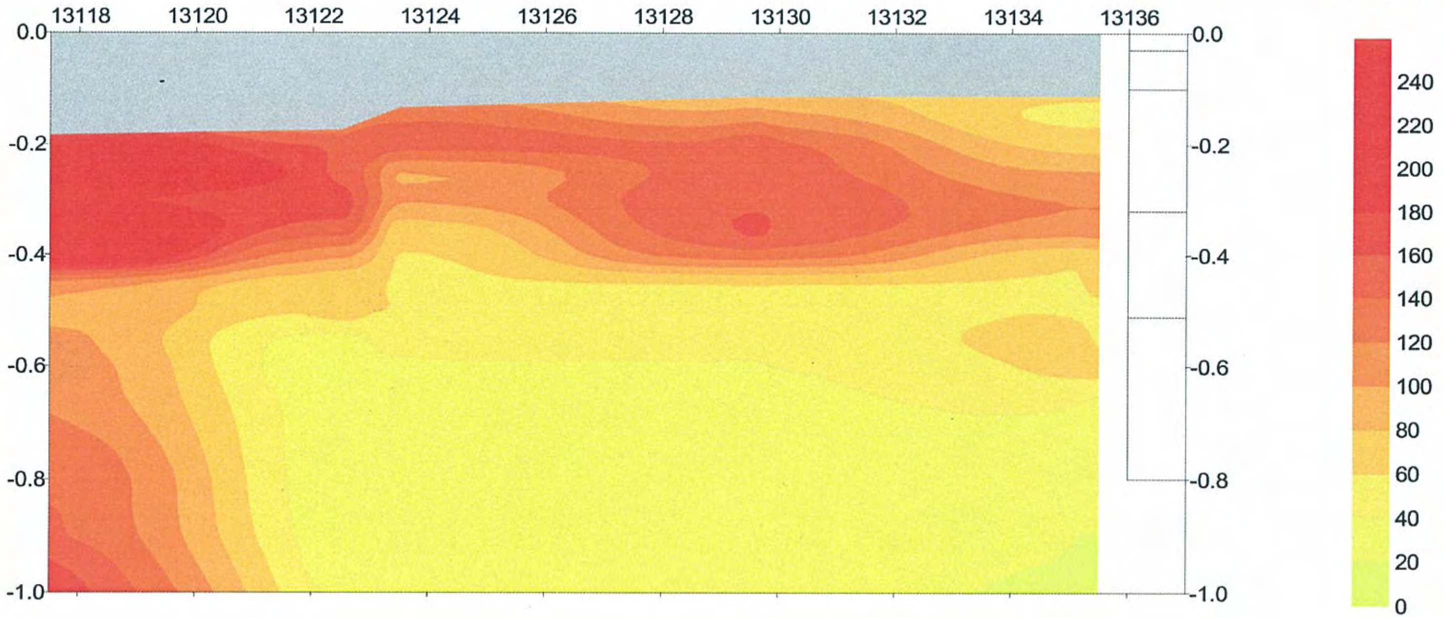
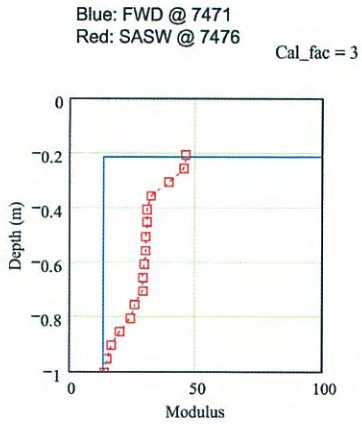
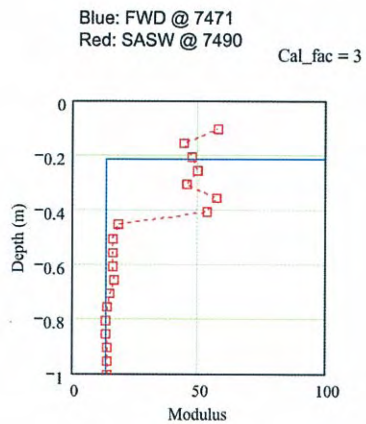


Figure D2 iGEO Site No. 4, Site BM12, SH005, RS169, Taupo. Sampling rate 1000Hz, Source small ball peen hammer.





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Test Pit 2, Ch 7200

0 to -0.030	Bitumen bound chipseal
-0.030 to -0.120	Sandy gravel up to 40 mm
-0.120 to -0.200	65 mm sandy gravel
-0.200 to -0.510	Pumiceous sand

Test Pit 1, Ch 8370

0 to -0.030	Bitumen bound chipseal
-0.030 to -0.110	AP40 aggregate with sandy fines
-0.110 to -0.140	Old Bitumen bound chipseal
-0.140 to -0.270	Crushed sandy gravel up to 65 mm in size
-0.270 to -0.550	Moderately packed pumiceous sand with gravel
-0.550 to -0.700	Moderately packed pumiceous coarse sand

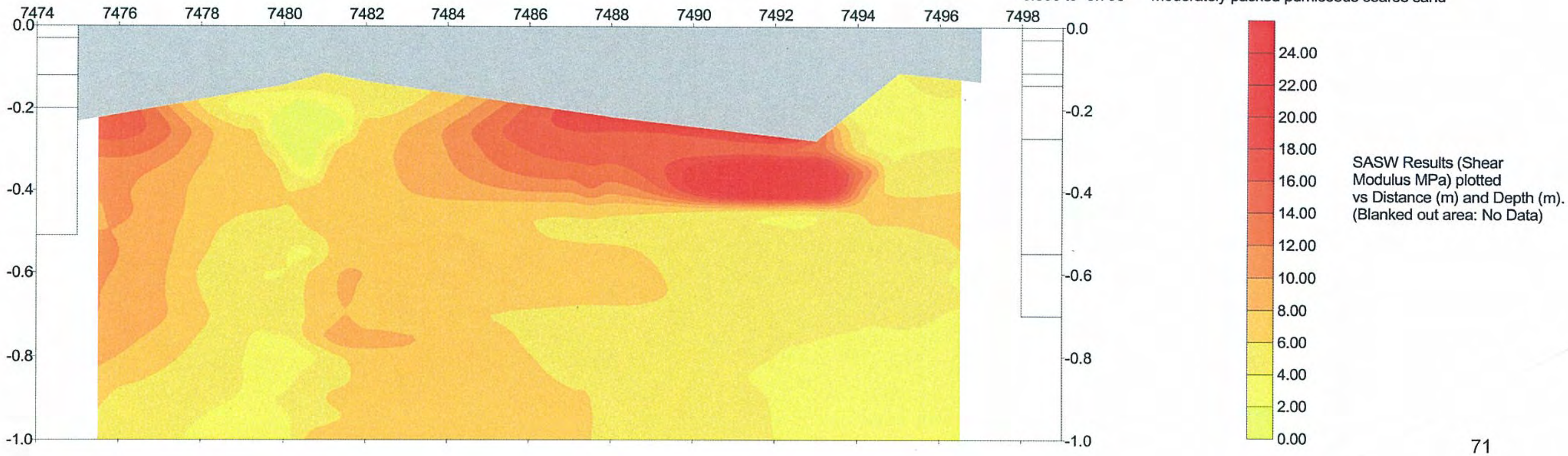
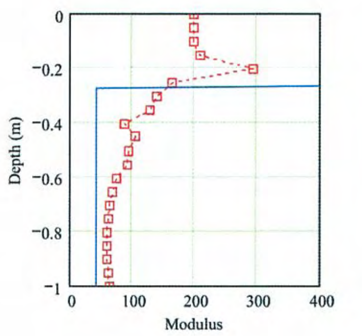
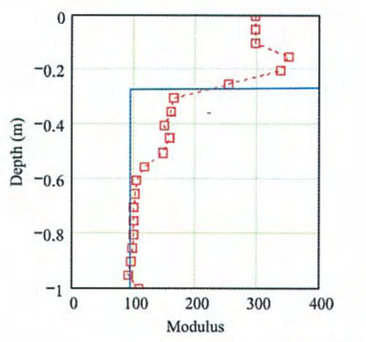


Figure D3 iGEO Site No. 6, Site BM14, SH029, RS50, Tauranga. Sampling rate 4000Hz, Source small ball peen hammer.





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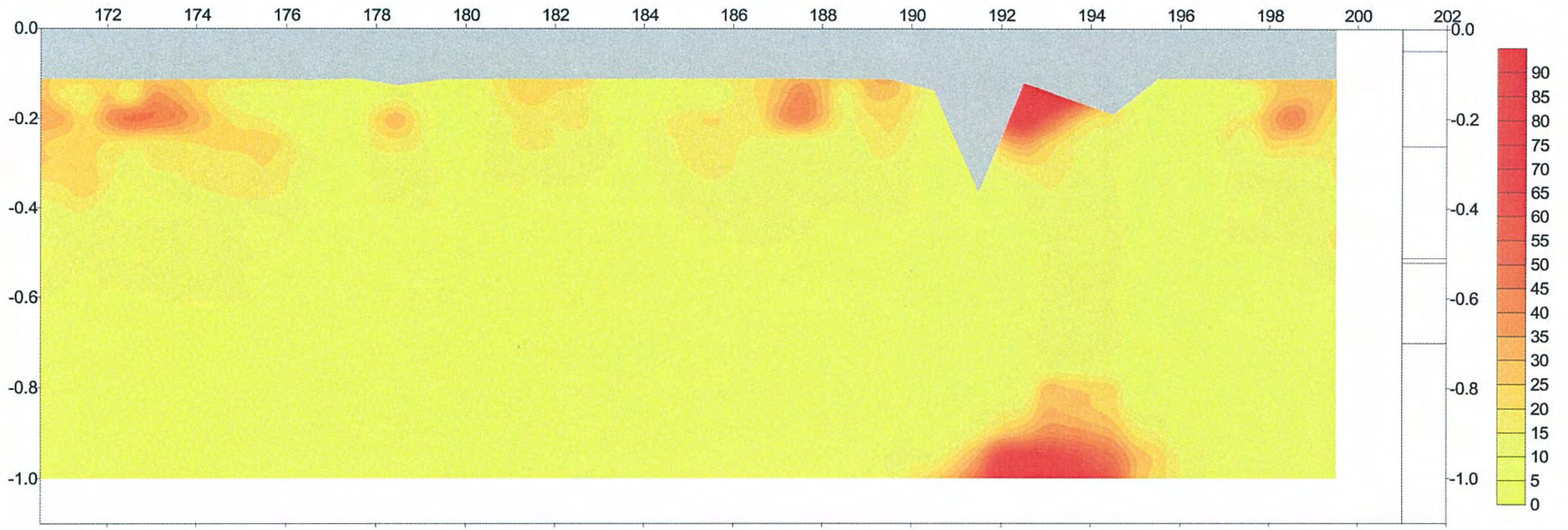
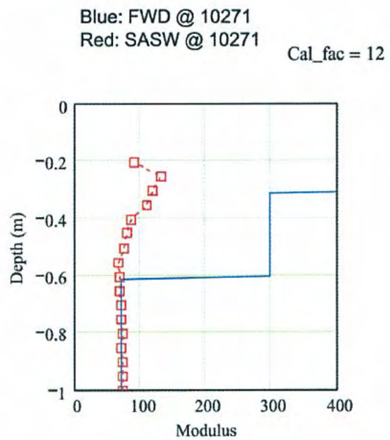
Test Pit 1, Ch 220

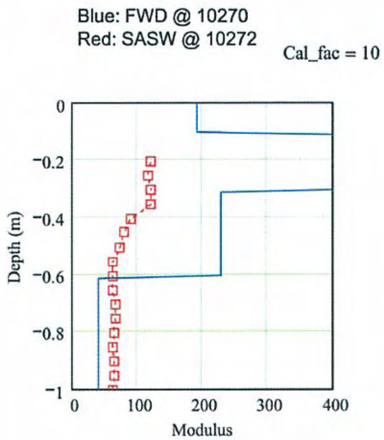
0 to -0.050	Bitumen bound Chipseal
-0.050 to -0.260	AP40 aggregate with Sandy Silt fine
-0.260 to -0.510	AP65 aggregate
-0.510 to -0.520	Old Chipseal
-0.520 to -0.700	AP40 aggregate with Sandy Silt fine
-0.700 to -1.100	Orange Clayey Silt



SASW Results (Shear Modulus MPa) plotted vs Distance (m) and Depth (m). (Blanked out area: No Data)

**Figure D4 iGEO Site No. 7, Site BM21, SH035, RS250, Gisborne.
Sampling rate 1000Hz, Source small ball peen hammer.**





Test Pit 2, Ch 10070

0 to -0.030	Bitumen bound chipseal
-0.030 to -0.190	AP40 sandy gravel with sandy silt fines
-0.190 to -0.200	Old chipseal layer
-0.200 to -0.380	Sandy gravel with sandy silt fines
-0.380 to -0.580	AP65 sandy gravel with sandy silt fines
-0.580 to -0.800	Silty fine sand to sandy silt
-0.800 to -0.970	Orange silt with sand and clay fines

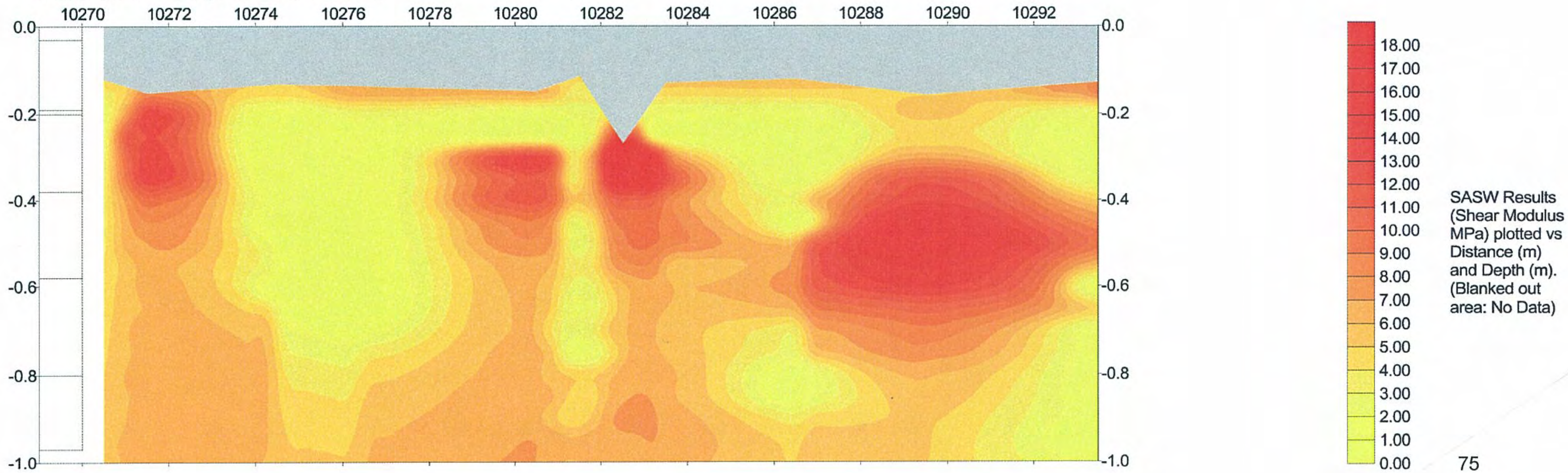


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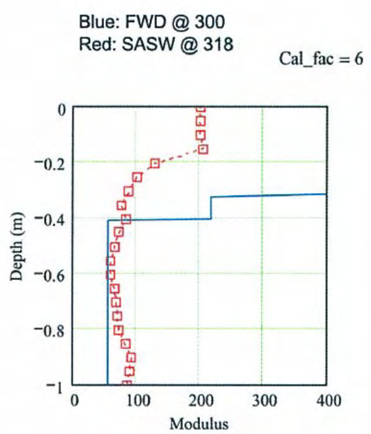
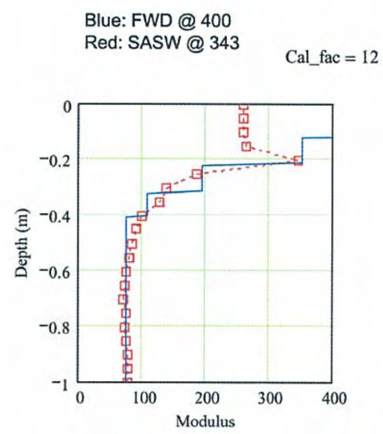
Figure D5 iGEO Site No. 16, Beach Road (WHA), Thames-Coromandel District. Sampling rate 4000Hz, Source small ball peen hammer.





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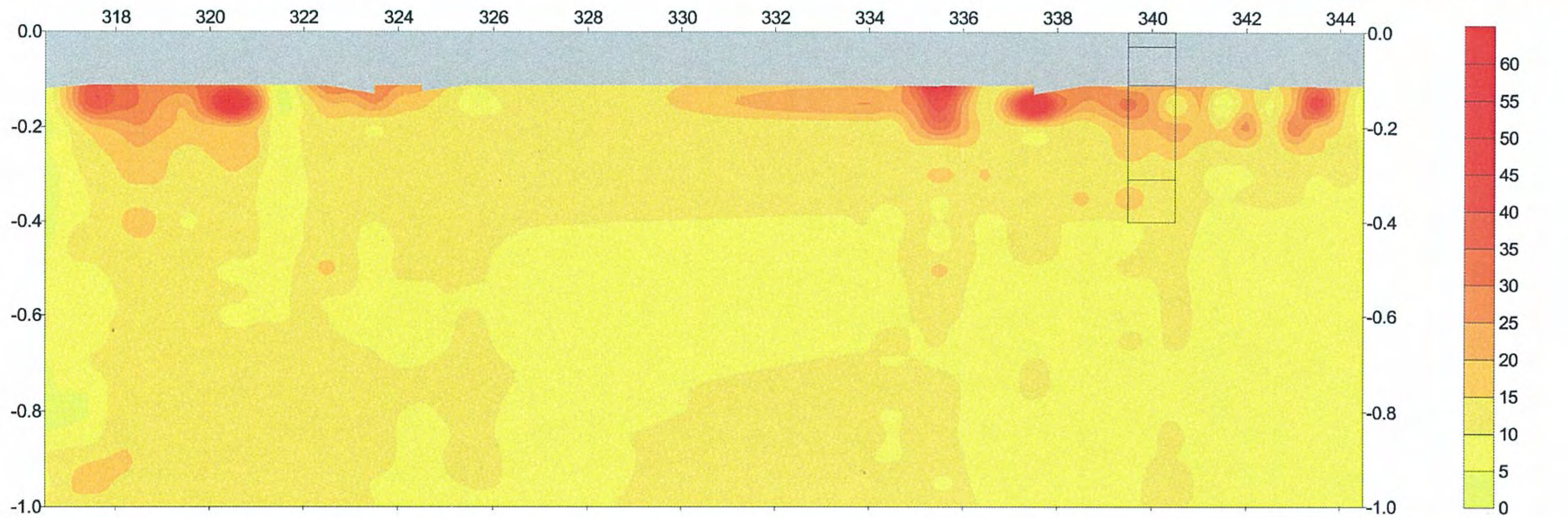
Test Pit, Ch 340

0 to -0.030	Grade 5 Chipseal
-0.030 to -0.110	Basecourse: Dense, unweathered greywacke, AP40
-0.110 to -0.310	Sub base: very dense, slightly weathered greywacke, WHAP 65
-0.310 to -0.400	Subgrade: Dense, slightly silty medium sand



SASW Results (Shear Modulus MPa) plotted vs Distance (m) and Depth (m). (Blanked out area: No Data)

Figure D6 iGEO Site No. 18, Pauanui Boulevard, Thames-Coromandel District. Sampling rate 4000Hz, Source small ball peen hammer.

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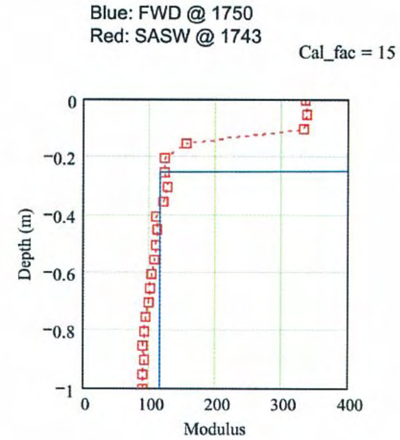

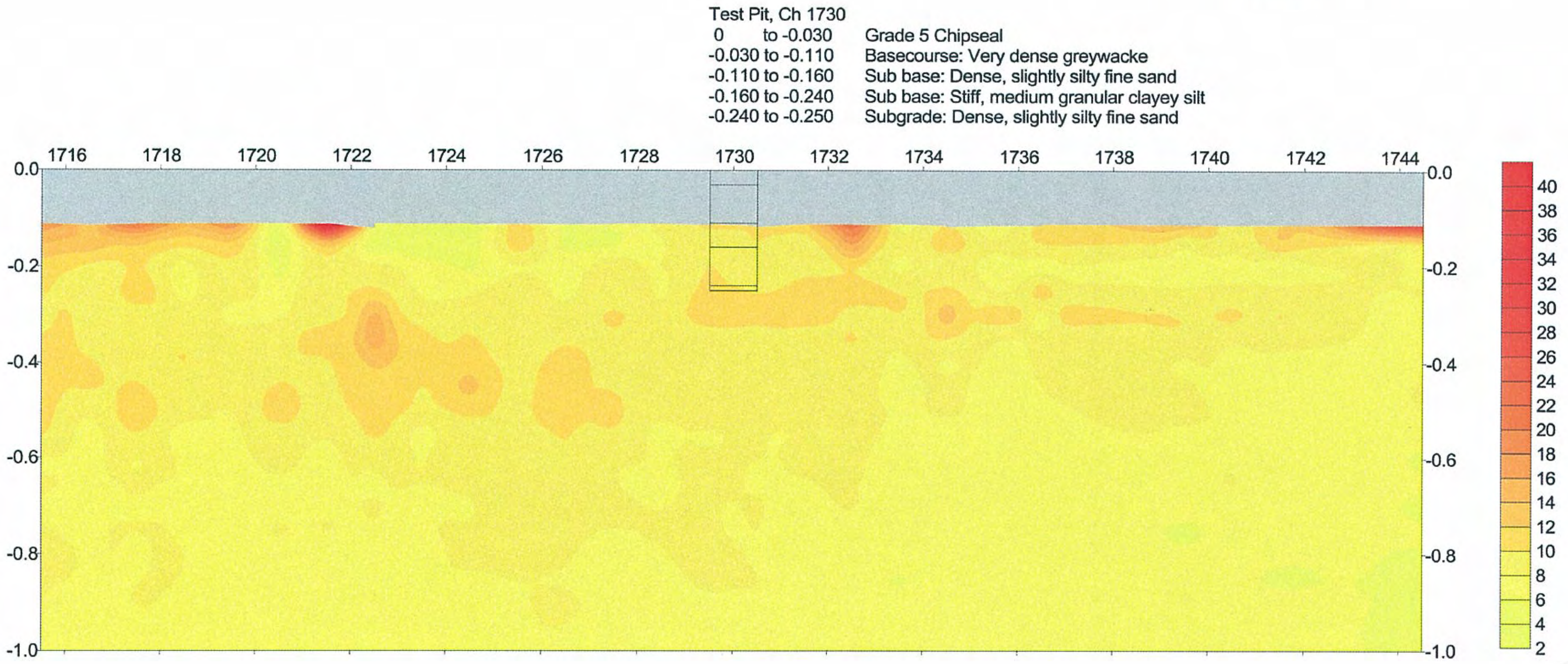

SASW Results (Shear Modulus MPa) plotted vs Distance (m) and Depth (m). (Blanked out area: No Data)

**Figure D7 iGEO Site No. 20, Jellicoe Road, Auckland.
Sampling rate 1000Hz, Source small ball peen hammer.**

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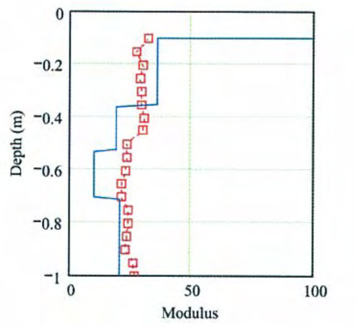
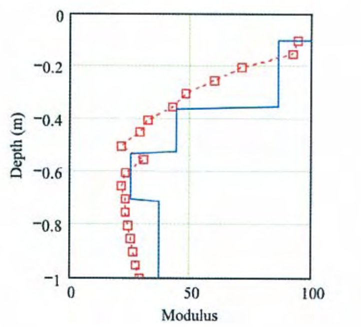
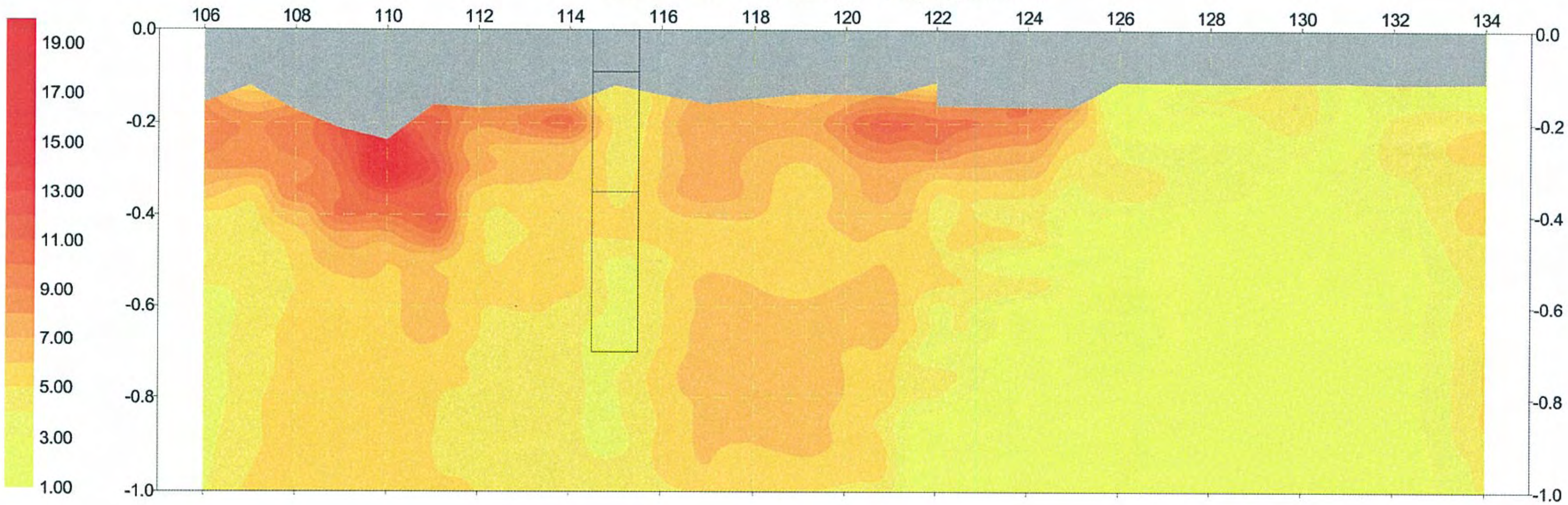




Test pit 2, Ch 115
0 to -0.090 Asphaltic concrete
-0.090 to -0.350 Silty Gravel, fine to coarse, GAP40
-0.350 to -0.700 Silty Gravel, fine to very coarse, GAP65



SASW Results (Shear Modulus MPa) plotted vs Distance (m) and Depth (m). (Blanked out area: No Data)