PAVEMENT DEFLECTION MEASUREMENT & INTERPRETATION FOR THE DESIGN OF REHABILITATION TREATMENTS

Transfund New Zealand Research Report No. 117

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AN IMPORTANT NOTE FOR THE READER

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EXECUTIVE SUMMARY

A mechanistic design procedure has been adopted by Transit New Zealand for rehabilitation projects on New Zealand roads. Deflection testing and back-analysis of the deflection bowl induced by a standard wheel load provide the principal parameters for mechanistic design.

Most structural evaluation using deflection testing has been carried out with the Falling Weight Deflectometer, with minor work being done with the instrumented Benkelman Beam. Considerable experience is now available world-wide with the interpretation of deflection bowls, but most of this work has been related to pavements with thick asphaltic surfacings. Relatively little information is available regarding the back-analysis of unbound granular pavements with thin asphaltic or chipseal surfacings, which are in common use in New Zealand. To address this problem, research was initiated in 1996 to study about 30 pavement sections, most being of unbound granular construction together with a small number of sections containing either thin asphaltic surfacing or cement-stabilised basecourse. A wide range of subgrade types have been included and there are also comparative studies of conditions before and after rehabilitation treatment, to verify the mechanistic designs.

This, the first of two reports for this project, is intended to complement existing documents, in particular the AUSTROADS Pavement Design Guide, its New Zealand Supplement and NZIHT (NZ Institute of Highway Technology) course notes for mechanistic analysis. The objective is to provide New Zealand practitioners with concepts and parameters applicable to local conditions, as well as to provide a more detailed appreciation of key principles that are applied in mechanistic analysis. Quality assurance for both the field data and the design results is addressed. A set of examples and results for different overlay methods is also supplied to show how to obtain insight into pavement behaviour, to examine distress mechanisms, and to select the most appropriate rehabilitation treatment.

A companion report providing more specific detail and addressing the 30 pavement sections under study is the Transfund New Zealand Research Report "Pavement Evaluation and Deterioration Modelling for New Zealand Conditions" which is in preparation.

ABSTRACT

A mechanistic design procedure has been adopted by Transit New Zealand for rehabilitation projects on New Zealand roads. Deflection testing and back-analysis of the deflection bowl induced by a standard wheel load provide the principal parameters for mechanistic design.

Research was initiated in 1996 to study about 30 pavement sections, most being of unbound granular construction together with a small number of sections containing either thin asphaltic surfacing or cement-stabilised basecourse. A wide range of subgrade types have been included and there are also comparative studies of conditions before and after rehabilitation treatment, to verify the mechanistic designs.

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1. INTRODUCTION

A mechanistic design procedure has been adopted by Transit New Zealand for designing rehabilitation treatments for New Zealand roads. A computer program such as CIRCLY (Wardle 1980) is used to analyse the reaction of various pavement rehabilitation designs (modelled as multiple layers of linear elastic materials) under a standard wheel load. Other programs such as ELMOD include allowance for non-linear elastic material. Strains within various critical layers are computed for each rehabilitation design being considered. The designs which are acceptable are those which meet or exceed specific performance criteria for asphalt, cemented bases and subgrade layers. Mechanistic design has the advantage of allowing the design of a range of rehabilitation treatments including: strengthening the existing pavement layers (stabilisation or other means); granular overlay; asphalt overlay; or any combination of these.

The requirement to determine the elastic material properties of each pavement layer for mechanistic design is now a principal issue for the pavement designer. One method to determine the elastic modulus of the pavement materials is to use either the Falling Weight Deflectometer (FWD) or the instrumented Benkelman Beam, with appropriate software. The FWD applies a load to the pavement and deflections are measured directly under the load and at set distances from the load (i.e. the deflection bowl is recorded). These recorded deflections combined with information on the load, layer thicknesses and material types are processed by back-analysis software to estimate the modulus of each pavement layer. Some software, in addition to automatically determining the moduli of the pavement layers, will determine the overlay depth required for the future design traffic. This report describes the use of the FWD, analysis procedures, and interpretation of the computed pavement layer moduli and overlay depths to aid in determining an appropriate rehabilitation treatment, by following the procedures detailed in Section 10 of the *New Zealand Supplement to the AUSTROADS Pavement Design Guide* (Transit New Zealand 1997).

Most documentation on deflection testing relates to structural asphaltic pavements. This report draws on local experience with unbound granular pavements used on roads throughout New Zealand, as well as material from Ullidtz (1987), Sweere (1990), the AUSTROADS Pavement Design Guide (1992) and the New Zealand Supplement to the AUSTROADS Guide (Transit New Zealand 1997). It is intended for use by practitioners, and it addresses only the main concepts and their application. Greater detail and results of ongoing research are supplied in a companion report, *Pavement Evaluation and Deterioration Modelling for New Zealand Conditions* (Tonkin & Taylor, in prep).

2. REHABILITATION DESIGN METHODS

2.1 General

In July 1995 Transit New Zealand Authority approved the adoption of the AUSTROADS pavement design procedures as described in the document *Pavement Design - A Guide to the Structural Design of Road Pavements* (AUSTROADS 1992). This Guide superseded the existing Transit New Zealand (1989) State Highway Pavement Design and Rehabilitation Manual (SHPDRM).

A New Zealand supplement to the AUSTROADS Pavement Design Guide was produced by Transit New Zealand in November 1995, and revised in July 1997, to address pavement design issues which are unique to New Zealand.

The method for rehabilitation design adopted by Transit New Zealand is described in Section 10 of this New Zealand Supplement. It describes a mechanistic procedure for the design of rehabilitation treatments and replaces the procedures for unbound granular pavement design described in Chapter 10 of the AUSTROADS (1992) Pavement Design Guide.

Before the release of the July 1997 revision of the New Zealand Supplement, other AUSTROADS rehabilitation design methods were being trialled on New Zealand roads and these are briefly described in the following sections of this report.

2.2 AUSTROADS (1992) Pavement Design Guide

Chapter 10 of the AUSTROADS (1992) Pavement Design Guide describes a method for the design of unbound granular or asphaltic concrete overlays. The design method is based on the following two deflection parameters, D_0 and D_{200}:

D_0 = the maximum (central) deflection generated by the dual tyre of a standard 8.2 tonne axle.

D_{200} = the deflection measured 200 mm from the point at which the maximum deflection was produced (in the direction of travel).

The deflections are used to determine the Curvature Function (CF):

$$CF = D_0 - D_{200}$$

The AUSTROADS (1992) method is still used in Australia but it has not been adopted by Transit New Zealand for unbound granular overlays.

2.3 General Mechanistic Procedure (GMP)

The GMP procedure (ARRB 1994) uses deflection bowls, combined with information of the pavement structure, the condition of pavement materials, and computer programs, to determine the appropriate thickness of an asphalt or granular overlay needed to remedy the structural deficiencies of existing pavements.

Although this procedure has not been adopted by AUSTROADS, the concepts are widely accepted and are used in the mechanistic design procedure for rehabilitation treatments described in Section 10 of the New Zealand Supplement (Transit New Zealand 1997).

2.4 AUSTROADS Simplified Mechanistic Overlay Design (ASMOL)

The ASMOL design procedure (ARRB 1994) was developed from the GMP to cater for conventional highway traffic loading on pavements constructed with fine-grained subgrades. Pavement modelling was used to develop equations to estimate critical strains from the measured deflection bowls, layer thicknesses and test temperatures. Procedures were developed to correct the estimated critical strains to the values at the weighted mean annual pavement temperature. Another series of equations was developed to reduce the existing strains to tolerable design levels. In practice the method requires limited knowledge of the pavement layering, and usually three deflection parameters, namely D_0, D_{300} and D_{900} (for unbound granular pavements) where the subscripts refer to the distance in millimetres of the measured deflection from the point of maximum deflection. The 1997 ASMOL revision replaces D_{300} with D_{200} for some pavement types (ARRB 1997, Moffatt et al. 1997).

ASMOL was partially revised in 1997, but the procedure is still regarded as interim and has not been adopted by AUSTROADS.

2.5 RRU Bulletin 79 Design

NRB RRU (Road Research Unit) Bulletin 79 (Shepard 1989) provides guidelines for selection, design and construction of thin flexible bituminous surfacings for roads in New Zealand. While its recommendations for asphaltic concrete are now superseded, it specifies deflection criteria for friction surfacing mix (FSM). The latter is not addressed specifically by the AUSTROADS Pavement Design Guide, or by the New Zealand Supplement.

FSM is more flexible and less expensive than asphaltic concrete. It can tolerate reasonable pavement deflections and provides excellent skid resistance.

RRU Bulletin 79 recommends deflection criteria for friction course suitability in relation to traffic usage as given in Table 2.1.

Table 2.1 Deflection criteria in relation to Annual Average Daily Traffic (AADT).

Traffic (AADT)	D_0 (Maximum 95% ile)	D_{250}/D_0 (Minimum 95% ile)
> 5000	1.1 mm	0.62
500-5000	1.6 mm	0.54
< 500	2.4 mm	0.47

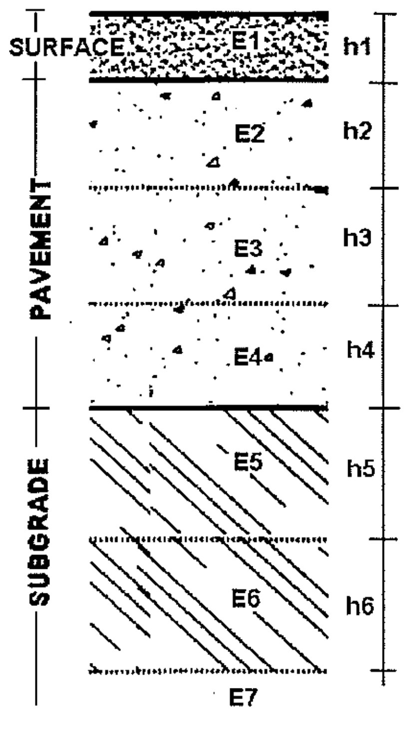
D_0 and D_{250} are the Benkelman Beam deflections at 0 mm and 250 mm offsets. Information on the basis of the criteria has been difficult to find. However, experience reported by Tonkin & Taylor (in prep.), suggests that the guidelines are good indicators although replacement of the AADT categories with corresponding ESA (equivalent standard axle) loadings would be more meaningful.

2.6 New Zealand Supplement to the AUSTROADS Guide

2.6.1 Mechanistic Design Procedure

Transit New Zealand adopted a mechanistic design procedure for rehabilitation treatments that is described in Section 10 of the New Zealand Supplement (Transit New Zealand 1997). This describes a mechanistic procedure, where the designed rehabilitated pavement (e.g. existing pavement plus an overlay) is modelled as multiple layers of linear elastic materials (Figure 2.1).

Figure 2.1
Existing pavement modelled as multiple layers of linear elastic materials (E1, E2 ... E7 = Elastic moduli of layers 1, 2...7; h1, h2 ...h6 = thickness of layers 1, 2...6).



Using CIRCLY (Wardle 1980), or a similar program, an 8.2 tonne dual-tyred axle (ESA = 1) is applied, and the compressive vertical strain at the top of the subgrade and the horizontal strain at the bottom of any bound layers are computed. The total number of allowable ESAs to failure of the rehabilitated pavement is then calculated for each layer (subgrade and any bound layers) using a strain criterion of the form:

$$Max\ ESAs = \left(\frac{const}{strain}\right)^{exp} \tag{1}$$

The constants (const, exp) are different for each material type (i.e. asphalt, cemented base or subgrade). The rehabilitation treatment is acceptable when the allowable ESAs for all layers are less than the future design traffic loading.

For the rehabilitation design of flexible unbound granular pavements, the principle criterion is the design vertical compressive strain at the top of the subgrade (ϵ_{des}) as defined by Equation 2 in this report (Equation 10.3 in NZ Supplement 1997):

$$\epsilon_{des} = \epsilon_{cvs} (N_F/N_P)^{-0.23} \tag{2}$$

where:

N_F/N_P = ratio of future traffic to past traffic

ϵ_{cvs} = compressive vertical strain in the subgrade computed under the existing pavement prior to rehabilitation for each individual point:

or ϵ_{cvs} = $\bar{x} - fs$ (3)

where:

$\bar{x}$ = mean of the existing vertical compressive strains at the top of the subgrade computed for all the layered existing pavement structures, developed with similar subgrade soil types.

s = standard deviation of the existing vertical compressive strains at the top of the subgrade computed for all the layered existing pavement structures developed with similar subgrade soil types.

f = pavement condition factor which shall be 0 unless it is considered that more or less than 50% of the road section has reached a terminal serviceability condition (e.g. rut depth >20 mm), in which case the appropriate value of f (derived from the normal distribution) is given in Table 2.2.

Table 2.2 Appropriate values of pavement condition factor (f) for Equation 3.

f	% Road Section in Terminal Serviceability Condition
-1.5	5
-1.2	10
-0.84	20
-0.53	30
-0.25	40
0	50
0.25	60
0.53	70
0.84	80

Equation 2 considers the pavement's past performance and may not always be appropriate where a major change in road use is expected (e.g. a rural road that has only previously carried light traffic is upgraded to a standard capable of supporting heavy vehicles), or where estimating the ratio of future to past traffic is extremely difficult, or where the primary distress mode is not related to permanent strain in the subgrade. In these situations the AUSTROADS subgrade strain criterion is used as defined in Equation 4:

$$\epsilon_{des} = 0.008511(N_F)^{-0.14} \text{ or } \epsilon_{des} = 0.0093(N_F)^{-1/7} \qquad (4)$$

where:

ϵ_{des} = limiting design vertical compressive strain at the top of the subgrade,
N_F = design future traffic (ESA).

The alternative equation (with rounded coefficients) is recommended by Moffatt & Jameson (1998) based on back-analyses, using the dual wheels on both ends of the standard 8 tonne axle as well as a revised sub-layering procedure for granular materials (Section 4.3.7).

In addition to the rehabilitated pavement being required to spread the load to satisfy the appropriate subgrade strain criterion, the pavement materials need to have sufficient strength to resist the shear forces imposed by the design traffic loading. The CBR (California Bearing Ratio) of the pavement material gives an indication of the available shear strength and this is checked where the existing pavement materials are suspected to be of poor quality. Section 10.3 of the New Zealand Supplement (Transit New Zealand 1997) describes in detail the procedure to follow to check for shear strength.

The FWD used with appropriate software is an efficient tool to aid the pavement designer in determining an appropriate rehabilitation treatment using the New Zealand Supplement method and is discussed in Section 2.6.2.

2.6.2 Example of Rehabilitation Design Incorporating Past-Performance Method

The advantage of using Equation 2 for the subgrade strain in the mechanistic design of rehabilitation treatments is that it makes maximum use of precedent, i.e. the past traffic loading which has demonstrably been sustained, is used to predict future performance.

An illustration of the past-performance method for assessing rehabilitation treatments is shown in Figure 2.2. Using back-analysis of deflection bowls, the subgrade strains are calculated for a pavement which has reached the end of its design life. The number of strain repetitions to date are estimated from the pavement age and historic traffic data. By plotting these parameters onto the diagram showing recognised strain criteria (Ullidtz 1987), the actual strain susceptibility of a specific subgrade may be compared with that expected for "conventional" soils.

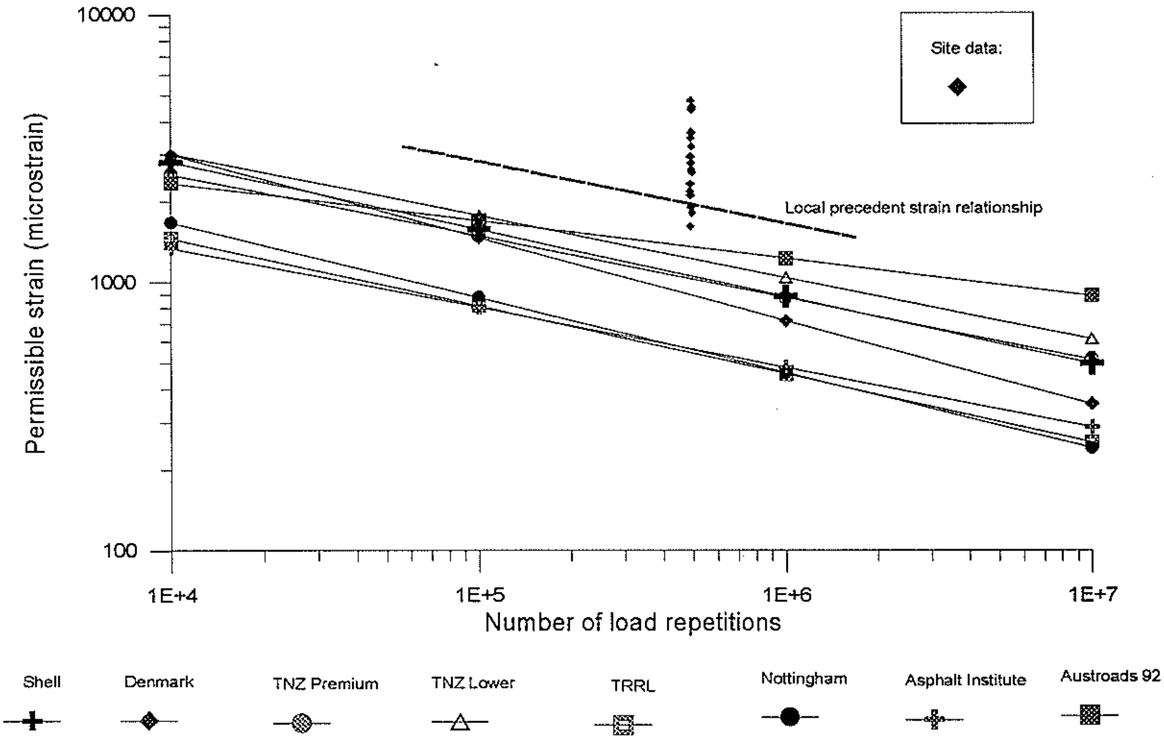


Figure 2.2 Deflection-bowl analysis using precedent strains for determining "local" strain criteria (from Ullidtz 1987).

If the conditions at the time of testing can be shown to be typical of those occurring historically, and the serviceability of the pavement has not been significantly affected by routine maintenance, then a "local precedent" design criteria can be established, as shown in Figure 2.2. The ordinate for the design relationship can be assessed assuming a normal distribution and the appropriate factors from Table 2.2. If the strains in the road do not follow a normal distribution, then an appropriate percentile can be selected graphically (depending on the proportion of the road exhibiting a terminal condition), and the gradient should be parallel to the recognised strain criterion.

The strain criteria gradient can be shown to be the reciprocal of the traffic loading equivalence (Ullidtz 1987). The latter has normally been regarded as approximately a 4th power relationship (derived from the AASHO Road Test, 1961). However AUSTROADS has produced a 7th power relationship, as the result of an indirect back-analysis of CBR-pavement thickness design curves. For this reason, the 4th power law may perhaps be regarded as having a slightly more substantive origin and has been adopted by in the New Zealand Supplement for deriving local precedent strain criteria. Additionally, the 4th power law (strain gradient of 0.25) is less likely to result in unconservative projections of sustainable traffic loadings.

Analyses of the past performance of several pavements built on unweathered volcanic ash in New Zealand have been carried out recently, using the above method. Preliminary indications are that a subgrade strain criterion of 1.5 to 1.75 times higher than that used for "conventional" soils can be invoked. Unweathered volcanic ash appears to provide unusually high resistance to permanent strain accumulation, probably attributable to the very high shear resistance provided by its sharply angular grains.

3. FALLING WEIGHT DEFLECTOMETER (FWD) & INSTRUMENTED BENKELMAN BEAM

3.1 General

Back-analysis of a measured deflection bowl is a widely accepted method for estimating the elastic properties of the existing pavement materials as required for the mechanistic design of rehabilitation treatments. Both the FWD and instrumented Benkelman Beam can be used to measure the deflection bowl. The FWD has been used in this research to measure deflection bowls on pavements throughout New Zealand. The instrumented Beam is briefly mentioned in this report for comparison with the FWD.

3.2 Equipment

The FWD has been developed from the "déflectométre à boulet" originally devised by Bretonniere (1963). A force pulse is applied to the road surface by a specially designed loading system which represents the dynamic short-term loading of a heavy wheel load. This produces an impact load of 25-30 ms (millisecond) duration, and a peak force of up to 120 kN (adjustable). The deflection bowl response of the pavement is measured with a set of 7 precision geophones at a range of distances from the loading plate. FWD equipment is produced by three main manufacturers, two Danish and one Swedish, each with essentially similar field recording systems.

The Benkelman Beam, instrumented for automatic recording of the full bowl shape, measures responses under a slower and variable loading time. As the wheel load is positioned close to the point of maximum deflection during set up, the effective load duration is longer at close offsets than at the more distant points.

General layouts of both the FWD and instrumented Benkelman Beam are shown in Figures 3.1 and 3.2.

3.3 Supporting Software

Software specific to each FWD is supplied to capture field data, and to display it via a laptop computer to the driver/operator. The primary information obtained is the stationing, temperature, time history of loading and deflection (Figure 3.4) although only the peak values for each test are normally stored. The Dynatest system (one of three products readily available) has been used in New Zealand and produces plots showing the full time histories as well as peak values. Surface moduli are also plotted by the field software as this parameter can be determined explicitly as detailed in Section 3.7 of this report.

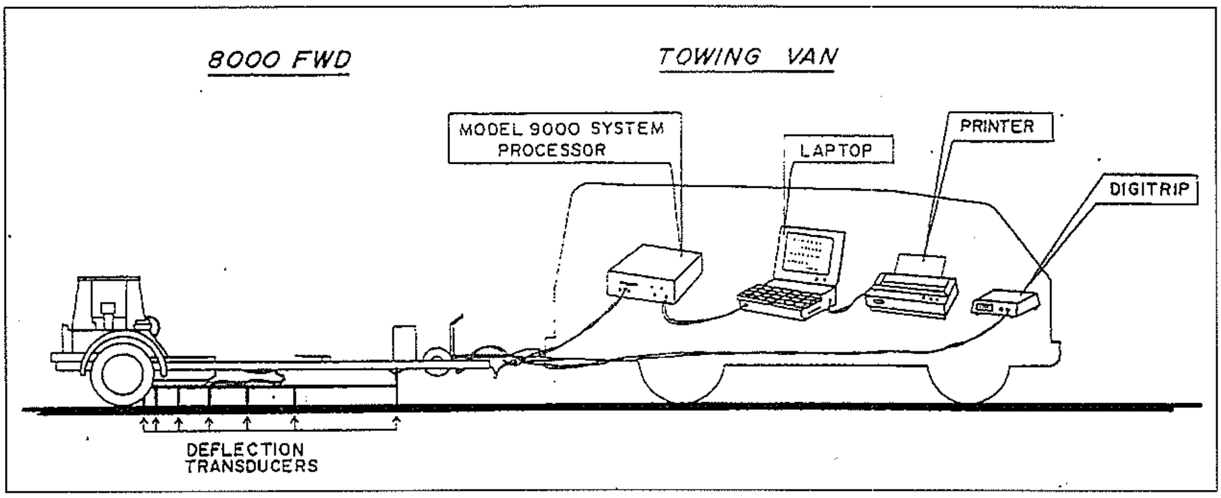


Figure 3.1 Falling Weight Deflectometer assembly.

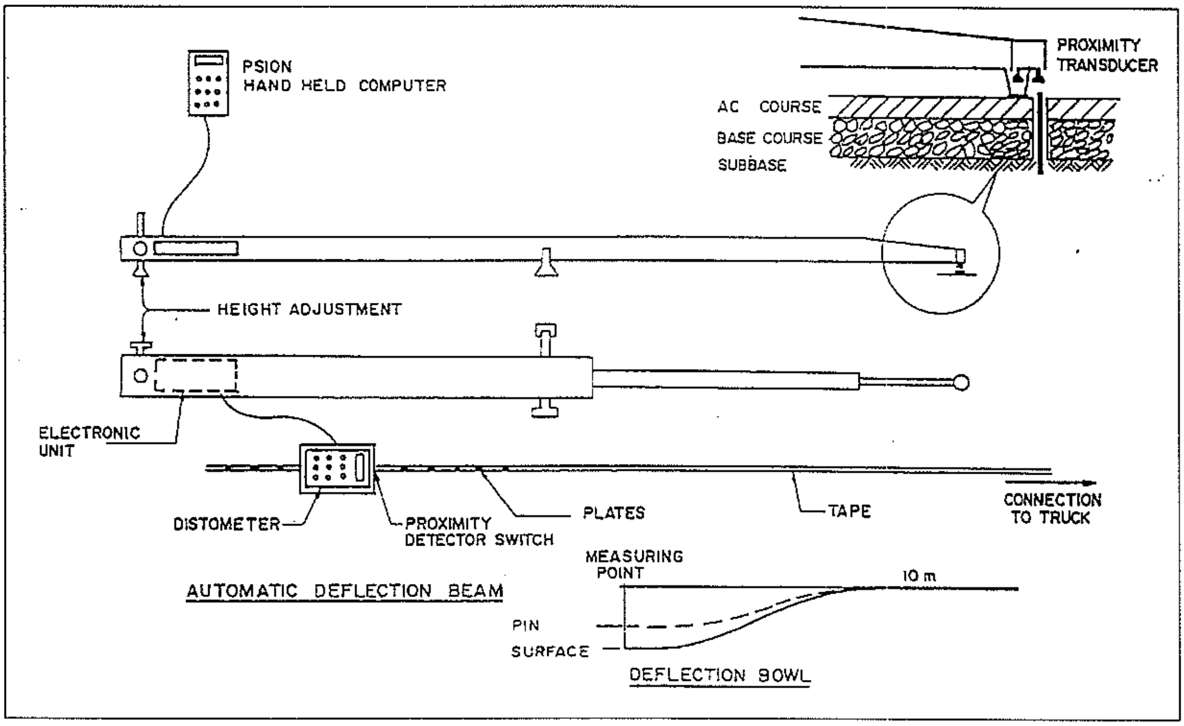


Figure 3.2 Instrumented Benkelman Beam assembly.

3.4 Comparison between FWD & Instrumented Benkelman Beam in Relation to a Moving Wheel Load

The ideal duration of a pavement test load should correspond to that of a moving wheel velocity of 60 - 80 km/h. This velocity is important because it affects the load duration and therefore the measured deflections which relate to the visco-elastic characteristics of the asphalt layers and the elasto-plastic response of the subgrade.

The response of pavement structures to the FWD, the Beam, and to loading by a heavy truck wheel has been compared on several instrumented test roads (Ullidtz 1973). In that research stresses, strains and deflections were measured under comparative conditions. As a result of the design of the FWD loading system, the responses under the FWD and moving wheel load are practically identical. On the other hand, Ullidtz has shown that no simple correlation exists between the Benkelman Beam and the moving wheel load. The relation is very dependent upon the specific visco-elastic responses governed by the dynamic characteristics of the asphalt layers and subgrade.

It is concluded that if the deflection bowl is measured under an FWD test, and the theory of elasticity is then used to determine the moduli of the individual layers that would produce the same deflection bowl, then the resulting layer moduli will be representative of the pavement materials under moving traffic loads. Because of its longer loading period, the instrumented Benkelman Beam cannot be used as directly.

Using a dynamic loading device is clearly preferable. Ideally the analysis should also be dynamic and research has been continuing into this aspect. As yet however there is no widely recognised dynamic analysis procedure, partly because of the computational time required (Ullidtz & Coetzee 1995).

A comparison between the central deflections of the Benkelman Beam and FWD is important in order that the substantial body of experience and empirical relationships obtained with the Benkelman Beam can be used as a broad check on interpretations made using the full deflection bowls measured by either the FWD or Instrumented Beam.

There is no universal comparison because the ratio of Benkelman Beam to FWD central deflection is a function of the pavement composition (elastic properties of the pavement materials and the subgrade). It is however possible to obtain consistent ratios on any one pavement type. Paterson (1987) reports:

> *The loading applied by FWD is currently considered to be more similar to traffic loading in both the load and time domains than either the Benkelman Beam test (which applies similar loads at creep speed) or the light-loading, high frequency devices. Under similar applied loads, the ratio of FWD to Benkelman Beam deflections ranges from 0.8 to 1.35 for asphalt-surfaced pavements. Thus as a reasonable first approximation, in the absence of*

specific local correlations, is to equate the FWD deflection (after correction for the applied load) to the Benkelman Beam deflection.

Paterson apparently drew his conclusions from the work of Tholen et al. (1985) who collated data from a number of projects using different pavement types but found no correlation.

To examine the theoretical relationship between the two loading devices, calculations were carried out using CIRCLY (Wardle 1980) and also the finite element program FLEA (University of Sydney 1994) as a check.

A total load (40 kN) was applied initially to two discrete circles spaced 330 mm between centres, and the deflection was computed midway between the "tyres" to simulate the dual wheels of the Benkelman Beam truck. The same 40 kN load was then applied over a 300 mm circular area with a central hole to simulate the FWD loading plate. The deflections between the dual wheels and directly under the FWD loading plate were computed for comparison.

Both methods of analysis produced a theoretical Beam : FWD central deflection ratio of much less than 1 (slightly dependent on layer moduli). This was a surprising result in view of the generally accepted higher correlations. It is important to appreciate that these analyses relate to a continuum (i.e. a material which is continuous rather than the assemblage of discrete particles as found in a granular layer). Therefore the theoretical results may be expected to be more appropriate to very dense pavements (with low deflections) than unbound granular layers.

Also as part of ongoing research in New Zealand, Beam : FWD central deflection ratios were determined for one unbound granular pavement with thin friction course surfacing and for one structural asphaltic pavement (at the CAPTIF test track) giving respective Beam : FWD ratios of 1.05 and 1.22 respectively. The CAPTIF data, obtained from recent research at the University of Canterbury, allowed precise positioning of both Beam and FWD, producing a high correlation.

Using data from Tholen et al. (1985), together with the local information, there appears to be a slight trend for greater Beam : FWD ratios with greater overall deflection (Figure 3.3). This result is not unexpected when the difference between the loading times and mass-inertia effects are considered. This result is discussed in Section 3.6.4.)

The data supports the conclusions by others that there is no real correlation (even when plotted logarithmically) and that site-specific correlations should be carried out. This correlation should preferably be made by direct reading. Indirect correlations could be carried out using a program such as CIRCLY, but limited experience suggests that such theoretical approaches can yield Beam : FWD ratios which are lower than achieved in practice. As an interim guide, the following approximations taken from Figure 3.3 are suggested :

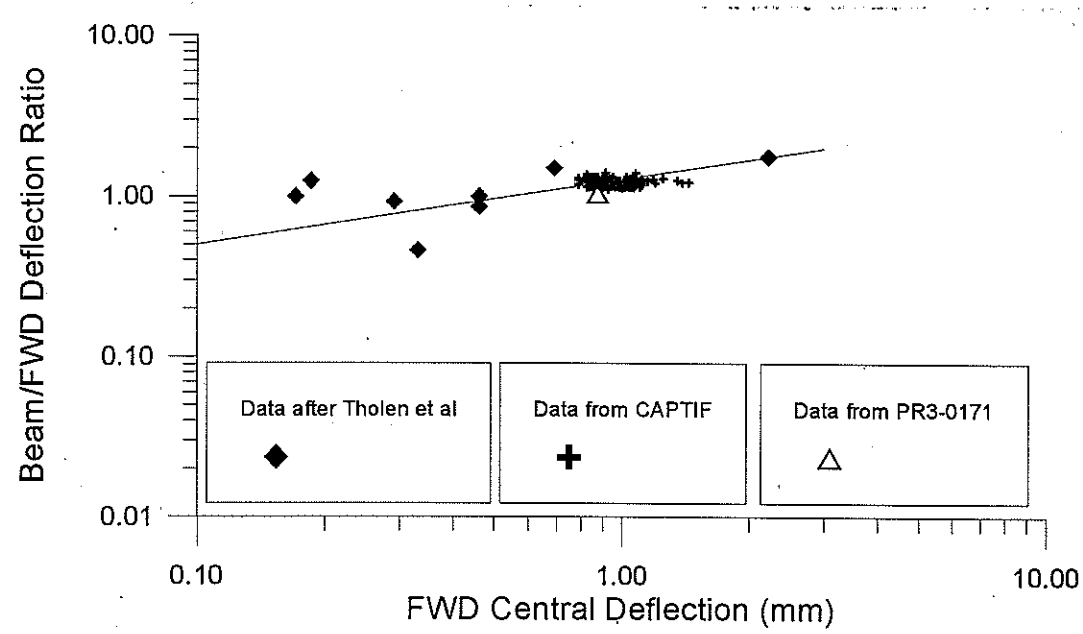
- Where deflections are less than 1 mm, under a 40 kN FWD impact load, adopt a Beam : FWD ratio of about 1.1.

- Where deflections exceed 1 mm, the ratio is likely to be in excess of 1.1, and related to deflection as defined by Equation 5:

$$\text{Beam : FWD ratio} = 1.1 \times (\text{FWD deflection in mm})^{0.4} \tag{5}$$

This relationship is intended only for use with simplified overlay design methods, which require Benkelman Beam deflections (AUSTROADS 1992, Chapter 10). For determining the elastic properties of the existing pavement for mechanistic design of rehabilitation treatments, deflection bowls measured with either a Benkelman Beam or an FWD with no adjustment are used. When back-analysing the deflection bowls, the user informs the program of the loading geometry that was used to obtain the deflections. However conversion between FWD and Benkelman Beam deflections is unnecessary when mechanistic analysis is adopted for pavement rehabilitation design.

Figure 3.3 Comparison of Benkelman Beam and FWD central deflections (using a 41 kN load). (Tholen et al. 1985; CAPTIF University of Canterbury; PR3-0171 this project)



3.5 Accuracy

Because no reference point (or support) is needed for the FWD deflection bowl measurement, the deflections can be measured with high accuracy. Ullidtz (1987) indicates a typical accuracy of 0.5% ± 1 μm, and New Zealand experience supports the claim. This accuracy is necessary because the subgrade modulus must often be determined from deflections of only 20-30 μm. The accuracy of the geophones can be readily checked at any time in the field by setting all sensors vertically above one another in a special test frame in order to confirm identical amplitudes and responses.

The accuracy of the FWD deflections is further ensured by carrying out measurements two to three times at each point to assess repeatability. This will allow the effects of different loadings to be evaluated and identify any external factors such as passing vehicles which may have affected results.

The Benkelman Beam test has somewhat lesser accuracy and repeatability in practice, owing to the effects of proximity of its supporting legs, load reversal and accuracy in repositioning. For this reason, only one test is normally carried out at each position.

3.6 FWD Test Procedures

3.6.1 General

During normal operation, the total test sequence is controlled from the driver's seat of the towing van, and the results are automatically stored on disk, for later uploading and processing. Generally 200 to 300 points may be tested during one day, i.e. up to 15 lane kilometres of testing (at 50 m centres) at project level or more for network level appraisals.

3.6.2 Loading

The FWD load is normally adjusted in the range of 35 to 50 kN, to produce maximum deflections towards the upper limit of the geophone capacity (i.e. about 2 mm). Alternatively, as at least one seating load pulse and 2 or 3 recordings are made at each site, a sequence of 35, 40 and 50 kN impacts may be automatically applied to examine stress dependence more closely. The effective impact is changed by varying the drop height (pre-selected from a set of proximity sensors adjacent to the falling weight guide mechanism).

A standard 8.2 tonne dual-tyred (inflation pressure of 550 kPa) axle is the applied load for the Benkelman Beam test. It is not necessary to adjust the load as, in practice, there is no upper limit for measuring deflection.

3.6.3 Selection of Offset Distances for Deflection Bowl Measurement

Deflection measurement positions are controlled manually. On the FWD, geophones are clamped in the required positions.

With the instrumented Benkelman Beam a positioning tape with small metallic plates at the required offsets is attached to the rear of the loading truck and plate locations are sensed with a proximity transducer as the truck moves.

Recommended offset distances for determining the elastic properties of a pavement depend on the overall stiffness of the pavement layers. For a typical unbound granular pavement, deflections would be recorded at: 0, 300, 450, 600, 900 1200, and 1500 mm distances from the centre of the load.

For a very thick granular pavement, cement-stabilised basecourse or thick asphaltic concrete pavement, greater spacing may be required to ensure that the three outermost measurement points are governed by subgrade response, as explained in Section 3.7.3 of this report.

If bowl shapes are recorded at offsets other than those required, specific values may be determined using interpolation, provided the full bowl shape is reflected. The FWD applies its load on a semi-rigid plate, and trials show that the deflections at 200 and 250 mm offsets can best be calculated using a curvilinear interpolation routine (e.g. Lagrange method) with the assumption that the maximum deflection occurs continuously over a 60 mm-radius circle.

3.6.4 Field Recording

The FWD records the geometry of the deflection bowl and the maximum impact load from a stress sensor above the loading plate. The FWD may also be set to record the full time history of stress and deflection, by sampling at 0.2 ms intervals. An example of the latter is presented in Figure 3.4.

Figure 3.4 Typical FWD record of geophone displacement (microns) v. time (milliseconds).

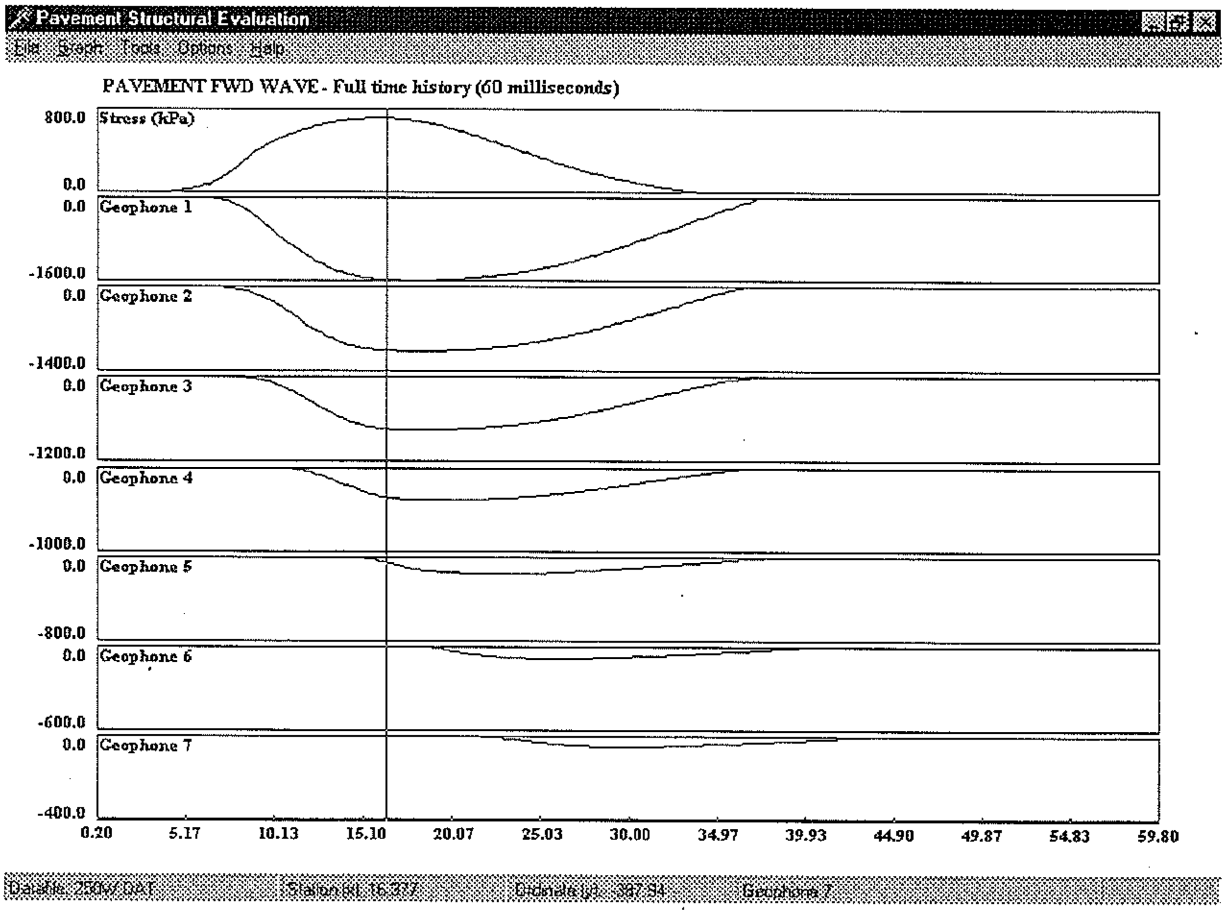


Figure 3.4 shows that the outer geophones hardly begin to respond before the stress pulse reduces to almost zero. It is clear that the mass inertia of the pavement layers above the subgrade makes a significant contribution to the deflection bowl response to impact loading. Considerable theoretical investigation of this effect has been carried out, comparing the frequency response functions obtained from FWD load-time histories with those calculated using sophisticated elasto-dynamic models of layered systems. However so far, the implementation of these models for pavement design has proven too demanding for routine evaluation (Ullitdz & Coetzee 1995, Stolle & Peiravian 1996).

3.6.5 Unbound Basecourse with Chipseal Surfacing

Testing of unbound basecourse is normally carried out in the left wheelpath at 50 m intervals, or closer where anomalies are detected. Closer spacing is also used on short sections to obtain a minimum of 30 tests for analysis. When testing in the opposite lane, test locations are staggered evenly between those in the initial lane to give coverage at 25 m centres. With the FWD, at least two tests are carried out at each site with checks (discussed in Section 3.7) made for repeatability, consistency of bowl shapes and surface moduli. Beam readings are not normally repeated, but test spacing may be closer to give some compensation for this lack of repeatability.

3.6.6 Asphaltic Concrete

Testing of asphaltic concrete or friction course is similar to procedures for unbound basecourse except that the temperature of the surface layer is measured regularly, and results are entered on the FWD file.

3.6.7 Seal Extension

Testing of unsurfaced (loose gravel) roads or subgrades is quite practical with the FWD as repeated tests at a given point are carried out in quick succession until consistent results are obtained. Testing is carried out as for unbound basecourse. The Beam is limited to very firm surfaces as local heave between the loaded dual wheels can easily invalidate results, and repeat testing of each site is not normally carried out because the precise re-location of the dual tyre is time-consuming.

3.6.8 Widening, New Construction and Construction Monitoring

Testing for widening is carried out in the same way as for seal extension except that testing is carried out in the area of widening rather than in the existing wheelpath. Tests in the left wheelpath may also be useful for determining the effectiveness of the existing design, and estimating likely equilibrium values for subgrade moduli beneath the new road widening.

For new construction or construction monitoring, testing is carried out as for seal extension, but judgement regarding likely seasonal changes of moisture content is required during analysis. New pavements also show relatively low moduli for the basecourse (and sub-base) even though they may be thoroughly compacted. Further densification with substantial improvement in basecourse moduli will occur in an unbound granular pavement during the first 10,000 to 20,000 ESA of trafficking.

Somewhat longer trafficking is required to achieve full densification beneath a structural asphaltic surfacing or in sub-base materials.

3.7 Quality Assurance & Interpretation of Deflection Bowls

3.7.1 Repeatability

Repetition of tests in the same position is carried out routinely for FWD surveys. Usually, results will be within a few percent, i.e. inconsequential in relation to differences between adjacent test points. The FWD automatically displays the successive deflection bowls graphically for identification of anomalies, and rejection or further repetition of the test.

3.7.2 Rational Deflection Bowl Shapes

A normal deflection bowl will give decreased deflection with increasing offset distance. The Dynatest FWD prints a warning message at the time of test if this criterion is not met, and the test may be rejected, then repeated. Readings are also rejected if any of the geophone readings are affected by vibrations which are occasionally significant when a heavy vehicle passes while the test is in progress.

3.7.3 Surface Moduli Plot, Subgrade Modulus, CBR and Soil Type

The most effective means for quality assurance of the data collected in the field is to inspect the surface moduli plot corresponding to each test drop in the sequence at each test point. The surface modulus is the "weighted mean modulus" of an equivalent half space of a material with uniform modulus. This concept of "overall apparent stiffness" at any point is important both for the operator's understanding of the pavement and for the designer, as discussed below.

The surface modulus (not to be confused with the modulus of a surface layer) is calculated directly from the surface deflections using Boussinesq's equations:

$$E_o(0) = 2\,(1-\mu^2)\,\sigma_o\ \ a/D(0), \text{ and} \tag{6}$$

$$E_o(r) = (1-\mu^2)\,\sigma_o\ \ a^2/(r\,D(r)) \tag{7}$$

where:

$E_o(r)$	=	surface modulus at a distance r from the centre of the loading plate,
μ	=	Poisson's ratio (usually set equal to 0.35)
σ_o	=	contact stress under the loading plate
a	=	radius of the loading plate, and
D(r)	=	deflection at the distance r.

The subgrade modulus plot (E_o versus r) provides at the time of test:

(i) an estimate for subgrade modulus (or CBR, Equation 12);
(ii) immediate determination of whether the subgrade modulus is linear elastic or non-linear, giving an indication of likely soil type;

(iii) confirmation of the adequacy of the geophone settings (as shown in Figures 3.5, 3.6, 3.7);

(iv) an approximate but direct appraisal of whether overlay is likely to be required.

These points are illustrated in Figures 3.5 to 3.9.

Figure 3.5 shows an example of a surface modulus plot from a pavement with linear elastic subgrade, as evidenced by the outer three geophones showing essentially the same surface modulus. At relatively large distances (generally more than 600 mm) from the loading plate, all compressive strain will occur in the subgrade rather than in the pavement layers which lie outside the stress bulb. For this reason the outer deflections will not be influenced by the pavement structure, i.e. the surface modulus will tend towards the modulus of the subgrade alone. In the example given in Figure 3.5, the subgrade modulus is about 300 MPa. Linear elastic materials tend to be sands and gravels, hence the subgrade at this test site is likely to be a compact sand or gravel.

Figure 3.5 Surface modulus plot with linear elastic subgrade modulus.

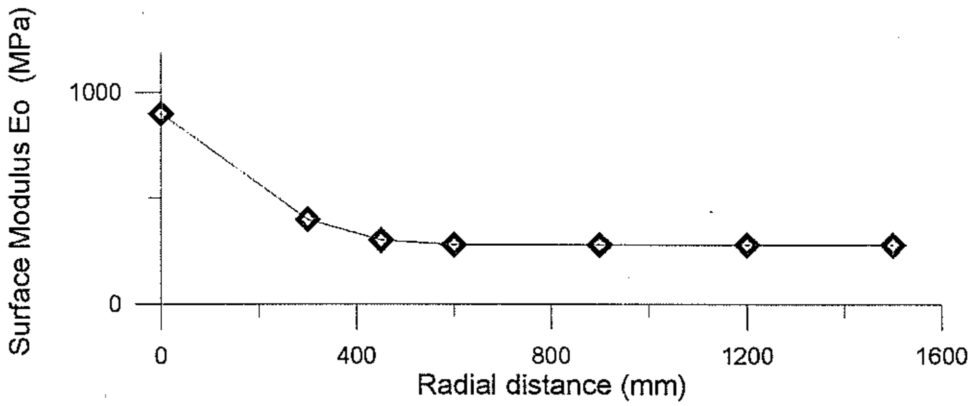


Figure 3.6 shows an example of a surface modulus plot from a pavement with moderately non-linear elastic subgrade. The three outer geophones show an apparently increasing modulus at increasing distance (i.e. decreasing stress). The subgrade modulus is approximately 80 MPa which, with the non-linear response, suggests it is a firm silt or clay. Results which show moderate or high subgrade moduli, together with highly non-linear response, may represent poor drainage at the top of the subgrade. Very low CBR together with strongly non-linear response are indicative of soft clays or peat.

Figure 3.6 Surface modulus plot with non-linear subgrade modulus.

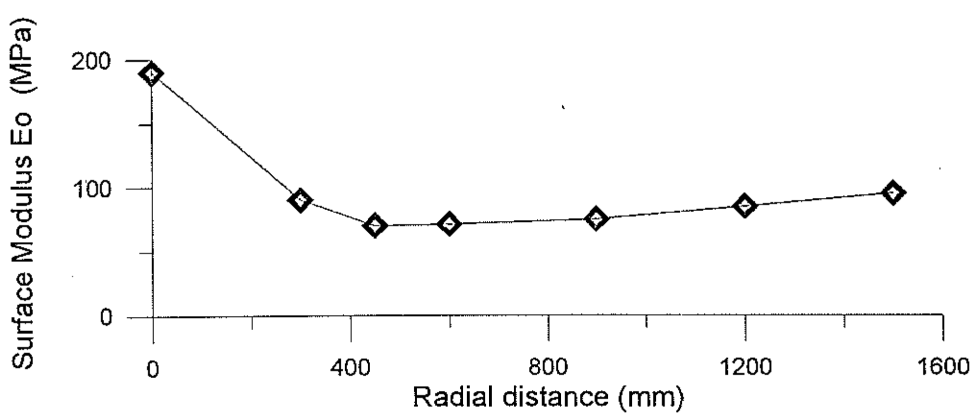
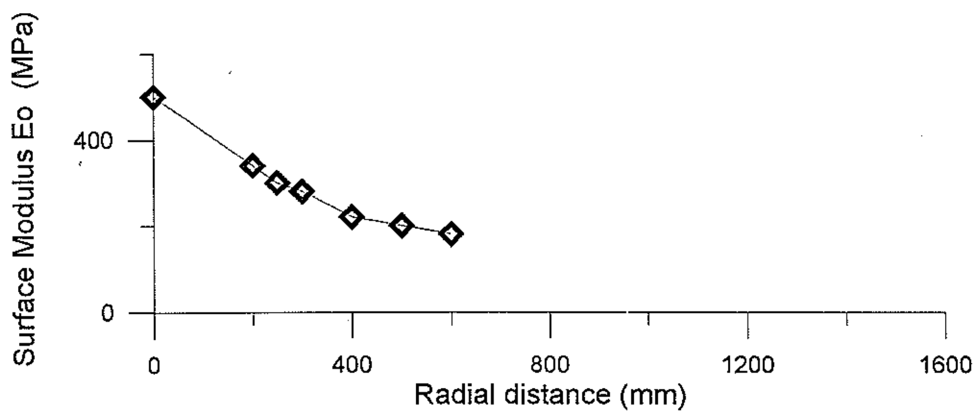


Figure 3.7 shows that the outer geophones are recording from progressively softer materials at depth, i.e. there may be softer soils beyond the range of the geophone assembly. The geophone spacing is evidently too close to the loading plate on a thick, stiff pavement. In this case the geophone spacings would normally be increased so that at least the three outer geophones define a linear segment on the surface modulus plot.

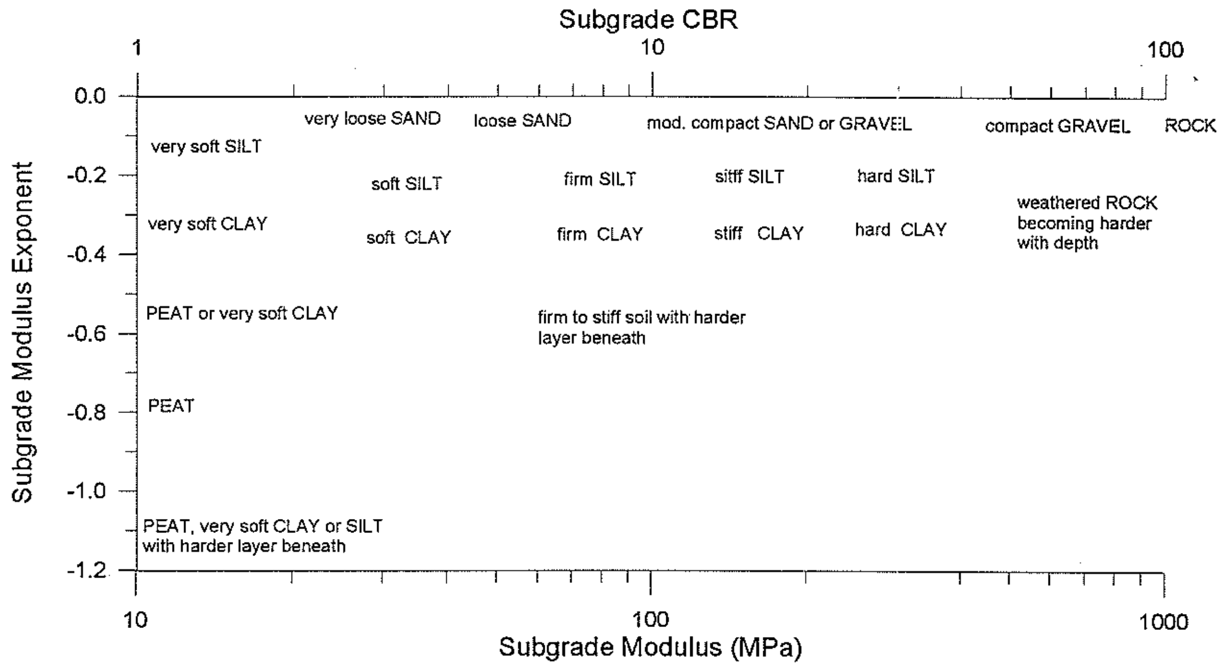
Figure 3.7 Subgrade modulus plot where geophones are too close.



By using both the subgrade modulus non-linearity exponent (Equation 8) and the subgrade modulus, an approximate soil type identification may be made, as indicated in Figure 3.8.

If sub-layering of the subgrade has been adopted (e.g. as determined by CIRCLY) a qualitative appreciation of the degree of non-linearity may be gained from inspection of the variation between successive sub-layer moduli. The regions in Figure 3.8 are not closely defined because thin layers (not obviously influencing the deflection bowl) or lateral variations in soil type will affect the exponent to various degrees. With thick pavements, or pavements with very stiff (bound) layers, the subgrade moduli tend to have reduced non-linearity and the soil type is subsequently more difficult to differentiate.

Figure 3.8 Identification of subgrade soil type from deflection bowl parameters.

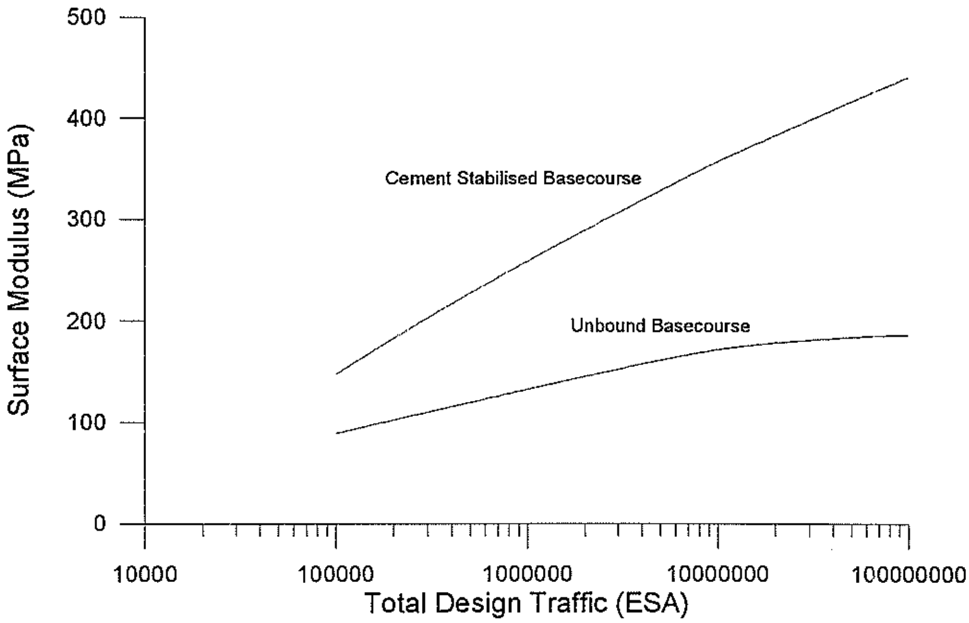


The chart in Figure 3.8 uses the approximate Shell relationship (SHELL 1978) showing the isotropic subgrade modulus as equal to 10 times the CBR. The AUSTROADS relationship is slightly different as discussed in Section 4.3.4 of this report.

Because the surface modulus is computed directly (no iterations or layer information are required) with the field software, this parameter can be readily inspected in the field as testing progresses. Also a preliminary appraisal of structural adequacy of the pavement can be made from Figure 3.9. Provided the design ESA for the road is known, the curves show the approximate intervention levels at which overlay will be required for either unbound granular or cement-stabilised pavements. The main advantage of knowing structural adequacy is in a network appraisal where test centres may be at generally 100 to 200 m centres. When a structural deficiency can be identified in the field from the surface modulus, then test spacing can be immediately adjusted to define the limits of the weak section.

The relationships in Figure 3.9 are intended for the FWD (where load varies) and are derived simply from Equation 6 and the simplified overlay design method (AUSTROADS 1992, Figure 10.3). For the Benkelman Beam where load is constant, the AUSTROADS Figure 10.3 may be used directly.

Figure 3.9 Preliminary assessment of structural adequacy of a pavement.



For the designer, apart from identifying soil type and possible subsurface drainage problems, a further function of the subgrade modulus plot is quality control during processing. The surface modulus plot is normally inspected so that irregular deflection bowl shapes can be rationally assessed and discounted if they are inappropriate. It is usually straightforward to identify bowls which, for instance, have been located over a culvert or approach slab, or have one geophone suspended over a pothole.

4. ANALYSIS OF PAVEMENT DEFLECTIONS

4.1 General

The shape of the deflection bowl allows detailed structural analysis of the pavement. Basically, the outer deflections define the stiffness of the subgrade while the bowl shape close to the loading plate allows analysis of the stiffness of the near-surface layers. A broad bowl with little curvature indicates that the upper layers of the pavement are stiff in relation to the subgrade. 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. With the critical layer identified in this manner, existing or potential distress mechanisms can be identified and therefore the most fitting treatment may be designed. Examples are given in Section 6.4 and in Appendix 1 of this report.

4.2 Software

4.2.1 General

A large selection of software is now available for determining the stresses, strains and deflections within a layered elastic system. A back-analysis procedure is therefore generally adopted to find moduli from an observed deflection bowl. The basic procedure comprises iterations, making adjustments to layer moduli until the computed deflections match the measured deflections. When the multi-layered elastic model is established (Figure 2.1), a forward-analysis is carried out to determine strains for a modelled rehabilitation treatment such as overlay. Some packages, e.g. EFROMD2 and CIRCLY are supplied as separate programs while others such as ELMOD combine both back- and forward-analyses into a single program.

Ullidtz & Coetzee (1995) summarise the properties of a range of layer moduli back-calculation programs. Most of the forward-analysis programs (including CIRCLY, BISAR and MODULUS) are based on multi-layer elastic theory with numerical integration or finite element analysis (FLEA), while a few programs (e.g. ELMOD) are based on the Odemark-Boussinesq transformed section approach. There are however many users of the latter form of software because of its rapid processing time.

Comparisons of the results obtained for the same deflection data analysed with different programs are given by Lytton (1988) and Ullitdz (1987). The adopted seed moduli can affect outcomes but most differences will arise from operator choice of consistent layer thicknesses. Misjudgement in the latter during back-analysis will tend to cancel out when determining overlay thicknesses in the forward-analysis. However, appropriate model layering is important when evaluating likely distress mechanisms. Features and advantages of some software packages are discussed in Sections 4.2.2 to 4.2.5 of this report.

4.2.2 EFROMD2 & CIRCLY

EFROMD2 (Elastic properties FROM Deflections) was developed by the ARRB (1994). It uses CIRCLY (Wardle 1980) iteratively to provide elastic layer moduli corresponding to a given deflection bowl.

Field data from either the FWD or Instrumented Benkelman Beam may be used, and the program will apply one or two loading circles accordingly. The program also corrects for secondary effects if the Beam support points are affected by the deflection bowl.

When an appropriate model of the existing pavement has been established, then CIRCLY is used again in the forward-analysis to evaluate rehabilitation options. For materials where the modulus is strongly dependent on stress level, sublayering is recommended to improve the accuracy of modelling.

Seed moduli are required for EFROMD2, and maximum/minimum credible moduli can be specified. CIRCLY uses numerical integration and is one of the few programs which will accommodate materials with anisotropic moduli. It is very versatile and can include complex loading patterns.

EFROMD2 and CIRCLY are both recommended by AUSTROADS for mechanistic analysis of pavements.

4.2.3 MODULUS

MODULUS, provided by the Texas Transportation Institute (TTI), fits the deflection bowl to a library of bowl shapes with corresponding layer stiffnesses. This fitting procedure greatly increases the speed over iterative numerical integration methods. MODULUS was recently selected as the back-analysis program of choice by the Strategic Highway Research Program (SHRP), and it can therefore be expected to gain increasing support in the United States. However it allows only isotropic moduli to be considered.

4.2.4 ELMOD

ELMOD (Evaluation of Layer Moduli and Overlay Design) is supplied by Dynatest (1989). It carries out back- and forward-analyses within the one program, originally using the Odemark-Boussinesq transformed section approach (Ullidtz 1987). The program has recently been upgraded to include the capacity for deflection basin fit, and it can also provide results based on numerical integration methods. A facility is incorporated to find the appropriate adjustment factors so that Odemark-Boussinesq solutions will fit more closely with numerical integration methods if required. The upgrade also allows modulus limits to be applied.

Unlike most other software, it has the capacity to analyse non-linear subgrade moduli as stress dependent (rather than depth dependent from sublayering), and has been widely used in Europe, Asia and North America. ELMOD will analyse only isotropic materials.

4.2.5 Limitations and Advantages of Software Features

Anisotropy

Historically, most empirical strain criteria (e.g. SHELL 1978) have been associated with back-analysis of isotropic materials, principally those involved in the AASHO Road Test. It is therefore necessary to ensure that forward-analysis relates to the same assumptions. The AUSTROADS strain criterion is based on back-analysis of CBR-pavement thickness design curves, assuming anisotropic moduli, and therefore the same anisotropy should be used for overlay design. This assumption limits the available software for AUSTROADS mechanistic design to CIRCLY, unless appropriate translations are adopted. Further discussion is given in Section 4.3.4 of this report.

Seed Moduli, and Moduli Limits

Most programs require seed moduli to begin the back-analysis iterations. This provides another area where the modelling results will be operator-dependent. Maximum and minimum credible moduli can also be input. Where moduli are unconstrained, unrealistic solutions will draw attention to the problem and layer thickness will need to be adjusted further.

Speed of Execution

ELMOD processes a specified series of points, all having the same layer thicknesses, very rapidly as a batch.

EFROMD and CIRCLY require that test points be analysed individually by the operator, making it more suitable for detailed design. Usually representative points giving a range of low and high strength pavement materials and subgrades are selected for analysis.

Non-linear Moduli

Only a few of the available packages provide for analysis of non-linear moduli. Ullidtz (1987) considers this feature to be of particular importance:

> *Many subgrade materials are highly non-linear, and if this is neglected very large errors may result in evaluation of the moduli of the pavement materials.....It should be noted that in a non-linear material the modulus increases with distance from the load, both in the vertical and in the horizontal direction. If one of the linear elastic programs is used to calculate the pavement response then the vertical increase in modulus may be approximated by subdividing the layer into a number of layers with increasing modulus, or by introducing a stiff layer at some depth. But this will not imitate the horizontal increase in modulus, and the deflection profiles derived will be quite different from those found on a non-linear material.*

Dynamic Analysis

More realistic analysis methods that address dynamic loading have been developed for research but are little used in practice (Ullidtz & Coetzee 1995). Additional parameters would need to be defined and measured, e.g. visco-elastic properties and densities. Part of the problem is that the mechanistic procedure is an analytical-empirical one. The induced strains are determined analytically but an empirical relationship is still used to determine allowable strains. If true dynamic strains are calculated this would simply shift the problem to that of determining a new allowable dynamic strain criterion.

Comparison of Theoretical Models with Real Strains

All the mechanistic design methods in general use assume that the loading is static, the materials are in uniform, continuous and homogenous layers, and they have simple stress-strain relationships. Also, the calculated strains apply to a continuum. However, pavements are comprised of a series of discrete particles which will experience much lower strains within individual particles and much higher strains at particle contact points. In other words "correct" analysis methods can provide only an average of the combination of strains which occurs in practice.

To put the difference between currently used mechanistic analysis programs in perspective, and to consider the implication of material variability inherent in pavement engineering, only a 1-m shift along the road for any given FWD test point, for example, is likely to produce greater variation in moduli results than the variation that is related to choice of any of the recognised software packages.

4.3 Calculating Layer Moduli

4.3.1 Basic Calculations

When back-calculating layer stiffnesses, the deflection bowl is initially analysed in conjunction with assumed or measured layer thicknesses to determine moduli, stresses and strains in each layer.

Because most of the measured deflection is dominated by the nature of the subgrade, it is important that its stiffness is accurately modelled, otherwise back-analysis to provide the upper layer moduli can produce disproportionately large errors. Accordingly Brown et al. (1986) and Ullitdz (1987) suggest that the subgrade should be characterised by a non-linear elastic model, taking into account the stress dependency of that layer.

Some packages provide for approximate analysis of non-linear subgrades by generating additional sub-layers with gradational elastic properties. ARRB (1994) suggest that in this case (e.g. when using EFROMD2) the subgrade should be modelled as four sub-layers with thicknesses, from top to bottom, of 250, 350, 500 mm, and then infinite thickness.

The ELMOD package requires only one subgrade layer because it uses the deflections to calculate C and n in the non-linear subgrade modulus relationship:

$$E = C(\sigma_z/\sigma')^n \qquad (8)$$

where:
C and n are constants,
σ_z is the vertical stress and
σ' is a reference stress.

The reference stress is introduced to make the equation correct with respect to dimensions. E (modulus of elasticity) and C then both take dimensions of stress. This approach allows quick and accurate modelling with the additional benefit discussed earlier, in that the subgrade soil type may be broadly identified. The exponent "n" is a measure of the non-linearity of the subgrade modulus. If "n" is zero the material is linear elastic (e.g. hard granular materials). Soft cohesive soils may be markedly non-linear with n being between –0.3 and –1. The moduli of an upper stiff layer and of an intermediate layer, if present, are then determined through an iterative process using the total central deflection and the shape of the deflection bowl under the loading plate. The subgrade modulus at the centre line is adjusted according to the stress level. The outer deflections are then checked and a new iteration carried out if necessary.

To provide the most realistic model, a preliminary analysis is normally carried out using the available data. A check is then made for consistency with visual examination and expected performance in the region. After incorporation of all findings and inclusion of any further field work, re-analysis is carried out for detailed design. Calculations for specific conditions, e.g. layer thickness, rigid bases, anisotropy, and subgrade CBR, are described in the following Sections 4.3.2 - 4.3.5.

4.3.2 Layer Thickness Sensitivity

The Odemark-Boussinesq method primarily considers the *stiffnesses* of the various layers rather than *moduli* directly, i.e. for isotropic layer moduli, the overall layer stiffness defined by Equation 9 is determined:

$$h^3 E / (1-\mu^2) \qquad (9)$$

Therefore when back-analysing to find the layer modulus (E) from an assumed layer thickness (h), a small error in layer thickness will translate to a large error in modulus. The same sensitivity occurs in the other analysis methods (e.g. CIRCLY) which use numerical integration. The expression is relatively invariant to the ranges of Poisson's ratio (μ) found in practice. It is, however, important to consider the general order of magnitude of layer moduli as results will not be precise. This comment does not apply to subgrade moduli because these values are determined explicitly and results will generally be reliable. Also in the later stage (when determining overlay requirements) the stiffness rather than the layer modulus is used, and hence the design overlay thickness is affected minimally by reasonable assumptions regarding layer thicknesses.

4.3.3 Rigid Base Condition

An apparently non-linear subgrade modulus (or linear elastic sub-layers becoming stiffer with depth) could be incorrectly inferred from the surface modulus plot as a result of a very stiff layer occurring at depth. For this reason noting any outcrops and the regional geology is important. If rock is present within about 3 m of the pavement surface an "infinitely stiff" boundary must be used in the model. If this is not done, overlay results can be unconservative. Some software packages (e.g. ELMOD and MODULUS) provide options for computing the depth to a rigid base automatically from the response of the outer geophones.

4.3.4 Anisotropy

Anisotropic pavement materials (with a vertical to horizontal modular ratio, E_v / E_h, of 2) are suggested for design by AUSTROADS (1992, Table 6.4). However, few analysis methods other than CIRCLY allow for anisotropy. Also, there is substantial worldwide experience founded on analyses which have assumed isotropic conditions.

To allow valid comparison of results from those software programs which use isotropic moduli, and from CIRCLY when a degree of anisotropy of 2 is used, it is necessary to determine the applicable modulus constant (K_{i-a}) in the relationship:

$$E_{v,n=1} = K_{i-a} \cdot E_{v,n=2} \tag{10}$$

where $E_{v,n}$ is the vertical modulus with modular ratio of "n".

Logically it would be expected that the equivalent isotropic modulus ($E_{v,n=1}$) for a material with modular ratio $n = E_v / E_h = 2$ must be somewhere between the extremes:

$$\text{i.e.} \quad 0.5 < K_{i-a} < 1 \tag{11}$$

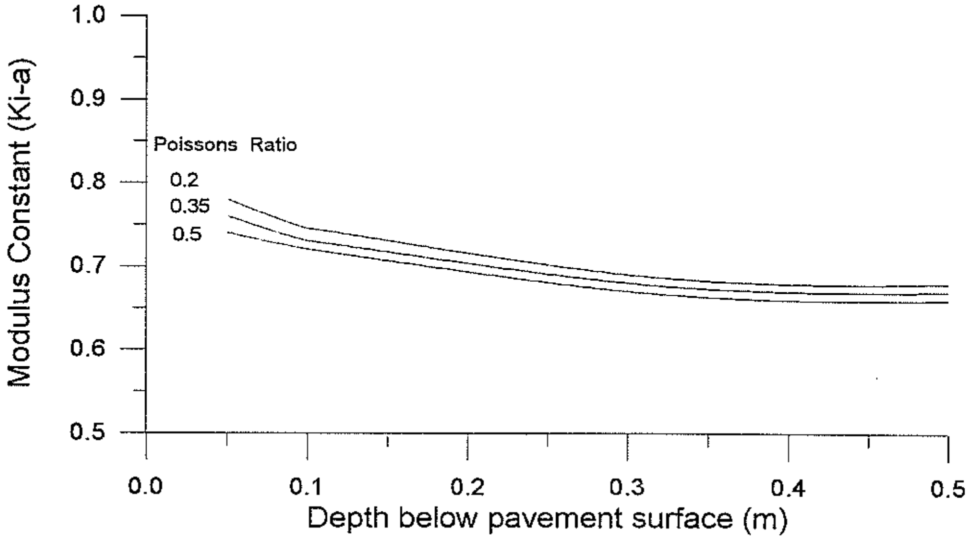
The analytical solutions for anisotropy are given by Ullitdz (1978, Table 3.2). The comparison between pavement structures which are anisotropic and their isotropic equivalents cannot be determined directly. However the equations given by Ullitdz can be solved iteratively to provide the theoretical relationships. The constant K_{i-a} is found to be independent of stress but is very slightly dependent on the depth below the surface, Poisson's ratio, and the loaded area. The relevant data for highway situations are shown in Figure 4.1.

For subgrade material (at depth of say 0.3 to 0.5 m or more, and Poisson's ratio of 0.45) a value of 0.67 for K_{i-a} provides a practical equivalent, i.e. a subgrade with anisotropic modulus ($E_{v,n=2}$ = 100 MPa) could be modelled as a material with 67 MPa isotropic modulus.

For basecourse material (say 100 to 150 mm thick with Poisson's ratio of 0.35), K_{i-a} will be about 0.75, i.e. a typical M/4 modulus of about $E_{v,n=2}$ = 500 MPa is equivalent to a material with isotropic modulus of 375 MPa.

The issue does not arise with cemented materials or asphalt for which AUSTROADS indicates that isotropic moduli should be used.

Figure 4.1 Modulus constant as a function of Poisson's ratio, and depth.



Little information is presented in the AUSTROADS Guide on sensitivity of analyses to anisotropy. Anisotropy remains as one factor in the stiffness expression which is determined by the back-analyses and cannot be deduced explicitly. In the anisotropic model three other variables (Poisson's ratio and layer thickness as well as modular ratio) are still necessary to assume, in order to determine in-situ vertical modulus. Adding the capability for variable anisotropy has been considered for a future ELMOD upgrade, but is not receiving high priority. Ullitdz (pers.comm.) comments:

> *Including anisotropy would introduce one more unknown parameter, and a parameter that is very difficult to measure, but it would be uncertain whether this would bring you closer to or further away from the actual stresses and strains in the pavement.*

The anisotropy used by AUSTROADS has significant implications with regard to allowable subgrade strains, as discussed in Section 4.3.4 of this report.

4.3.5 Estimating Subgrade CBR

The CBR test imposes high strain, plastic deformation, in marked contrast to the loading applied to the subgrade to determine resilient modulus which imposes low strain, elastic conditions. Hence there is little reason to expect good correlations between CBR and resilient modulus, and any values inferred from a mean value relationship could be in error by a factor of 2 or more (AUSTROADS 1992).

Furthermore, most cohesive soils have highly stress-dependent moduli, i.e. their stress-strain curves are non-linear. Typical responses of various subgrade soil types are shown on Figure 4.2 which encompasses the range of stresses and strains imposed on subgrades under a pavement subjected to 1 ESA. The modulus (i.e. slope of the stress-strain curve) for a given cohesive soil, evidently varies by a factor of 2 or 3 depending in the effective load spread (i.e. depth to the subgrade and stiffnesses of pavement layers). Modulus-CBR correlations must therefore be taken as indicative only. In addition if the CBR is estimated with the Scala penetrometer then the variation in predicted modulus is compounded by a further factor of about 2 (Scala 1956).

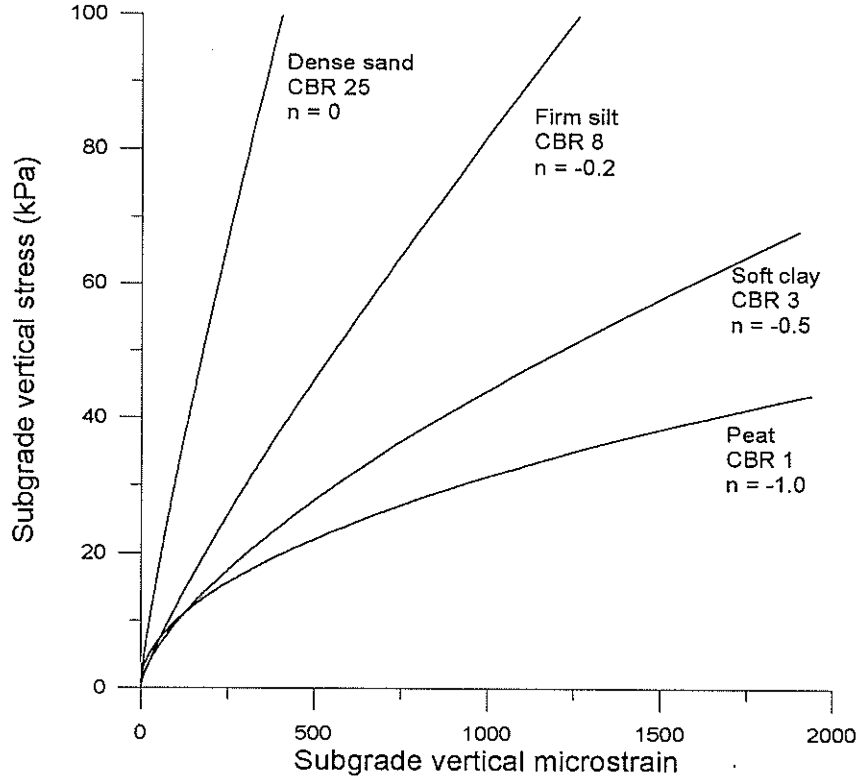


Figure 4.2 Typical subgrade moduli and stress-dependency found from FWD back-analyses.

The implication for design is that it is important to focus on modulus (and its degree of non-linearity) for evaluating pavement distress mechanisms and rehabilitation design options, leaving the CBR parameter to its more appropriate role in the design of new pavements (AUSTROADS 1992, Figure 8.4, where it is still preferable to use soaked CBR test values).

Note that the E-CBR relationship for the subgrade, used by AUSTROADS, is given by Equation 12.

$$E_v = 10\text{ CBR},\ E_h = 5\text{ CBR} \qquad (12)$$

(because modular anisotropy of 2 is adopted).

Therefore from the discussion on anisotropy (Section 4.3.4), the equivalent isotropic modulus of the subgrade implied by AUSTROADS is:

$$E_{\text{isotropic}} = 6.7\text{ CBR} \qquad (13)$$

This assumes that the subgrade is at a depth of about 300 mm and has a Poisson's ratio of 0.45 although, as seen from Figure 4.1, the relationship is relatively insensitive.

Equation 13 is clearly more conservative than relationships adopted by other organisations for estimating the subgrade modulus from CBR. The compensating consequence of this difference is that the AUSTROADS subgrade strain criteria (derived by back-analysis of subgrade CBR design curves: Jameson 1996) is somewhat less conservative than strain criteria recognised by other organisations.

CBR relationships for sub-base and basecourse materials are discussed in Section 4.3.7 of this report.

4.3.6 Validity of Back-Calculated Elastic Pavement Material Properties

A number of sensitivity analyses are required to gain an appreciation of any pavement modelled as multiple layers of linear elastic materials. Layer thicknesses are normally varied over the likely range or found from test pits, and the resulting moduli and required overlays are compared.

To obtain maximum reliability, the multiple layer pavement structure should meet the following conditions (Ullitdz 1987, Dynatest 1989):

(i) The structure should contain only one stiff layer ($E_1 / E_{subgrade} > 5$). If the structure contains more than one stiff layer, they should be combined for the purpose of structural evaluation.

(ii) Moduli should be decreasing with depth ($E_i / E_{i+1} > 2$).

(iii) The thickness of the uppermost layer should be larger than half the radius of the loading plate (i.e. usually larger than 75 mm). For three layer structures, the thickness of the uppermost layer should be less than the diameter of the loading plate (i.e. less than 300 mm usually) and the thickness of layer 1 should be less than that of layer 2.

(iv) When testing near a joint or a large crack or on gravel road, the structure should be treated as a two-layer system.

If the structure does not comply with these limitations, the analysis can still be used but the precision will not be as high.

Other checks on model validity may be made by comparing moduli with values typically found in materials of a similar nature. Standard recommendations are given in AUSTROADS (1992, Table 6.4). New Zealand experience, so far, indicates that this table may be quite conservative for pavements constructed in accordance with current Transit New Zealand specifications.

4.3.7 Unbound Granular Materials

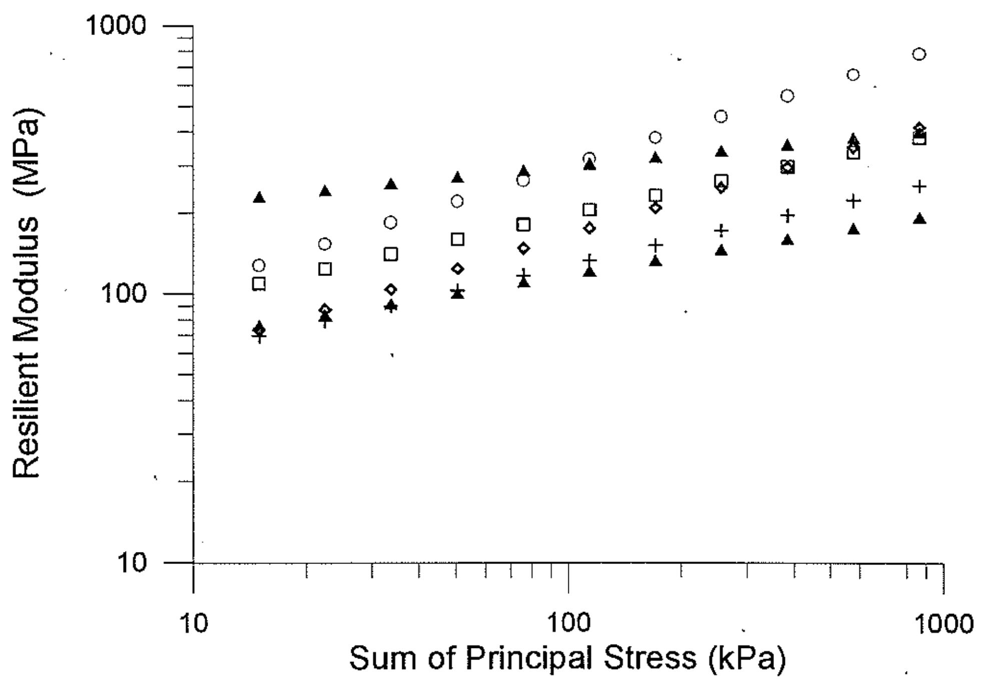
A complication in pavements with unbound granular surfacing is the non-linearity of the basecourse modulus. Brown & Pell (1967) suggested the use of the now widely adopted relationship:

$$E = K1\,\theta^{K2} \qquad (14)$$

where: θ is the sum of the principal stresses at maximum deviatoric stress, and
K1, K2 are material parameters.

To express the relationship of modulus of unbound granular materials to their degree of compaction and stress state, typical values for K1 and K2 are given by Sweere (1990). Some of these typical values (that closely comply with TNZ M/4:1995 grading and crushing resistance) are given in Figure 4.3.

Figure 4.3 Resilient moduli (MPa) v. mean stress (kPa) for sound basecourse (after Sweere 1990). (Symbols indicate different basecourses)



These values show that a non-linear elastic model would be preferable for basecourse materials. However, for the widely used linear elastic models, Sweere recommends as a first approximation that thick granular basecourses be divided into sub-layers to minimise the effects of stress dependency of the back-calculated moduli. At some future time, a rigorous finite element method that fully characterises this range of values is likely to be adopted by practitioners, but no such procedure is in general use as at 1998. Meanwhile the approximations will need to be kept in mind while using the widely recognised packages currently available, as the latter still do provide practical working models for analysis and design.

Considering the principal stresses under an ESA loading, at the top and bottom of a 125 mm layer of unbound basecourse, Sweere's data suggest a range of moduli that are mainly between about 200 and 300 MPa. These values are isotropic and relate to freshly compacted laboratory samples. However substantially higher values are typically obtained on good quality basecourses that have experienced either repetitive loading in the laboratory (Jameson 1991) or sustained trafficking in the field.

It is important to appreciate that the modulus of any unbound layer is not simply a function of the component material, but is also dependent to a large degree on the stiffness of the underlying material. In a multi-layer system, Heukelom & Foster (1960), using linear-elastic analyses, found that the ratio of the E modulus of an unbound base layer E_i to that of the underlying soil E_{i+1} was limited to $E_i / E_{i+1} < 2.5$. Their rationale was that an unbound material cannot be properly compacted on a soft subgrade. Alternatively, if a stiff dense layer is placed on a yielding foundation, then tensile strains will develop and the upper layer will de-compact. Heukelom & Foster supported this practical explanation theoretically, showing that tensile horizontal stresses would develop at the bottom of layer "i" if the E_i / E_{i+1} ratio exceeded 2.4. Under repeated loading these stresses would lead to de-compaction of the overlying unbound layer until its stiffness reduced to a limiting value at which tensile stresses would not occur.

Subsequently the Shell Pavement Design Manual (1978) used the concept of modular ratio limitations in successive unbound layers in the relationship:

$$E_i / E_{i+1} = 0.2\, h_i^{\,0.45} \quad \text{and} \quad 2 < E_i / E_{i+1} < 4 \qquad (15)$$

where h_i is the thickness (in mm) of the overlying layer.

Subsequently, Brown & Pappin (1985) found, using more rigorous non-linear finite element analyses, that the above limitations were too restrictive and they reported:

$$1.5 < E_i / E_{i+1} < 7.5 \qquad (16)$$

AUSTROADS (1992) Design Manual requires granular materials that are placed directly on the subgrade to be sub-layered using, as constraints, sub-layer thicknesses that must be approximately in the range of 50-150 mm and a ratio of moduli of adjacent sublayers that does not exceed 2.

Moffatt & Jameson (1998) provide the following recommendations for granular materials placed directly on the subgrade (with no stiff cemented sub-base):

(a) divide the granular materials into five (5) layers of equal thickness,

(b) determine the vertical modulus of the top sublayer as the minimum of the value indicated in AUSTROADS (1992) Table 6.6, and that determined using

$$E_{V\ \text{top of base}} = E_{V\ \text{subgrade}} \times 2^{(\text{total granular material thickness}/125)}$$

(c) calculate the ratio, for moduli of adjacent sub-layers from

$$R = (E_{V\,\text{top of base}} / E_{V\,\text{subgrade}})^{1/5}$$

(d) calculate the moduli of each sub-layer successively from the known modulus of the underlying layer, beginning with the subgrade.

The above relationships are intended for forward design. However back-analysed moduli should be checked to ensure that a reasonable pavement model has been obtained when carrying out sensitivity analyses of different layer thicknesses. Clearly, only unbound layer moduli are restricted in this manner as the moduli of bound materials are influenced much less by the stiffnesses of underlying layers.

Because the modulus of any layer is strongly influenced by the underlying layer, correlating the modulus of a granular layer with CBR is not reliable. For an approximation (based on observations of moduli determined on basecourses that have a known CBR of at least 80), it is suggested to use the following relationship to estimate the CBR of an unbound granular basecourse material:

$$E_v \text{ (MPa)} = 5 \text{ CBR} \qquad \text{for } E_v/E_h = 2 \qquad (17)$$

where E_v = vertical modulus;

E_h = horizontal modulus

The equivalent relationship for an isotropic basecourse (from Section 4.3.5 of this report) is approximately:

$$E_{isotropic} \text{ (MPa)} = 4 \text{ CBR} \quad \text{for } E_v/E_h = 1 \qquad (18)$$

Sweere (1990) presents data which are consistent with the above relationships (to within a factor of 2) provided that the applied stresses (sum of principal stresses) are about 750 kPa. However, the constant of proportionality in the above equations decreases by a factor of 4 as the applied stresses reduce to 50 kPa. For sands (e.g. sub-base materials) the constant of proportionality was found to be about 3 to 4 times higher than for gravels. Therefore by fortuitous cancellation, the above equations should apply (very approximately) either for basecourse close to the wheel load or for sandy sub-base at depth.

Moduli for granular materials are clearly very sensitive to test conditions requiring close replication of in-service density, grading, applied stresses, and underlying support for meaningful measurement of modulus or correlation with CBR.

4.3.8 Seasonal Effects

The back-analysis of a deflection bowl provides results for the specific moisture condition at the time of testing. Seasonal variations in moduli must therefore be considered before calculating residual life and overlay requirements. Software packages vary in the way seasonal effects are incorporated. One option is to increase deflections by a multiplier in the range of 1.1 to 1.6 if measurements are not carried out during a wet period. Another approach is to assume an annual sinusoidal variation in moduli between a maximum and minimum value (usually the subgrade modulus alone would be varied but the factor could be applied to all unbound layers, with similar end results).

In a long-term study of deflection changes with season in Australia, Rallings & Chowdhury (1995) found a generally sinusoidal variation in peak deflection each year, and concluded that a seasonal adjustment factor of 1.1 would be appropriate for deflection measurements made between mid-summer and the end of autumn. The data they obtained include both "wet" and "dry" rainfall areas and there is clearly more seasonal fluctuation of deflection in the case of the dry areas. If the design condition for the subgrade is taken towards the wetter state rather than at the median condition, then an adjustment factor of about 1.3 would be indicated by the data.

Another similar study undertaken at Delft University (Van de Pol et al. 1991) produced comparable sinusoidal seasonal fluctuations in subgrade moduli, from FWD measurements taken over a 2 year period. However, no specific guidelines for assessing seasonal effects generally were indicated.

A considerable degree of judgment will be required to assess seasonal adjustment factors for specific sites. Factors listed in Table 4.1 are suggested as provisional guides for temperate climates, such as New Zealand. This table draws on the above references and is supported by studies in progress. The subgrade moisture condition at the time of testing should be assessed relative to expected seasonal ranges in that locality.

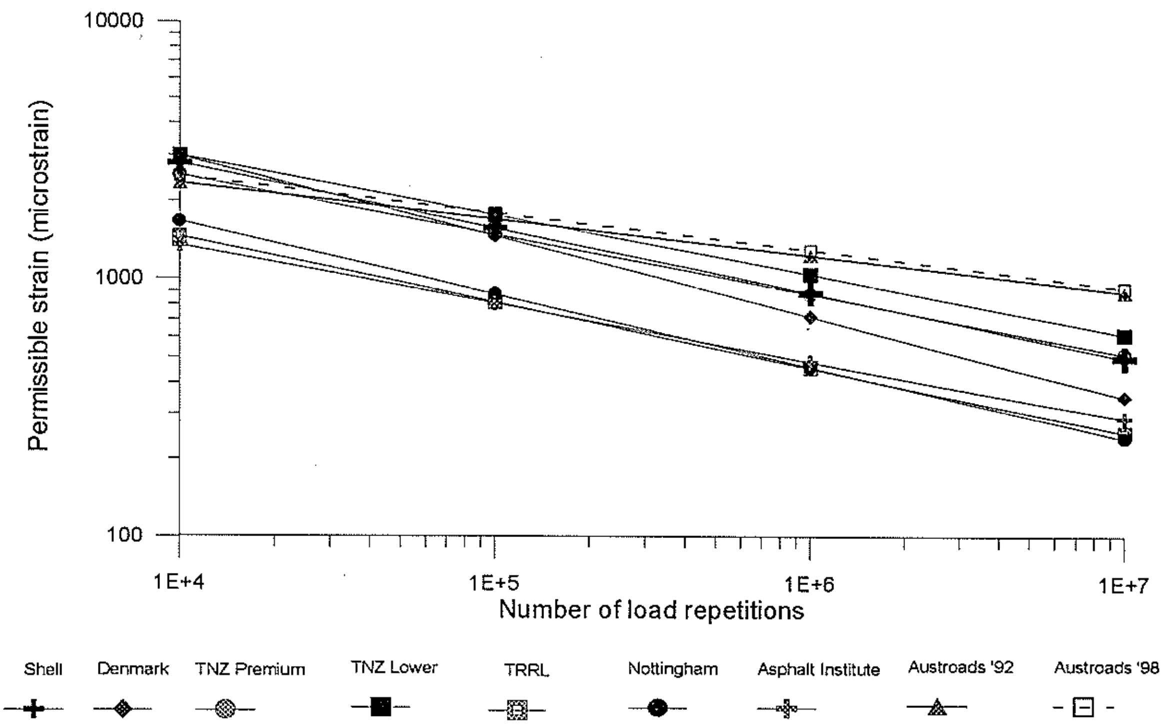
Table 4.1 Seasonal adjustment factors to apply to deflection testing, at two extremes of rainfall, and at four moisture conditions of subsoil (after Rallings & Chowdhury 1995, Van de Pol et al. 1991).

Mean Annual Rainfall (mm)	**Subgrade Moisture Condition (at time of testing)**			
	Very Wet	**Wet**	**Dry**	**Very Dry**
	Seasonal Adjustment Factor			
500	0.95	1	1.15	1.3
1000	0.95	1	1.10	1.2

5. RESIDUAL LIFE

Residual life, i.e. the number of ESAs that can be accommodated by a pavement before it is no longer serviceable, can be estimated by comparing the existing roughness with a terminal roughness condition, and using established relationships for allowable material strain versus number of load repetitions. Figure 5.1 shows a number of strain criteria for unbound materials from a range of different sources. Most methods are based on the AASHO Road Test and the criteria are applied to the subgrade only, although the Denmark relationship is used on all unbound layers and is only an implicit strain criterion (based on stress and modulus according to Ullidtz 1987). The AUSTROADS relationship is not based on the AASHO Road Test and passes well above all others as the number of repetitions increases, i.e. it is significantly less conservative for high traffic loadings.

Figure 5.1 Comparison of alternative subgrade strain criteria (from Ullidtz 1987).



The procedure for determination of residual life from empirical data relating to the AASHO Road Test is clearly simplistic as it is based only on roughness progression. Therefore prediction will be less reliable when other factors govern the pavement life.

Alternative residual life predictions, based on the AASHTO structural number approach, are given by Paterson (1991). Where only roughness is available, the remaining life may be determined from:

$$Ri_t = 1.04\, e^{mt}\, \{RI_0 + 263\,(1+SNC)^{-5}\ NE_t\} \tag{19}$$

where:

Ri_t = roughness at pavement age t (m/km international roughness index IRI)
RI_0 = initial roughness
SNC = structural number modified for subgrade strength
NE_t = cumulative ESA at age t (million ESA/lane)
t = pavement age since rehabilitation or construction (years)
m = environmental coefficient (0.023 for wet non-freeze climate)

The appropriateness of these two predictive methods for unbound granular pavements is the subject of ongoing New Zealand research (Transit NZ Research Project PR3-0171). Preliminary indications are that the AASHO method (AASHO 1961) tends to give slightly optimistic but useful predictions for New Zealand unbound granular pavements, while the AASHTO structural number approach (Paterson 1991) may produce excessively optimistic residual life predictions. Both residual life determinations appear to be good relative predictors for comparison or ranking of pavements of similar construction within a given area (e.g. in network surveys), but absolute life predictions should be regarded with caution until calibrated to local conditions.

6. MECHANISTIC DESIGN OF REHABILITATION TREATMENTS

After completion of the deflection bowl analysis and determination of layer moduli, rehabilitation options are evaluated by means of a forward-analysis program such as CIRCLY. A suitable overlay thicknesses can be applied, and a check made to confirm that strains within all layers are acceptable.

6.1 Adjustment of Back-Calculated Moduli for In-Service Conditions

6.1.1 Unbound Granular Materials

Because the moduli of granular layers are stress-dependent, an adjustment to back-calculated moduli is required if the stresses imposed at the time of testing are significantly different from those that will be applied in service conditions.

ARRB (1994) recommend the following adjustments:

$$E_{i-s} = E_{i-m} \cdot (\text{in-service mean stress / measurement mean stress})^{K} \tag{20}$$

where: E_{i-s} = modulus (MPa) of granular layer "i" for the in-service condition,
E_{i-m} = the modulus (MPa) as measured
K = a constant selected from the range 0.3 (low quality sub-base material) to 0.5 (high quality basecourse material)

The stresses at the mid-depth of each layer need to be obtained from the analysis and an appropriate correction applied. If the loading used for measurement is equal to (or slightly less than) the in-service stress, then no correction is required (and minimal conservatism is the result in this case).

6.1.2 Subgrade Materials

Where non-linear elastic subgrade moduli have been approximated in a sub-layering process (e.g. EFROMD2 or CIRCLY), the moduli should be adjusted as follows for the forward-calculation (ARRB 1994):

$$E_{i-s} = E_{i-m} \cdot (300\text{ MPa} - \text{in-service deviatoric stress}) / (300\text{ MPa} - \text{measurement deviatoric stress})^{P} \tag{21}$$

where: E_{i-s} = modulus in MPa of subgrade sub-layer "i" for the in-service condition,
E_{i-m} = modulus in MPa as measured,
P = a function of subgrade CBR (Table 6.1).

Table 6.1 Subgrade stress-dependency exponent relating subgrade CBR to function P (ARRB 1994).

Subgrade CBR	P
2	8
3	6
4	5
5	4
7	2
10	0.5
15	0

For a program (e.g. ELMOD) where the subgrade moduli are back-calculated as stress-dependent non-linear materials, the forward-analysis uses the same modulus / stress relationship (Equation 8) with the calculated exponent for that test point. (ELMOD carries this out automatically as it combines both back- and forward-analyses in the one program.)

If a linear-elastic forward-analysis program such as CIRCLY is to be used with stress-dependent moduli (e.g. obtained from ELMOD), then the standard set of subgrade sub-layers (Section 4.3 of this report) should be used. The in-service stresses should then be calculated, and the equivalent linear-elastic modulus for each sub-layer be determined from Equation 8.

6.2 Moduli for Overlay Materials

6.2.1 Unbound Granular Basecourse

The resilient moduli of various overlay materials are given in Table 6.4 of the AUSTROADS Pavement Design Guide.

New Zealand research, described in the companion report to this project (Tonkin & Taylor, in prep.), has found that these values are realistic design values for thin pavements but may be somewhat conservative for stiff pavements. Unbound granular overlays produce moduli which are consistent with the values suggested by AUSTROADS when first constructed. However, where strains in the underlying layers are small, basecourse moduli may increase by 50% after in-situ densification has occurred from trafficking.

The New Zealand Supplement (Transit New Zealand 1997) requires that the modulus used for an unbound granular overlay shall be the same as the modulus determined for the top basecourse layer. This assumption is reasonable because, for stiff pavement structures, a higher modulus for the unbound granular material will be used. Also the overlay modulus should not be less than the underlying existing basecourse modulus.

6.2.2 Bound Overlays

Alternative rehabilitation treatments, such as asphaltic overlay or cement-stabilisation of the basecourse layer, are considered by modelling the pavement with appropriate parameters (AUSTROADS 1992, Table 6.4b; NZ Supplement 1997). The moduli of cement-stabilised basecourses used in New Zealand have been found to be highly variable. Examples are given in Appendix 1.

Further details on mechanistic design and modelling of rehabilitation treatments with worked examples are given by ARRB (1994), NZIHT (1996), Wardle (1980) and RTA (1994).

6.3 Presentation

Software packages produce a range of display outputs, but most include options that can be transported either directly or indirectly into spreadsheets for subsequent graphing to suit individual project requirements.

The advantage of spreadsheet files is that FWD information can be readily supplied on diskette and viewed graphically to facilitate appraisal by the designer. A display can show the inferred moduli and relevant parameters as well as a comparison of overlay requirements or depth of basecourse stabilisation using the mechanistic procedures described in the New Zealand Supplement (Transit New Zealand 1997). It is generally useful to compare the overlay design methods using both the AUSTROADS subgrade strain criterion and the two methods which use a past-precedent strain criterion.

The visual condition assessment and known performance of local materials must then be used as a check on the appropriateness of the preliminary analytical model. Any inconsistencies must be addressed, the layer thicknesses adjusted in accordance with the destructive test information, and a final model developed.

An example of a final report presentation of parameters is given in Figure 6.1, showing a number of parameters plotted against road chainage. Reading up from bottom of the page, the parameters are:

- The layer thicknesses used in the model and the actual dynamic deflections (corrected to standard temperature for an 8-tonne ESA loading).
- The subgrade strain ratio and subgrade modulus non-linearity exponent. The subgrade strain ratio is the strain at the top of the subgrade divided by the allowable strain (obtained from either AUSTROADS or New Zealand Supplement) for the proposed traffic loading (in ESA). (The original AUSTROADS strain criterion has been used in this case)

The subgrade modulus non-linearity allows identification of likely soil type in the subgrade and an indication of whether poor subsurface drainage could be a factor.

- The critical layer, i.e. the layer that governs the design life of the pavement according to the adopted strain criterion.
- The design traffic (in millions of ESAs) and results of the structural analysis, giving the moduli for each layer: basecourse (if unbound granular chipseal, or asphalt if structural), sub-base and subgrade.

  The resilient modulus scale is shown on the left, while the equivalent CBR is shown on the right margin. Colour coding is used to allow the various layers to be identified readily (see Key below Figure 6.1).
- The interpretation and design guides are at the top of Figure 6.1.

  Each point shows the remaining life (AASHO method in bar graph and AASHTO structural number method as a line graph) and calculated overlay (AUSTROADS or New Zealand Supplement method as required).

  Where cement stabilisation of the existing basecourse is being considered, the necessary depth of stabilisation is shown using the tensile strain criterion given by the New Zealand 1997 Supplement.

To analyse sensitivity to layer thicknesses, a separate back-analysis will be required. To evaluate other changes, a forward-analysis only is needed and this could be carried out using the spreadsheet supplied in Appendix 2 of this report. This will allow consideration of variations in ESA, overlay modulus or thickness, alternative strain criteria, and basecourse stabilisation.

When a satisfactory model is obtained, the individual results should be grouped into structurally uniform sub-sections to show practical intervals for which individual forms of treatment may be specified for construction. This important step ensures a cost-effective approach to ensure that the design life is achieved without using superfluous overlay. The emphasis is placed on obtaining comprehensive in-situ test data so that sections which are structurally deficient can be clearly delineated from areas which require no strengthening. This avoids the over-design that can result where a single form of treatment is applied to an extended length of pavement.

The Figure 6.1 example was obtained for a road in which shallow shear was the principal distress mode, i.e. the AUSTROADS strain criterion rather than precedent subgrade strain methods should be applied. (Examples with results for the New Zealand Supplement methods are given in Appendix 1.) The ELMOD software was used in this instance, but EFROMD2 together with CIRCLY will produce the same set of parameters except for the subgrade modulus exponent (n). Limitations of the various analysis methods are given in Section 4.2 of this report.

The above road could be interpreted in four subsections, as listed in Table 6.2.

Figure 6.1 Pavement structural analysis from FWD survey of state highway section (SH1 RP 0/0-3.1, project 1 6/94).

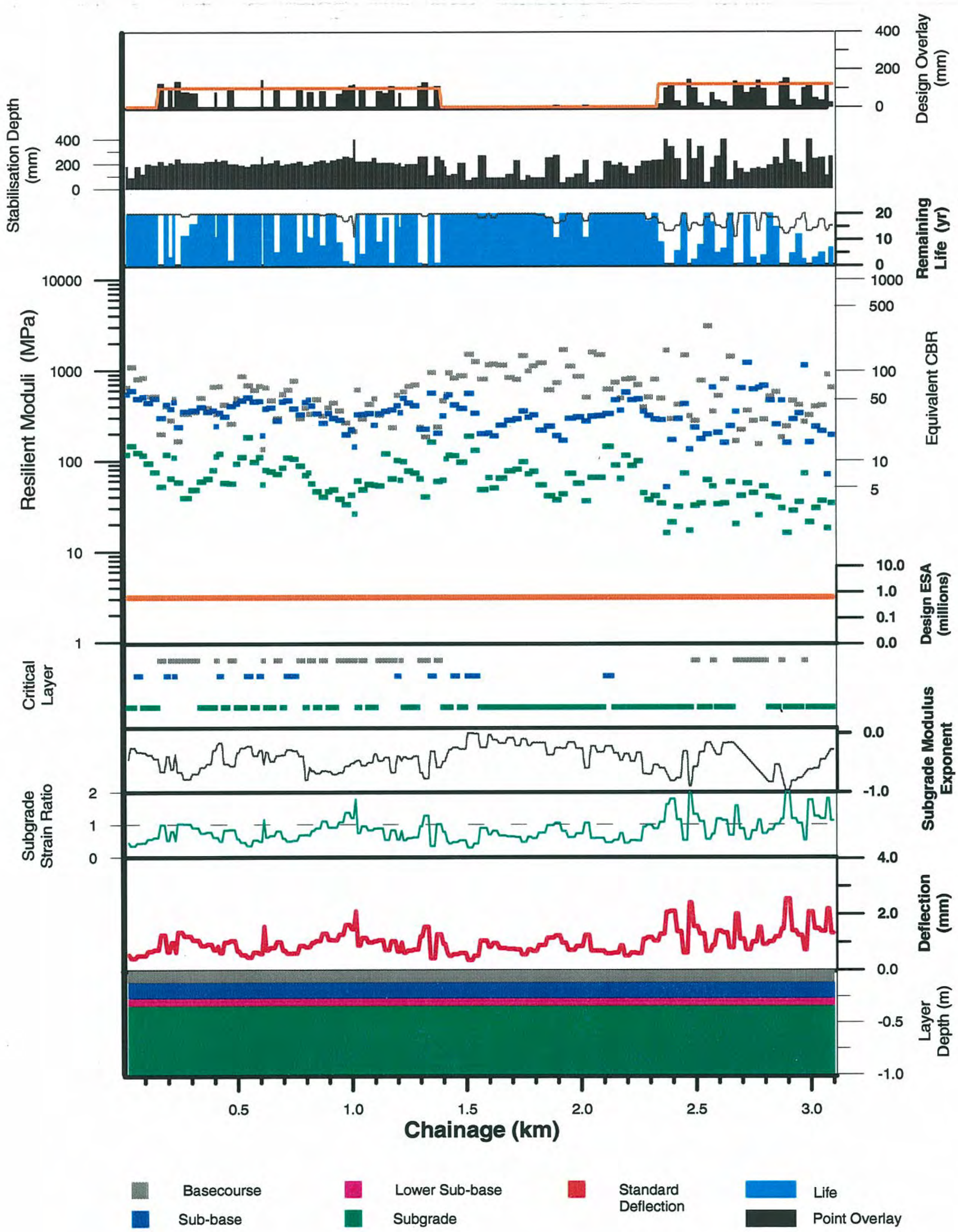


Table 6.2 Sub-sectioning for uniform intervals of the road analysed for Figure 6.1.

Sub-section	**Chainage (km)**		**Layer 1 Modulus (MPa)**		**Subgrade CBR**		**n**	**Critical Layer**	**Overlay (mm)**
	From	**To**	**Median**	**10%ile**	**Median**	**10%ile**			
1	0.00	0.16	517	515	17	12	-0.4	4	0
2	0.16	1.39	389	211	12	6	-0.5	1	100
3	1.39	2.35	808	489	9	4	-0.3	4	0
4	2.35	3.10	399	181	4	1	-0.8	1	120

Subsection 1 (up to Chainage 0.16): shows relatively high strength basecourse and subgrade. No surface distress was apparent. A four layer model (including the subgrade) was adopted. The subgrade strain ratio is much less than 1, i.e. strains are already much lower than those required by AUSTROADS and hence no overlay is required.

Subsection 2 (to Chainage 1.39): shows much greater variability in the basecourse modulus and includes some very low values. Layer 1 is shown to be critical, i.e. the analysis indicates that in several places the basecourse will be experiencing higher strains than in the subgrade, and it clearly has potential for shallow shear. (The latter was markedly evident from visual survey.) Using the AUSTROADS strain criterion, an overlay of 100 mm of unbound basecourse is required.

Subsection 3 (to Chainage 2.35): shows only minor structural deficiency and it is evident that the basecourse modulus is uniformly high. No structural overlay is needed. The subgrade strain ratio is slightly less than 1, i.e. strains are only marginally less than required. (For new pavements a subgrade strain ratio much less than 1 gives a measure of the over-design incorporated.)

Subsection 4 (to Chainage 3.10): analysis of this indicates that the subgrade CBR is lower than elsewhere on all 4 subsections and the basecourse modulus is also poor and variable. The greatest strains are occurring in the subgrade in part and in the basecourse for the remainder. The subgrade modulus non-linearity exponent (n) is unusually low, suggesting that the potential for improving subsurface drainage should be checked in this subsection.

Where precedent subgrade strain information is required, the appropriate strain ratio can be selected from the graph (Figure 6.1) for any subsection, and the actual precedent strains calculated directly from the AUSTROADS subgrade strain relationship (Equation 4 in this report).

6.4 Design Review

At completion of deflection testing and visual assessment, all raw data and a preliminary interpretation should be reviewed by the designer in order to assess the need for and location of destructive tests (e.g. coring, test pits and penetration tests).

In the Figure 6.1 and Table 6.2 example, where shallow shear was evidently the principal distress mode in this road, the test points showing the lowest basecourse moduli (or where basecourse strains are higher than subgrade strains) should be selected for test pitting and CBR testing in accordance with Section 10.3 of the New Zealand Supplement. A test pit at about Chainage 1.0 would identify the weakest basecourse and also confirm the typical subgrade CBR for the first 3 subsections. For subsection 4, basecourse CBR should be investigated around Chainage 2.75 but, in this case, care would be needed to identify the more adverse areas visually as the results show marked fluctuation in stiffnesses. The subgrade CBR here could be expected to be significantly lower than at the test pit site at Chainage 1.0.

Re-analyses for final design are normally carried out to incorporate the destructive testing information. Finally, geometric constraints need to be considered (e.g. kerb and drain levels), and then comparisons may be made to determine the most cost-effective treatment, i.e. local digouts, overlay, cement stabilisation, or reconstruction. In this example, costs for overlays of 100 to 120 mm of TNZ M/4:1995 would be compared with those for cement stabilisation of about 250 mm to give the same design life. However the example shows some points where very deep stabilisation would be required, and there the subgrade may be too weak for this option.

6.5 Case Histories

A number of pavement evaluations using mechanistic analysis are presented in Appendix 1. These show both the AUSTROADS overlay design methods and the two precedent strain methods from the New Zealand Supplement. Included are rehabilitation designs of existing pavements, recent overlays and new construction. The ELMOD software was used, but similar results could be obtained using EFROMD2 and CIRCLY. A summary of the interpretation which can be made is given in the text accompanying Appendix 1.

The examples show:

- New basecourse prior to shakedown, with typical moduli for local M/4 basecourses
- Cement-stabilised basecourses
- Unbound basecourse on pumice sand, showing volcanic subgrade characteristics
- Friction course with localised distress

- Unbound basecourse on peat, showing peat subgrade characteristics
- Failure in recent unbound basecourse, with identification of pavement distress mechanisms
- Old basecourse with part overlay, for before and after overlay comparison
- Shoving in basecourse over hard rock, to show limitations of simplified methods

Email versions, with software that provides graphical viewing for these data, are available from the author. These files allow enlargement of specific sections or display of parameters. Further interpretation of the data can also be carried out using the spreadsheet analysis program shown in Appendix 2. This provides, in a transparent form, the Odemark-Boussinesq equations for stresses and strains in a layered elastic structure subject to a 1 ESA dual tyre. A printout of the spreadsheet formula (taken from Ullidtz, 1987) is given in Appendix 2.

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APPENDICES

Appendix 1
CASE HISTORIES OF STRUCTURAL EVALUATION OF PAVEMENTS

Interpretation of FWD Data & Examples using AUSTROADS Mechanistic Procedures

The following eight graphs show structural evaluations made recently (1996-97) from Falling Weight Deflectometer (FWD) surveys on a range of pavement types used in New Zealand. These evaluations are a selection from 30 trial sections on roads located throughout the country. The sections are currently (1998) being examined using mechanistic analyses (as Transit New Zealand Research Project PR3-0171, Tonkin & Taylor in prep.) As well as monitoring changes in the pavements, each section is analysed using a range of overlay methods including those given in the AUSTROADS Pavement Design Guide (1992), and the methods documented in the later New Zealand Supplement (Transit New Zealand 1997). Results obtained using the two methods may be compared on the graphs.

The overlays analysed and depicted in the graphs are:

File H6: New basecourse prior to shakedown
File A3: Cement-stabilised basecourse
File A7: Unbound basecourse on pumice sand
File BR: Friction course with localised distress
File H3: Unbound basecourse on peat
File N5: Failure in recent unbound basecourse
File RV: Old basecourse with part overlay
File CO: Shoving in basecourse over hard rock
(Horizontal axis is distance of stations from origin, in kilometres)

The analyses carried out on each of these overlays, and shown in the graphs, include the relevant selection from the following:

Layer 1 modulus (MPa)
Layer 2 modulus (MPa)
Subgrade CBR
Subgrade modulus exponent
AUSTROADS 1992 (Simplified Ch.10) overlay (mm)
(Ch. 10, in AUSTROADS 1992)
HDM modified structural number (identified as SNC or SN)
Friction course suitability
FWD corrected deflection (mm)
AUSTROADS (GMP - rigorous) overlay (mm)
AUSTROADS General Mechanistic Procedure (1992)
AUSTROADS 1994 (ASMOL) overlay (mm)
AUSTROADS Simplified Mechanistic Overlay in ARRB (1994)
GMP (AASHO) residual life (yr)
GMP critical layer

TNZ '97 precedent strain overlay (mm)
NZ Supplement to AUSTROADS (1997), Equations 10.3, 10.4
TNZ '97 SHPDRM overlay (mm)
NZ Supplement to AUSTROADS (1997), Equation 10.3 alone
GMP overlay (mm)
General Mechanistic Procedure using AUSTROADS (1992) strain criterion
Subgrade strain / GMP allowable strain

The parameters discussed in the main report are shown in the graphs of this Appendix if relevant to the interpretations made for the individual sections.

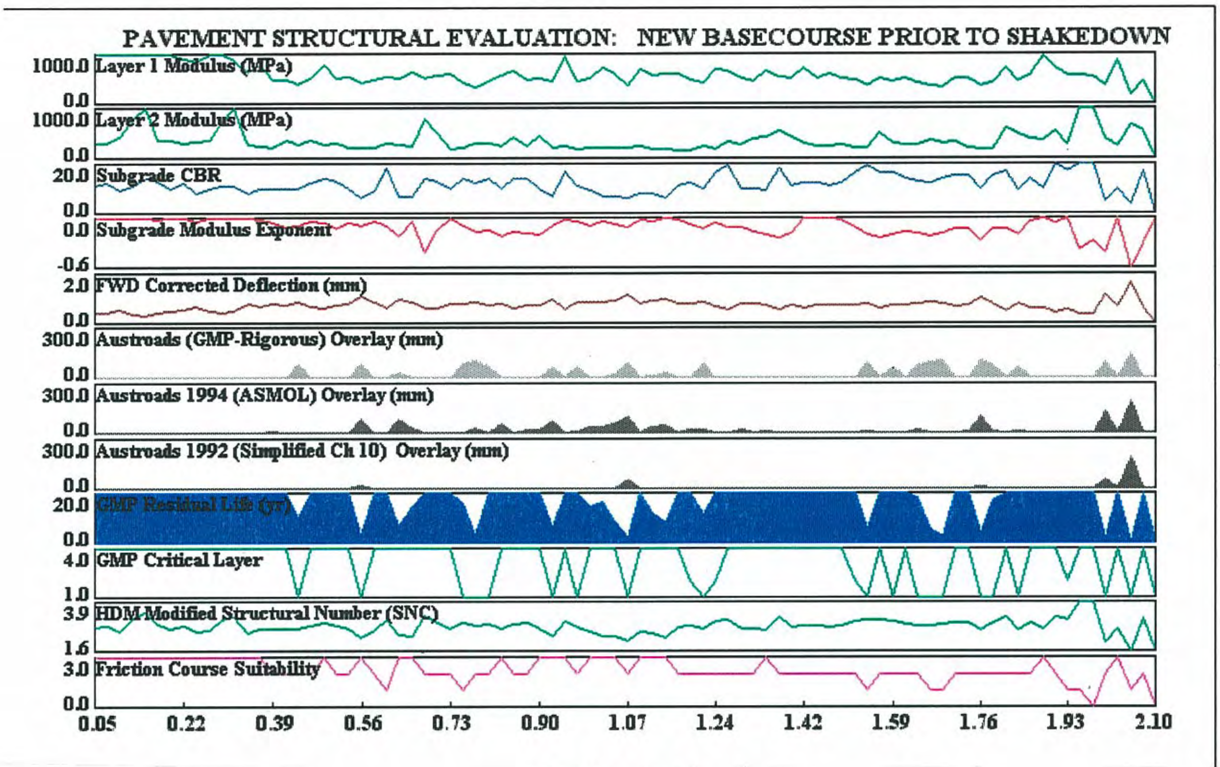
The text accompanying each graph gives examples of the information that can be obtained from the analyses to gain some insight into the structural behaviour of the pavement, likely reasons for distress (apparent or latent), and appropriate rehabilitation options.

The demonstration files are available in electronic format with graphical viewing software for examining the data in greater detail, and display of other parameters. (Instructions are given in the README file.) The graphs on the following pages show screen captures from the viewing software.

Also available on electronic format is the spreadsheet (shown in Appendix 2 of this report) which carries out a forward mechanistic analysis, calculating stresses and strains in a multi-layer elastic pavement.

To obtain the demonstration files in electronic format by email, contact the authors of this report, using the following email address:
gsalt@tonkin.co.nz

File H6: NEW BASECOURSE PRIOR TO SHAKEDOWN



This is a newly constructed section of highway. The pavement is unbound basecourse and stabilised sub-base on fine grained subgrade.

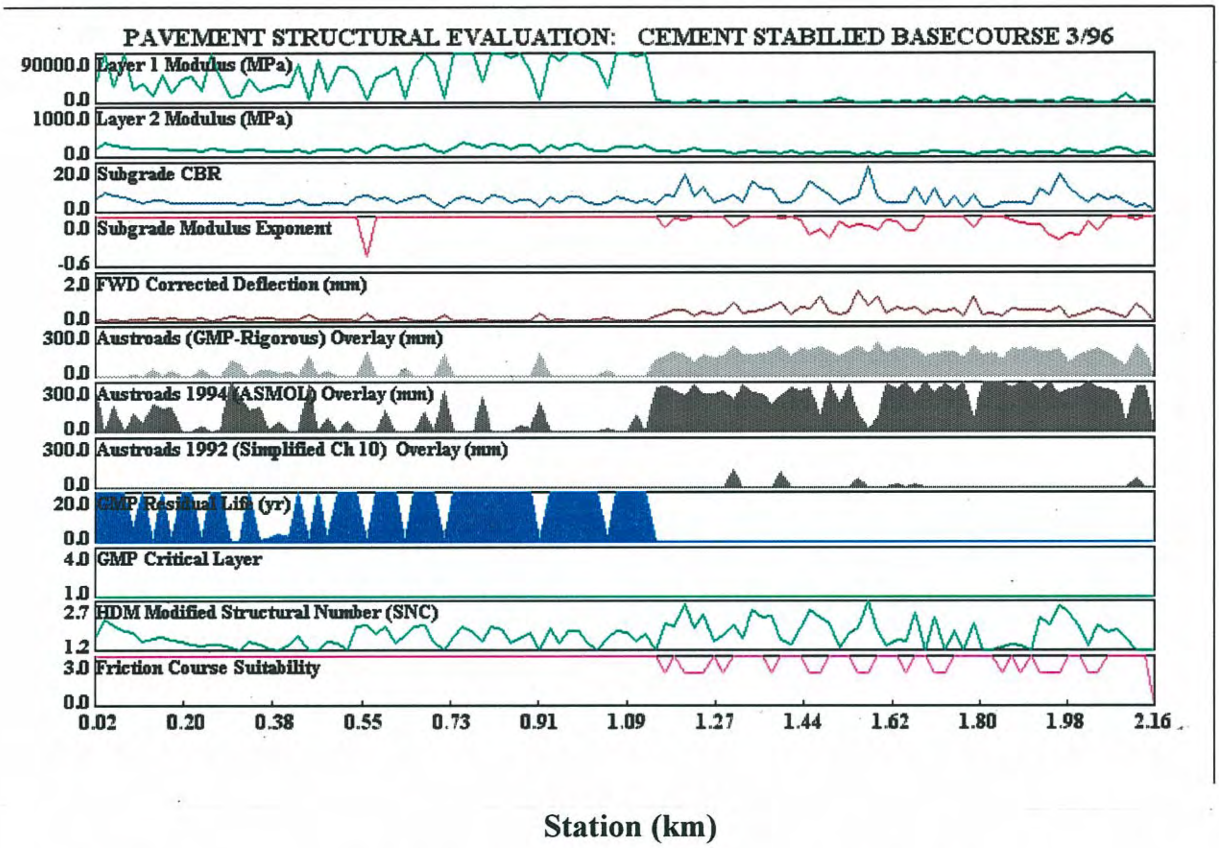
The tests were taken soon after construction. In practice, even with heavy construction compaction, unbound pavement layers will densify considerably in the first 10,000 to 20,000 ESA of traffic. At this site, good moduli (500 MPa for the most part) have been achieved in the basecourse, and the uniformity is relatively good also. The CBR is relatively high (mostly more than 120).

The residual life graph and calculated overlay requirements may be unduly pessimistic as no densification of the pavement layers from trafficking has taken place. Likewise the critical layers (those where limiting strains are experienced first) are, at the time of testing, inferred to be the basecourse and upper sub-base. (Reliable design aims to make the subgrade the critical layer.)

This highway will be re-tested to examine and quantify the moduli improvement in the near-surface layer after modest trafficking (i.e. >20,000 ESA).

The TNZ overlay methods are not relevant in this instance as minimal traffic had used the road at the time of testing.

File A3: CEMENT-STABILISED BASECOURSE



This is a quarry road with three different forms of cement-stabilised basecourse, each constructed at different stages with different methods. The average basecourse moduli are 40,000 MPa (to Station 0.7), 80,000 MPa (to Station 1.2) and 2,500 MPa for the remainder. The study of this section is addressing performance of the variously stabilised materials, as they are all subject to the same traffic.

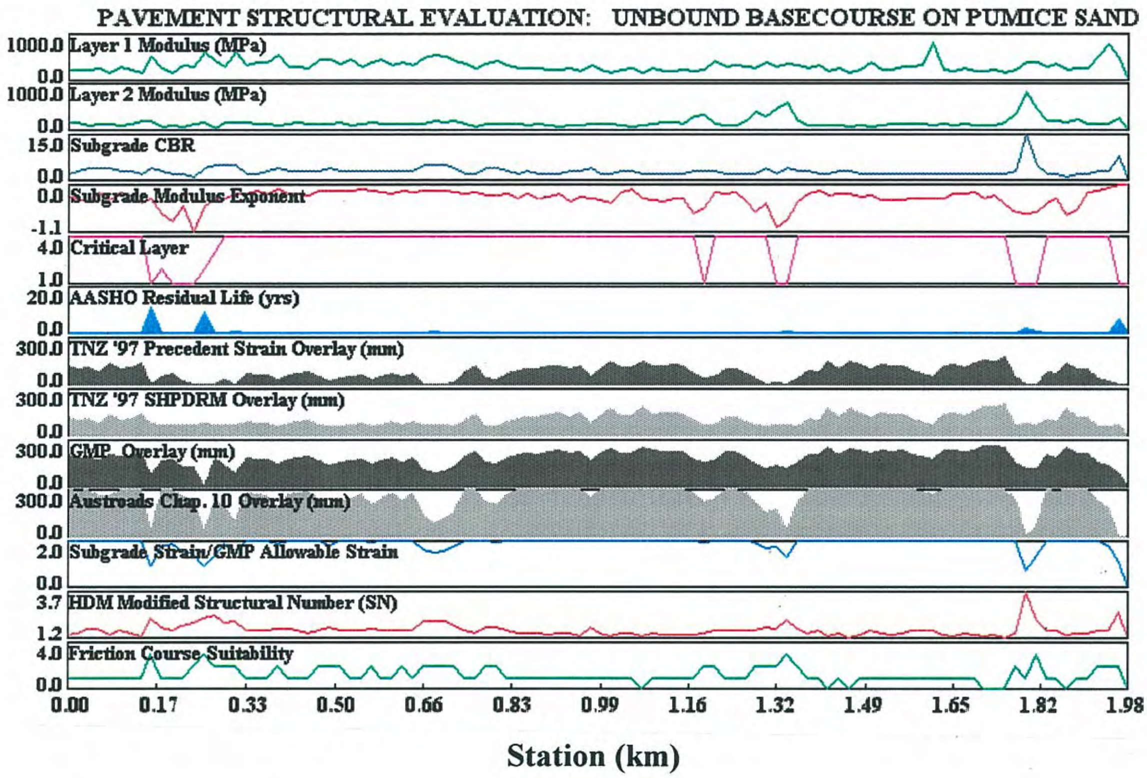
It is interesting to note the wide variation in basecourse moduli achieved within each section. This is a common finding with other stabilised pavements included in this study.

Preliminary interpretation using the three AUSTROADS alternative methods show considerable variation between analytical methods, i.e. the AUSTROADS Simplified analysis presents a markedly more optimistic picture of all three sections.

The GMP suggests that Layer 1 will be critical in all sections, i.e. tensile strains in the base of the stabilised layer will govern subgrade strains, as normally expected in this form of construction.

The New Zealand Supplement methods are not shown as they are not applicable to cement-stabilised basecourses where subgrade strains are not critical.

File A7: UNBOUND BASECOURSE ON PUMICE SAND



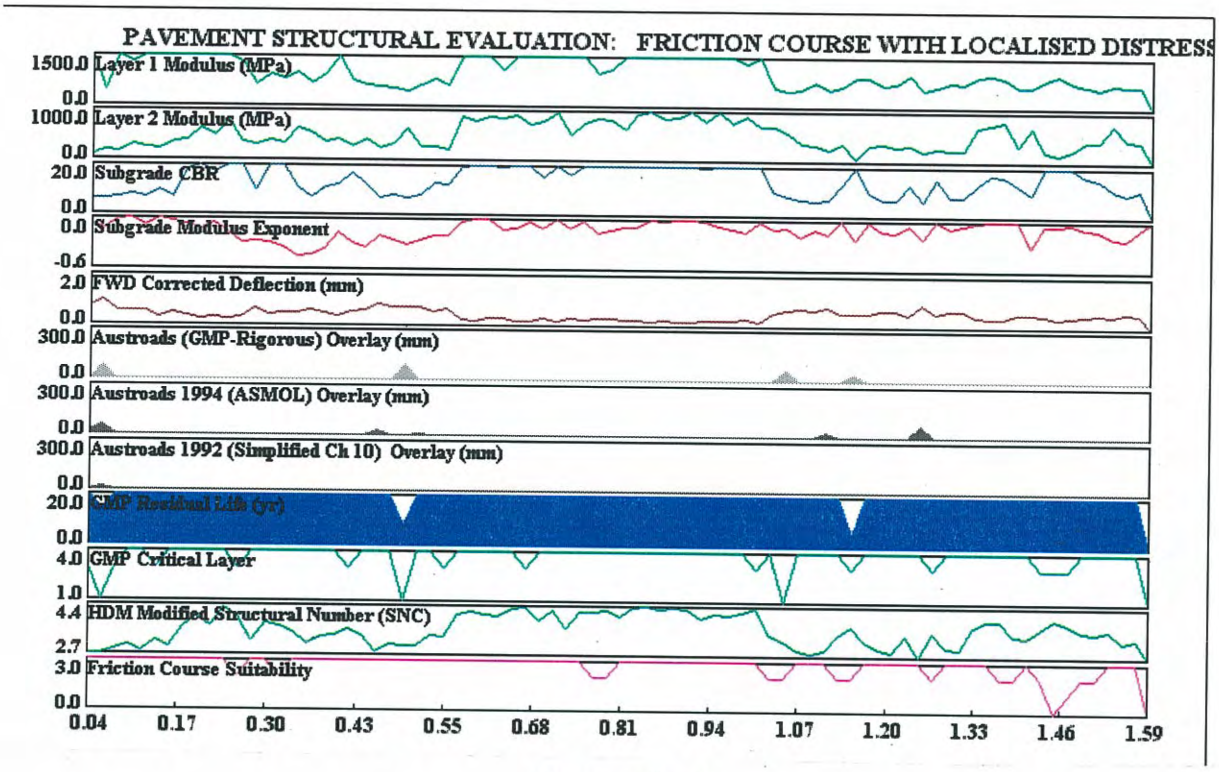
This section of highway is failing and is being rehabilitated. It has conventional unbound basecourse and sub-base over pumice sand which characteristically exhibits relatively low modulus. It is of interest that it is subjected to high traffic loading, has relatively uniform moduli in each layer and, using some design methods, requires substantial thicknesses of overlay.

In this case, the AUSTROADS (Chapter 10, 1992) design calls for extremely thick overlays. GMP requires slightly thinner overlays but it is evident from the subgrade strain ratio plot that the strains in the subgrade will be in most cases twice those permitted by AUSTROADS GMP. The Transit New Zealand Supplement (1997) methods show substantially thinner overlay, more in keeping with the expectation for pavement performance in this area.

The suggestion is that pumice subgrades generally do not perform conventionally, and applying considerably more optimistic criteria may be appropriate than are advocated by the AUSTROADS design, because Australian experience with unweathered volcanic soils is limited. This aspect is being addressed in the study on New Zealand volcanic soils recorded in this report.

The rehabilitation options include straight overlay, or stabilising the existing basecourse followed by lesser thicknesses of unbound granular overlay.

File BR: FRICTION COURSE WITH LOCALISED DISTRESS



Station (km)

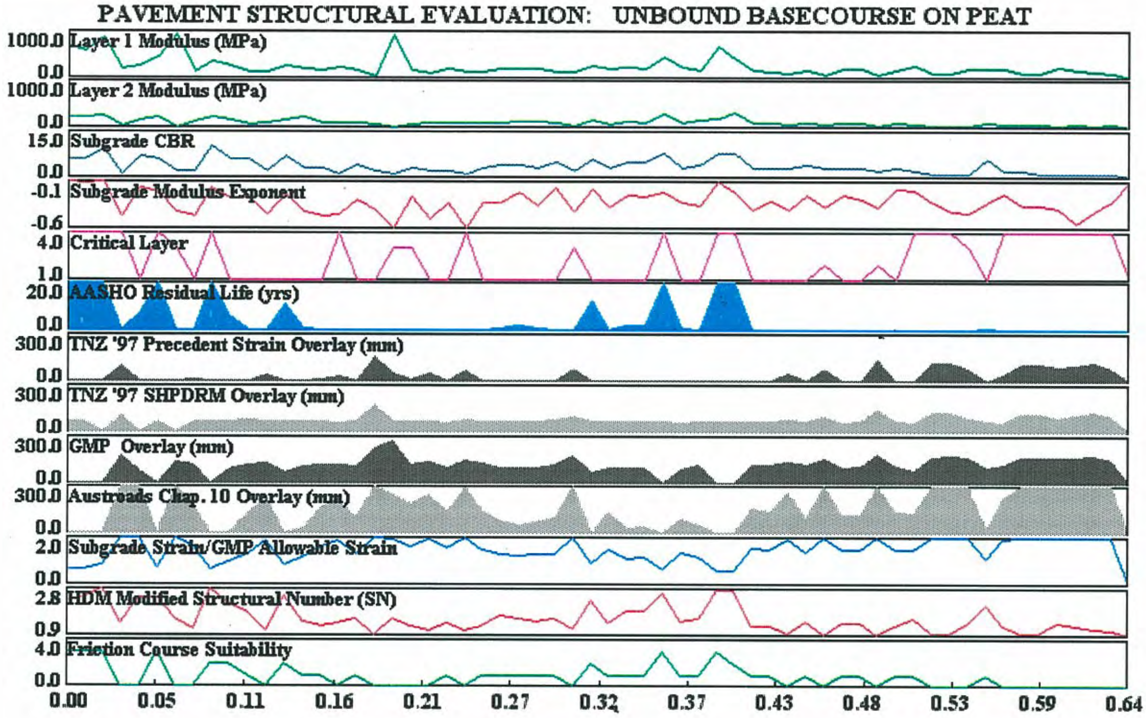
This is an unbound granular pavement, recently surfaced with friction course where the state highway passes through a city. The relatively thin friction course (30 mm) cannot be differentiated in the back-analysis as a separate layer, so it is combined with the basecourse as layer 1, producing an average modulus of nearly 1000 MPa.

The sub-base modulus and subgrade strength are both relatively high also, indicating it is a stiff pavement.

The New Zealand Supplement (1997) overlay methods are not generally applicable to friction course surfacings and are not in the graph. GMP indicates that little problem with vertical strains occurs anywhere in the pavement structure. Another method is therefore required to determine the life of the friction course.

This example is intended as verification of the RRU Bulletin 79 criteria for suitability of pavement for application of a friction course surfacing. Categories assigned are unsuitable (0), suitable for low AADT (1), medium AADT (2), or high AADT (3). Actual categories from the field testing are shown in lowermost graph. This heavily trafficked pavement is already showing some evidence of cracking, confined to the areas identified as category 2, i.e. performance is consistent with the recommended criteria. One objective of the continuation of this project will be to transfer the current AADT categories to more meaningful ESA equivalents.

File H3: UNBOUND BASECOURSE ON PEAT



Station (km)

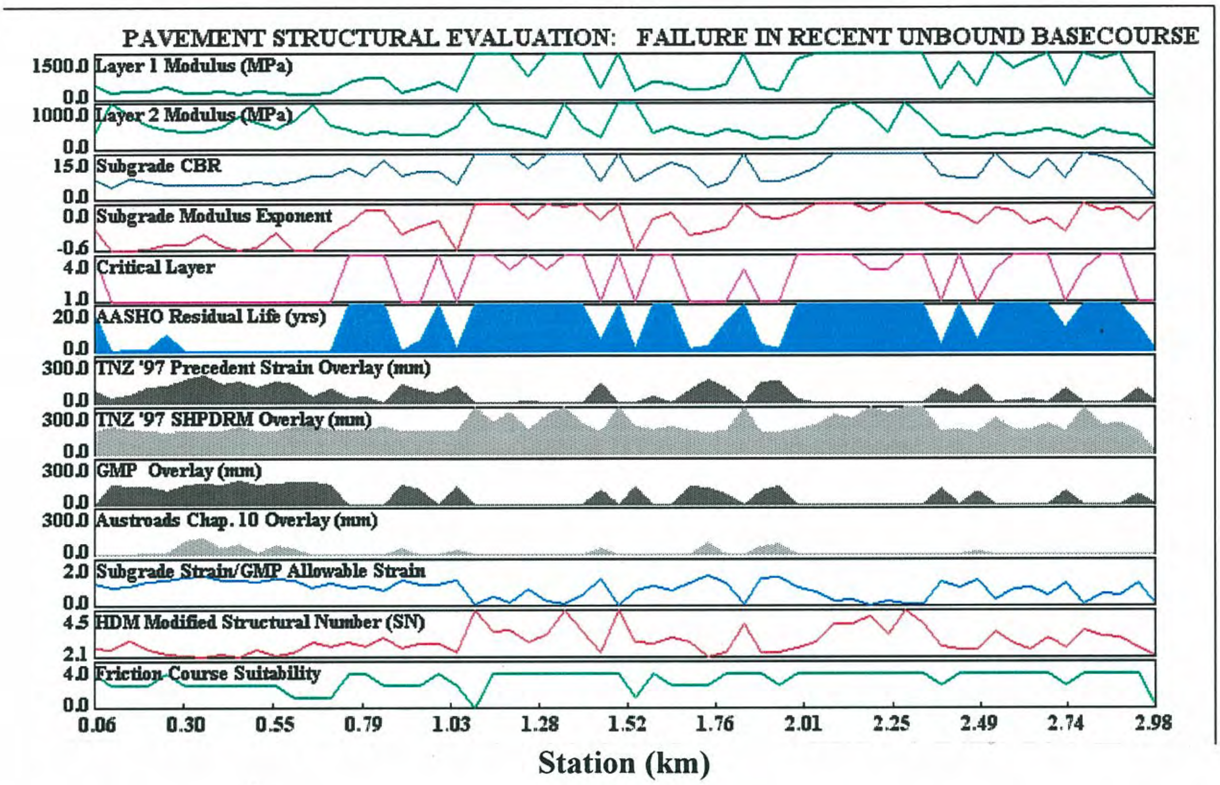
This road is an unbound granular pavement with poor profile shape. It is being used as a research section by the consultants responsible for its rehabilitation. The basecourse shows some very low moduli and consequently the critical layer in many places is found to be the surfacing. (When layer 1 governs, rapid deterioration with shallow shear can be expected.)

The subgrade modulus is very low (design CBR 1-2) and markedly non-linear (i.e. exponent less than -0.3, typical of peat).

The AUSTROADS (1992) Chapter 10 simplified design method shows large overlay requirements but the GMP indicates about 100 mm less, a common finding on very weak subgrades with non-linear moduli.

Neither of the New Zealand Supplement (TNZ 1997) methods are applicable because most of the strains are highest in the basecourse layer, but they are still useful to define a lower bound to likely overlay requirements. The TNZ 1997 method tends not to discriminate road sections which are relatively strong and require no overlay from those that are weak, giving a relatively uniform overlay requirement rather than targeting the structurally inadequate sections.

File N5: FAILURE IN RECENT UNBOUND BASECOURSE

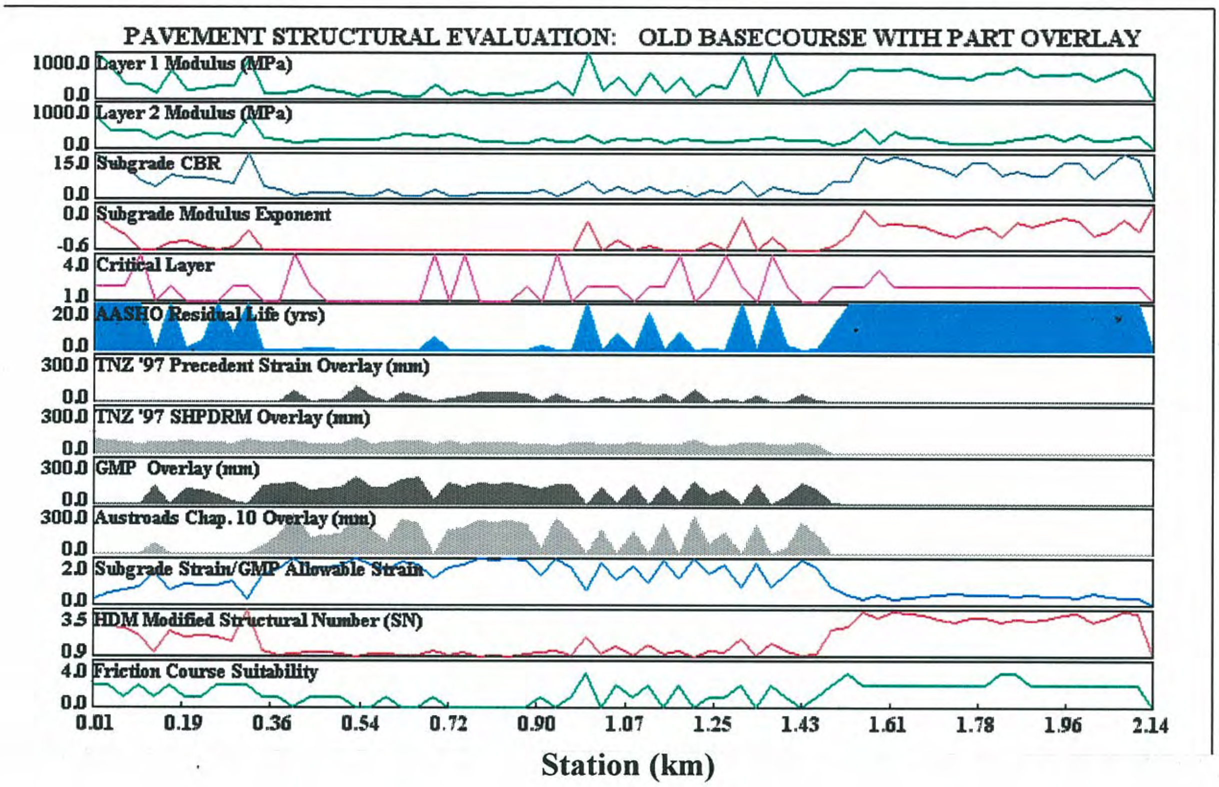


These tests are from a road with steep grade, with cuts and fills through mudstone. Extensive deformation of the recently rehabilitated pavement has occurred. As far as Station 0.5, the surface layer has a very low modulus and this is found to be the critical layer. (Continuing rapid loss of shape from shallow shear deformation would be expected without intervention.) The remainder of the section shows some very large surface moduli, suggesting that some Asphalt Concrete or surface stabilisation may be present. Some destructive testing (e.g. shallow test pits) is appropriate to identify the failing materials.

The moderately low CBR with non-linear subgrade modulus (over the first 0.5 km) points to the likelihood of very wet conditions at the top of the mudstone subgrade. Springs are likely in this terrain and subsoil drainage would need to be addressed. Beyond Station 0.5 km, CBR shows marked variation as the pavement crosses from cut to fill. This is one of several examples encountered where new construction has failed and the designers have found difficulty identifying the distress mechanism from visual survey alone. In each of these cases a low subgrade modulus exponent has been a useful tool, distinguishing problem areas from those that are performing well.

The comparison of overlays shows unusually poor agreement between the analysis methods. AUSTROADS simplified method indicates a need for overlays up to 300 mm while the GMP requires less than 200 mm. SHPDRM is out of step with the other methods. These contrasts reinforce the opinion that design checks should include more than one method of design.

File RV: OLD BASECOURSE WITH PART OVERLAY



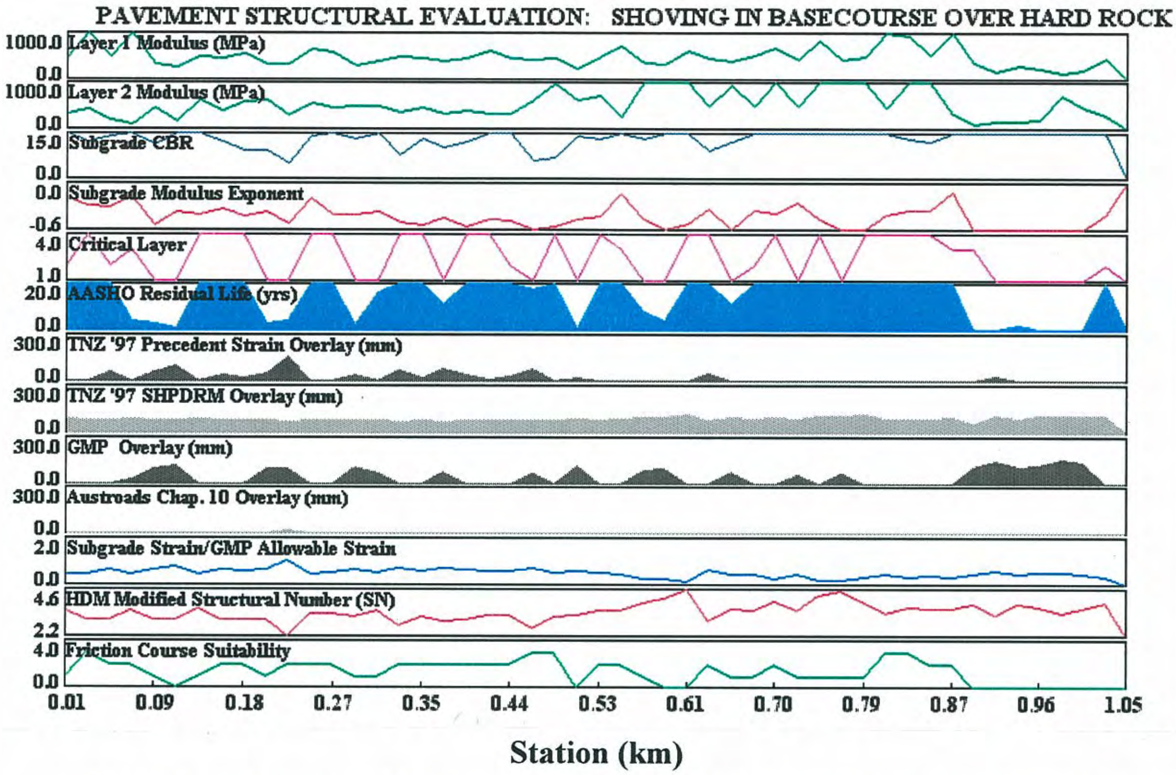
This section shows an old unbound pavement up to Station 1.5, followed by a section which has been rehabilitated with digouts and thick overlay of M/4 specification (Stations 1.5-2.14). The low CBR and very non-linear subgrade modulus beneath the old pavement indicates very soft clay or possibly peat. Assuming uniform conditions at depth originally, clearly the CBR beneath the overlain portion has been enhanced considerably. This will be related in part to the consolidation beneath the surcharge of pavement, but also because the original subgrade modulus is strongly non-linear. Non-linearity is slightly reduced beneath the overlaid pavement.

The sawtooth pattern of fluctuating moduli in the basecourse layer from Stations 0.95 to 1.43 imply that there has been shallow reconstruction on one side of the road. (Test points are displayed from alternate sides of the road.)

The residual life shows that the overlay has been quite effective and good construction techniques are indicated by the very uniform modulus achieved in the overlaid portion (Stations 1.5-2.14, Layer 1).

The first half of the section (Stations 0.0-1.43) is about to be overlaid also, and comparative FWD testing will be carried out as part of the ongoing research study.

File CO: SHOVING IN BASECOURSE ON HARD ROCK SUBGRADE



This section shows an old unbound pavement which is underlain by hard rock at relatively shallow depth, as shown by rock in box cuts. Cement stabilisation of the surface layers was carried out some years ago. Now the layer moduli suggest that, while some stabilisation has taken place in the sub-base, this stabilisation has been of little benefit in the upper basecourse.

The relatively high CBR values, together with very low subgrade modulus non-linearity exponents, indicate the likelihood of a hard layer at depth. At present, significant distress is evident with localised shallow shear taking place, especially severe at Stations 0.9-1.0. The GMP identifies the shallow shear by showing that Layer 1 is critical and the significant overlay requirements at the relevant locations.

The AUSTROADS (1992) interpretation calls for virtually no overlay which clearly conflicts with the real situation. The lack of validity of this model is perhaps not surprising in this instance because the method is based only on central deflection. Central deflection alone does not recognise that very little of the deflection component will arise from the subgrade (hard rock). This means that high strains occurring in the surface layer cannot be distinguished.

This case shows the need to recognise that very unconservative design can occur if either of the simplified overlay methods are adopted, when incipient basecourse shoving (which may not be evident from visual survey) is developing in pavements underlain by hard subgrades. The Transit New Zealand methods are not appropriate as subgrade strains (in the rock foundation) are not governing the pavement life.

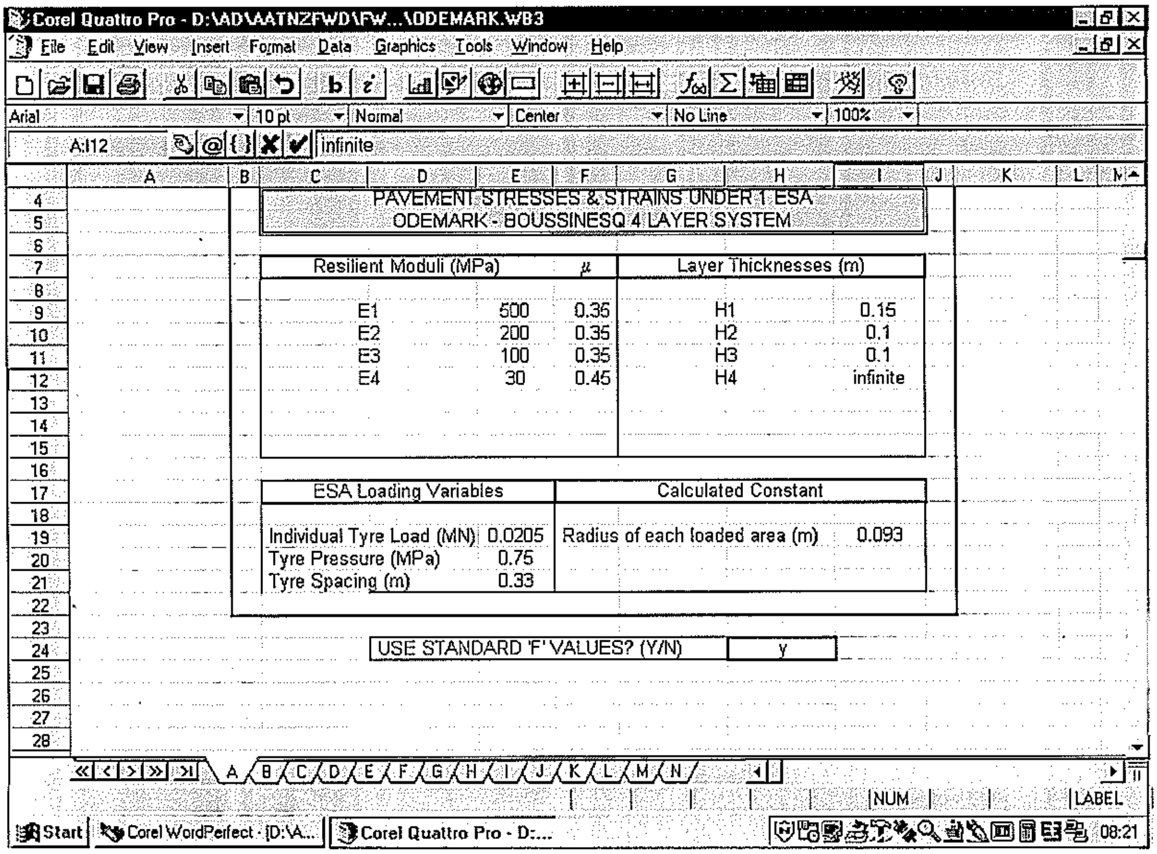
Appendix 2
PAVEMENT ANALYSIS SPREADSHEET

Structural Analysis of Stresses & Strains in a 4-layer Pavement

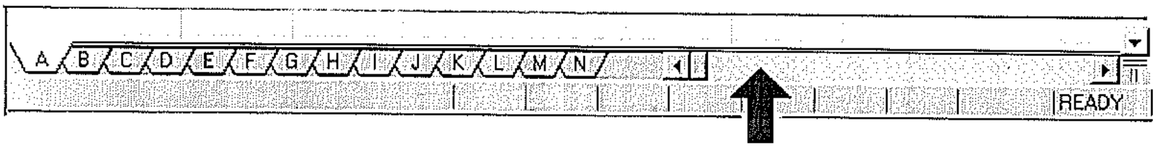
This spreadsheet uses the Odemark-Boussinesq equations method to compute stresses and strains in a conventional pavement. Any number of linear elastic layers that are homogeneous and isotropic may be modelled.

The equations have been taken directly from Chapter 2 of *Pavement Analysis* by Per Ullidtz (1987), which defines the variables used in the spreadsheet prepared in Excel (or Quattro Pro) programs.

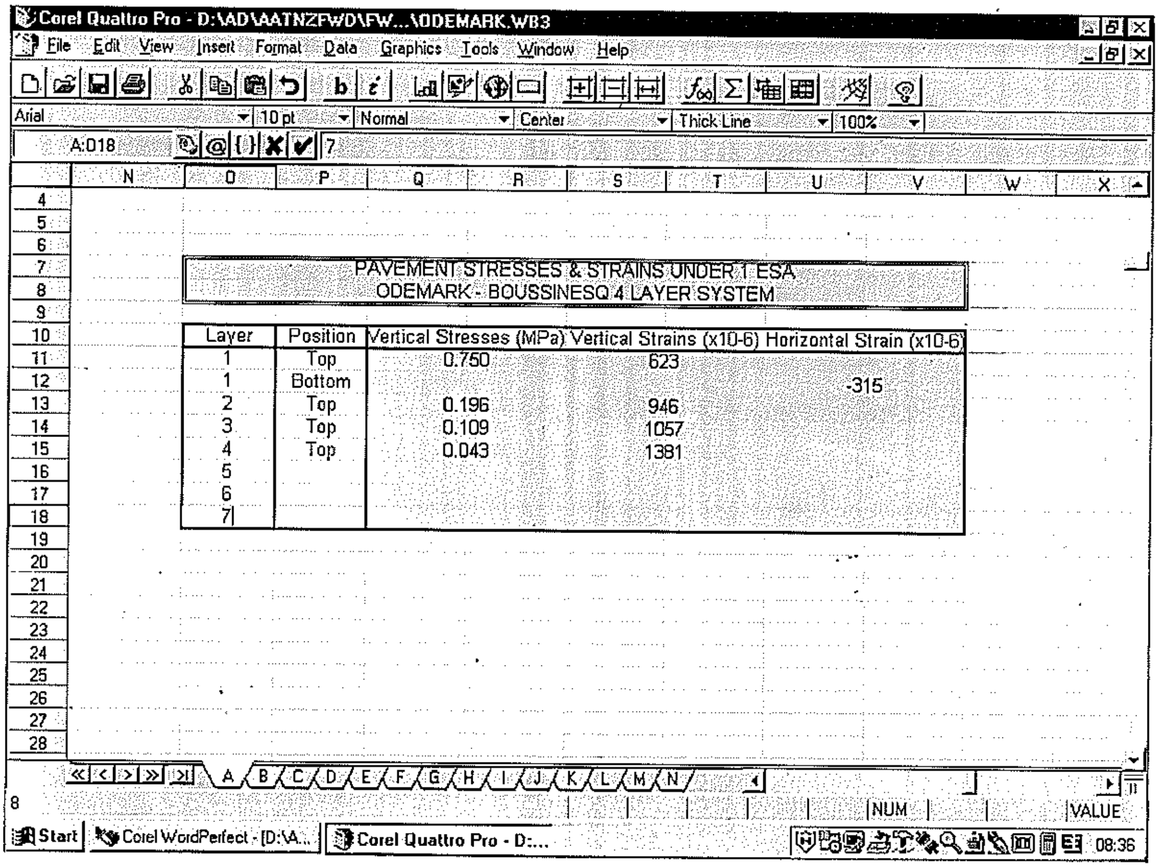
The spreadsheet (available from the author by email, file name ODEMARK.XLS) comprises a main program (Odemark equations) and a subroutine (Boussinesq equations) that is called five times.
First, the subroutine calculates the horizontal tensile strain at the base of the top layer. Then it is called a further four times to calculated the stresses and strains at the top of each of the four layers.
The initial input values (layer thicknesses, moduli, and Poisson's ratio) are keyed in on Sheet A, and the associated output values are given on the right-hand side of the screen. Effectively, Sheet A is the front-end, and Sheet B is where all the input values pass through a series of calculations to be returned back to Sheet A.
Sheet B is, therefore, not required by the user to see or access, but can be used for modification or assurance of the program.
An example of the input for a 4-layer structure follows:



The program goes through the calculations almost instantaneously and the output can be found by clicking once, at the point shown here with the marker.



This is an example of the output screen:



The input screen contains the standard 1 ESA loading geometry which may be modified if required. Also present are Odemark's *f* values (Ullitdz 1987). These can also be changed to ensure that the results correspond with ELMOD, CIRCLY, MODULUS or other packages. Output of allowable ESA is also given, on the basis of the AUSTROADS strain criteria for the subgrade. (If desired, allowable ESA for the other unbound layers or a cemented surface layer can also be read from the display.)

Although the original spreadsheet has been checked, the user is advised to carry out independent verifications, and to be satisfied that the results are applicable to individual situations.

Spreadsheet Formulae

Initially the program starts off with Odemark *f* values for the first layer with values from Sheet A:

SUBROUTINE ODEMARK

f1	1
f2	0.8
f3	0.8

call z e(1) 0.35 vstress(1) vstrain(1) dmy

The above line calls for the Boussinesq subroutine and the calculations are made using the following formulae:

z	0.001
E	@MAX(A:$E$8,0.001)
u	+A:$F$8
az	+1+($G$6/I7)^2
za	+1+(I7/$G$6)^2
ez	(1+I9)*($G$4/I8)*((I7/$G$6)/(I11)^1.5-(1-2*I9)*((I7/$G$6)/(I11)^0.5-1))
sz	+$G$4*(1-1/(I10)^1.5)
er	((1-I9)/(2*I9)*(I13-I8*I12)-I9*I13)/I8
R	(I7^2+$G$5^2)^0.5
ct	+I7/I15
eza	(1+I9)*$G$3/(2*@PI*I15^2*I8)*(3*I16^3-2*I9*I16)
ez	+I12+I17
sza	+3*$G$3/(2*@PI*I15^2)*I16^3
sz	+I13+I19
eta	(1+I9)*$G$3/(2*@PI*I15^2*I8)*(-I16+(1-2*I9)/(1+I16))
era	(1+I9)*$G$3/(2*@PI*I15^2*I8)*(-3*I16^3+(3-2*I9)*I16-(1-2*I9)/(1+I16))
er	@MIN(I14+I21,I14+I22)
R	(I7^2+($G$5/2)^2)^0.5
ct	+I7/I24
ez'	+2*(1+I9)*$G$3/(2*@PI*I15^2*I8)*(3*I16^3-2*I9*I16)
sz'	+6*$G$3/(2*@PI*I15^2)*I16^3
et'	+2*(1+I9)*$G$3/(2*@PI*I15^2*I8)*(-I16+(1-2*I9)/(1+I16))
er'	+2*(1+I9)*$G$3/(2*@PI*I15^2*I8)*(-3*I16^3+(3-2*I9)*I16-(1-2*I9)/(1+I16))
ez	@MAX(I26,I18)
sz	@MAX(I27,I20)
er	@MIN(I29,I28,I23)

It has now completed the calculations for the vertical stress and strain at the top of the first layer. It resets the Odemark *f* values and starts again:

```
ffactor  +A:$J$8/A:$J$18*A:$E$8/@MAX(A:$E$9,1)

f1       @IF($B$9<10,0.96+0.83*A:$J$18/A:$J$8*A:$E$9/A:$E$8,1.13-0.0565*
         @LN((A:$J$8/A:$J$18)^2*A:$E$8/A:$E$9))

he1      +$D$11*A:$J$8*(A:$E$8/A:$E$9)^(1/3)

call     he1 e(2) 0.35 dmy dmy tenstrain(1)
```

It has now calculated the horizontal tensile strain at the base of the first layer. It now proceeds to calculate the vertical stresses and strains at the tops of the second, third, and fourth layers:

```
f1       @IF(A:S26="Y",1,0.99-0.07*A:$J$8/A:$J$18)
he1      @IF(A:S26="Y",$B$17*A:$J$8*(A:$E$8/A:$E$9)^(1/3),$B$17*A:$J$8*
         (A:$E$8/A:$E$9)^(1/3))

call     he1 e(2) 0.35 vstress(2) vstrain(2) dmy

f2       @IF(A:S26="Y",0.8,1.04-0.176*@LN(A:$E$9/A:$E$10))
he1      @IF(A:S26="Y",$B$22*A:$J$8*(A:$E$8/A:$E$10)^(1/3),$B$22*A:$J$8*
         (A:$E$8/A:$E$10)^(1/3))
he2      @IF(A:S26="Y",$B$22*A:$J$9*(A:$E$9/A:$E$10)^(1/3),$B$22*A:$J$9
         *(A:$E$9/A:$E$10)^(1/3))

call     he1+he2 e(3) 0.35 vstress(3) vstrain(3) dmy

f3       @IF(A:S26="Y",0.8,0.96-0.176*@LN(A:$E$10/A:$E$11))
he1      @IF(A:S26="Y",$B$28*A:$J$8*(A:$E$8/A:$E$11)^(1/3),$B$28*A:$J$8*
         (A:$E$8/A:$E$11)^(1/3))
he2      @IF(A:S26="Y",$B$28*A:$J$9*(A:$E$9/A:$E$11)^(1/3),$B$28*A:$J$9*
         (A:$E$9/A:$E$11)^(1/3))
he3      @IF(A:S26="Y",$B$28*A:$J$10*(A:$E$10/A:$E$11)^(1/3),$B$28*A:$J$10*
         (A:$E$10/A:$E$11)^(1/3))

call     he1+he2+he3 e(4) 0.35 vstress(4) vstrain(4) dmy
```

The final values are taken from the ends of each subroutine calculation and only the necessary values are sent back to the solution display on the right-hand side of Sheet A.