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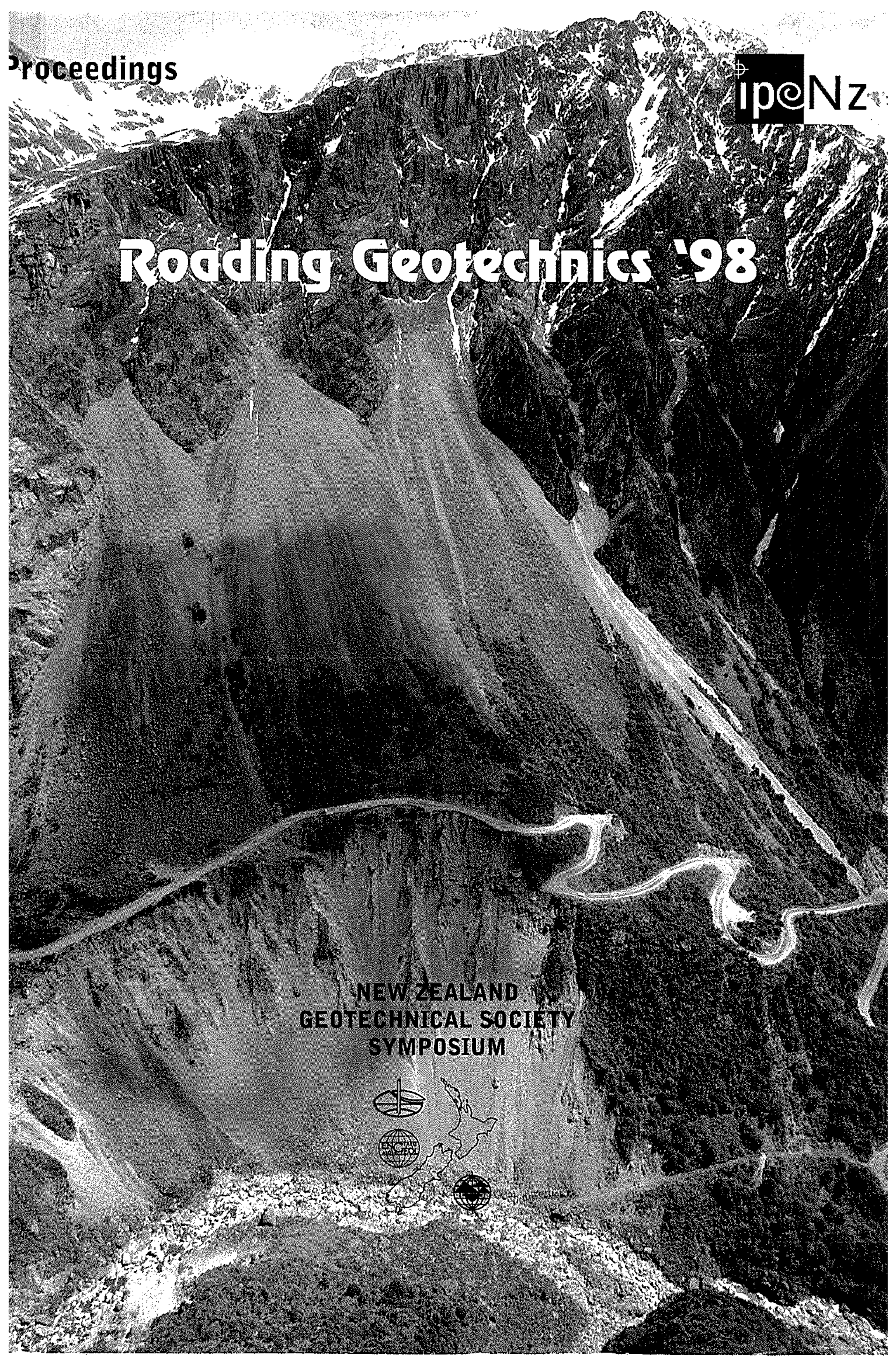

Roading Geotechnics '98

NEW ZEALAND GEOTECHNICAL SOCIETY SYMPOSIUM

Roading Geotechnics '98

AUCKLAND, JULY 1998

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FOREWORD

This was the twelfth symposium held by the New Zealand Geotechnical Society. The theme of the symposium was the geotechnical issues associated with roading in NZ. It was held at the University of Auckland 3-5 July 1998. The symposium followed a series of Symposia held on an approximately three year cycle.

1996	Geotechnical Issues in Land Development	Hamilton
1994	Geotechnical Aspects of Waste Management	Wellington
1990	Groundwater and Seepage	Auckland
1986	Pile Foundations for Engineering Streucture	Hamilton
1983	Engineering for Dams and Canals	Alexandra
1981	Geomechanics in Urban Planning	Palmerston North
1977	Tunnelling in New Zealand	Hamilton
1974	Stability of Slopes in Natural Ground	Nelson
1974	Lateral Earth Pressures and Retaining Wall Design (Workshop)	Wellington
1972	Using Geomechanics in Foundation Engineering	Wanganui
1969	NZ Practices in Site Investigation for Building Foundations	Christchurch

The subject of this Symposium was selected by the Management Committee as an area of current interest, and has special relevance following the introduction of the AUSTROADS Design manual and NZ Supplements, and the shift to mechanistic pavement design. Also there have been recent increases in road funding and the number of large roading projects undertaken.

The keynote address was given by Mr Robin Dunlop, the General Manager for Transit NZ. This address outlined the future directions for roading and reforms in NZ.

Organising Committee
July 1998

TABLE OF CONTENTS

KEYNOTE ADDRESS

PAVEMENT DESIGN

PAVEMENT MATERIALS & GEOLOGICAL RISK ASSESSMENT

CASE STUDIES: MOTORWAYS & HIGHWAYS

TESTING AND PAVEMENT DESIGN PARAMETERS

RISK/SLOPE MANAGEMENT OF ROADS

ROADING AND GEOTECHNICS IN NEW ZEALAND IN THE 21ST CENTURY

Dr Robin Dunlop
General Manager, Transit New Zealand

Keynote Address for Roading Geotechnics 1998 Symposium

Friday 3 July, 1998

Dr Robin Dunlop was born in New Zealand and graduated from the University of Canterbury with a Bachelor of Engineering in 1968 and a PhD in Roading in 1972. His main focus of study while at University was slope stability. He is currently General Manager of Transit New Zealand, and previously held a number of positions in New Zealand Rail and Ministry of Works and Development, as well as working for six months in the Transport and Road Research Laboratory in Crawthorne, England. He has presented many technical papers in New Zealand and worldwide, including carrying out consultancy tasks and training sessions for the World Bank and International Road Federation on roading agency structures and management. He is the Immediate Past President of the Road Engineering Association of Asia and Australasia, a Council member of Austroads, a Director of the International Road Federation, and a Council member of the Central Branch of the New Zealand Institute of Management.

ROADING AND GEOTECHNICS IN NEW ZEALAND IN THE 21ST CENTURY

by Dr Robin Dunlop
General Manager, Transit New Zealand

Abstract

Technology will begin to drive the management and development of the roading network in the future as never before. A more commercial approach to road management will shape all public sector highway bureaucracies that now manage most of the world's road networks, and provide the incentives and opportunities to deliver better services in a more timely and efficient way.

Management of geotechnical structures will be able to benefit from the use of technology to the point that at least some countries over the next decade will seize the opportunity to commercialise their road networks and transform the way we manage the highway networks.

Community Desires

The following are the apparent community aspirations for travel:

- Mobility
- No surprises in the travel mode
- Value for money
- Safety in the use of the system
- Don't build transport infastructure in my backyard

When managing a road network, many components of a highway system need to be taken into account in order to achieve a community's aspirations.

Components of a Highway Network

Management of a highway network comprises many components and resource requirements, only one of which is the management of geotechnical features. In the past, considerable emphasis has been placed on pavements, but this is changing as consumers demand an understanding of the services available from highway infastucture providers, especially 24 hour/all year availability of the highway network.

The key components of any highway network should be:

- Charging/pricing policy
- Investment decision making
- Traffic management and modelling
- Safety of the network
- Managing entry into the network
- Provision of roadside furniture and amenities
- Asset management of the road reserve including maintenance, rehabilitation, and design of pavements and structures.

Management of Highways in the Future

The future highway infrastructure provider are likely to be responsible for setting prices and directly charging road users, including congestion pricing, managing safety and enforcement and determining investment levels based on road user demand.

Technology already exists to directly charge road users for actual use, but the cost is much higher than say traditional fuel taxes to collect. While a fuel tax is easy to collect and the costs of administration are low, there is no long term future for such a tax in highway funding. This is because vehicles are becoming more fuel efficient, resulting in reduced revenue in relation to kilometers travelled. Alternative fuels will play a more significant role in the future making the fair charging of these vehicles more difficult using a fuel tax. Also it is virtually impossible to charge for peak demand on a highway via a fuel tax. This in effect means that a person who can afford a fuel efficient car can at the present time travel on an urban motorway at peak time for probably 30% less than a person living in a rural area driving an older vehicle on an unsealed road.

These infrastructure providers are likely to be commercially orientated, and either government or private sector companies who will not only make the decisions on pricing but will determine service levels and investment in new infrastructure after consultation with users, but free of government intervention, just like many of the other utility providers eg. telecommunications. It is unlikely that these companies will have access to property taxes but could charge for property access. These companies will make profits and pay tax.

The highway infrastructure provider will ultimately be responsible for the safety of the network and the enforcement of safety standards. This will require a completely different culture and level of accountability. The government will still have a role in regulating a highway infrastructure provider, but hopefully such legislative provisions will be kept to a minimum.

A future highway infrastructure provider will be much more customer focused in that setting charges for services will bring about greater scrutiny by the road users. This will include the information made available to road users, the amenities offered, the safety and level of ride of the road, level of congestion, and in the case of heavy vehicle operators the level of load carrying capacity. Road user surveys as currently carried out by many countries will become considerably more important as the company develops its level of service for different pricing options. Given that most highway infrastructure providers will be monopolies, Government guidance will be required via the statement of corporate intent (contract) which is negotiated annually with the company. Profit levels should not be allowed to exceed normal industry standards of return on equity.

Over time these highway infrastructure providers will make extensive use of new technology to charge for road use by time, weight and location and to provide information to users and ultimately automated highways may become practical.

Traffic management within the existing road corridors will be a key focus of a commercial highway infrastructure provider with new investment decisions receiving considerable scrutiny by the board of directors. Traffic modelling will be much more important in terms of both price setting and predicting future investment needs.

Because a commercial highway company is likely to be charging its users directly the services it offers will play a major part in determining if the company is successful. To this end road furniture such as signs, rest areas and other related facilities will be very important in this commercial mode.

Safety management will receive even higher attention by a commercial highway company as users will again judge their performance by the safety record of the road. If safety is not taken seriously then court action will be a reality which will not only cost the company but create adverse publicity for the company.

Managing access to the network will also be important from both a safety and efficiency point of view. This means that a commercial highway company will need the skills to ensure that the whole network is managed in as efficient manner as possible. Likewise, making sure the network is available close to 24 hours a day, 365 days a year, will require a very good assessment of like geotechnical structures along the route.

While considered last in their brief analysis, pavement management, including maintenance, rehabilitation and reconstruction, is nevertheless very important and where most of the expenditure will go. Considerable expertise will still be required to manage the highway pavement.

As one can see by this futuristic highway infrastructure provider scenario, the role of the pavement manager is but one component in the highway network operations of the future.

Application of the New Technology to Highways

By examining the various components of the highway system described earlier in this paper, possible applications appropriate to NZ can be discussed.

Pricing and charging

GPS is probably the only economic way to charge vehicles for the use of a total network. This technology is available now, and can be used to collect road user charges for heavy vehicles and at the same time provide those vehicle operators with many other features such as logistical positioning data.

Investment decision making

The ability to charge directly for road use will enable capital projects to go ahead when conventional funding is not available. Also new technology will enable capital projects to be built with better information availability and safety features.

Traffic management and modelling

Intelligent transport systems (ITS) can right now provide many benefits from a traffic management point of view such as:

- Co-ordinated management of traffic lights, including priority for emergency vehicles
- Ramp metering on to major arterials, and control of bus lanes
- Variable speed signage to better manage traffic flow
- Changeable message signs to advise motorists of hazards ahead

Safety of the network

The latest technology advances enable vehicles to be provided with in-vehicle guidance systems, which can either give messages about hazards ahead or inappropriate driving, or actually override the driver and correct the vehicle's position on the road. Other safety features have already been described above.

Managing entry into the network

Technology such as GPS will enable control of vehicles on to a road network, either for traffic management reasons or for vehicle enforcement. Similar technology could also be used to manage property access on to highways.

Provision of furniture and amenities

GPS technology should be able to monitor the state of road signs and other furniture from a remote site thus improving both safety and asset management.

Asset management

Using GPS, monitoring of many features in the road will be possible, such as:

- Bridges for excessive stresses
- Slope stability
- Pavement construction using in situ detectors
- Moisture conditions in pavements

Geotechnics and the Resource Management Act

Returning now to the current issues of the day, it is important to look at what role the Resource Management Act 1991 (RMA) has had on geotechnics in roading. It is quite common now to find that resource consents are issued with conditions that involve geotechnics in one way or another. For instance, the construction of a bridge in a seashore environment would normally require the construction of a cofferdam around any pile site with strict control on any discharge of removed material including water. Slope stability features in many resource approvals.

Recent Projects

SH 1, Thorndon Viaduct

Transit is nearing completion of seismic retrofitting of the Thorndon Viaduct, which has included strengthening of the foundations and consideration of ground conditions and liquefaction and lateral spreading risks in an area of reclamation adjacent to a harbour edge. A major geotechnical investigation was undertaken, which included location of the active Wellington fault trace to sufficient accuracy to enable the span above the fault to be identified, and catch frames to be designed to hold the spans in the event of movement on the fault. The retrofit has included allowance for the ground performance in a major earthquake, except that, for the Aotea off ramp, economics have precluded upgrading which would require extensive ground improvement to address a local area of liquefiable sand. Murashev and Palmer (1998) have addressed the ground conditions in the general Wellington waterfront area, including the viaduct site and Powell and Billings (1994) have addressed the general upgrading.

SH 73, Arthurs Pass

In the Arthurs Pass area we have three projects under way, which involve substantial geotechnical input for projects with extremely challenging site conditions. These are:

- Otira Viaduct project
- Whites Bridge replacement
- Candys Bend - Starvation Point project.

A paper by Graham Ramsay to this symposium describes some of the issues and dilemmas faced with road slope stability evaluations in the rock avalanche slopes at the Otira Viaduct approaches, which include sections supported on reinforced earth embankments.

SH 1, Newlands Interchange

Construction of the Newlands Interchange in Wellington involved the design of abutments, using reinforced soil and ground anchors in an area of high seismicity - up to 0.6 g design accelerations, with rock anchors which were double corrosion protected and tested to verify design parameters.

A risk management approach was adopted to design high cuttings which would not jeopardise the integrity of properties at the top, with decision analyses to decide the best course of action.

SH 2, Silverstream - Widening at Manor Park

Road widening at Manor Park used an innovative method to widen the road through a tight corridor between nature reserve and railway. Used cuttings were stabilised by soil nailing and partial shotcrete grid with ponga-faced walls, integrating engineering and landscape requirements in environmentally sensitive nature reserve. Reinforced earth walls were incorporated along the railway line.

SH 6, Newmans Lookout, Upper Buller Gorge

The site of Newmans Lookout is approximately 20 km north of Inangahua Junction in the Upper Buller Gorge on SH 6. This highway, which is contained on an excavated bench above a steep 100 metre high slope forming the right bank of the Buller River, has been progressively threatened by river bank instability since the 1998 Inangahua earthquake initiated severe river bank erosion.

Geotechnical and river engineering studies in 1996 indicated the preferred reinstatement option was relocation of the highway close to the existing alignment by further excavation into the hillside, rather than large scale deviation of the original alignment. Monitoring of the active processes has been identified as an appropriate strategy for risk management at this location, which has enabled major earthworks to be deferred, resulting in optimisation of capital expenditure. Monitoring methods include surveying of the river bang erosion trends, tension cracks at highway level, and flood and ground water levels. Drainage drilling is being carried out to reduce ground water pressures at the critical highway level. Control of river erosion is being assisted by means of a rock groyne and progressive scalping of the river terrace opposite the eroding highway slope to ease flow velocities.

SH 6, Hawks Crag, Buller Gorge

The road at Hawkes Crag passes underneath an overhang that was created when the road was first constructed in the late nineteenth century. This feature has been enlarged three times since then to accommodate larger vehicles using the route. The route continues to be a restriction to some legally-dimensioned heavy commercial vehicles, so a further enlargement is being carried out.

This section of the Buller Gorge is influenced by many geological features. Initial investigations took the form of a site inspection and literature search of available geological data. The method proposed for securing the large overhang was to fix a line of rock bolts along the inside of the overhang to hold discrete blocks together. Subsequently inspection during design and prior to the construction allowed Transit to better identify features that would need to be secured, and to then target the rock bolting.

The project has been a significant consultation exercise for Transit, with many parties being potentially affected by the works. These included:

- Department of Conservation, as the site is inside a scenic reserve
- Fish and Game New Zealand, because the Buller River has a National Water Conservation Order on it
- New Zealand Historic Places Trust, because the site shows an example of pioneering road building techniques.

The project is now under construction.

SH 94, Homer Tunnel

About a year ago, a section of each end of the tunnel was widened to allow buses to pass in the tunnel. This was achieved using drill and blast techniques. The widened sections were left unlined, as is the rest of the tunnel. Some rock bolting was also employed to secure the unlined rock faces. This was not without difficulty, as the widened sections have a flat roof instead of an arch lacked support. Loose sections of rock in the roof not restrained tended to break away from the main rock body of the tunnel. Additional rock bolting, installed at night when the tunnel was closed to traffic, appears to have addressed the problem.

Albany to Puhoi Realignment Project (ALPURT) - Auckland

This is a large project, comprising 27 km of 3 to 4 lane motorway extension of the Auckland northern motorway. Several papers are to be presented on the ALPURT project at this symposium.

Concerning the geology, topology and cut slopes, the Albany to Redvale section geology is typical Auckland Waitemate Group, thicker than normal weathered soil profile, steep to undulating. Cut is up to 25 metres, all within weathered soils. For the Redvale to Silverdale section the geology is the slip-prone highly-fractured mudstone that causes trouble throughout Northland. Cut batters up to 15 metres are required.

Fill embankments up to 12 metres are planned. Special precautions are required to ensure proper settlement of the fills, and to eliminate post-commissioning settlements. Some foundations include reinforcement geotextiles and geogrids to enhance stability, without encroaching on to wetland or over the designation. These include settlement and piezometer monitoring. Special handling techniques are required for the earthworks.

Retaining walls include the use of proprietary soil-nailed walls with grass panel facing in two sensitive areas. Concrete panel and keystone block walls with steel ladder and geogrid soil reinforcing have been utilised for bridge abutments and culvert ramps.

Bridges incorporate bored friction/end bearing piles to bedrock, and reinforced earth and mechanically stabilised earth abutment pad footings have been used to found the bridges.

To date, investigations have produced some 120 boreholes, 120 machine-excavated pits, and associated field and laboratory tests. Helicopter access for the rigs was required in the steeper inaccessible country.

Conclusions

Technology will play a very significant role in the future management of our highways. The challenge for the geotechnical person is to recognise the opportunities available, and to move with these technological developments.

In the meantime, in order to deliver both cost effective projects and to satisfy resource consent conditions, we must be looking all the time to minimise costs without reducing quality. This is where new innovative solutions are needed. In particular, we must make considerably more use of lower graded material and recycling.

Techniques such as strengthening rock and earth structures are many, and so the designer must use his skills and knowledge to produce cost efficient solutions. Stabilisation techniques, including grouting and reinforced earth technology, have been - and will continue to be - a feature of further Transit projects.

This Roading Geotechnics Symposium provides an excellent forum to promote the knowledge and practice of geotechnical matters, which are essential to the success of civil engineering works. I wish you all an enjoyable and worthwhile symposium.

Disclaimer: Any views expressed in this paper are those of the author and do not necessarily reflect the views of either Transit New Zealand or the Government.

References

1 Powell, A. J., and Billings, I. J. *Thorndon Overbridge seismic assessments.* Proc 2nd Int Workshop on Seismic Design of Bridges, Queenstown 1994.

2 Murashev, A.K., and Palmer, S.J. *Geotechnical Issues associated with development of Wellington's Waterfront.* IPENZ Annual Conference Proceedings, 1998, vol 3 p45-50.

MECHANISTIC DESIGN OF FLEXIBLE PAVEMENTS

Peter Millar
Tonkin & Taylor Ltd

INTRODUCTION

This paper provides an overview of the current pavement design practices in New Zealand and identifies issues which are expected to arise as a result of major changes in the design procedures.

Transit NZ has adopted the AUSTROADS pavement design procedures as the basis for design in New Zealand. The AUSTROADS document "A Guide to the Structural Design of Road Pavements", (1992), including Transit New Zealand Supplement Notes (1997), now supersedes the previous New Zealand State Highway Pavement Design and Rehabilitation Manual (1989).

The introduction of the AUSTROADS preferred mechanistic pavement design procedures has fundamentally changed the method of design used in New Zealand. Whereas previously a limited number of design charts were used regardless of geology and ground conditions, the use of mechanistic design methods firmly places a high level of responsibility for performance on the pavement engineer. The pavement engineer is required to assess the applicable subgrade and pavement material engineering properties and apply these to the site specific operational conditions to develop the design. There is also a requirement in New Zealand for engineers to be conscious of the objectives of the Resource Management Act which encourages increased utilisation of marginal aggregates and recycled materials for which the performance characteristics and limitations need to be well understood.

These material assessment and mechanistic analysis methods are a natural extension to the skills of geotechnical engineers who are increasingly involved in pavement design projects. The associated risks, particularly with the advent of performance specified contracts, are also being increasingly directed to the geotechnical engineer in much the same way as structural engineers are subrogating foundation design risks. Geotechnical engineering is clearly not an industry for the faint hearted and it will be interesting to maintain a watching brief on successes and problems that may develop, particularly as there are as yet no clear precedents. Historically, the 85 percentile (or 1 Standard deviation above the mean), has been generally applied for acceptance standards on roading design. Local failures were expected and reluctantly accepted, such that litigation was infrequent and generally limited to clear cases of negligence.

PREVIOUS DESIGN APPROACHES

The principal methods of pavement analysis used historically have been the application of design charts (British Model) and equivalent thickness or structural number method (US Model). Both thick methods have been based on road tests undertaken by AASHTO in the 1950's, AASHO (1961).

NZ design methods have been traditionally strongly linked to the British model. Representative design charts have been developed from a series of elastic analyses for typical material stiffnesses and layer thicknesses using the Shell BISAR software Shell, (1978). Strains at the top of subgrade and near the base of the asphaltic concrete layer were assessed relative to adopted repetitive loading criteria. A comparison of some of the relationships that have been adopted for NZ subgrade strain criteria are shown in Figure 1. These include the early NRB S/4 specification, the TNZ State Highway Pavement Road Rehabilitation Manual (1989) (SHPDRM) criteria and are shown together with the less conservative current AUSTROADS criteria.

The high quality of base aggregate materials required by NZ specifications were considered to be adequate to preclude structural failure within these layers. Fatigue criteria were also applied to stabilised layers. Design charts are still available in AUSTROADS for checking non-critical pavement conditions.

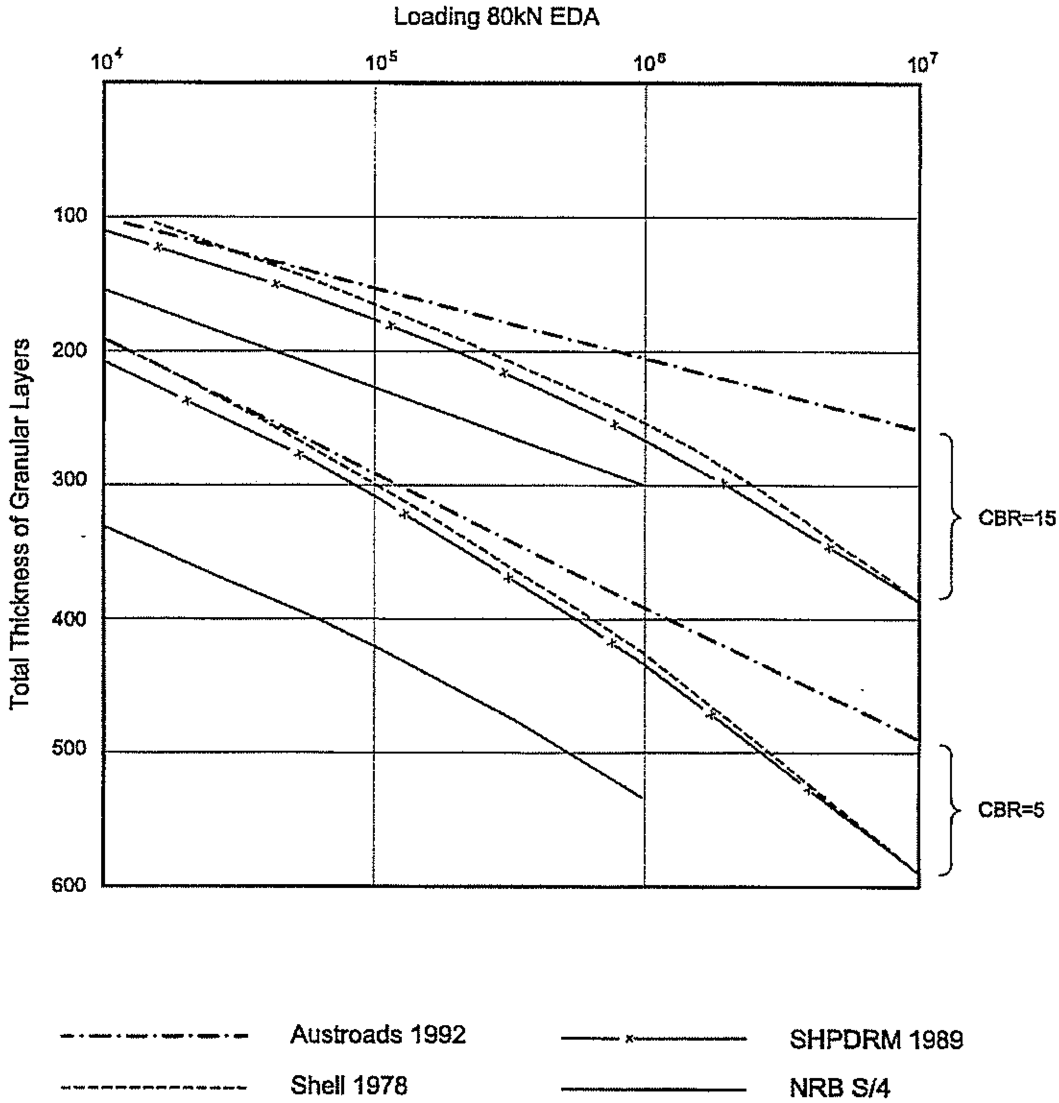


Figure 1.

COMPARISON OF DESIGN CURVES
FOR UNBOUND PAVEMENTS

The U.S. pavement design method AASHTO (1986) adopted the Structural Number equivalent pavement thickness concept in which the pavement performance is assessed as the product of a layer coefficient and thickness for each base and surfacing layer together with adjustments for serviceability, drainage, temperature and reliability coefficients.

The moduli are determined as a function of the third root of the moduli ratio of the pavement materials, similar to the use of Oedmarks theory for equivalent thickness of structural beams.

The relative merits of the British and U.S. based methods have been subject to extensive discussion in the technical literature. My experience with both methods in South East Asia and New Zealand is that the U.S. system is particularly useful for assessment of asphaltic concrete overlay thicknesses (using the concept of remaining life factors) while the British system is more simply applied to new pavements (and may in fact be non conservative for overlay design). It is somewhat puzzling therefore to appreciate why the US system has been so strongly supported in much of Asia where new pavements are the norm, while N.Z. has for so long relied on the British system, but new pavements are relatively rare.

AUSTROADS MECHANISTIC DESIGN METHOD

The AUSTROADS mechanistic design method recommends the use of structural analysis of pavements using CIRCLY or other validated multilayer elastic analysis programs. This enables the strains within structural layers to be evaluated using resilient moduli directly applicable for pavement materials and environmental conditions. More complex multilayer pavements including stabilised layers, geogrids and stress dependent material property characteristics can also be considered together with more accurate assessment of maximum strains within asphaltic concrete layers (as these generally occur above the base of the bitumen based layer. Sensitivity analysis may also be undertaken to assess the effects of variability of material properties which may be used in risk analyses, decision trees and whole of life assessments of optimised development of the pavement.

The major features of the AUSTROADS design procedure are:

(1) Pavement materials are homogenous and elastic with the subgrade and unbound base materials being considered as anisotropic

(2) Critical strains are assessed for

• asphaltic concrete	- horizontal tensile strain at the base of the layer
• unbound granular base	- structural life assumed to exceed life of other pavement materials
• cemented granular layer	- horizontal tensile strain at the bottom of the layer
• cemented subgrade soils	- horizontal tensile strain at the bottom of the layer, followed by period equivalent thickness of unbound aggregate
• subgrade	- vertical compressive strain at the top of the layer

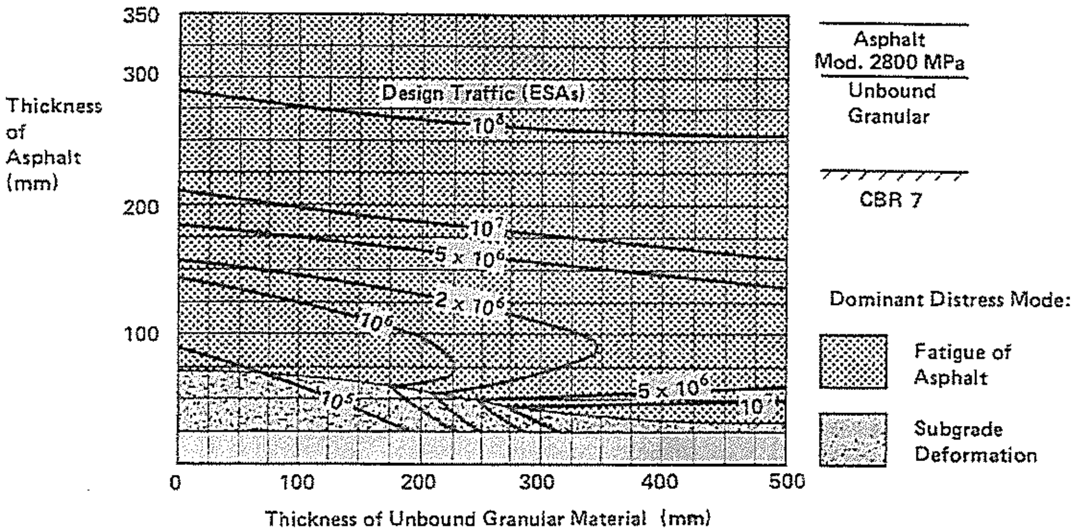
(3) Standard Axle (ESA) loading is a dual wheeled 8.2 tonne single axle represented by two equivalent area 92 mm radius loads separated 330 mm centre to centre and with a contact stress of 750 kPa.

In applying the mechanistic design method it is necessary to assess the allowable number of ESA for each distress mode (see (2) above). It is important to note that, for the design of asphaltic concrete (AC) pavement on stiff subgrades, fatigue failure of the A.C. may be highly sensitive to the depths of the unbound base and AC layers, as demonstrated in Figure 2. A sensitivity analysis should be undertaken where there are significant variations or uncertainties on material characteristics. The selection of appropriate material properties is discussed in a companion paper by Bartley (1998) and a proposed review of material requirements is set out in a paper by Arnold (1998).

The AUSTROADS design criteria have been supported for NZ materials by controlled testing at the CAPTIF Canterbury test track Pidwerbesky (1994) and Stevens (1993). Tonkin & Taylor Ltd experience is that the criteria are conservative for some volcanic soils which have high voids but have high cohesive strength eg Taranaki Brown Ash, Millar (1986a) and Salt (1998a).

However, the application of a single subgrade strain criteria is a simplification as it is clear that the cumulative effect of the level of strain on the resultant surface rutting will be highly dependent on the rafting effect of the depth of overlying materials. This does not appear to have been considered and, until the recent SHRP studies, the adopted criteria have all effectively been based on a series of tests undertaken by AASHO in the mid western U.S. Nevertheless, experience has proved that, for

Figure 2: EC5 - EXAMPLE DESIGN CHART 5 - from AUSTROADS (1992)



NOTE 1. Allowance to be made for construction tolerances
2. For explanation why more than one asphalt thickness is satisfactory refer to appendix F

moderately loaded pavements ($10^5 - 10^7$ ESA) the design criteria have generally provided satisfactory results. The AUSTROADS criteria has been modified to be less conservative at higher levels of vehicle loadings. However, under extreme loading conditions (often as a result of a large number of very heavy axle loads the criteria may be less reliable. In these conditions it is more appropriate to undertake designs utilising representative loadings rather than extrapolation using ESA (eg PAWL's as used for ports and container handling facilities).

OVERLAY DESIGN

Outline of Methods

Overlay design for pavement rehabilitation may be undertaken using a range of methods, the selection of which requires an appreciation of the pavement materials and the past performance of the road section. The methods which all rely on deflection testing and/or destructive tests are summarised in Table 1 and an example of their application is given in Figure 3.

Table 1: Overlay Design Methods for Pavement Rehabilitation

	Overlay Design Method	Basis
1.	AUSTROAD Simplified Deflection Method, Chapter 10	Surface deflection used irrespective of pavement construction.
2.	AUSTROADS General Mechanistic Proceedure (GMP)	Subgrade Strain analysis at each test point.
3.	TNZ Supplementary Equation 10.3 (SHPDRM)	Detailed analysis at each measuring point. Past performance of subgrade is used.
4.	TNZ Supplement Precedent Strain Uses equation 10.3 and 10.4	Statistical approach for variable ground conditions based on past performance of subgrade.

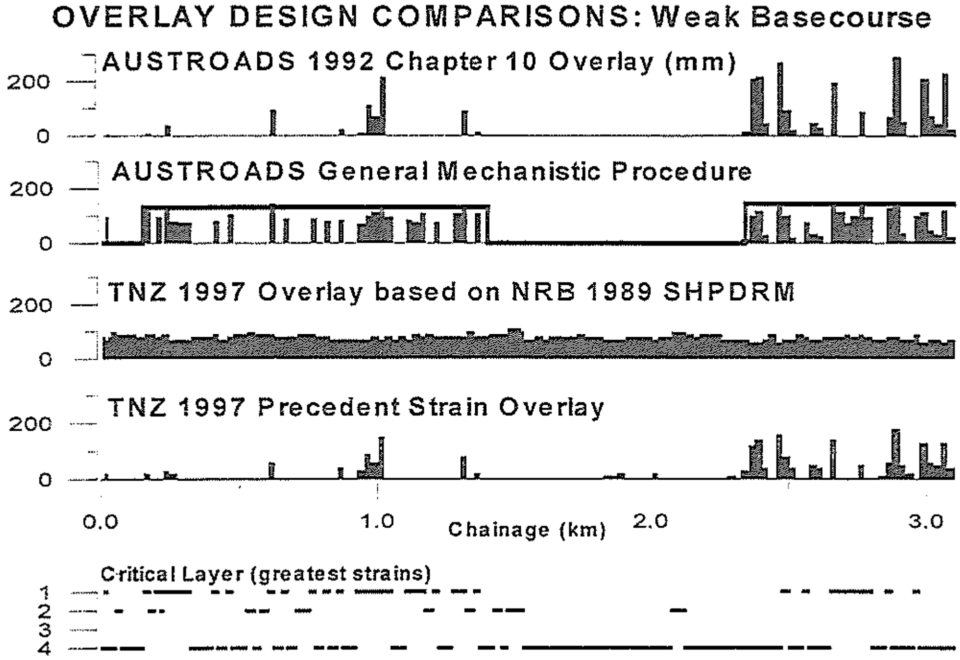


FIGURE 3 : OVERLAY DESIGN COMPARISONS - WEAK BASECOURSE

The AUSTROADS Simplified Deflection Method provides for assessment of structural overlays based on structural modelling or charts developed from pavement deflection measurements using the Benkelman Beam. The use of the deflection method is consistent with experience that the required overlay thickness is not particularly sensitive to the structure of the pavement. This is reflected in both the British overlay design charts published in TRRL 1132 and the successful application of the US overlay design method based on structural numbers, AASHTO(1992). The AUSTROADS design manual also includes a chart to assess the required thickness to reduce the curvative function to acceptable levels. However, this is not often the critical condition unless the pavement surface has deteriorated to the point where it has little residual structural integrity and consideration should be given to replacement. Experience by Tonkin & Taylor Ltd in the use of the deflection charts is that they should be used where the pavement material types and construction are well known and should be used with caution where hard subgrade or stabilised layers are present.

The AUSTROADS General Mechanistic Procedure and the NZ Supplement provide for overlay design by the more representative dynamic loading systems such as the Falling Weight Deflectometer (FWD). Back analysis of the deflection-based profile provides the best fit stress dependent resilient moduli of the pavement structural layers from which the critical strains can be determined under an equivalent structural axle (ESA) load or any other representative design loading. The critical strains may then be assessed relative to the appropriate AUSTROADS failure criteria to determine overlay thickness requirements rather than the design charts presented in the manual. The FWD test method provides information on the critical failure mechanism and is not dependent on good information on past history of loadings. However, experience is required in applying the result to some subgrade conditions such as brown ash, pumiceous soils and peat where the subgrade strain criteria may not accurately reflect the material performance. It is also essential that consideration be given to the effects of the timing of the testing as deflections are likely to increase during winter if subgrade softening occurs and may decrease under extreme conditions of full saturation or freezing.

The NZ Supplement includes two methods of design that are based on past performance. Rehabilitation treatment is assessed on the basis of the terminal compressive subgrade design strain (ε_{des}) which is a function of the ratio of historic (Np) to projected future traffic (Nf) multiplied by the existing vertical strain (ε_{evs}) using a standard wheel load (1ESA).

$$\epsilon des = \epsilon evs \left(\frac{N f}{N_p} \right)^{-0.23}$$

Where the quality of test information allows individual test points to be back analysed (such as for FWD tests), the required overlay may be assessed statistically using

$$\epsilon evs = \bar{x} - fs$$

where $\bar{x}$ = mean vertical strain for each section of similar soil type
s = standard deviation for each soil type
f = pavement condition factor (derived from normal distribution).

The TNZ methods are different from the SHPDRM (1989) in that they do not use the design charts of the earlier manual. When such charts for new pavement designs are applied to assess residual life, and the charts are based on the assumption of a continuously saturated CBR, then the performance of the base layers may be over estimated leading to an incorrect diagnosis of pavement distress and a potential underestimate of overlay requirements.

The application of the 4 methods is illustrated in Figure 3. They show some significant variations in results for overlay thickness. Experience by Tonkin & Taylor Ltd, (Salt 1998 (a) and (b)), has indicated that designers need to have a good appreciation of the limitations of the methods.

- The AUSTROADS Simplified Deflection Method is best suited to short sections of road where the pavement construction and history is well known. It should be avoided where a hard subgrade is present eg rock or stabilised soil and it may overestimate overlay thicknesses where volcanic soils are present.

- The General Mechanistic Procedure (GMP) may be used where little is known about the structural history but provides best results where the pavement structure is well defined. It is not dependent on knowledge of traffic history and developed software allows analysis of non-linear stress dependent properties and seasonal variations for whole of life assessment of long lengths of pavement. It may overestimate overlays for volcanic soils and should not be used where pavement subgrades are fully saturated or frozen.

- The TNZ (1997) methods are applicable where the traffic history and maintenance is well known and the basecourse quality is good. They should not be used where advanced degradation has occurred or shallow pavement shear is evident. They are particularly useful in volcanic soils or materials with unusual soil properties where precedent is a major factor in rehabilitation design.

The analysis of pavements using all methods as shown in Figure 3, highlights areas of difference that identify sections of pavement which may require further assessment. A combined approach is therefore considered most appropriate with the pavement engineer supplemented analysis with experience and judgement to determine the rehabilitation design.

RECENT DEVELOPMENTS

The application of the AUSTROADS General Mechanistic Design Procedure is subject to further modifications as experience is gained in its application both in NZ and Australia. Recently Moffat and Jameson (1998) addressed several aspects that are expected to be adopted. These are:

- full axle loading using two sets of dud tyres at 1.8 m spacing rather than one set. This is a relatively minor correction and generally only significant in deep asphaltic concrete pavements.

- Increased sublayering of unbound materials. Five layers are recommended by Moffat and Jameson 1998. The vertical modulis at the top of each sublayer is determined as the lower of those currently indicated in AUSTROADS (1992) or

  Ev top of base = Ev subgrade x 2 (total granular thickness/125)
  and the modular ratio of each sublayer (R) is assessed as
  R = (Ev top of base / Ev subgrade) $^{1/5}$

- Subgrade Strain review. The AUSTROADS (1992) subgrade strain criteria is based on back analyses of design Australian charts. A review of the analyses using a full dual wheel axle and the above sublayering procedure for granular materials has modified the subgrade strain criteria to:

  $$N = (9300/\mu\epsilon)^7$$

This results in very little difference for thick pavements but there is some divergence for the pavements, implying the earlier criteria was slightly conservative.

In his paper at this Symposium, Arnold (1998) will also outline proposed changes in TNZ specifications which will provide scope for the use of pavement materials previously excluded.

CONCLUSIONS

The use of mechanistic design procedures for pavements requires that designers have a good understanding of the design process, material properties and the structural behaviour of the pavement. This provides opportunities for engineers to optimise designs for specific loading conditions and to utilise materials which may have previously been considered marginal. It also permits engineers to test the sensitivity of design alternatives and to quantify risk due to variations in site conditions.

Rehabilitation design of pavements based on back analysis of structural performance allows definition of the critical failure mode. Design methods based on deflections and/or traffic history allow whole of life analysis of rehabilitation options to be carried out. There are limitations to each of the 4 methods of overlay analysis and pavement engineers need to ensure that the most applicable methods are used.

The mechanistic design procedures will continue to be modified as a result of increasing application and experience but I consider the adoption of the method is a very positive development which will provide greater understanding of pavement performance and best utilisation of our limited aggregate resources.

APPENDIX A: DEFLECTION TESTING

Deflection testing forms the basis of the AUSTROADS overlay design method. A brief outline of the use of deflection based methods is provided below.

Deflection testing of pavements has traditionally been undertaken using the Benkelman Beam. The original pivot joint was updated by the TRRL design which was adopted in NZ in the 1960's and included a lever arm with a sealed roller bearing joint. A series of NBR funded research projects in the early 1980's provided poor results in respect to repeatibility and the deflection methods were considered unreliable. Testing at Central Laboratories MWD (Millar, 1987) determined that the beams manufactured to the TRRL design had mechanical deficiencies that affected the accuracy and a modification was developed to improve the design.

Tonkin & Taylor Ltd developed an electronic deflection beam in 1989 that has no moving parts and has greatly improved accuracy. This instrument, which is shown in Figure A1 and is known as the Geobeam, has been widely used throughout NZ and S.E. Asia. It records 12 measurements of deflection as the vehicle moves away from the measuring point providing definition of the recovery bowl that allows overlay analysis using either the AUSTROADS simplified method or by best fit structural modelling (GMP).

It is worth noting here that the AUSTROADS curvature design charts do not specify whether the deflections used are corrected values but there are no published methods for correcting the D200 measurements and it is our experience that uncorrected values are used in these charts.

The Falling Weight Deflectometer was developed in the 1970's and has become a widely used structural condition monitoring test method. The equipment is shown in Figure A2 and provides a dynamic impulse load which simulates the passage of a loaded axle at realistic traffic speeds rather than the pseudo-static loading of a Benkleman Beam. It also measures the compression of the layers rather than the recovery of the surface which may include plastic components of deformation at the relatively slow speed of movement. The deflections profile is generally measured using a series of geophones set at appropriate offset from the loading point. Back analysis of the best fit layer stiffnesses is undertaken to develop the structural model from which the induced strains can be assessed relative to AUSTROAD criteria to determine the residual life. Overlay thicknesses required to extend the pavement life for projected traffic levels may also be determined as shown in Figure 3.

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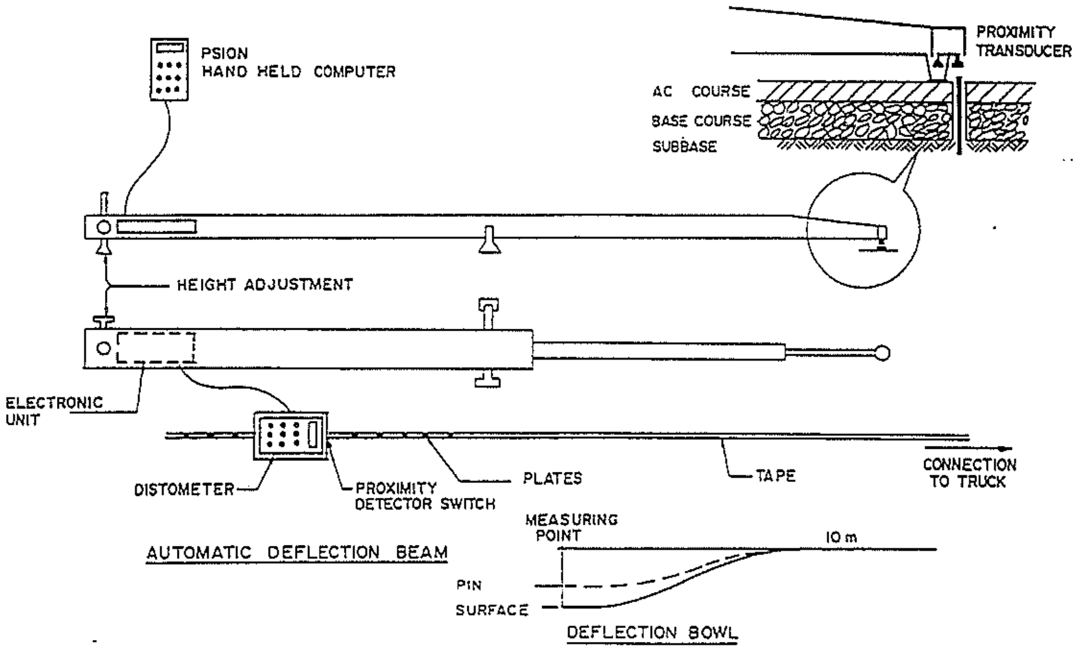

Figure A1: Instrumented Benkelman Beam Assembly

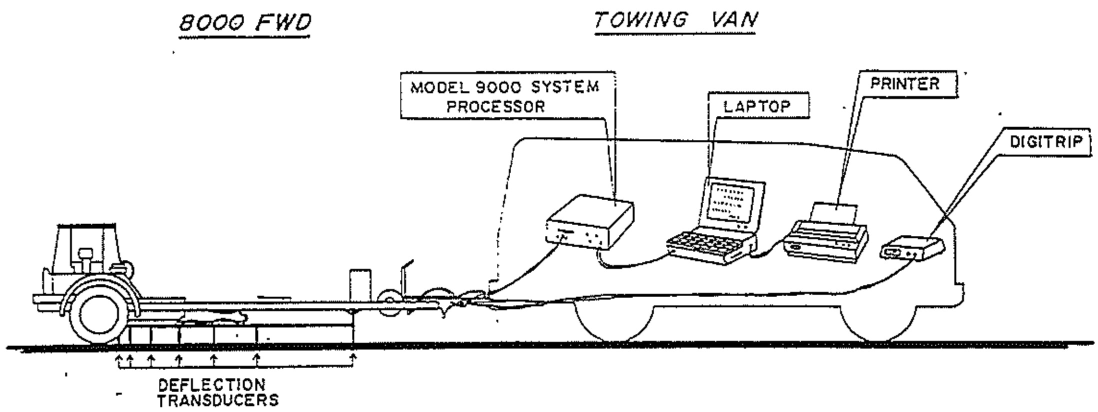

Figure A2: Falling Weight Deflecomter Assembly

APPLICATIONS OF MECHANISTIC PAVEMENT DESIGN IN NEW ZEALAND (II)

Case Histories Comparing Design Prediction with Post Construction Measurement and Analysis

Graham Salt, Tonkin & Taylor
William Gray, Opus International Consultants, Napier

SUMMARY

The AUSTROADS Pavement Design Guide has been used for the mechanistic design of pavement rehabilitation in New Zealand for several years. A number of projects have now been completed where pavement testing has been carried out before and after construction, giving the opportunity to compare design prediction with in situ performance achieved by the rehabilitation. Case histories involving different forms of construction in various parts of New Zealand are presented, quantifying the improvement in performance resulting from the use of AUSTROADS procedures. Data is also presented for typical in situ material properties of unbound granular overlays and cement stabilised basecourses.

1.0 INTRODUCTION

Rehabilitation design for unbound granular pavements in New Zealand is now based on mechanistic procedures given in the Pavement Design Guide (AUSTROADS, 1992) and the Transit New Zealand Supplement to the AUSTROADS Guide (TNZ, 1997).

Mechanistic analysis allows pavement rehabilitation design to be carried out from first principles, calculating stresses and strains in the existing pavement. The advantage to the designer is that he will obtain an understanding of the mechanics of the behaviour of each specific section of pavement, increasing the opportunity for innovative design. With insight into the relevant distress mechanism(s), selection of the most appropriate rehabilitation measure can then be made with assurance that the correct problem is being addressed and that the solution will be cost effective and provide long service.

2.0 REHABILITATION DESIGN METHODS

Deflection testing and back analysis (using layered elastic theory) of the deflection bowl induced by a standard wheel load provide the basic parameters for mechanistic design.

For unbound granular pavements the principal design criterion is limitation of the vertical strain at the top of the subgrade. Where a bound layer is present, an additional criterion (horizontal tensile strain at the base of the layer) is also applied. Trial pavements are modelled using alternative rehabilitation treatments to determine the most effective solution.

Five rehabilitation options used in New Zealand for unbound granular pavements with chip seal surfacing are: unbound granular overlay, friction course overlay, cement stabilisation of the existing basecourse, "upside-down" reconstruction and full depth granular reconstruction. Examples of each are given below.

3.0 CASE HISTORIES

3.1 General

Each case study is presented showing the following graphs:

(a) Unbound granular overlay requirements
(b) Cement stabilisation depth required
(c) Resilient modulus of the upper layer (usually basecourse)
(d) Resilient modulus of the subgrade
(e) Subgrade strain ratio

The unbound granular overlay has been computed using the General Mechanistic Procedure and the AUSTROADS subgrade strain criterion (AUSTROADS, 1992; TNZ, 1997) for unbound pavements, ie:

$$\epsilon_{des} = 0.008511(N_F)^{-0.14} \qquad (1)$$

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

Where a cement stabilised layer is present, the tensile strain at the base of the stabilised layer has also been checked, using the criterion given in TNZ (1997).

The depth of cement stabilisation required has been computed assuming the pavement will be rehabilitated using the same tensile strain criterion and qualifications given by ARRB (1996). ARRB recommends assuming a design modulus of 5000 MPa in the cement bound layer, and that the effective traffic loading (ESA) for cement stabilised material be taken as 10 times the actual ESA.

The residual modulus of the upper layer and the subgrade are both shown on the graphs as isotropic values. AUSTROADS (1992) recommends that cement stabilised basecourse should be modelled as a material with an isotropic modulus, whereas unbound layers should be modelled as anisotropic materials with the vertical modulus equal to twice the horizontal modulus. The AUSTROADS anisotropic vertical modulus for granular basecourse is found by dividing the isotropic modulus by 0.75. The AUSTROADS anisotropic modulus for the subgrade is found by dividing the isotropic modulus by 0.67 (using equations presented by Ullitdz, 1987). Moduli values quoted in the text, are in terms of the AUSTROADS convention.

The subgrade strain ratio is the maximum vertical compressive strain at the top of the subgrade when the pavement is loaded by 1 ESA, divided by the allowable strain (for the design future traffic) using the AUSTROADS subgrade strain criterion). This parameter is used to normalise the strains to give a more direct perception of the degree to which a subgrade is being overworked. A ratio of no more than 1 indicates that no strengthening overlay is required to provide the required design life. Ratios greater than 1 indicate that greater structural capacity is required, through overlay, stabilisation etc. If the subgrade strain ratio is much less than 1 in a newly overlaid pavement then some degree of overdesign may be indicated in an ubound pavement. The lives of pavements with bound layers are usually governed by horizontal tensile strains at the base of the bound layer.

3.2 Case 1: Unbound granular M/4 overlay.

Fig. 1 shows an unbound granular pavement on a flat alluvial plain. The subgrade is soft clay with high watertable. The first half of the road has been in service for many years and shows substantial rutting and loss of shape. The second half of the road was recently rehabilitated using some pre-overlay repair (digouts) then unbound granular overlay from design by TNZ (1989), State Highway Pavement Design and Rehabilitation Manual. Because the original pavement profiles for both halves of the road were similar, this example provides a representation of before and after rehabilitation conditions.

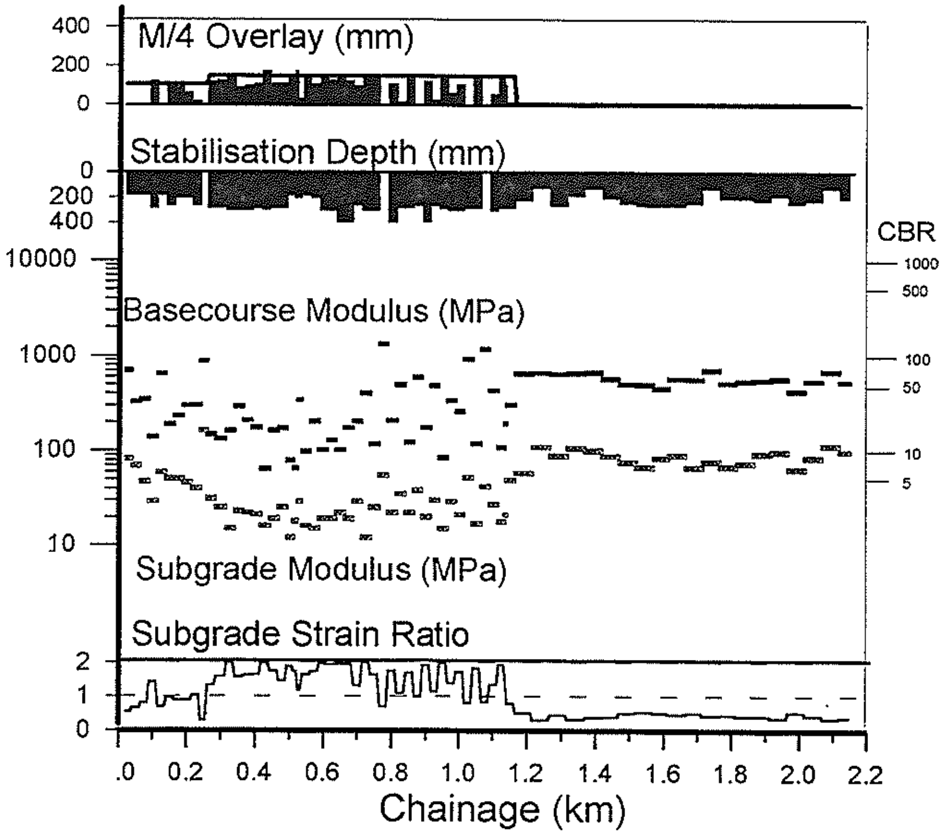

Figure 1 Unbound granular pavement on soft clay. Old basecourse to Ch 1.2, then overlaid with M/4 to Ch 2.2.

Looking first at the lowermost graph (subgrade strain ratio), analysis shows that the subgrade strain in the old pavement is up to twice the AUSTROADS allowable value. The rehabilitated section shows strains of less than half the allowable, ie there appears to be some element of overdesign. The next graph shows that the design subgrade modulus for the soft clay is about is about 20 MPa (ie CBR about 2). After rehabilitation, the subgrade modulus as increased considerably due both to surcharge consolidation effects of the overlay and non-linearity of the subgrade modulus (Ullidtz, 1987). If non-linearity effects are not taken into account, substantial over-design can result. The old basecourse has very poor uniformity and low modulus. The good quality well compacted M/4 overlay has achieved a minimum (5 percentile) vertical modulus of 550 MPa (assuming vertical to horizontal anisotropy of 2:1), ie at the top end of the range suggested by AUSTROADS (1992). The two upper graphs show that cement stabilisation depths would be excessive, and that a substantial overlay is required on the old section.

3.3 Case 2: Unbound Granular Overlay on Volcanic Ash Subgrade.

Rehabilitation of a pavement formed on a central Waikato ash subgrade is shown in Figures 2 and 3. Testing of the original pavement showed very high strains in the subgrade. The subgrade strain ratio (under the future traffic loading) was 2 or more for much of the section, but local experience has shown that the local volcanic ash can usually tolerate much greater strain than typical soils derived from sediments or weathering products. Analysis of past performance was carried out, using both comparison of precedent strain ratios and also the procedure given in the Transit NZ Supplement to the AUSTROADS Pavement Design Guide. The precedent subgrade strain ratio (ie based on past rather than future traffic) was found to be in the range of 1.5 to 2.

A similar result has been obtained for a number of pumice and ash sites in the North Island, ie based on past performance, these soils perform substantially better than would be expected using the AUSTROADS subgrade strain criterion.

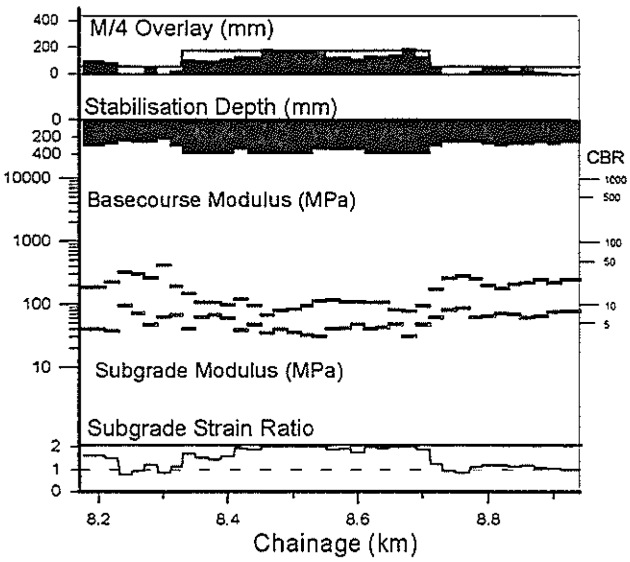

Figure 2 Unbound granular pavement on ash subgrade (before overlay).

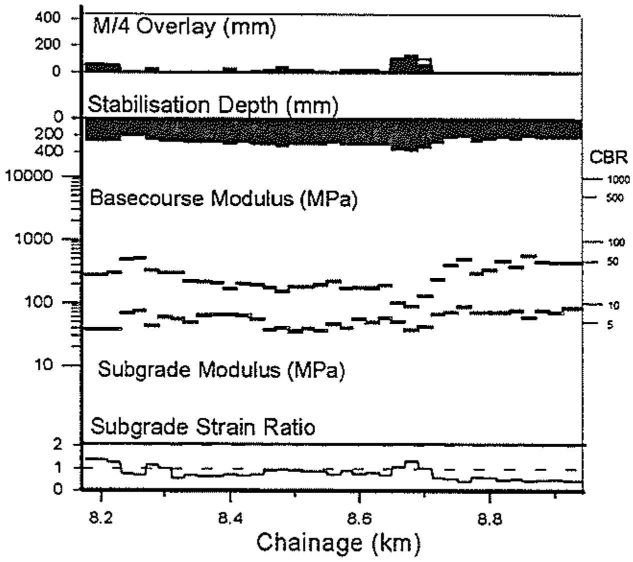

Figure 3 Ash subgrade after M/4 overlay.

The rehabilitation design was based on a value engineering decision including:

1. Past performance of the pavement
2. Providing a new primary basecourse layer (due to marginal strength within the existing pavement)
3. Providing sufficient total granular pavement depth for the future traffic loading

A "reseal and do nothing" trial was undertaken over the first 200m of the project length. The trial involved laying a section of polymer seal and a section of polymer seal underlain with a geotextile. Over the next section from Ch 8.32 to 8.72 a 150 mm M/4 overlay was adopted, reducing to 100 mm overlay for the remainder. The reseal was undertaken in April 1998 and the overlay in November and December 1997. To date the works are performing well, although it is too early to evaluate the appropriateness of the design considering that it has only carried 1% of its design load.

Repeat testing was carried out several months after completion of overlays. The pavement layers were difficult to model individually without constraining the subbase layer moduli, therefore the full depth of new basecourse and subbase layers were modelled as a single thick layer. Accordingly the "basecourse modulus" plotted is an average of all the pavement layer moduli. It is interesting to note the way that the basecourse moduli follow the subgrade moduli, reinforcing the characteristic of unbound layers that their moduli are generally limited by the support provided by the underlying soil layer. After overlay (Fig. 3), there appears to be a localised section, near Chainage 8.7 where the basecourse modulus is unusually low. It has been influenced in part by a low subgrade modulus but this is likely to be only part of the problem. The constructed overlay appears to be relatively thin or perhaps under compacted. Post -constuction testing in this manner provides a useful check on workmanship. Nevertheless, the subgrade strain ratios can be realistically compared before and after rehabilitation. In the parts that were overlain, the subgrade strains have successfully been reduced to close to the AUSTROADS allowable values, and on the basis of past performance, the life of the section (in terms of rutting induced by subgrade strains) should exceed the design requirement.

3.4 Case 3: Friction Course Overlay

Figs 4 and 5 show a heavily trafficked granular basecourse in a 50 kph zone, before and after friction course overlay. Testing of the original pavement shows that the subgrade strains are only marginally larger than allowable and about 100 mm of unbound granular overlay would be required. As there was only minor loss of shape, 30 mm friction course surfacing was adopted. (A bituminous overlay is structurally equivalent to an unbound granular layer which is approximately 3 times thicker.) The post construction testing shows only minor reduction in subgrade strains (as expected) but most of the road should provide the intended design life.

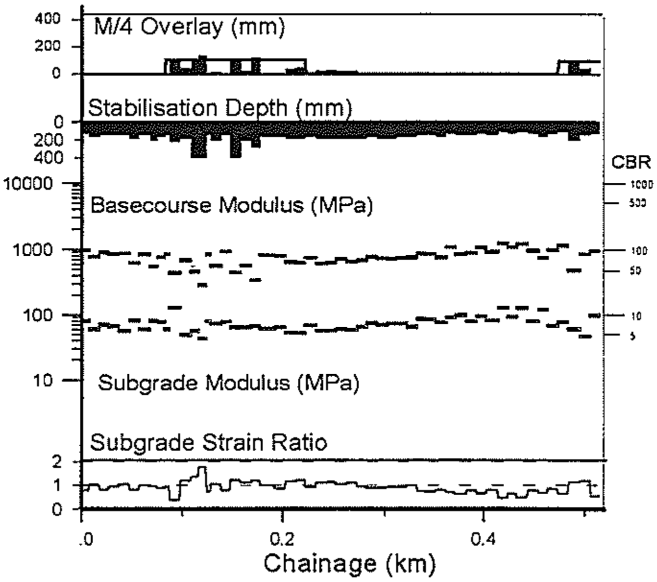

Figure 4 Unbound pavement before rehab.

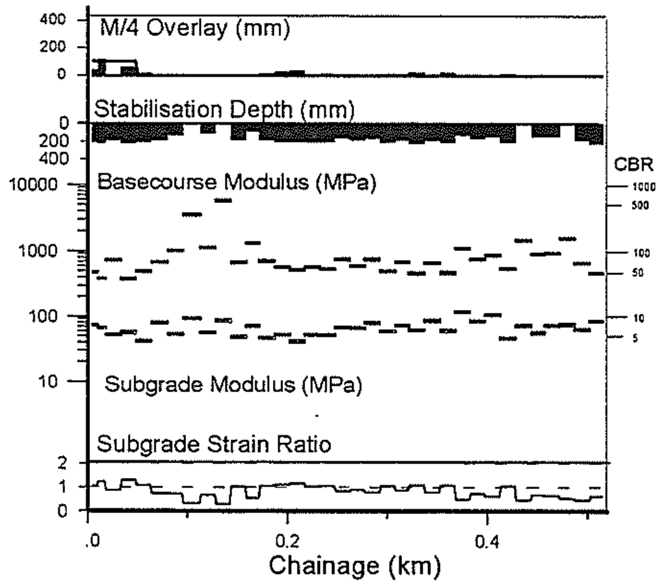

Figure 5 After friction course surfacing.

3.5 Case 4: Cement Modified Pavement Recycling

Figures 6 and 7 show the pre and post construction testing of a site in northern Hawkes Bay. Here the existing pavement was recycled by milling cement into the existing top surface layers. In addition, top up M/4 material was included beyond Chainage 4 where the original subgrade strains were high (often twice the AUSTROADS allowable values. Both the subgrade and basecourse in the original pavement had highly variable moduli.

The design concept for the pavement recycling projects, of which this site is one example, is to provide an alternative to full depth rehabilitation. Recycling aims to provide up to ten years of service, after which an unbound granular overlay or other rehabilitation treatment would be expected. The recycling process was targeted at sites where shallow shear was the principal cause of distress.

The process involves milling and relaying the cement-modified material to a depth of 200 mm. Because the subgrade was less protected at the far end of the site, this section was first overlaid with 100 mm of M/4 aggregate. In this case the milling/relaying process then incorporated this overlay, and the existing seal and top surface basecourse into a modified top surface layer. At the other end of the site the existing top surface layers only were milled and relaid. The full section was then chip-sealed.

The post construction testing has shown that subgrade strains have become much more uniform over the site and are for the most part much lower than those allowed by the AUSTROADS strain criterion. Isolated test points show that tensile strains at the base of the bound layer are above allowable values. We expect this modified layer to crack. However, cement contents and the construction process are intended to produce micro-cracks, rather than block cracking. The modified pavement surface has a more stable bitumen/binder ratio (as compared to the original very unstable top surface layer) with the addition of cement binder. This layer appears to provide a better distribution of the wheel loads over the subgrade and lower pavement materials.

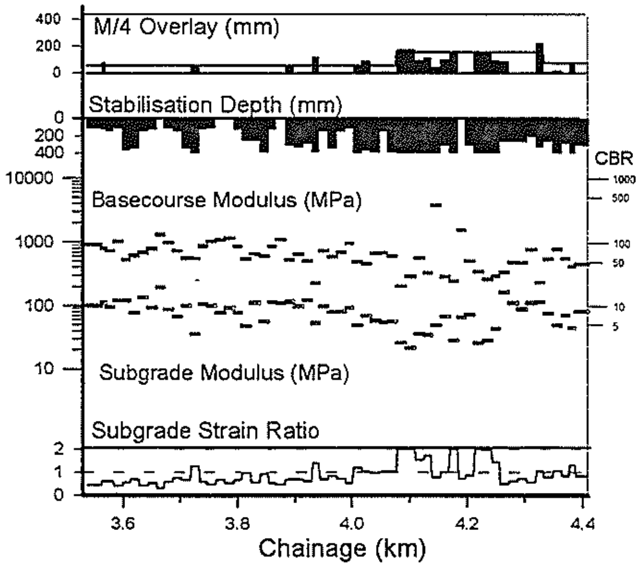

Figure 6 Unbound pavement before rehab.

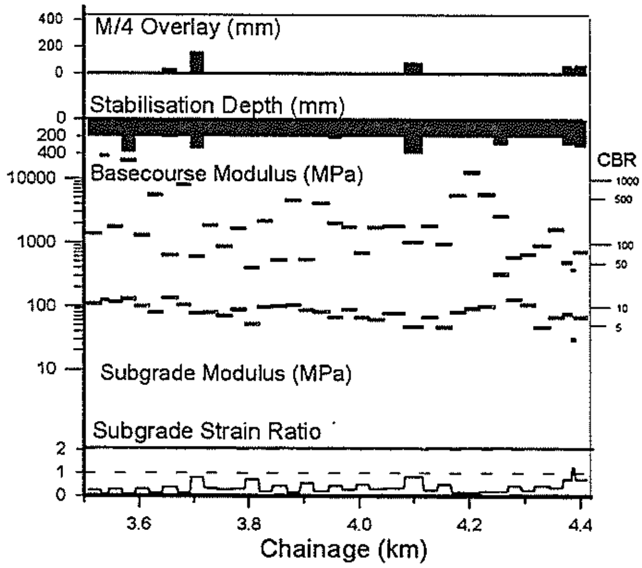

Figure 7 After cement stabilisation.

Although the recycling projects have only been undertaken in the last three years, the results so far are encouraging. The development of any block cracking is a key issue that will be monitored.

3.6 Case 5: Upside Down Pavement Reconstruction

Figures 8 and 9 show analyses of deflection bowls from before and after construction of an "upside down" pavement (unbound M/4 basecourse over cement stabilised subbase consisting of reused aggregate and subgrade) overlaying a soft silt subgrade. The original pavement had high subgrade strains, and was exhibiting shallow shear (from poor basecourse materials) and deep-seated rutting. The upside down pavement concept aimed to reuse as much of the existing basecourse as possible. Stabilisation was adopted to construct a compacted layer from the existing thin basecourse (poorly graded alluvial gravel) and the silt subgrade, providing a sound subbase. The stabilised subbase was then overlaid with M/4 aggregate. The unbound granular overlay provides a stable surface that can be shaped and sealed. It also helps prevent any cracking in the subbase from extending through to the surface.

The subgrade strains are very low, mostly less than half of the AUSTROADS allowable values. Limiting the horizontal tensile strains in the bound layer is a key factor in the design. Over the last 10 years two similar pavements have shown some signs of distress. The failure mechanism is localised block cracking and shear failure. In both cases the as constructed pavement depth was found to be less than the specified depth. However these cases suggest that more conservative design may be required for construction tolerances.

The majority of pavements built this way (many kilometres) are performing well. They can be adapted to both rural and urban situations.

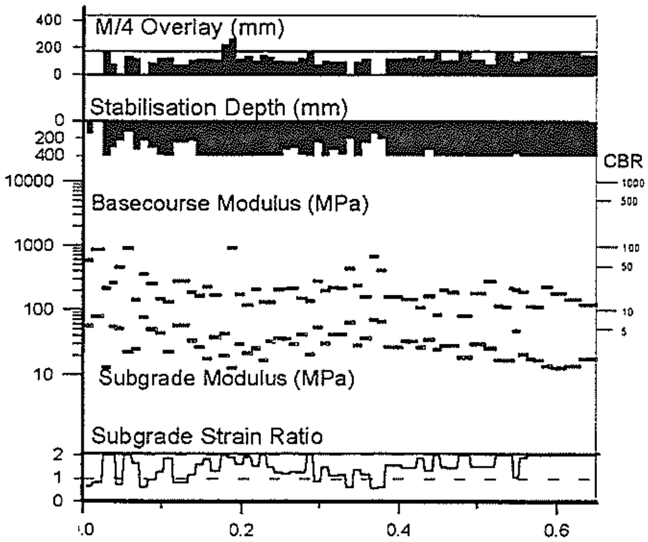

Figure 8 Unbound pavement before rehab.

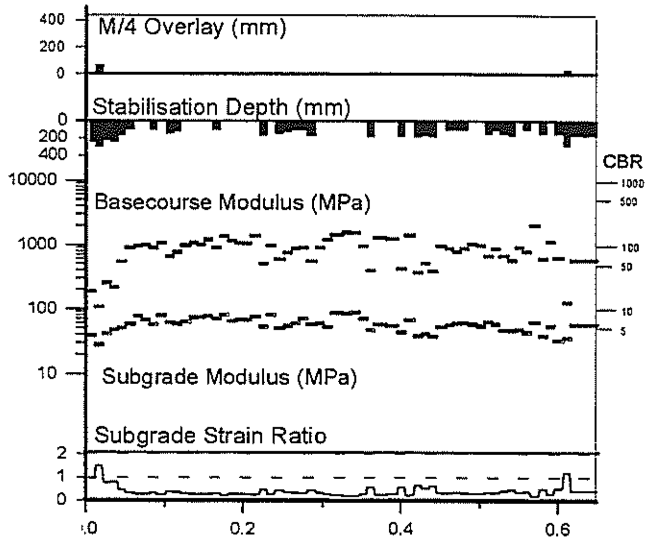

Figure 9 After "upside down" reconstruction.

3.7 Case 6: Full Depth Granular Reconstruction with M/5 Basecourse

Fig. 10 shows an example where only the post-reconstruction case is available. Testing was carried out immediately after completion, so minimal "shakedown" or compaction of the surface layers through trafficking would have occurred. The subgrade was lime stabilised, and an M/5 basecourse (alluvial gravel with less than 70% broken faces) was used.

The firm foundation resulted in a relatively stiff pavement structure and a design (5 percentile) vertical modulus of 490 MPa was achieved in the basecourse. (Greater values would be expected after compaction trafficking). This value is consistent with AUSTROADS (1992) recommendations. Subgrade strains are very uniform and close to allowable AUSTROADS requirements, indicating efficient design.

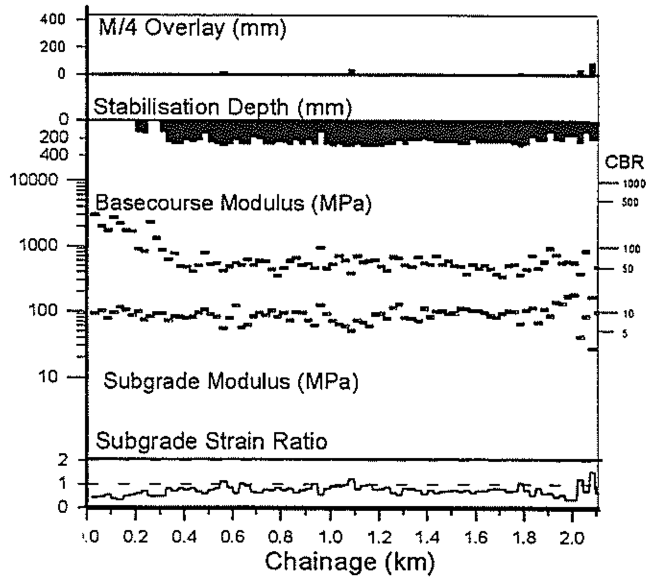

Figure 10 New M/5 Reconstruction

CONCLUSIONS

Rehabilitation treatments using AUSTROADS mechanistic design procedures are generally achieving the intended reductions in subgrade strains. Provided that the accepted strain criteria are appropriate, long term performance should be assured. Ongoing monitoring of these cases is intended.

Post construction testing provides a guide to the effectiveness of conventional or non-conventional rehabilitation treatments, quantifying the degree of over or under-design.

North Island volcanic ash subgrades appear to provide an exception to the standard AUSTROADS strain criterion. Rehabilitation treatments are being proposed which will allow subgrade strains up to twice the recommended values in these soils. The Transit NZ Supplement to the AUSTROADS Design Guide provides a precedent based method that should provide assurance of long term performance, but further case histories need to be followed up with post-construction verification and long term monitoring.

Limited in situ testing of unbound basecourses (M/4 or M/5) suggests that the typical moduli recommended by AUSTROADS (1992) are quite appropriate design values for New Zealand conditions.

For cement stâbilised pavements, case histories demonstrate that in practice, moduli of the stabilised materials are often highly variable, ranging from under 1000MPa to over 20,000MPa within one construction length. However, subgrade vertical strains can be reduced to very low values (ie associated rutting distress should be minimal). This is due to the good loadspreading ability of the cemented layer provided there is an adequate overall pavement depth. Appropriate strategies for addressing the effects of tensile fatigue, ie cracking, are hence most significant and the staged rehabilitation approach - such as planning for a later stage of unbound granular overlay can be most cost effective.

With ongoing projects for which the Cement Treated Basecourse concept is being considered, it is intended to target lower cement contents, and lower unconfined strengths to help achieve controlled micro-cracking rather than block cracking. Even with micro cracking a stable layer with good load spreading ability can be achieved.

ACKNOWLEDGEMENTS

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ENGINEERED PAVEMENTS

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SUMMARY

In July 1995 Transit New Zealand adopted the AUSTROADS Pavement Design Guide. To design a pavement or rehabilitation treatment a computer program like CIRCLY is used. The pavement is modelled as multi-layered linear elastic materials and stresses and strains at critical layers caused by the application of a standard wheel load are computed. Pavement Engineer's now have the ability to design a pavement out of locally available materials (stabilised or otherwise) if their linear elastic properties are known. However, Transit New Zealand still restrict pavement materials to those that meet the requirements of the recipe based specifications TNZ M/4 and M/3. To allow the use of alternative materials, a project has been initiated to develop a specification (*Performance Requirements for Unbound Pavement Materials*) that specifies minimum requirements of a basecourse and sub-base in terms of its shear strength; load spreading ability and durability. This will allow pavements to be "engineered" out of any materials that meet the strength and durability requirements.

Disclaimer

The views expressed in this paper are entirely personal and should not be relied upon as representing those of Transit New Zealand. Matters discussed in this paper are still in developmental stage and are subject to change.

1. INTRODUCTION

Pavement design has been relatively simple for the Pavement Engineer. The pavement thickness is determined by a chart and the pavement materials used are required to comply with the appropriate road controlling authority specifications. In comparison with our cousins in the geotechnical engineering field there is little "engineering" involved for the design of pavements. However, behind the development of the thickness design charts and pavement material specifications a significant amount of engineering, experience, and research has been undertaken.

Recently, user friendly computer based pavement design programs are being used for design of pavements using mechanistic analysis. These programs analyse the reaction of a wheel load on the pavement to compute stresses and strains within the pavement and subgrade. This has resulted in Pavement Engineers gaining a better understanding in the mechanics of the pavement and the materials used. However, the design is still based on determining the cover of granular material required to protect the subgrade and the choice of pavement materials is limited to those complying with the material specifications.

The current design methods (i.e. thickness design chart or mechanistic design) provide adequate pavements that will last the required design life. Difficulties occur when the availability of high quality granular materials that comply with the specifications are in scarce supply. Pavement Engineer should source alternative locally available materials that could be used in the pavement either with or without a stabilising agent at a reduced cost.

This paper presents how proposed performance specifications for materials and construction will allow pavements to be "engineered" out of any materials for a specific purpose (e.g. car park or motorway).

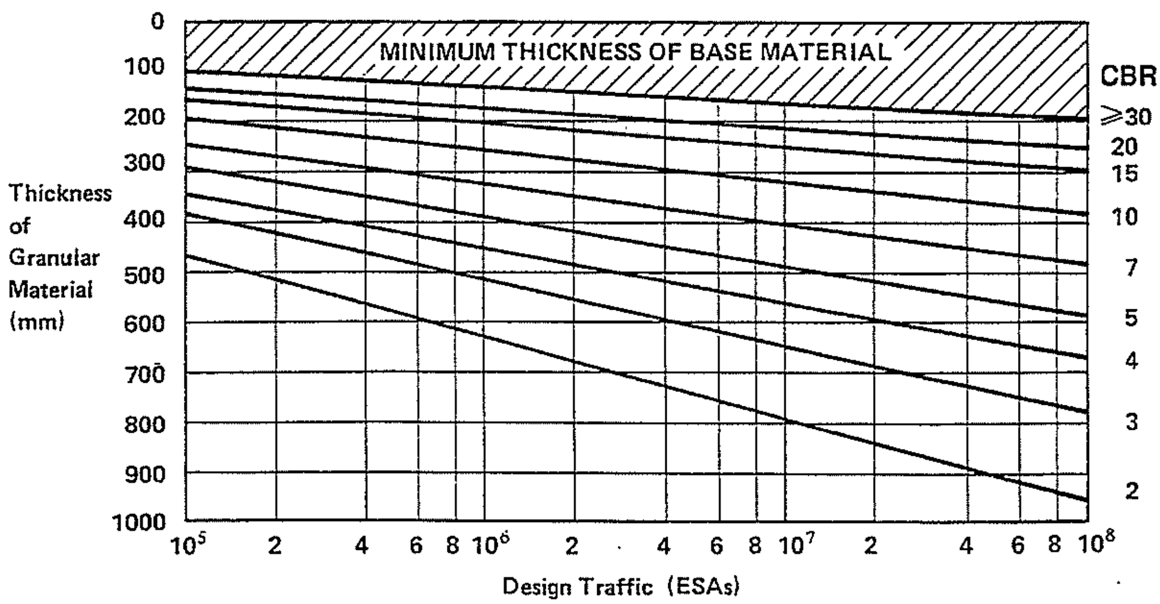
2. PAVEMENT THICKNESS DESIGN FOR THIN SURFACED UNBOUND PAVEMENTS

2.1 Background

The pavement thickness design for flexible pavements is traditionally determined from already established design charts. These design charts require knowledge of design traffic and the subgrade California Bearing Ratio (CBR) to determine the required thickness of granular cover.

The pavement thickness design charts used before the adoption of Austroads reside in the State Highway Pavement Design and Rehabilitation Manual (SHPDRM) (Transit, 1989). These charts in the SHPDRM were developed using mechanistic analysis based on the design methodology detailed in the Shell Pavement Design Manual (Shell, 1978).

Figure 1 - Austroads Thickness Design Chart for Thin Surfaced Flexible Pavements (Figure 8.4, Austroads Guide)



In July 1995 the Transit 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 Guide). This Guide superseded the existing Transit State Highway Pavement Design and Rehabilitation Manual (SHPDRM). A New Zealand supplement to the AUSTROADS Pavement Design Guide (NZ Supp.) was produced in November 1995 and since revised in July 1997 to address issues which are unique to New Zealand.

Since the adoption of the AUSTROADS Guide a different but similar pavement thickness design chart to those in the SHPDRM is used (Figure1). The development of this chart is empirical and can be traced back to the Californian State Highways Department CBR method of pavement design (Porter 1942).

The AUSTROAD Guide also details a mechanistic analysis procedure for determining the pavement layer thicknesses.

2.2 Mechanistic Analysis

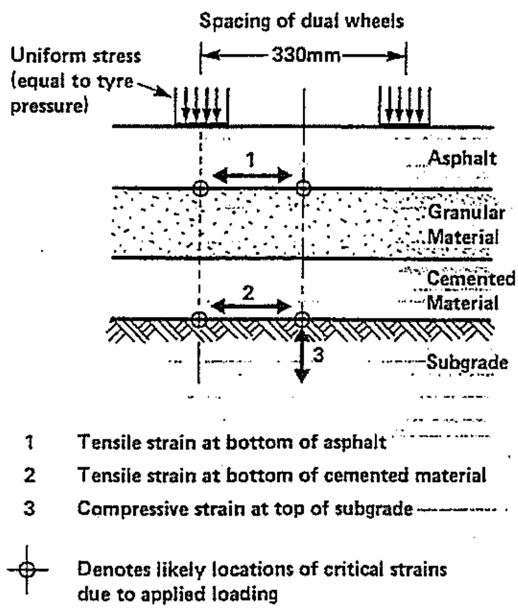
In mechanistic analysis, the pavement is modelled as multiple layers of linear elastic materials (Figure 2) subjected to an equivalent standard axle (ESA) load in a computer program like CIRCLY (Wardle, 1980). The vertical compressive strain is computed at the top of the subgrade which is then used to determine the maximum allowable ESAs (Equivalent Standard Axles) using the Austroads Subgrade Strain Criterion (Equation 1). The pavement thickness is increased until the maximum allowable ESAs is greater than the design ESAs.

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

where:
const = 0.008411
exp = 7.14
strain = vertical compressive strain at the top of the subgrade

A similar method is used to determine the thickness of any bound layers. The tensile strain is computed at the bottom of the bound layer which is then used to determine the maximum allowable ESAs (Equivalent Standard Axles) using a Equation of the same form as Equation 1 with different values for the constant and exponent.

Figure 2 - Pavement Modelled as Multiple Layers of Linear Elastic Materials for Mechanistic Analysis.



The advent of computer programs like CIRCLY have provided the Pavement Engineer with a powerful tool. Various pavement types for which a design chart had not been produced can now be assessed. Stresses within the pavement can also be computed to aid in material selection and design.

3. DESIGN and SELECTION of UNBOUND PAVEMENT MATERIALS

3.1 Requirements of Unbound Pavement Materials

Pavement materials are required to:

- spread the wheel loads to reduce the load on the soft underlying subgrade;
- not fail in shear with the application of wheel loads;
- have minimal deformation, where most of the deformation occurs in the subgrade;
- not deteriorate structurally over the design life;
- adequately hold and support the surfacing; and
- not be detrimental to the performance of the surfacing (e.g. cracking).

The requirement to adequately spread the load over the subgrade is currently ensured by providing adequate pavement thickness as determined using the thickness design procedures discussed previously. All the other requirements listed are satisfied by using aggregates that comply with an appropriate specification.

If material specifications were non-existent geotechnical and pavement engineering expertise will be required to select a material that could meet the above requirements.

3.2 Current Method

The current method to design (or select) an appropriate unbound granular basecourse aggregate is to simply specify compliance with the specification for basecourse aggregate TNZ M/4 (1995). This specification is a recipe for quarries to make a basecourse that has proven over time to meet the requirements detailed in Section 3.1 above. Effectively, the Pavement Engineer does not design unbound granular basecourses.

Mechanistic analysis can be used to determine the required thickness to protect the subgrade using any type of pavement materials. However, pavement materials not complying with an appropriate specification could fail by shear, rapidly degrade, cause pre-mature rutting, and/or cause surfacing failure.

There is a need, particularly in the Auckland region to use an alternative material in the road base to prime quality aggregate that complies with TNZ M/4. The resource of prime quality TNZ M/4 is rapidly diminishing as it is being continually specified for everything from under footpaths, car parks, cul-de-sacs and roads.

3.3 Australian Method

In Australia the basecourse material is required to comply with a recipe based specification similar to New Zealand. However, basecourse materials that comply with the specification can often be at locations that are too far away to even be considered as a viable option. This has resulted in Pavement Engineers becoming actively involved in material design. The Pavement Engineer is required to locate a material locally available that with or without the addition of a stabilising agent will have sufficient strength and durability to provide adequate performance.

Some of the methods used to assess alternative materials with or without a stabilisation agent are listed below:

- the Pavement Engineers previous experience with the same or similar material;
- a unbound base material is required to have a CBR>60 for low volume roads or a CBR>80 for other roads;
- a bound base material is required to have an Unconfined Compressive Strength > 1.0 MPa.
- permanent deformation test using the Repeated Load Triaxial (RLT) apparatus;
- use of the Texas static triaxial test; and
- simple shear box test.

As discussed further, it is proposed that Pavement Engineers in New Zealand take a more active role similar to the Australians in design of materials. However, consideration of the differences in climate between the two countries will be required.

3.4 Proposed Future Developments in New Zealand

3.4.1 Performance Based Specification for Unbound Road Base and Sub-base Aggregates

Currently under development is a Transit New Zealand Performance Based Specification for Unbound Road Base and Sub-base Aggregates. This specification sets out end-result requirements to ensure a unbound material will satisfy the performance requirements stated in Section 3.1. Essentially, this will allow any unbound material that meets durability and strength requirements to be used in place of TNZ M/4. A summary of the requirements specified in the performance based specification are as follows:

Unbound Materials

To allow the use of lightly stabilised materials a definition of a unbound material was defined as a material that will not develop shrinkage cracks > 1mm in width. The material is required to undergo a linear shrinkage test and the results are used to estimate the crack width.

Durability (requirement to not deteriorate over design life)

Durability of the unbound material will initially be ensured by retaining the crushing and weathering rēsistance requirements in TNZ M/4 (1995) that have already been proven.

Strength (requirement not to fail by shear and have minimal deformation)

The in-service shear strength and deformation will be evaluated with the Repeat Load Triaxial (RLT) test equipment. This test equipment applies a pulsating stress on the base-course sample to simulate the passage of one set of duals with a standard 8.2 tonne axle load. Aggregate is tested in the RLT apparatus for 100,000 load cycles and the permanent deformation during the test is recorded. The test conditions are saturated undrained at the maximum and minimum stress conditions expected in service (two RLT tests required). CIRCLY can be used to estimate the in service stress conditions.

The important results from the RLT tests are the rate in change in permanent deformation for both the high and low stress tests. Geomechanic principles are applied to these results to estimate the total deformation within a 150 mm thickness of aggregate caused after the application of the design traffic loading.

A total deformation limit has currently not been decided but based on analysis of material complying with TNZ M/4 it is likely to be initially in the order of 5 mm. If the calculated total deformation exceeds the limit of say 5 mm then the material cannot be used for the situation analysed. The Pavement Engineer will then have the option of either using a different material or placing another layer of stronger material on top to reduce the level of stress.

Using this method of analysis for strength will allow weaker materials with a higher rate of permanent deformation to used where the traffic volume is low. Conversely, a high strength material with a low rate of deformation will be required for high traffic volume roads.

This specification will "open the door" for allowing alternative materials to TNZ M/4 and lead the way for pavement materials to be "engineered" for each specific application.

3.4.2 Performance Based Specification for Construction of Flexible Unbound Pavements

In parallel with the development of the performance specification for unbound materials a performance based specification for construction of flexible unbound pavements is being developed. This specification requires the Contractor to design the pavement materials and depths based on the site information given in the tender. If the tender information is proved to be incorrect then the Contractor will be paid for a re-design and material quantities will be altered. If quantities reduce then the unit rate will be re-negotiated. There will also be a 12 month maintenance requirement for the pavement and surfacing. The Engineer's role will essentially be an auditor to check the pavement design, ensure the pavement is constructed according to the Contractors tender, and materials used comply with the performance specification for materials.

This specification will give the Contractor an opportunity to undertake the pavement design and select appropriate materials. It is envisaged that in areas where prime quality aggregate complying with TNZ M/4 is scarce, Contractors who are innovative in sourcing alternative materials will have a competitive edge. Effectively, the market will drive the design and selection of alternative materials with significant cost savings.

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PRACTICAL ASPECTS OF BASECOURSE CONSTRUCTION CONTROL

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Basecourse construction for roads is typically undertaken in accordance with the Transit NZ B/2 specification. The objective in construction is to achieve a pavement system which is adequately compacted and sufficiently robust to provide the required levels of service. Adequate compaction is important to avoid post construction pavement rutting.

Compaction control is usually managed in terms of Maximum Dry Density (MDD) criteria and field density or voids control. There are a number of variables which impact on the reliability of test data (eg the use of the NDM) and its interpretation. These are identified and their significance is described, including the influence of different aggregate sources.

A strategy is described for the systematic control of basecourse construction.

BACKGROUND

With the introduction of TNZ B/2:1997 the procedures for monitoring and acceptance of basecourse layers have changed. The concepts of plateau density with primary and secondary compaction processes are not specified. With the focus on MDD it is essential that engineers and technicians fully understand the implications of this specification criteria and the factors which influence compliance.

The aim of this paper is to assist those involved with pavement construction to better understand the methods used to measure aggregate/basecourse parameters and to assist with evaluation of the test results. Areas of uncertainty and sources of error are identified. Suggestions are provided to reduce these uncertainties.

It is beyond the scope of this paper to comment on the specified compaction criteria and their relationship with the performance of constructed granular pavement.

PROPERTIES OF AGGREGATES

Consideration of the fundamental component properties of a pavement aggregate is essential in order to understand and control placement and compaction during construction. We need to resort to first principles in soil mechanics to ensure a clear grasp of the concepts.

A placed aggregate consists of:

- Solids - all of the aggregate particles (of mass m_s and volume V_s)
- Voids - the space between the aggregate particles (volume V_v)
- Water - which partially infills the voids (of mass m_w and volume V_w)
- Air - which partially infills the voids (of mass zero and volume V_a)

The total volume, V, can be expressed as:

$$V = V_s + V_v \qquad \text{where } V_v = V_a + V_w$$

The total mass, m, comprises:

$$m = m_s + m_w$$

Densities can be expressed as follows:

Solid particle density, $\rho_s = \dfrac{m_s}{V_s}$

Water density, $\rho_w = \dfrac{m_w}{V_w} = 1.0$

Dry density, $\rho_d = \dfrac{m_s}{V} = \dfrac{m_s}{V_s + V_v}$

Wet (Bulk) density, $\rho_b = \dfrac{m_s + m_w}{V}$

Conventional terminology defines:

Water content $w = \dfrac{m_w}{m_s} \, x \; 100\%$ where $m_s = m - m_w$

Void ratio $e = \dfrac{V_v}{V_s}$

Degree of saturation, % $s = \dfrac{w\rho_s}{e} . 100\%$ or $\dfrac{\rho_d \, . \, 100\%}{1 - \dfrac{\rho_d}{\rho_s}}$

Total Voids $V_v = \dfrac{(V_a + V_w)}{V} \, 100$ or $\left(1 - \dfrac{\rho_d}{\rho_s}\right) 100$

Total voids is usually expressed as a percentage.

Total Voids $= \dfrac{V_v . 100}{V} \, \%$ where $\dfrac{V_v}{V} = n$, the porosity.

Aggregate Grading

Where the maximum aggregate size is larger than that permitted in the laboratory test it will be necessary to calculate the equivalent (theoretical) dry density which will be the target field dry density.

The theoretical dry density, ρ_{dt}, is calculated:

$$\rho_{dt} = \frac{1}{\dfrac{F}{\rho_s} + \dfrac{(1-F)}{\rho_d}}$$

where:

F = fraction above 19mm (or 40mm)
ρ_s = solid density of the stones (ie ρ_d for this fraction F)
ρ_d = dry density of the fraction $(1 - F)$ tested

This calculation assumes that the mortar (ie the fraction tested) will neatly fill the voids among the larger stones. For this to be achieved the aggregate needs to be well graded without segregation. This is practical where the split is at 4.75mm (cf USBR Earth Manual) but may not be as suitable where the split is at 19mm. Derivation of ρ_{dt} is presented in Appendix 1.

Example:

For an aggregate with an MDD tested (2.01 t/m³) based on 58% passing the 19 mm sieve and a solid density of 2.81 t/m³, the theoretical MDD is 2.28 t/m³.

AGGREGATE DENSITY TESTS

Laboratory Tests

Laboratory compaction density tests are undertaken using:

NZS 4402:1986 Test 4.1.3	Vibrating Hammer Test (max particle size 37.5mm)

The test provides a Maximum Dry Density (MDD) for the compacted aggregate which is a function of the applied energy. TNZ B/2 utilises the vibrating hammer test. Other compaction tests are 4.1.1 and 4.1.2 NZ Standard and Heavy compaction respectively.

Laboratory solid density tests are undertaken using:

	NZS 4407:1991 Test 3.7.1	The solid density of Aggregate particles - Pycnometer method for particles passing the 19mm test sieve
and,	NZS 4407:1991 Test 3.7.2	The solid density of Aggregate particles - Immersion in water method for particles passing the 19mm test sieve

Aspects of the solid density test are discussed.

FIELD COMPACTION CONTROL PROCESS

Control of basecourse compaction based on TNZ B/2: 1997 typically requires the following aggregate parameters:

(i) the Maximum Dry Density (MDD) using vibratory compaction NZS 4402:1986: Test 4.1.3, a laboratory based test. This density parameter must be established prior to the construction of the basecourse.

(ii) the field insitu wet (bulk) density and water content using nuclear densometer, NZS 4407:1991: Test 4.2.1.

(iii) the solid density of aggregate particles, a laboratory based test , NZS 4407:1991: Test 3.7. (See discussion below re ASTM solid density testing)

(iv) the insitu water content determined using the laboratory oven dried method, NZS 4407:1991: Test 3.1. This enables the correlation between NDM measured water contents and oven dried water contents to be established

(v) % Saturation determined on the basis of the insitu wet (bulk) density using NDM (NZS 4407:1991: Test 4.2.1), insitu water content using laboratory oven dried (NZS 4407:1991: Test 3.1) and the solid density of aggregate particles

(vi) field density as % Maximum Dry Density (MDD) determined on the basis of insitu dry density calculated with the corrected oven dried water content and the maximum dry density (MDD) achieved by vibratory compaction NZS 4402 1986: Test 4.1.3.

TEST ERRORS AND CONFIDENCE

When evaluating test data for aggregate compaction control, it is essential to understand the accuracy of the data and the source of possible errors. Errors discussed here are based on an insitu bulk density of 2.35t/m³ and a (laboratory) water content of 6%, which would be typical of field compacted aggregate in a pavement.

(a) Water Content

With most NZ aggregates the nuclear densometer (NDM) will not provide the correct water content. Errors will depend upon individual densometers and the aggregate type, ie greywacke, andesite, basalt, dacite etc. To obtain the "correct water" content samples should be taken at regular intervals to determine the relationship of nuclear densometer water content to oven dried water content. Tests with no oven dried water content can then be corrected by this ratio. It would not be good practice to take samples every test as this would create potential "potholes" in the finished basecourse surface.

The likely water content error is ± 2% of water content which equates to a ± 0.8% change in % saturation and ± 0.1% change in dry density

ie 2.35/1.06 = 2.217 t/m^3
2.35/1.061 = 2.215 t/m^3

(b) Solid Density of Aggregate Particles

This is a laboratory based test (NZS 4407 : 1991 test 3.7.1 and 3.7.2) with an error of ± 0.4% (ie reported to ± 0.01 t/m^3). This error equates to 1.3% change in saturation and 0.4% dry density.

There is a need to undertake the measurement of solid density on a consistent basis. With vesicular aggregates this is particularly important. It is important that a consistent method is adopted for this test. ASTM C127:1993 and C128:1993 test for three conditions:

- apparent (dry solid density of the mineral content, starts with an oven dried sample, volume is on a mass basis)
- bulk (dry solid density of particles including the rock voids, $V = V_{SSD}$)
- bulk at SSD (wet solid density based on surface saturated dry (SSD) condition after 24 hour soaking, V_{SSD})

These conditions give different solid density values, see Table 1.

Source	State (ASTM C127/128:1993)		
	Apparent	Bulk	Bulk SSD
Greywacke 1	2.80	2.63	2.70
Greywacke 2	2.75	2.69	2.71
Greywacke 3a	2.70 *		
Greywacke 3b	2.72 *		
Greywacke 4	2.69 *		
Basalt 1	3.07 *		
Basalt 2	2.87 *		

Table 1 Solid Density (t/m^3) results to ASTM 127/128:1993

NZS 4407:1991 (Test 3.7.1) * is effectively an apparent solid density test (in terms of ASTM) where the sample is oven dried to determine solid mass.

Samples should be prepared in a surface saturated dry (SSD) state for testing, ie a similar condition to that in which the aggregate will be present in the field.

Adopting a Bulk SSD solid density will enable subsequent derivation of true total voids for the placed aggregate, where the total voids are the matrix voids (excluding any vesicular voids). It will best reflect the wet condition of the aggregate in the field.

When testing for solid density it is necessary to split the sample at 4.75mm. Invariably the solid density for the fine fraction is less than for the coarse fraction. Solid density is grading dependent but generally this is small (± 0.02 t/m^3) .

Care is required to ensure that test results are representative of the aggregate, ie how repeatable is the solid density measurement (cf Greywacke 3 in Table 1). ASTM C127/128 indicates that the interlaboratory precision is ± 0.03 t/m^3.

(c) **NDM Wet (Bulk) Density**

Typically a newly calibrated nuclear densometer will have an error of ±1% (this is based on calibration data supplied with machines from authorisation calibration agencies). This could easily extend to 2% with older machines or machines without current calibration certificates. A 1% error in bulk density will equate to 1% change in measured dry density and 4% change in % saturation.

(d) **NDM Operator error**

We need to be aware of the potential for operator error particularly in the NDM bulk density testing. This is influenced by the preparation of the testing base, levelling, and ensuring correct standard counts. Selection of the water content sample is critical, ie have we taken a representative sample of the depth of aggregate being tested.

From ASTM D2922:1996 Standard Test Methods for Density of Soil and Soil-Aggregate in place by Nuclear Methods (Shallow Depth) states that for the backscatter method of determining wet density we can expect, when using one operator, a variation of .054 t/m^3 (wet density in the order of 2.00 t/m^3) between two tests and similarly, for interlaboratory testing, a variation of 0.107 t/m^3.

This variation for one operator equates to 2.7% variation in wet density and for interlaboratory variation it is 5.4%.

Cumulative Error

The total possible error is the sum of the above mentioned errors, ie for % saturation the total error is 6.1% and 1.1% for dry density.

In total the possible error is in the order 10% for saturation and 3% for dry density.

	% saturation	dry density
Water content	± 2 %	± 0.1 %
Solid density	±1.3 %	± 0.4 %
NDM wet density	± 4 %	± 1 %
NDM operator	±2.7 %	± 2.7 %
Total	> 10 %	> 3 %

Often specifications include criteria in terms of total voids. Total voids are defined as the percentage of total voids, V_v (= $V_a + V_w$), to the total volume, V. This value is influenced by the above stated errors. The total voids value could vary by 8% (18.1% to 19.6%) with a 2 % change in solid density (2.70 to 2.75 t/m^3).

TESTING ISSUES

(i) **MDD NZS 4402:1986 Test 4.1.3**

This test is not tightly specified. Results coming out of various laboratories indicate difficulty in determining reasonable compaction curves where the technician can state with certainty a maximum dry density and optimum water content.

For a given aggregate source and grading individual laboratories can produce consistent MDD results (± 0.02 t/m^3).

Issues of concern are:

- Specification of Vibratory Hammer - a wide power range is allowed and this can influence results by 18% (eg 1.90 to 2.25, 2.04 to 2.17 t/m^3 going from a 750 to an 1100 watt hammer). NZS 4402 allows 600 to 1200 watts.
- Determining water contents - before or after the sample is compacted?
- Type of mould base - perforated or not
- Aggregate grading - this does have an influence on the MDD result. Experience suggests that it may be small < 1%
- MDD values should be corrected to theoretical density where materials exceed 37.5 mm.

In order to establish the "optimum water content", the water content of the test samples needs to be established before compaction testing. With higher water content samples free water is released and the post test water content may not be representative. Typically to achieve the laboratory MDD it is necessary to start with a sample wetter than the "optimum water content". In order to measure a true density and water content the test should be halted at the point when the sample starts to yield free water. The optimum water content is typically just below the point where free water is yielded.

Figure 1 illustrates the variation between the two approaches:

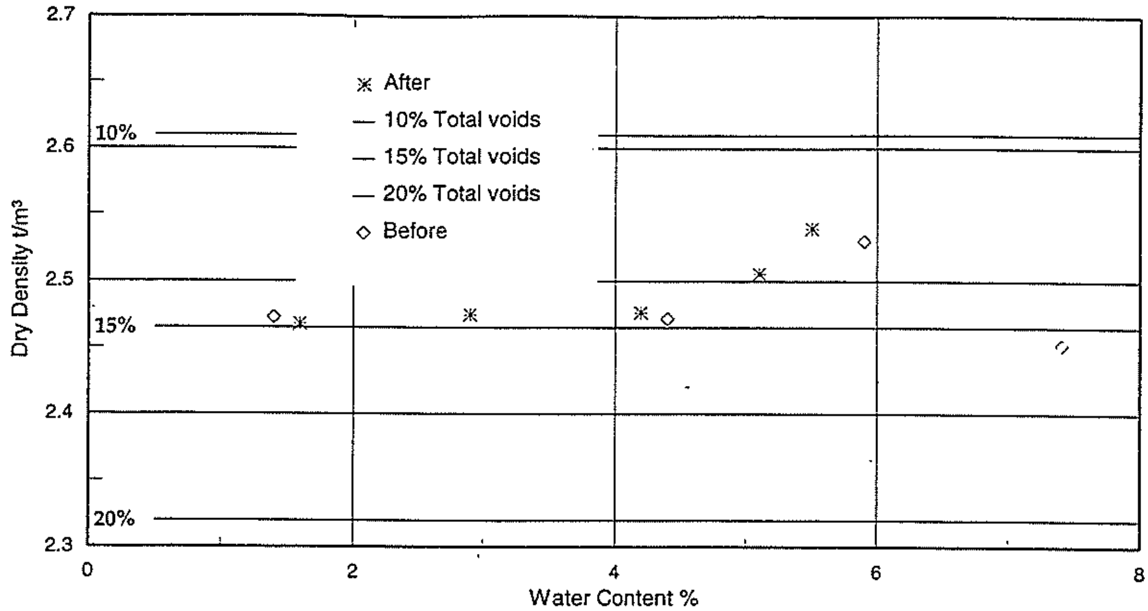

Figure 1 MDD test result

Maximum Dry Density (MDD) NZS 4402 Test 4.1.3 should be specified with a vibrating hammer power of1100 to 1200 watts where this criteria is used.

(ii) Total Voids and MDD

Typically the total voids at MDD are about 15 %. If the total voids are greater than 20% the test data may be suspect.

In some cases the dry density required to achieve 20 % Total Voids is greater than 100% MDD. Part of this problem could come from the errors listed above. More significantly this indicates that 98% MDD may not be an adequate specification criteria for compaction to ensure the required pavement performance.

(iii) Solid Density

In the past many laboratories have tested the solid density at surface saturated dry condition with the view that the absorption of the aggregate will not interfere with the density value (and hence Total Voids). From the discussion above the ASTM bulk SSD solid density should be adopted.

With the % saturation in TNZ B/2 the solid density method based on bulk SSD will yield the effective saturation and total voids of the aggregate matrix.

(iv) Representative Number of Tests

In each situation it is important that sufficient samples are taken and tested to ensure that test results are representative. Variability in samples and testing both contribute.

PROPOSED SPECIFICATION CRITERIA

Specification criteria for basecourse construction are continually debated. There is a need to review the criteria adopted to ensure that pavements provide the end result performance expected.

Having considered all of the factors influencing the measurement of aggregate properties, and the sensitivity of the vibrating hammer MDD test, careful consideration of the compaction criteria is warranted. From our experience there appears to be real merit in adopting a target total voids criteria based on local performance knowledge. This criteria will be relatively insensitive to aggregate type/source providing bulk SSD solid density values are adopted (ie ASTM C127/128:1993).

Experience suggests that acceptable total voids as percentage will be between 10% (below which significant aggregate breakdown occurs during construction) and 20% (above which the aggregate is obviously "loose"). It is beyond the scope of this paper to investigate the most appropriate absolute criteria.

PROPOSED TESTING PROCEDURES

With the issues outlined above it is clear that well defined methods and procedures are required to minimise potential errors.

(1) Solid Density

- Test in terms of ASTM C 127/128:1993
- Adopt the ASTM bulk SSD solid density value
- NZS 4407 is not an appropriate test where voids and degree of saturation of the aggregate matrix are to be calculated.

(2) Field bulk density (by nuclear densometer (NDM))

- Calibration of nuclear densometer must be current, ie yearly, preferable with 6 monthly checks. Take daily standard counts and record data stating running averages.
- Carefully prepare test base/aggregate surface.
- Take multiple readings at each location and average to measure density.
- Recover samples of aggregate for water contents, (and ensure that samples are taken from correct depth).
- Ensure that number of test locations are adequate for area being tested (for the site sampling strategy).

3) Total voids

- Establish appropriate specification criteria

SUMMARY

Factors which influence the measurement of aggregate properties during pavement construction have been outlined in this paper. There is a need for clarification/better definition in the application of test methods. The vibrating hammer test presently allows a significant variation in MDD test result which is influenced by the hammer power rating. Where voids and the degree of saturation are to be calculated we recommend that the aggregate solid density is measured on the basis of the bulk SSD solid density.

Specification criteria for basecourse construction are continually debated. There is a need to review the criteria to be adopted to ensure that pavements provide the end result performance expected.

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APPENDIX 1 : DERIVATION OF THEORETICAL DRY DENSITY, ρ_{dt}

The dry density of a composite sample, ρ_{dt} will reflect the solid density, ρ_s, of the coarse fraction (stones) $(m_{s1} = F.m)$, and the dry density, ρ_d, of the fine fraction $(m_{s2} = (1- F)m)$.

$$\rho_{dt} = \frac{m_{s1} + m_{s2}}{V} = \frac{m_s}{V}$$

where:

$m_{s1} = F.m$

$m_{s2} = (1 - F)m$

and F = fraction of oversize (stones) in sample

$$= m_{s1} = \frac{m_{s1}}{m_{s1} + m_{s2}}$$

$$\Rightarrow \qquad m_{s1} + m_{s2} = \frac{m_{s1}}{F}$$

$$\rho_{dt} = \frac{m_{s1} + m_{s2}}{V} = \frac{m_{s1} + m_{s2}}{V_{s1} + V_{s2}}$$

$$= \frac{m_{s1} + m_{s2}}{\dfrac{m_{s1}}{\rho_{s1}} + \dfrac{m_{s2}}{\rho_{s2}}}$$

$$= \frac{\dfrac{m_{s1}}{F}}{\dfrac{m_{s1}}{\rho_{s1}} + \left(\dfrac{m_{s1}}{F} - m_{s1}\right)\dfrac{1}{\rho_{s2}}}$$

$$= \frac{\dfrac{1}{F}}{\dfrac{1}{\rho_{s1}} + \left(\dfrac{1}{F} - F\right)\dfrac{1}{\rho_{s2}}}$$

$$\rho_{dt} = \frac{1}{\dfrac{F}{\rho_{s1}} + \dfrac{(1-F)}{\rho_{s2}}}$$

where:

$$\rho_{s1} = \frac{m_{s1}}{V_{s1}} \equiv \rho_s$$ [the solid density of the rock (stone)]

$$\rho_{s2} = \frac{m_{s2}}{V_{s2}} \equiv \rho_d$$ (the dry density of the fine fraction)

cf USBR Earth Manual p43.

Evaluation of Pavement Response Models Using Measured Pavement Response Data

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SUMMARY

The outputs from four computer based pavement response models has been compared with the measured pavement response from two types of pavements. The pavement models; ELSYM5, VESYS, MICHPAVE and CIRCLY were setup to model two pavements that have been tested at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF). The models were chosen to represent the type of model that has been the basis for pavement design in New Zealand and overseas. The input parameters for the models are outlined and the material characterisation methods are described. The process required to model a non-linear material in a linear pavement model is discussed. The results of the predicted pavement strains vary from 0.10 to 0.58 of the measured pavement strain. Some of the problems encountered in modelling thin surfaced flexible unbound granular pavements with simple pavement models have been identified.

1. INTRODUCTION

This paper describes the application of four software packages for the analysis of pavements. Two pavements are analysed in each of the packages and the outputs are compared with the measured responses from the actual pavements. The four programs are ELSYM5, a multi-layer linear elastic theory based package, VESYS IIIAM, a finite element based linear elastic theory package, MICHPAVE, a finite element based non-linear elastic theory based package and CIRCLY, a finite difference based linear elastic program.

2. BACKGROUND

Both the previous Transit New Zealand design process, using the State Highway Pavement Design and Rehabilitation Manual [1] and the current process, the AUSTROADS 1992 Pavement Design Guide [2], adopted in 1995 are based on a combination of empirical knowledge and linear elastic theory. While linear elastic theory has been satisfactory for structural asphalt or concrete pavements, the non-linear response of granular materials and soils requires a more advanced pavement response model. Until recently there has only been a limited amount of data collected from either experimental or in-service roads available to either calibrate or verify the fundamental outputs from the pavement models for New Zealand conditions. Researchers as early as 1972 recognised that granular and cohesive materials exhibited non linear behaviour [3], and that linear elastic theory was only suitable for pavements containing structural layers of asphaltic concrete. Dunlop et. al. [4] also recognised this fact but commented that suitable models were not easily run on the computing power available at the time. A major shortfall of linear elastic theory is that materials are able to develop tensile strains which is impossible for unbound granular material.

Transit New Zealand (and its predecessor, the NRB) and the University of Canterbury have been cooperating in accelerated testing of full-scale pavements since 1969. The original pavement testing machine developed by Paterson [5] was used for a number of research projects before it became unserviceable in 1983. In 1986, the upgraded CAPTIF was commissioned. A further upgrade of the facility was carried out in 1997 to update the operation of the facility.

CAPTIF is housed in a hexagon-shaped building that is 30 m wide and 6 m high. An annular concrete tank, 1.5 m deep and 4 m wide, confines the bottom and sides of the track. The track has a median diameter and circumference of 18.5 m and 58.1 m, respectively. Normal field construction and compaction equipment is used to construct the subgrade and pavement structure in a concrete walled annular tank.

The main feature of CAPTIF is the Simulated Loading And Vehicle Emulator (SLAVE). SLAVE was designed

The penetrometer and CBR testing confirmed the visual assessment of very low CBR values in the range of 1.5 to 3.0%. The Loadman also indicated low resilient modulii in the order of 15 to 20MPa. Moisture content/dry density relationships gave maximum dry densities of 1800kg/m^3 at 15% moisture content under N.Z. Standard Compaction.

The low densities in the subgrade soils gave rise to some concern that consolidation would occur over time due to the pavement loadings and construction traffic. However, the size of the area and the high initial ground water levels precluded compaction by vibration or preconsolidation when, in a saturated condition, one passage of an excavator was sufficient to cause the sandy silts to heave severely.

Options to minimise the effect of consolidation were considered and are described further. Consolidation testing was not undertaken.

4.2 Loadman Testing

The Loadman Falling Weight Deflectometer(1) was used in the early stages of the investigation to assess its potential to provide resilient modulii information for design purposes and its potential as a construction monitoring tool. This device measures the deflection of a 10kg weight using an accelerometer located in the head of the instrument. The measured deflection is related to the resilient modulus.

The modulii measured during the testing of 10-30MPa were consistent with the normally accepted empirical conversion of Modulus (MPa) = 10x (CBR%). However, subsequent testing on fill areas where the sandy silts had been compacted to maximum dry density at just above optimum moisture content also indicated resilient modulii of around 25MPa.

These compacted soils were just slightly weaving under construction traffic due to elevated pore pressures immediately after compaction. The CBR was clearly much higher than 2.5% and penetrometer testing indicated a CBR greater than 6%.

The explanation for this is believed to lie in the level of saturation of these soils. In both the low density saturated soils and the compacted soil the percentage of air voids would have been close to zero. In this situation it is suspected that the deflection measured by the Loadman Falling Weight Deflectometer was influenced mainly by the rise in pore pressure as the weight hit the surface. The contact between the soil particles appeared not to control the deflection.

For this reason the Loadman is considered unreliable in saturated soils, however, testing in unsaturated soils gave results which appeared to be much more reliable.

5. DESIGN CONSIDERATIONS

Preliminary design indicated that very thick pavements, i.e. up to 2000mm, would be required. However, deflections of up to 6mm under wheel load were predicted. This required further investigation.

The site investigation had shown very low CBR values and subgrade modulus. Further, the alluvial silts comprising the subgrade were of low density and potentially subject to consolidation.

The construction program did not allow time for pre-loading and consolidation and the high water table prevented consolidation of the subgrade by conventional vibrating or static rollers.

Due to the relatively uniform fall along the site from west to east and the presence of the main stormwater outfall at the north-east lowest corner, it was decided to install an extensive network of sub-surface drainage at the earliest possible time to enable the lowering of the water table throughout the site. Once the subgrade soils were unsaturated consolidation by construction plant would be possible.

The layout of the container handling areas allowed surface drainage to channels located in conjunction with kerbs separating the rail track and pavement areas. All drainage manholes were located between the rail tracks. This allowed all pavement areas to be free of dish channels, sumps, manholes and other surface drainage structures. These structures, if incorporated into heavy duty pavements, result in high impact loadings due to the surface irregularities and seriously reduce the quality of the riding surface.

To provide good surface drainage minimum transverse gradients of 2% were adopted for concrete block surfacing. This would be relaxed slightly for asphalt paving.

The decision on the type of surfacing was deferred and pavement designs developed which could use 80mm interlocking concrete block paving or 100mm asphalt paving.

Initial cost estimates indicated that a full depth unbound granular pavement would be the most economical option, particularly as there was a suitable sandy gravel available on site for the lower sub-base materials. However, it was recognised that thick pavements would lower the subgrade level into the softer and wetter soils at depth under the site. Control of deflections to acceptable levels was also necessary.

6. PAVEMENT LOADINGS

As with any industrial pavement, it was difficult to estimate the likely passages of vehicles and the distribution of axle loadings. The total tonnages handled by the facility were easily established and the distribution of container weights was assumed to match that of the published British Ports Association distribution[2]. The projected number of repetitions of loading was built up by considering the method of working the facility and accumulating the likely axle loads and frequencies over the most heavily loaded pavement areas.
Determination of the loadings and frequencies for design purposes is imprecise and involves assumptions. This problem is further compounded by the rapid advances in container handling plant and changes in the type of materials and goods which will be handled by the facility. For the purposes of design, a design life of 20 years and a 5% compound growth rate was adopted.

7. PAVEMENT STRUCTURAL DESIGN

Preliminary design was undertaken using four different methods. Three of these were essentially mechanistic design procedures and the fourth an empirical design procedure. Two of the procedures were specific to concrete block pavement. For the purposes of preliminary design interlocking pavement was considered equal to an equivalent thickness of asphalt.
The four methods used were CIRCLY[3], LockPave[4], British Ports Association[2] and the IB 027[5] method. The design axle loads exceeded the IB 027 nomographs and some extrapolation was required.
The preliminary design suggested that very thick unbound granular pavements were required because of the soft subgrade conditions. CIRCLY was the only method which could predict surface deflections under the design axle loadings and these were examined because it was anticipated that high deflections would occur.
CIRCLY predicted deflections of up to 6mm under the highest design axle loadings on a 1900mm total pavement thickness. This was considered realistic given that all methods were predicting similar pavement thicknesses. However, the granular pavement thicknesses and deflections were sensitive to the subgrade modulus adopted for design.
The high predicted deflections required further consideration. Whilst the radius of the deflection bowl was large, the high deflections could result in damage to pavement structures such as kerb and channel and, in extreme cases, could result in damage to the surfacing. The method of operation of the facility would result in heavy wheel loads close to the kerbs and channels.

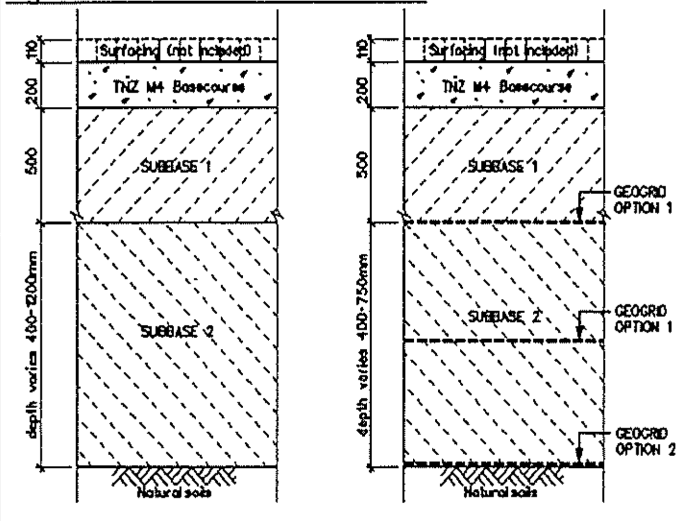
8. TEST PAVEMENT

The layout of the site and construction sequence allowed the construction of a test pavement which could be tested using similar axle loadings to those which would occur in service. The intention was to incorporate this test pavement into the final pavement structure.
A range of pavement depths and configurations was selected including several which incorporated various layers of Geogrids. The grid used was Tensar SS30[8]. Typical test pavement sections are shown in Figure 1.
It is emphasised that the primary purpose of this pavement was to calibrate the mechanistic model provided by the program CIRCLY with regard to the pavement modulii and deflections. It was not considered necessary, or feasible in the time available, to investigate the performance of the pavement under repeated loading.
The total test pavement area consisted of eighteen 10m x 10m bays. These were carefully constructed and extensively tested as construction of the test area proceeded.
After construction, the test pavements were tested using a forklift and loaded container. The front axle load was 58T for the test.
The University of Canterbury's GeoBeam was used to measure the rebound deflection for this testing but, unfortunately, due to electronic failure and limited availability of the forklift, the full range of testing on each pavement could not be completed. However, sufficient tests were undertaken to enable comparisons between the various pavement configurations.
The rebound deflection bowl was very large and both supports of the GeoBeam were within the bowl. In order to correct the readings for support deflections, the assumption was made that the deflection bowl shape remained constant as the wheels moved away from the measuring point. This had the effect of significantly increasing the measured deflections.
These corrected deflections were compared with the deflections predicted by CIRCLY and good agreement was found.

It had been intended to use the bowl shape measurements determined with the GeoBeam to back-calculate the modulii of the pavement using the computer program EFROMD2[6]. This proved unsuccessful as the measured bowl shape was quite different from that determined by EFROMD2. It is understood that EFROMD2 uses a subset of the CIRCLY program to determine and match bowl shapes.

In investigating this problem, the bowl shapes predicted by CIRCLY were then studied and manually compared with the measured shapes. It was found that CIRCLY was predicting deflections which were much too high in the range of 2 to 10m from the axle load. This suggests that CIRCLY may not have been calibrated for heavy loads and deflections at a distance from the load point.

Figure 1 - Test Pavement Construction



The test pavements incorporating Geogrid did not perform as well as anticipated in helping to control surface deflections. Previous experience with Geogrid on more lightly loaded and thinner pavements had been satisfactory. Little information is available on the performance of Geogrids in heavy duty pavements and even the manufacturers could not supply detailed technical information or design methods. Houlsby & Jewell[7], amongst others, have undertaken studies and provided design methods, however, these were all based on the prevention of excessive rut depths in pavements which could tolerate rutting.

This test pavement provided the opportunity to investigate the performance of Geogrid in controlling surface deflections in more detail under heavy loading and in thicker pavements. Preliminary calculations indicated that the level of strain in the grid may not be sufficient to mobilise the strength of the grid and it was suspected that the grid layers positioned higher in the pavement would not be effective in controlling deflection or improving the compacted density of the metalcourse. Layers of Geogrid close to the surface could be effective in controlling shear failure in weak metalcourse layers, however, the high strength granular materials available on this site were considered unlikely to fail in shear and Geogrid at high levels was not incorporated.

The suppliers of the grid expressed the opinion that the grid should be in contact with a fully crushed granular material to maximise the friction between the grid and the metalcourse. A thin layer of crushed granular material over the grid could not easily or economically be placed because of the very soft subgrade conditions and because ample supplies of well graded alluvial gravels (max. size 65-100mm) were available on site as a lower sub-base.

During compaction of the lower layers, slightly higher densities and resilient modulii were obtained when Geogrid was placed directly on the subgrade. This was because of an anvil effect where the grid restrained the lowest layers of metalcourse and allowed the succeeding layers to be compacted against them.

The deflections measured on the pavements which incorporated Geogrid at the bottom generally had slightly lower deflections than the equivalent pavement without grid. However, this reduction was small. No improvement in deflection was found in the pavements containing grid in the lower and mid layers.

Back analysis of the pavement using the program CIRCLY with the measured deflections and with metalcourse and subgrade modulii adjusted to match the deflections allowed the strains in the metalcourse to be calculated ignoring the grid. These strains were found to be low and insufficient to mobilise the strength of the Geogrid.

The testing confirmed that high deflections could not be controlled by deep granular pavements on very soft subgrade.

9. CEMENT MODIFIED PAVEMENT

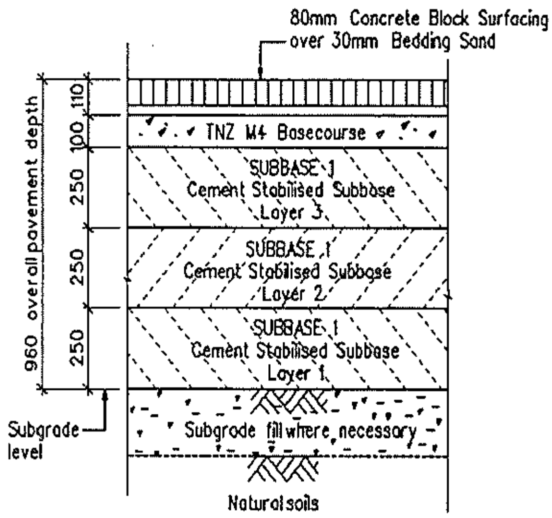
A further test pavement was designed and constructed to a depth of 1200mm using cement treated aggregate with 2.9% cement by weight. This pavement was constructed at a shallower subgrade depth and, as a result, higher subgrade CBR and modulii were obtained. The mean subgrade CBR was 3-5%.

The bowl shape in this pavement was expected to be large and a rebound deflection measuring beam known as a Southdown Beam[9] was used. This beam design was developed by N.Z. Rail some years ago for testing heavy pavement. It has an overall length of 6.0m but operates on the same principle as a Benkleman Beam.
In order to correct for the expected deflection at the beam supports a precise bar code digital level and a series of short staves was used. The staves were positioned along the beam and at the supports. The level readings were recorded digitally to 2 decimal places of a millimetre with approximately the same accuracy as the dial gauge readings.
This technique allowed correction of the beam readings and gave a measure of absolute bowl shape. However, it was a relatively slow technique and weather conditions interfered with some of the readings.
Deflections on this pavement after correction were in the order of 1 to 1.5mm under a 58T axle load. Back analysis of this pavement was undertaken to determine modulii. Because of the large bowl shape all four wheels of the forklift axle had to be included in the analysis. The pair of wheels remote from the measurement were shown to significantly influence the deflections.

10. FINAL PAVEMENT DESIGN

The testing of the cement stabilised pavement provided confidence to investigate the use of cement modified granular materials further. Three repeated load tri-axial tests were carried out on samples of the aggregate stabilised at 2.0, 3.0 & 4.0% cement by weight and a range of confining pressures. These were found to be in agreement with generally published modulii for cement treated metalcourse and agreed with the back calculated modulii.
Design of cement treated pavements is normally based on the tensile fatigue capacity of the pavement materials. Formula for determining this capacity is published in the AustRoads manual and modified in the N.Z. supplement[11] to AustRoads Pavement Design[10] also gives relationships for calculation of fatigue criteria.
Review of these relationships and comparison with the AustRoads formulae indicate differences in allowable number of repetitions of several orders of magnitude. The original fatigue relationship was developed by Odemark in 1949[12] and is still in use. LockPave appears to use the Odemark relationship.
The cement modified pavement was designed using LockPave and the axle loads and repetitions developed for the granular pavements. A comparison was made using the CIRCLY program and the AustRoads criteria. This resulted in a total pavement thickness of approximately 1200mm compared with the LockPave thickness of 830mm excluding surfacing.
The origins of the AustRoads fatigue relationships appear to be somewhat empirical and developed for roading pavements rather than heavy duty pavements. For this reason the LockPave design method was used for the final design. It has been proven in the design of many heavy duty pavements.

Figure 2 - Final Pavement Design



After detailed consideration of the pavement surfacing options and a life cycle costing comparing asphalt and 80mm thick interlocking concrete block paving, the latter was selected for the heavy duty pavements. Asphalt will be used for the areas confined to road vehicles only.
The final pavement design is shown in Figure 2. A 100mm thick layer of high quality basecourse conforming to the TNZ M4 Specification will be used over the cement treated sub-base. Its purpose is to provide a levelling course over the sub-base, to prevent bedding sand loss in the event of wide cracking of the cemented sub-base and to prevent saturation of the bedding sand.
Fatigue considerations and tensile capacity of the cement stabilised base were the major considerations. Subgrade stain criteria were not critical in this type of pavement.

11. FACTORS IN SELECTION OF PAVEMENT SURFACING

Dentated interlocking concrete blocks (ICB's) were selected in preference to rectangular concrete blocks for this application. Shackel[13] has shown that better interlock between the blocks allows a reduction in block thickness from 100mm rectangular to 80mm interlocking blocks.

Life cycle costing of asphalt and ICB surfacing resulted in similar present day costs when maintenance, laid cost and allowance for value at the end of the design life were taken into account. It was recognised that life cycle costing is sensitive to the assumed maintenance costs and remaining value assigned to the pavements at the end of the design life.
Other factors considered were the greater resistance to rutting and deformation of ICB's and their higher resistance to damage from hydraulic oils, container corner castings and mechanical damage.
The greater luminance of concrete blocks as compared with asphalt may result in brighter working conditions within the transfer sheds, and therefore less artificial lighting will be required. This aspect was a consideration but not a major factor in selection of the blocks.

12. CONCLUSION

The design of this pavement provided the opportunity to investigate options which would not normally be economically feasible, either from a design or construction viewpoint. Due to the magnitude of this project the investigation, design and testing resulted in significant cost savings over original estimates.
The contract documentation was sufficiently flexible to enable the design to develop whilst the early stages of construction were under way. The relationship between the project design and management team, the client and the contractor was carefully developed to aid in selecting the most economical design and to enable the use of salvaged aggregate from the site and alluvial gravels from a series of borrow pits.
Extensive sub-surface drainage was essential in lowering the water table at least 1m below sub-grade level and in maintaining this over the life of the pavement.
Consideration of deflections under design axle loadings was necessary as the soft subgrades resulted in high deflections which were not adequately controlled by deep unbound granular pavements.

13. ACKNOWLEDGEMENTS

The author wishes to acknowledge consent of Tranz Rail Ltd. to publish this paper and the support of Tranz Rail Ltd., Arrow International Ltd., the Project Managers, and Works Civil Construction Ltd., the contractors, in the provision of assistance and facilities during the design and testing of the pavements, and Ground Engineering Ltd. who provided the Geogrid for the test pavement.

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Evaluation of Pavement Response Models Using Measured Pavement Response Data

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1. INTRODUCTION

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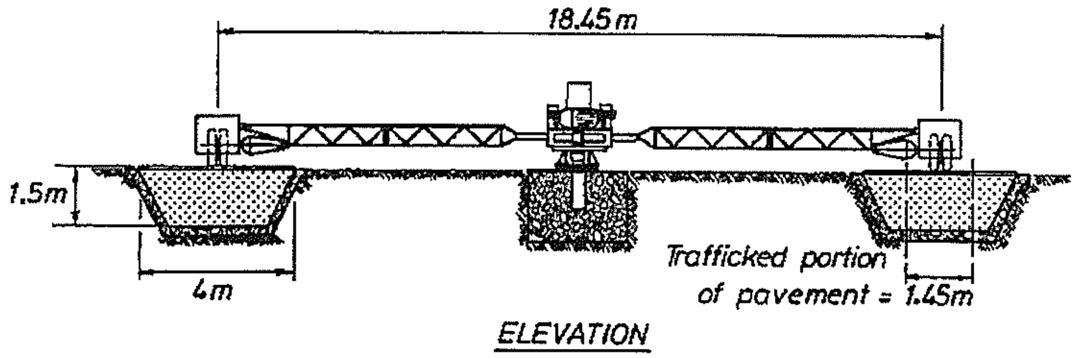

Figure 1 Cross-section of SLAVE and Test Track.

for the accelerated testing and evaluation of subgrades, pavements and surfacings by replicating the effect on the pavement of actual road traffic conditions. An elevation view of SLAVE is presented in Figure 1. A sliding frame within the central platform is moved horizontally a maximum of 1 m (from stop to stop) by two hydraulic rams; this radial movement produces multiple wheel paths. Further details of the facility are available in Pidwerbesky [6].

Pavement design in New Zealand concentrates on limiting the vertical compressive strain at the top of the subgrade to prevent plastic deformation in the subgrade. Plastic deformation in the subgrade shows up as longitudinal rutting in the pavement surface. Because of the importance of limiting the subgrade strain, research at CAPTIF places a high level of importance in measuring the vertical compressive strain in the different pavement layers. This is done using Bison Soil Strain sensors, which have a resolution of ± 50 μm/m. The sensors use the principle of inductance coupling between two free-floating, flat, circular wire-wound induction coils coated in epoxy, with a diameter of 50 mm. One of the two discs acts as the transmitter coil, creating an electro-magnetic field which induces a current in the receiving coil. The magnitude of the induced current is inversely proportional to the spacing between the two coils (Figure 2). The gauge length is the separation distance between each paired coil. The main advantage of the Bison strain coils is that they can be installed during the formation of the subgrade and the overlying unbound layers, thereby minimising the disturbance to the materials and thus providing a more representative value of the strains being measured without interfering with the materials. Bison strain sensors have been successfully used in a number of research projects ([5],[7],[8],[9]).

The data acquisition system is a modified prototype of the Saskatchewan Soil Strain/Displacement-measuring system (SSSD) developed by Saskatchewan Highways and Transportation, Regina, Canada. The system uses a dedicated computer containing a specially built General Purpose Input/Output (GPIO) board, circuit boards, rectifiers, amplifiers and assembler code written specifically for this application. Each sensor in an array is scanned simultaneously when triggered, every 30 mm of vehicle travel, so that a continuous bowl shape of strain versus distance travelled is obtained.

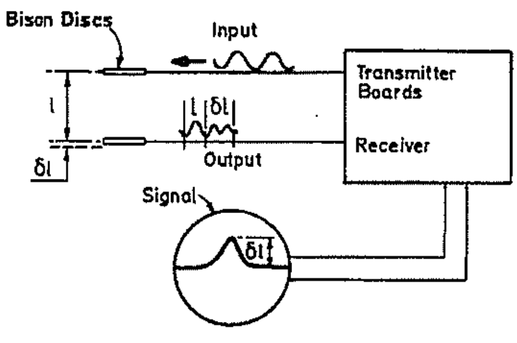

Figure 2 Principle of Bison Strain Coils.

3. PAVEMENT MODELS

Four pavement models were chosen for this study. The models are ELSYM5, VESYS, MICHPAVE and CIRCLY. ElSYM5 is a simple linear elastic program that was developed by the Federal Highway Administration (USA) in 1978 [10]. It is limited to 5 pavement layers. As with all linear elastic theory packages, it assumes that:

(i) materials are homogeneous and isotropic
(ii) there are no shear stresses at the surface

(iii) materials are linear elastic and their behaviour is defined by the elastic modulus and Poisson's ratio.

The VESYS package was also developed by the FHWA [11]. VESYS uses finite element code to model the asphaltic concrete layer of the pavement structure as a semi-infinite linear quasi visco-elastic structure resting on an elastic half space. For the modelling of unbound granular pavements, VESYS acts as a slightly more sophisticated version of ELSYM5. MICHPAVE is a nonlinear finite element program that uses simple pavement loading and infinite layer assumptions in the horizontal directions. MICHPAVE uses three material models for asphaltic concrete, granular and cohesive materials. These different material models will be covered later. MICHPAVE was developed by the University of Michigan [12]. CIRCLY was developed by the Commonwealth Scientific and Industrial Research Organisation, Australia [13]. CIRCLY is a simple finite element program that uses the basis of linear elastic theory to calculate stresses, strains and displacements. One advantage of CIRCLY is that it can model the materials in an anisotropic manner and also model different loading effects, namely traction, breaking and cornering.

The non-linear behaviour of the pavement materials can be accounted for in linear models by an iterative manner by using the following method:

(i) Initial moduli values are assumed for the pavement structure.

(ii) The pavement is analysed and the resulting stresses in the pavement layers are used to determine new values for the moduli using a calibrated resilient modulus model for the material.

(iii) The pavement is analysed again and the computed stresses are compared with the values from those used to determine the resilient modulus, if the two values are in reasonable agreement then the procedure is stopped, otherwise steps (ii) and (iii) are repeated until an acceptable match is obtained.

This method was adopted for the ELSYM5, VESYS and CIRCLY models.

4. MATERIAL MODELS

Devine [14] reviewed a number of material models used to determine the resilient modulus. These models use the results from repeated load triaxial tests to fit a mathematical model to the data. The models generally are a function of the confining and deviator stresses. The simplest model is the K-theta model proposed by Biarez [15] in which the resilient modulus is related to the bulk stress by the logarithmic equation

$$M_R = k_1 \theta^{k_2} \tag{1}$$

where

$$\begin{aligned} M_R &= \text{Resilient Modulus} \\ k_1, k_2 &= \text{experimental values} \\ \theta(\text{bulk stress}) &= \sigma_1 + 2\sigma_3 \\ \sigma_1 &= \text{axial stress} \\ \sigma_3 &= \text{confining stress} \end{aligned}$$

The K-theta model is used in the MICHPAVE model for granular materials, but the equation which gave the best fit to the measured triaxial test data was the following equation which was recommended in the SHRP P46 protocol [16]

$$M_R = k_1 \sigma_d^{k_2} (1 + \sigma_3)^{k_5} \tag{2}$$

where

$$\begin{aligned} k_1, k_2, k_5 &= \text{experminental variables} \\ \sigma_d &= \sigma_1 - \sigma_3 \end{aligned}$$

In the MICHPAVE model, a bilinear resilient modulus model is used for the cohesive material. The equation is given below and the influence of the experimental coefficients can be seen in Figure 3.

$$M_R = \begin{cases} k_2 + k_3[k_1 - (\sigma_1 - \sigma_3)] & \text{for } k_1 > (\sigma_1 - \sigma_3) \\ k_2 + k_4[(\sigma_1 - \sigma_3) - k_1] & \text{for } k_1 \le (\sigma_1 - \sigma_3) \end{cases} \tag{3}$$

where

$$k_1, k_2, k_3, k_4 = \text{experimental values}$$

Over the last four years a number of repeated load triaxial tests have been performed on the materials used for

Table 1. Properties for the test pavements.

Layer	Material	Thickness (mm)	Dry Density ρ_d (kg/m³)	Moisture content (%)
Pavement 1				
Asphaltic Concrete	16 mm AC	88	2103	NA
Basecourse	TNZ M4 AP20 crushed rock	200	2130	2.7
Subgrade	Silty clay	1212	1934	11.4
Pavement 2				
Asphaltic concrete	7 mm AC	25	NA(1)	NA
Basecourse	TNZ M4 AP20 crushed rock	250	2250	3.7
Subgrade	Silty clay	1225	1806	10.2

Note (1) The asphaltic concrete layer in pavement 2 was assumed to have no structural strength

constructing CAPTIF pavements. These tests were carried out in the Transportation Laboratory in the Department of Civil Engineering at the University of Canterbury. The test procedure following the SHRP P46 protocol. For the modelling work described in this paper, the density and moisture content values were determined for the basecourse and subgrade for the test pavements. The as built values were compared with the matrix of test conditions for the triaxial testing and the results from the triaxial test that best matched the as built conditions were used to calibrate the different resilient modulus models. The two pavements that were used in this study were the flexible asphalt pavement that was used in the OECD DIVINE experiment and a thin surfaced flexible unbound granular pavement that is currently being tested. The properties of the two pavements are shown in Table 1. A least squares approach was adopted to determine the coefficients required for the resilient modulus equations. The coefficients for the different models are shown in Table 2.

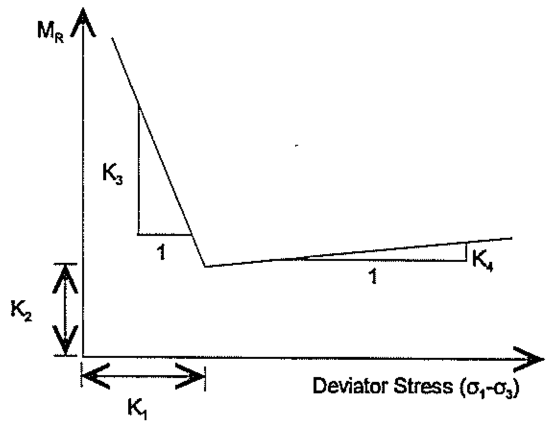

Figure 3. Bilinear resilient modulus model.

The basecourse and subgrade layers pavement layers were split into two layers each for modelling and the horizontal and vertical stresses were determined at the mid point of each sub layer. These values were then used to calculate the modulus value for the next iteration. This cycle continued until there was less than 5% change between the different moduli between the iterations. If a tensile horizontal stress was calculated by the model, then a horizontal stress of 0 kPa was used to determine the modulus for the next iteration. For the CIRCLY modelling, an factor of two between the vertical and horizontal directions was assumed for the anisotropic behaviour of the materials.

5. RESULTS

The results for Pavement One are shown in Tables 3 and 4. All four models predicted strains that are well below the measured strains. The three models that required the manual iterative method described earlier reached a satisfactory solution after 5-7 iterations. The strains predicted by VESYS were the best, but even those were

Table 2. Coefficients for the material models.

	SHRP Model				K-Theta Model			Bi-Linear Model				
Layer	k_1	k_2	k_5	R^2 value	k_1	k_2	R^2 value	k_1	k_2	k_3	k_4	R^2 value
Pavement 1												
Asphaltic Concrete	Interpolated resilient modulus: 2260 MPa @ 23°C											
Basecourse	8.206	0.656	0.139	0.93	2558	0.75	0.83					
Subgrade	196	-0.35	0.184	0.97				1.45	1550	157	25.8	0.83
Pavement 2												
Basecourse	8.206	0.656	0.139	0.93	2558	0.75	0.83					
Subgrade	196	-0.35	0.184	0.97				1.45	1550	157	25.8	0.83

between 0.58 and 0.29 times the measured strains. No model predicted the vertical compressive strain at the top of the subgrade better than 0.29 (VESYS), the other models were 0.10 to 0.19.

The modelling for Pavement Two was more complex as the thin asphalt surfacing was assumed to contribute no strength to the pavement structure. The ELSYM5 and VESYS models failed in this case as they repeatedly predicted large horizontal tensile stresses in the basecourse layers. These tensile values made the calculated modulus values to unreliable to use. The MICHPAVE and CIRCLY models were able to model the pavement successfully. CIRCLY reached a solution after 5 iterations. The results from the second pavement are shown in Tables 5 and 6. The strains predicted by MICHPAVE were the best for this pavement, but those were between 0.52 and 0.24 times the measured strains. MICHPAVE predicted the vertical compressive strain at the top of the subgrade to be 0.31 times the measured value and CIRCLY predicted a value 0.12 times the measured value.

Table 3. Final Moduli values for Pavement One.

Layer	Thickness (mm)	Final Modulus (MPa)			
		ELSYM5	VESYS	MICHPAVE	CIRCLY
Asphalt	88	2260	2260	2260	2260
Basecourse 1	100	524	372	336	392
Basecourse 2	100	284	168	221	248
Subgrade 1	200	163	46	147	177

6. DISCUSSION

The pavements models that were used in this study all predicted strains in the pavement structure that were at best 50% of the measured values. This could be for a number of reasons, of which some are; errors produced by the models due to simplifications and assumptions made in the design of the models, errors in the assumptions made when determining the input parameters to the models, ie moduli values, the use of theories that are not applicable to the complex interactions of the materials involved and the difficulty in relating laboratory based material characterisation methods to field tests. In recent years, considerable advances have been made in computing power, allowing the use of complex pavement models through the use of finite element methods and higher level material models. The next step is to either adopt or develop a more complex pavement model that better represents the actual behaviour of the pavement and a suitable method of characterising the material parameters for the model.

7. CONCLUSIONS

Four pavement response models have been evaluated by using two pavements from the CAPTIF facility. The models have been briefly described and the methodology that was used to select the input parameters and modelling process was outlined. The predicted pavement strains have been compared with the actual strains measured. There was a poor match between the predicted and the measured pavement strains. The predicted strains varied from 0.10 to 0.58 times the measured value. The factors that could influence the results are discussed, and the need for a more realistic pavement model is identified. This work provides a link between the data collected from full scale test pavements and the fundamental outputs from pavement models.

Table 4. Predicted and measured pavement strains for Pavement One.

Layer	Depth below surface (mm)	Pavement strain (microstrain)				
		ELSYM5	VESYS	MICHPAVE	CIRCLY	CAPTIF
Asphalt	88	235 (T)	431 (T)	353 (T)	380 (T)	774 (T)
Basecourse	238	619 (C)	1179 (C)	789 (C)	637 (C)	2050 (C)
Subgrade 1	338	576 (C)	1538 (C)	1014 (C)	529 (C)	5250 (C)
Subgrade 2	438	408 (C)	1077 (C)	686 (C)	411 (C)	3100 (C)

Note: C = compressive strain and T = tensile strain

8. ACKNOWLEDGEMENTS

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Table 5. Final Moduli values for Pavement Two.

Layer	Thickness (mm)	Final Modulus (MPa)	
		MICHPAVE	CIRCLY
Basecourse 1	125	607	604
Basecourse 2	125	315	488
Subgrade 1	200	147	276
Subgrade 2	1000	147	148

Table 6. Predicted and measured pavement strains for Pavement Two.

Layer	Depth below surface (mm)	Pavement strain (microstrain)		
		MICHPAVE	CIRCLY	CAPTIF
Basecourse	200	915 (C)	657 (C)	1750 (C)
Subgrade 1	300	1366 (C)	550 (C)	4450 (C)
Subgrade 2	400	808 (C)	440 (C)	3350 (C)

Note: C = compressive strain and T = tensile strain

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SHAKEDOWN ANALYSIS AND DESIGN OF UNBOUND PAVEMENTS

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New Zealand.

SUMMARY

Current design procedures for unbound pavements are based largely on consideration of the elastic (i.e. stiffness) properties of the basecourse(s) and subgrade. These properties define the "recoverable" part of the pavement deformation. Little attempt is made to include the plastic (i.e. strength) properties of the pavement layers, or to model the various actual failure mechanisms of the pavement. The response of an elastic/plastic structure to repeated loads is described by shakedown theory. In the past this theory has been successfully used to model the fatigue behaviour of frame structures and pressure vessels and the wear of metal surfaces. This paper describes ongoing research into the possible use of shakedown theory in pavement design. The basic concepts are outlined together with the most recent calculations of the critical design shakedown load. It will be shown that this theory leads naturally to a theory of rut formation as well as predictions of sub-surface slip and other failure mechanisms. The influence of design parameters such as the strength, stiffness and depth of the basecourse material as well as the consequences of water infiltration are discussed.

1. INTRODUCTION

The application of empirically established theoretical models of geomaterials to pavement design has lagged behind most other branches of geotechnical engineering. Perhaps this is mainly due to the fact that the loads are applied repeatedly and the resulting damage is built up over a long period of time, in marked contrast to problems involving foundations or slopes. The current design procedures for unbound pavements employed in New Zealand and elsewhere are essentially based on the knowledge of elastic properties of the basecourse(s) and subgrade. The conventional argument for this approach is that a successfully designed pavement should clearly not produce significant permanent strains. For example Croney [1] states that:" The shear strength of soil is not of direct interest to the road engineer... the soil should operate at stress levels within the elastic range... The pavement engineer is, therefore, more concerned with the elastic modulus of soil and the behaviour under repeated loading". This philosophy is in marked contrast to most structural design procedures, such as the design of frame structures, foundations, slopes etc., which are designed against failure by using a plastic analysis to determines the critical failure loads and then ensuring that the structure operates below a specified safety margin of these critical conditions.

In the extant pavement design procedures the situation is further confused by the methods used to determine the elastic moduli. The stiffness of the subgrade is normally determined by some power law relation between the stiffness and the CBR value [2], despite the fact that many experimental and theoretical studies have shown that the CBR measurement depends on the plastic as well as elastic properties of the soil [3, 4, 5]. Indeed for stiff soils the CBR value is a measure of undrained shear strength and is essentially unrelated to the elastic properties [5, 6, 7]. The nonlinearity of the elastic properties of the basecourse is well recognised and described by the use of the resilient modulus [8], which depends on the mean effective stress, the deviator stress, accumulated plastic strain and suction. Brown [9] defines the resilient modulus as the ratio of the repeated deviator stress to the recoverable (resilient) axial strain in a triaxal test.

This definition acknowledges the existence of non-recoverable (plastic) strains, but nevertheless these are ignored in the development of the subsequent models. There is hence a clear need to develop a pavement model which includes permanent (plastic) as well as recoverable (elastic) strains and which predicts the various type of pavement failure, such as rut formation, surface and subsurface slip and crack formation, all of which are governed by the strength properties of the pavement material. This need has recently been eloquently argued by Brown [9] in the 36^{th} Rankine Lecture to the British Geotechnical Society.
Finally it should be remarked that any kind of irreversible deformation occurring in the basecourse aggregate, be it due to particle rearrangement or particle crushing, are examples of plastic deformation and that these occur at stress levels well below ultimate failure. This is well recognised and modelled in the more sophisticated soil theories, such as critical state theories [10] or generalised "cap" models [11].

2. SHAKEDOWN MODELS OF PAVEMENTS

In an unbound pavement, the temperature and loading rate dependent, viscous surface bituminous layer plays no structural role, so that the pavement can be regarded as consisting of a number of rate-independent, elastic-plastic layers. The theory of the response of such structures to repeated loads is known as shakedown theory [12], which has been extensively applied to frame structures and pressure vessels and more recently to model the wear resistance of metal surfaces to repeated rolling and sliding loads [13, 14].
Four types of response of an elastic/plastic structure to cycles of loading are illustrated schematically here in Figure 1.

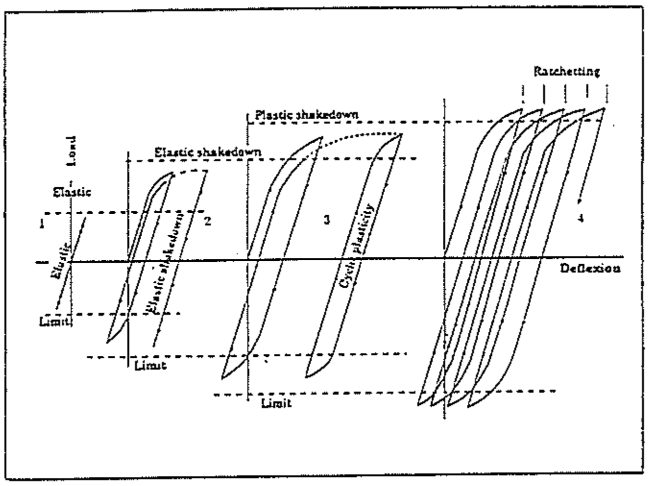

Figure 1. The four types of response of an elastic/ plastic structure to repeated loading cycles.

1) At sufficiently low loading levels the response is purely elastic and no permanent strains occur
2) At higher loading levels, the response is initially plastic but after a finite number of load application the response is purely elastic and no further permanent strain occur. When this happens.the structure is said to have "shaken down".
3) At still higher load levels, the ultimate response cycle may be of the form of a closed loop, analogous to low cycle fatigue; a state known as "cyclic or alternating plasticity".
4) Alternatively the permanent strain may go on increasing indefinitely; a response known as "ratchetting". In a pavement such a response would be demonstrated by the formation of a rut or the generation of slip planes.

The critical load level below which shakedown occurs, but above which permanent strains continue to occur is called the "shakedown load". The appropriateness of the shakedown load as the key design parameter for pavements seems first to have been suggested by Sharp and Broker [15], who estimated this load using a two-dimensional, plane strain, model in which the wheel is modelled by an infinitely long cylinder. These authors assumed that the material in the various pavement layers deformed plastically when the Mohr-Coulomb condition was achieved.

These calculations have been extended by Raad et al [16, 17, 18], but in all cases a two dimensional model was assumed and the estimate of the shakedown load was obtained by consideration of the stresses – the so called lower bound approach of Melan.
However Collins and Cliffe [19] showed that by employing the alternative kinematic approach due to Koiter, it was possible to consider much more realistic three-dimensional models in which the wheel load is applied over a circular area and the deformation is localised beneath the wheel. This analysis was extended to include layered models and two-wheel loads by Collins and co-workers in [20 - 24]. In all these calculation however failure was assumed to occur by subsurface slip in the direction of travel, paralleling the analysis in [14] for metal surfaces. More recently Collins and Boulbibane have been presented the results of calculations in which failure is assumed to be a result of rut formation [25, 26]. This development has made the analysis more realistic and is our principal concern here.

3. RUT FORMATION MODELS

It is not appropriate here to describe in detail the procedures used for calculating the shakedown load-these can be found in the references cited above. In essence the estimates are obtained by postulating some form of failure mechanism and equating the elastic and plastic rates of working in this mechanisms. The resulting value of the load is then an upper bound to the actual shakedown load, so that if a number of possible mechanisms are studied, that producing the smallest load estimate is the most likely to occur. Two broad classes of mechanisms are shown in Figure 2(a) and (b). In the first, failure occurs by slipping in the travel direction in a V-shaped channel. Such failure would be observed by the formation of surface shear cracks at the edges of the channel. Calculations demonstrate that in such failures the channel edge occurs very close to the edge of the loaded zone. Figure 2(b) shows one of many possible rut formation mechanisms in which the aggregate material moves downwards and sideways. A number of such mechanisms have been studied, c.f. [25, 26] and the optimum solution is that illustrated here. This mechanism bears some similarity to the well known bearing capacity solution for a footing [27], but is different in that the displaced material disturbs the surface very close to the loaded area, rather than over a region 2-4 times the radius of the loaded area as is the case in the foundation problem. It is to be emphasized that this model only predicts the onset of rutting and does not attempt to model the complete development of the rut.
The results are most conveniently presented in terms of a non-dimensional load factor

$$\mu = \frac{P}{\pi C a^2} \tag{1}$$

where P is the applied normal shakedown load, a is the radius of the circular loaded region and C is the cohesion of the basecourse.

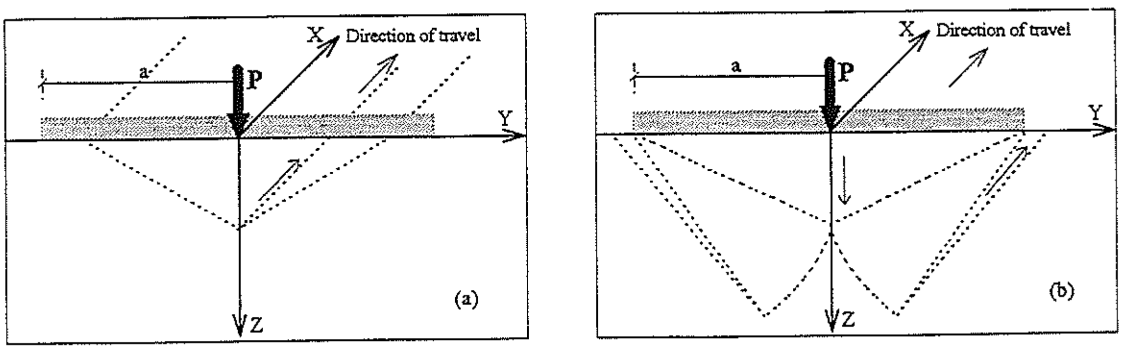

Figure 2. Slip and rut formation mechanisms.

In a uniform homogeneous pavement model, μ will be a function of μ_l the surface loading friction coefficient, ϕ the internal friction angle of friction of the basecourse material and the form of the loading distribution, here assumed uniform. However, It will not depend on the stiffness of the material. In a layered model, the dimensionless load factor will depend additionally on the *ratios* of the layer cohesions and stiffnesses, the values of the angle of internal friction and of Poisson's ratio in each layer and the ratio of the layer depth to the radius of the loaded area.

The variation of μ with μ_l for the two types of mechanisms is shown in Figure 3 for a single layer with $\phi = 35°$. It is seen that for $\mu_l < 0.2$ the rutting mode is the failure mechanism, whilst for larger values of μ_l the slip mode is predicted to occur. Figure 4 shows the variation of μ with ϕ for a purely normal load ($\mu_l = 0$) for the rutting failure mode.

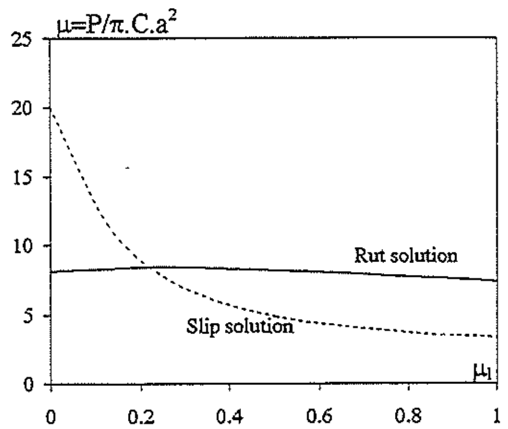

Figure 3. Variation of load factor with surface friction for slip and rut solutions.

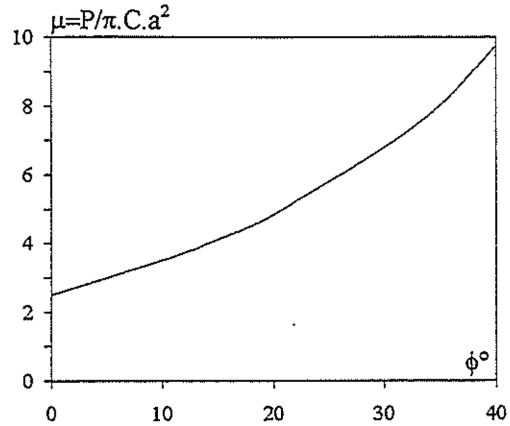

Figure 4. Variation of μ against friction angle for rutting failure.

Detailed calculations for two layered pavements have also been performed, some of the results have been presented in [25].

In reality rutting failures frequently occur when the pavement has a relatively weak subgrade. Some preliminary results of calculations for this situation are shown in Figure 5 in a form suitable for design purposes. The subgrade is modelled as a purely cohesive material with $\phi = 0°$ and C=40 kPa. The radius of the loaded area is 0.14m, the design load is 40 kN and the ratio of the basecourse to subgrade stiffness is 3. The graph shows the critical basecourse

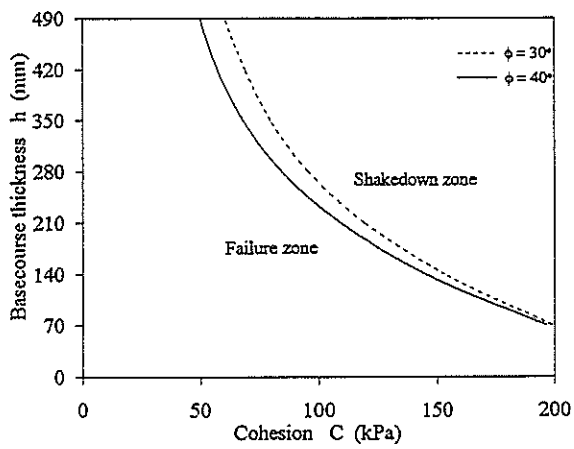

Figure 5. Variation of basecourse thickness against basecourse cohesion for various friction angles.

thickness at which shakedown will just occur for values of basecourse cohesions upto 200 kPa and basecourse friction angles of 30° and 40°. If a pavement is designed with a depth less than these critical values it is predicted to eventually fail.

4. DISCUSSION AND CONCLUSIONS

The models described above assume a Coulomb model for the various pavement layers determined by their cohesions and friction angles. Values of these have been determined experimentally by G.C. Duske in a triaxal apparatus under monotonic loading for a variety of New Zealand aggregate materials (Downers basalt, Stevensons greywacke, Hauraki greywacke, Kings andesite and Napier M5 greywacke) and are given in Appendix B to [24]. The subgrade can perhaps best be modelled as a perfectly cohesive material. The undrained shear strength can be estimated from its CBR value or preferably by direct field tests. The computation performed to date hence enable some preliminary design charts to be constructed. However, before some sample test track are constructed it is necessary to investigate a number of other aspects of the model.

We have already investigated a number of these, including the effect of dual wheel loads and the self weight of the pavement material. Other model modifications currently being investigated include in

(a) Three-dimensional failure modes in which the basecourse materials is displaced both forwards and sideways.

(b) The effect of lateral constraints or unsupported edges of the pavement and of lateral friction as occurs when vehicles take bends.

(c) The presence of pore pressures or suctions in the pavement by using the principle of the effective stress.

(d) Improving the material description of the basecourse aggregate and subgrade material, by using non-associated plasticity models and incorporating volumetric hardening/softening effects by employing more modern models such as those of critical state soil mechanics.

5. ACKNOWLEDGEMENTS

The authors are grateful to the PGSF and Transit New Zealand for financial support and to Mr Frank Bartley and Dr Ross Peploe of Bartley consultants for many helpful discussions.

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THE DAWN OF A NEW STONE AGE FOR AUCKLAND

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and R.G. Compton (Winstone Aggregates Ltd).

SUMMARY

The importance of a quality aggregate supply cannot be understated. In order to facilitate the projected growth of the greater Auckland Urban Area, the current high demand for aggregate in the Auckland Area is expected to continue. The urbanisation of areas historically used for aggregate supply is forcing the closure of many quarries. This paper discusses alternative sources of aggregate supply and the net result of supplying aggregate from more distant sources subject to increased environmental controls. The importance of identifying and protecting aggregate resources is highlighted and the potential impact of these changes for road planners, designers and contractors discussed.

1. INTRODUCTION

The purpose of this paper is to provide an overview of the changes that are expected to occur in the source and supply of aggregate for the greater Auckland Urban area. The importance of aggregate resources to the region is generally not fully appreciated, primarily because they are used by contractors rather than by the public. The greater Auckland urban area is one of the fastest growing population centres in Australasia and the availability and supply of aggregate for construction is vital to sustain this growth.

2. AGGREGATE DEMAND

In 1996, about 7.5 tonnes of aggregate was produced for every man woman and child in the Auckland Region compared to 6.5 tonnes in 1991[1]. Given Auckland's continued economic growth increasing aggregate demand is likely to continue. Estimates indicate that by 2001, up to 8.5 tonnes of aggregate will be produced per year for every person living in the greater Auckland Urban Area. Given that current consumption rates continue, the demand for aggregate in Auckland is expected to increase from about 7.5 Million tonnes per annum in 1997 to 12 Million tonnes per annum by 2020.

3. POTENTIAL SOURCES OF AGGREGATE

Six geological units within the Auckland Region and surrounding country contain rock generally suitable for roads, construction and concrete manufacture. These are:

- Albany Conglomerate
- Tangihua and Waitakere Andesite
- Auckland Volcanic Field
- South Auckland Basalts
- Greywacke

[1] University of Auckland, Department of Architecture, Property and Planning: 1996: "The infrastructure need of Auckland for the next five to ten years". Research Study prepared for Winstone Aggregates Limited, October 1996.

The general distribution of each of these broad geological units is illustrated on Figure 1. While extensive areas of these rock types are shown, it is important to understand that only a small proportion of these prospective areas are likely to contain resources of sufficient size and quality for economic extraction. Further, constraints such as land use, land ownership and natural and cultural heritage affect the viability of developing/operating a particular site.

4. AVAILABLE RESOURCES

4.1 Albany Conglomerate, Tangihua and Waitakere Andesite

Geological units to the north of Auckland that have the potential to supply quality aggregates comprise the Albany Conglomerate and the Tangihua and Waitakere Andesites. While the Albany Conglomerate is mapped over wide areas, much of this land is unsuitable for quarrying. Further, the Albany Conglomerate requires extensive processing to separate the rounded gravels from fine grained or "muddy" rock matrix. The rounded nature of the gravels and grain size limit aggregate use. Existing quarries in the Conglomerate include the Wainui and Coatesville Quarries.

Outcrops of Tangihua and Waitakere Andesite are limited in number and extent. The rock type is locally weathered and highly fractured. Quarries operating in this rock type include the Waitakere Quarry and the Flat Top Quarry.

Based on present knowledge of the area, the potential to develop other sites in these rock units is limited and the likelihood of identifying a major resource is low.

4.2 Auckland Volcanic Field

Basalt rock from the Auckland Volcanic field has historically been the major source of aggregate for Auckland since early settlement. The rock comprises strong fine to medium grained basalt, providing high quality aggregate. A number of large quarries have been developed in the volcanic field such as Lunn Avenue, the Wiri Quarries, East Tamaki and Roscommon, in addition to a number of smaller quarries located about the various volcanic centres.

In theory, in excess of 630 Million bank cubic metres of basalt rock remain available within the Auckland Volcanic Field. However, urban development and public reserves will prevent extraction and this volume of rock is unlikely to be available. For example, the Lunn Avenue Quarry at Mt Wellington was a green field site when in opened in 1936, but is now surrounded by urban development. Other areas such as Matukuturua, a large basalt scoria resource in the Wiri area, which have been zoned and set aside specifically for quarrying for over 25 years, are also under threat. Matukuturua has been designated as open space and a waahi tapu protection area in the new proposed Manukau District Plan. Ironically, the present heritage value of the land is the result of it having been set aside in private ownership for future quarry use.

It is expected that no new quarry development will be permitted within the Auckland Volcanic Field and that the existing quarries within the urban area will close within the next 2 to 6 years.

4.3 South Auckland Basalts

The South Auckland Basalts include lava and scoria cones associated with the many eruptive centres in the Papakura and Franklin Districts. The basalts and scoria from these many eruptive centres have the potential to provide a significant volume of high quality aggregate. Quarries in the South Auckland Basalts include the existing Ridge Road and Bombay Quarries and the proposed Quarry at Pokeno. However, the South Auckland Basalts are several hundred thousand years older than the Auckland Volcanic Field and have been subjected to a longer period of weathering. At many locations the basalts are deeply weathered and the depth to suitable rock makes quarry development uneconomic. Further more, many of the eruptive cones have been identified as having distinctive native vegetation and habitat value.

While the South Auckland Basalts have the potential to supply a large volume of aggregate, rock weathering and land use constraints can be expected to limit the number of locations where economically recoverable aggregate resource may exist.

4.4 Greywacke

Greywacke is a commonly used term to describe hard inter bedded mudstone, siltstone, sandstone and conglomerates of Jurassic and Triassic age throughout New Zealand. Greywacke rock underlies much of the Hunua Ranges to the south east of Auckland. The siltstone and sandstone rock units have the potential to make high quality aggregate depending on the degrees of weathering and fracturing of the rock. Existing quarries extracting Greywacke sandstone include the Hunua, Whitford and Drury Quarries.

Large areas of Greywacke rock exist to the south east of Auckland, but much of this area is dominated by mudstone rock sequences. Further a large proportion of the Hunua ranges is regional park and protected by municipal water supply catchment. Areas outside of the regional park are highly desired for both rural residential and intensive farming, and have been subdivided into many high value lots requiring a quarry developer to deal with many landowners.

Potentially large reserves of aggregate exist within the Hunua Ranges subject to site specific geology and land use restrictions.

Greywacke reserves to the north of Auckland could also be considered. However, transport distances are large and it would be generally uneconomic to supply Auckland from these sources with the possible exception of the North Shore and Albany areas.

5. CHANGES TO AGGREGATE SUPPLY

5.1 General

As noted above the Auckland Region has experienced rapid growth over recent years and the demand for aggregate has seldom been higher. The most important single source of aggregate within the Auckland Urban Area over recent years has been the Lunn Avenue Quarry at Mt Wellington, supplying between 1.2 and 2 Million tonnes annually. However, the Lunn Avenue Quarry may close by the end of 1999 unless an 18 month to 3 year extension is granted as part of a resource consent application. Other major quarries that have supplied the Auckland market such as Wiri and Roscommon are also reaching the end of their economic life.

It is anticipated that the existing accessible basalt and scoria resources within the Auckland Urban area, with the exception of Three Kings and a hand full of small operations in Manaukau City, will be exhausted within two to six years. The existing large quarries at Hunua, Drury and Bombay will be required to make up the bulk of the shortfall of about 4 Million tonnes per annum, at least in the short term. New quarries such as the proposed Winstones Quarry at Pokeno, once developed and in full production, will contribute significantly towards meeting the aggregate demand. Given that Pokeno and expansion at Wainui and Flat Top Quarries replace the shortfall caused by the closure of Lunn Avenue, by the year 2005 a further five quarries producing some 500,000 tonnes of aggregate will be still be required, or production from existing quarries further increased.

However, it is becoming difficult to find prospective aggregate resources in the Auckland Region and surrounding country which are not constrained by planning and environmental conflicts. The trend for new quarry sites becoming more distant from the Auckland Urban Area centre is inevitable.

5.2 Transportation

As aggregate sources become more distant from Auckland, the cost of transportation will rise. Based on data supplied by Winstone Aggregates Ltd, the cost of transportation from Auckland based quarries currently represents about 30% of the average delivered cost in the Auckland Region. If the pending shortfall in aggregate is sourced at distances from the market equivalent to those of Hunua and Pokeno, the cost of road transportation will rise to about 45% and 55% of the average delivered cost respectively. In terms of today's prices, if all aggregate were delivered by road from Pokeno to central Auckland, the delivered cost would increase by about 58%.

With the exception of the proposed quarry at Pokeno, few quarry sites are located close to rail and road transport is expected to be the principal mode of transportation. After cost, the next major impact of transporting aggregate by road from more distant sources will be on our roads. It is estimated that currently about 60 to 70 truck and trailer units carrying aggregate from outside of the region travel into Auckland per day. If all the projected aggregate demand in the year 2010 is supplied from outside of the region, up to 360 fully loaded truck and trailer units will be travelling up and down Auckland's motorway each day.

5.3 Environmental Issues

The demands of the Resource Management Act (RMA) and the environmental controls that form an integral part of any modern quarry development, have required many quarry operators to improve operations and environmental management procedures. Blast vibrations, dust, sediment control, traffic movements and rehabilitation/restoration are issues that are required to be managed and controlled more rigorously than in the past.

Until the introduction of the RMA the costs associated with improved environmental controls and those associated with opening a new prospect have tended to be taken as incidental, and have not impacted significantly on the cost per tonne. In more recent times, that is since the introduction of the RMA, the costs of environmental compliance and enhancement, and those associated with consent applications and renewals for existing sites have been significant. As more new quarry sites are developed, these costs are likely to be substantial. Further the time required to gain consents needs to be considered. Assuming a new quarry site is within an area where quarrying is an approved activity, the lead time to gain the required consents could be up to three years. In an area where appropriate land use zones and control are not in place, then it could take up to 10 years to gain consents, if it were at all feasible.

6. CONTINUITY OF SUPPLY

The existing quarries to the north, south and south east of Auckland in conjunction with the proposed quarry at Pokeno should be able to supply Auckland's aggregate needs over the next few years. However, as noted in the above sections, aggregate demand is expected to rise and in the long to medium term, there will be a need to develop additional quarrying sites. To ensure continuity of supply in an environment where new quarry developments are limited and generally opposed by landowners, careful planning and utilisation of these resources is required. It is critical that at District and Regional Council level, the importance of protecting aggregate resources is understood and included in legislation to avoid severe increases in costs to consumers.

The utilisation of alternative aggregates by designers and contractors to maximise the use of marginal aggregates should continue to be investigated. For example, the use of stabilised road sub base and base course layers as an alternative to M4 aggregate. The use of alternative aggregate materials in road construction is being facilitated by recent changes to Transit NZ specifications on road construction.

7. CONCLUSION

The current high demand for aggregate in the Auckland Area is expected to continue with demand increasing from about 7.5 Million tonnes per annum in 1997 to 12 Million tonnes in 2020. The urbanisation of areas historically used for aggregate supply is forcing the closure of many quarries. Within two to six years only Three Kings Quarry and one or two smaller quarries are expected to remain in operation within the Greater Auckland Area. Increased production from existing quarries and new quarries located to the north, south and south east of Auckland will be required to meet the projected shortfall in aggregate supply. The potential for a new large quarry north of Auckland is limited apart from expansion at existing quarries.

The net result of supplying aggregate from more distant sources will be a significant increase in transportation costs. Costs are also expected to rise in response to increased environmental controls required to operate a quarry and for new prospects to recover the cost of land purchase and the consent process.

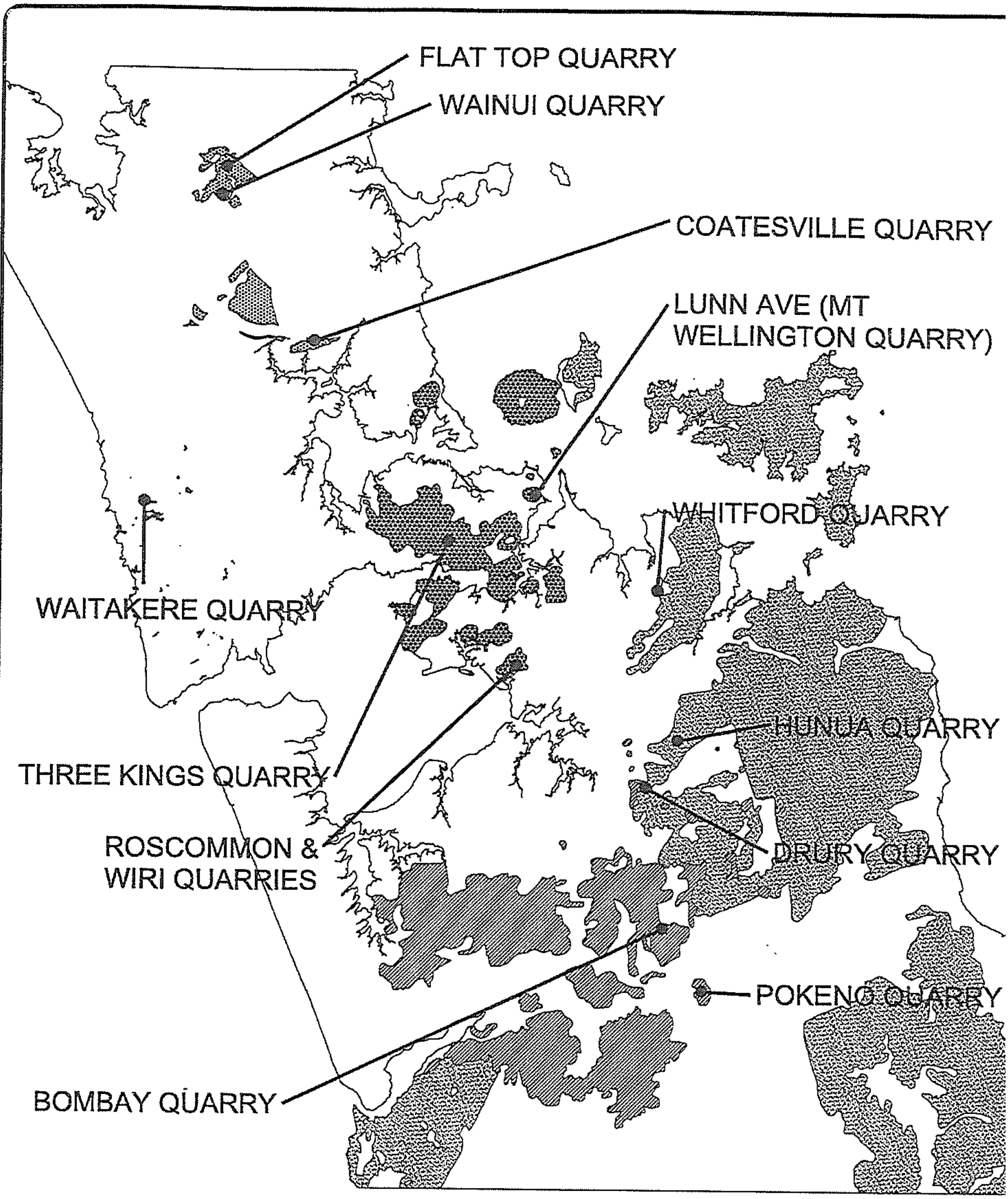
It is concluded that:

- Aggregate sources to the south and south east of Auckland need to be identified and protected from further urbanisation and inappropriate land use designation

- Quarrying companies, local authorities and the users of aggregates need to ensure that aggregate resources are maximised

- Failure to do this may result in history repeating itself in 10 to 20 years time leading to a short fall in supply resulting in the sourcing of aggregate from even further afield with corresponding increases in cost.

Road network planners need to factor into their planning the projected increases in the cost of aggregate supply in the Auckland Urban Area. Road designers, engineers and construction companies have a role to play in maximising the utilisation of product from the quarries. Recent changes to Transit NZ Specifications provide opportunities to use alternative aggregate sources to offset the projected increase in aggregate cost.



Key

- Albany Conglomerates
- Waitakere/Tangihua Andesite
- Auckland Volcanic Field
- South Auckland Basalts
- Greywacke

Illustration of Potential Sources of Aggregate in Auckland Region
Figure 1

Notes:
1. Drawing digitised from NZGS geology maps – "Waitakere", "Helensville & Whangaparoa", and "Helensville" (1989; 1:50,000) and "Auckland" (1987; 1:250,000). Extent of all geology approximate only.

ASPECTS OF EARTHWORKS DESIGN IN PUMICE BRECCIA STATE HIGHWAY 5 UPGRADING, ROTORUA

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Worley Consultants Ltd., Auckland

SUMMARY

Recently completed upgrading of State Highway 5 at Earthquake Flat south of Rotorua involved major excavation and filling in geological materials known locally as pumice breccia. Design of cut slopes in the breccia took advantage of its favourable geotechnical characteristics.

The deposit is an unweathered sequence of massive, unwelded ignimbrite flows (Earthquake Flat Ignimbrite) overlain by and interbedded with shower-bedded airfall ash (Rifle Range Ash). More recent ash beds, some weathered, mantle the topography.

The breccia is free-draining, compact and relatively uniform in distribution and geotechnical properties. Grain size is predominantly well graded, gravelly, medium and coarse sand with blocks of rhyolite rock to one metre in diameter. The deposit forms stable natural slopes and sub-vertical cuts stand to heights of about 30m.

Stability analyses aided the selection of a cut profile comprising 10m high batters and wide benches over total slope heights of 40m to 50m. Batters are steep to minimise potential erosion and benches are of sufficient width to allow clearing of slope debris by machine. The breccia forms a strong fill and can be handled and placed in wet weather.

INTRODUCTION

The volcanic deposits of the central region of the North Island present unique problems for road construction. The materials range from rock (lava and welded ignimbrite) to loose sand and silt (airfall ash), commonly in complex associations characterised by rapid vertical and horizontal changes in material types. Weathering introduces further complexity. Planning engineering works, particularly earthworks, in such terrain must therefore take account of the potential variability of earth materials and their probable wide range of geotechnical properties.

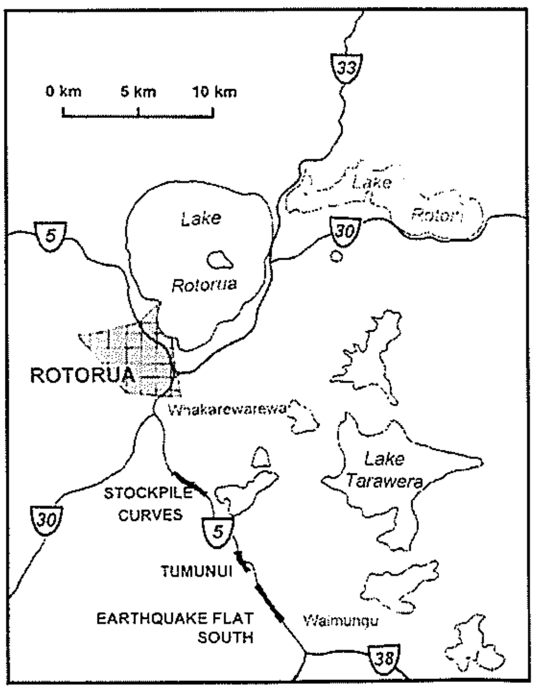


Figure 1 Location Map

Upgrading of State Highway 5 at Earthquake Flat, 15km south of Rotorua city (Figure 1), involved easing curves, road widening and constructing sections of new road to improve the alignment through a series of dangerous curves. The designers were Sigma Consultants of Rotorua and the owners are Rotorua District Council.

The project is located wholly in volcanic terrain and as such, all of the potential problems associated with that terrain were considered when scoping geotechnical investigations. Preliminary engineering geological mapping however, identified an extensive and apparently homogeneous pumiceous gravel as the main material type likely to be encountered in earthworks. This 'pumice breccia' is well known in the Rotorua region, principally for its consistent and advantageous geotechnical properties. Existing road cuts and borrow pits illustrate that the breccia stands sub-vertically to heights of 20m to 30m and the material is sought after as fill. Thus, contrary to early expectations of difficult conditions, the project was found to be located in volcanic terrain comprising a single material type having consistent and beneficial engineering properties.

This paper documents the development of geotechnical recommendations for design of cuts and fills in the pumice breccia. The sections of SH5 that were upgraded are known as Stockpile Curves, Tumunui and Earthquake Flat South (Figure 1).

ENGINEERING GEOLOGY

Geological Setting

Landforms in the project area are dominated by the Earthquake Flat Tephra Formation (previously the Earthquake Flat Breccia Formation, Froggatt and Lowe, 1990), a sequence of unwelded, pyroclastic flows (Earthquake Flat Ignimbrite) and interbedded airfall tephra (ash) units (Rifle Range Ash) erupted about 50,000 years ago from explosion craters centred on Earthquake Flat (Figure 2). The deposits form low-angle fans extending from the explosion craters to cover an area of about 110km^2 at a typical thickness of 50m to 60m.

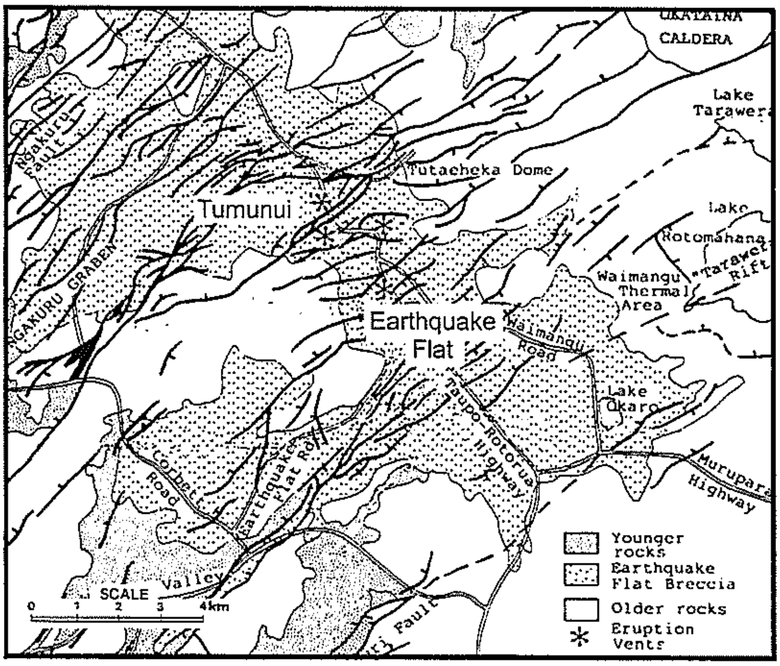


Figure 2 Fault Map
After Nairn and Hull 1987

The ignimbrite units comprise lapilli and blocks (coarse sand to cobble sizes) of pumice, slightly vesicular rhyolite and rhyolite rock fragments, in a sandy matrix of pumice, glass and crystals of quartz, feldspar, mica and heavy minerals. The rhyolite rock fragments are up to 1m in diameter. The ignimbrite units are generally massive and poorly sorted (well graded) but include weakly bedded components in places. The Rifle Range Ash is distinctly shower bedded.

Erosion processes have dissected the low-angle plateau to form steep-sided, flat-bottomed valleys and flat-topped ridges that have concordant heights. Post 50,000 years BP airfall ash units mantle the topography to depths of 2m to 3m. Northeast trending active faults of the Taupo Fault Belt cross the area (Figure 2) displacing the breccia and, to differing degrees, the overlying ash beds. Mapping and trenching on the nearby Paeroa Fault indicates fault displacements have occurred in the past 1800 years (Nairn and Hull, 1986). Fault traces are easily identified and they range from pronounced scarps (10m to 15m high) to small ripples traversing slopes.

Earthquake Flat itself is a roughly circular, caldera-like depression that has a flat floor and internal drainage. Surrounding slopes are steep, particularly those to the west abutting Tumunui rhyolite dome, and to the north along the road realignment. A major part of the engineering works involved cutting through the northern wall of Earthquake Flat (the Tumunui Cut) to achieve an acceptable horizontal alignment.

Slope Stability

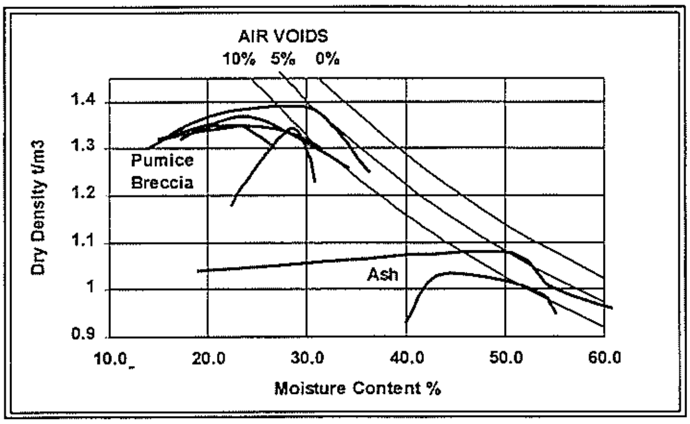
Natural slopes in the breccia typically are planar and steep (30° to 45°). Instability is minimal and generally limited to the mantle of ash beds. The numerous faults that cross the area do not appear to promote significant instability.

Road batters cut sub-vertically to heights of 15m in breccia perform satisfactorily although the materials are susceptible to erosion particularly where the breccia is loose such as in the more prominent fault zones. In general the expression of faulting in cut slopes is limited to a disturbed zone typically no more than a few millimetres in thickness. In addition, it is fortuitous that fault strike is SW-NE and that SH5 is mainly normal to strike. Both conditions are favourable to stability.

The ash in cuts is susceptible to frost heaving and ravelling but is relatively stable at heights of less than about 3m and when protected from running water. Above heights of 3m, sub-vertical cuts in ash have marginal stability.

The breccia has been borrowed extensively over the years for road construction leaving a number of borrow pits in the area. Most pits have sub-vertical walls that range in height to 30m (e.g. Red Quarry one kilometre south of Earthquake Flat). There is little evidence that the high walls are prone to collapse.

Some Geotechnical Properties of Breccia



Natural Water Content %	NZ Standard Compaction		Soaked CBR %
	OWC %	MDD t/m3	
BRECCIA			
23.7	*26.0*	*1.36*	*36*
ASH			
53.8	*46.5*	*1.06*	*6*

Figure 3 Material Properties

The breccia is a well graded gravely sand with a very low proportion of fines. Typical properties determined by laboratory tests are summarised on the table in Figure 3. Results for ash samples are included on the table for comparison.

Standard compaction tests (Figure 3) illustrate that the material can achieve near maximum density over a wide range of water contents (20% to 30%). That range is similar to the range of field water contents determined for drillcore specimens and inspection pit samples. Measured soaked CBR values average 36% for compacted samples. These results support the experience of local contractors that the breccia is an ideal material for fill construction as it requires little conditioning and can be handled in wet conditions and it has sufficient strength to be used as a lower sub-base layer in pavements.

Specimens of breccia core were sampled from one of the drillholes at the Tumunui Cut for assessment of strength. Effective stress strength parameters determined by consolidated, undrained triaxial testing are cohesion of 20kPa and friction angle of 44^0. The friction angle is similar to a value reported by Pender (1996) for pumiceous sand. The *in situ* dry density of the drillcore used for the triaxial test is 1.58t/m^3 which is about 15% above the maximum dry density (NZ Std Comp.).

CUT SLOPE DESIGN CONSIDERATIONS

Existing Practice

Design concepts for cut batters in granular volcanic materials in the mid North Island have for a considerable period been evolving towards systems of steep batters and wide benches. Steep batters minimise the exposure of slopes to rainfall while carefully graded benches allow for the collection and disposal of run-off away from batter surfaces. An objective of this approach is to prevent water running over unprotected and erodible surfaces and provide areas for the accumulation of slope debris.

Batter heights of existing cut slopes typically are in the range of 5m to 7m for slopes with overall heights of 20m to 30m. Individual slopes to heights of 10-12m have been cut successfully without benches.

This approach was adopted at Earthquake Flat for design of cut slopes. The intention was to utilise the favourable geotechnical properties of the breccia to minimise both cut volumes and the potential for erosion of slopes during heavy rainfall.

Earthquake Flat

The general design philosophy was refined by stability analysis, the aim being to maximise batter height (to minimise the number of benches) and maximise batter gradient. In addition, the road designer (Sigma Consultant, Rotorua) wished to avoid unnecessary shading of the road in winter to reduce the icing hazard. This latter objective essentially required easing cut slope gradients to minimise shading, an objective that was somewhat in opposition to the former two.

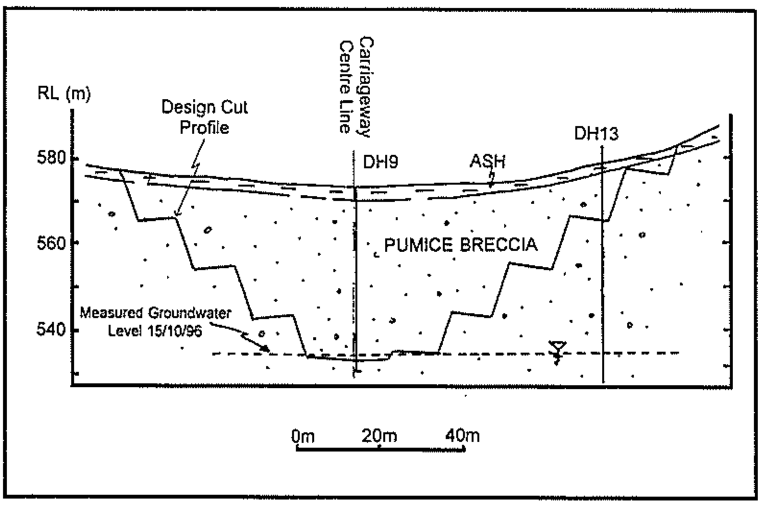
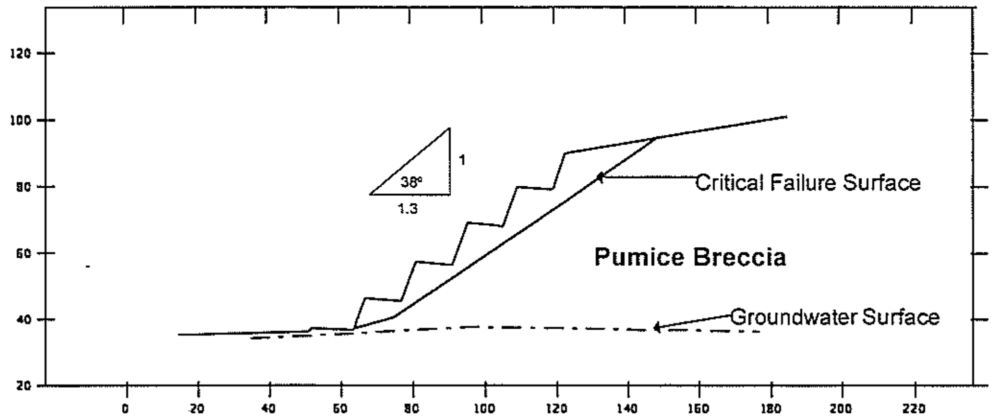


Figure 4 Conceptual Design Profile

The design concept that was selected is based on 10m high batters at 3 vertical to 1 horizontal (3:1) and 10m wide benches with a cross-fall of 1v:10h to the back of the bench (Figure 4). Selected ash was to be placed on benches to aid rapid re-vegetation. A 5m to 10m wide maintenance strip at the base of the slope allows for accumulation of debris spalling off batters and for subsequent collection of the material with minimal traffic interference. In addition the maintenance strip has the benefit of reducing shading. The benches also are debris accumulation areas and the 10m width allows access by machine to clear away debris.

The Tumunui Cut illustrated schematically in Figure 4 is typical of the profile adopted for cut slopes in pumice breccia. The cut is through the north wall of Earthquake Flat depression and involves overall cut heights of about 40m. Fully cored drillholes with Standard Penetration Tests suggested dense breccia for the full height of the cut except for the ash cover of about 3m. Measured static water level in drillholes is very low and simple drillhole permeability tests gives results of near 10^{-4} m/s. These groundwater observations are consistent with the numerous dry valleys and internal drainage in the area.



Horizontal Ground Acceleration	Static Conditions 0.0g	Serviceability 0.046g	Ultimate 0.275g
Factor of Safety Against Failure	1.8	1.6	1.1

Figure 5 Stability Assessment

The stability of the Tumunui Cut was modeled using the program GALENA and Sarma's method of analysis. Earthquake loadings were selected using the loading code, NZS 4203:1992, as a guide. A Building Classification Category IV was assumed based on a subjective assessment of the level of importance of the cut, and equivalent Ultimate Limit State and Serviceability Limit State accelerations were derived. Minimum factors of safety were calculated for the failure surface illustrated on Figure 5. The relatively shallow failure surface suggest that the 10m high batters are at about their maximum safe height.

CONSTRUCTION CONSIDERATIONS

Consent conditions for earthworks generally require contractors to adopt well defined procedures and practices to minimise uncontrolled run-off and sediment discharge to waterways. In most cases this requires that earthworks are halted in wet weather and materials and fill surfaces are dried prior to earthworks re-commencing. Such instances commonly involve materials with a significant proportions of fines or that are loose and sensitive.

The geological materials at Earthquake Flat are significantly different in their geotechnical properties compared to many volcanic deposits in the North Island. The compact breccia is permeable, resistant to erosion, and has minimal fines content. It can be handled and placed in fills during reasonably wet conditions with little drying. The low fines content results in minimal sediment in run-off.

The handling versatility of the pumice breccia, together with its consistent distribution and stability, are clearly indicative of an 'ideal engineering material'.

ACKNOWLEDGMENTS

Permission from Rotorua District council and Sigma Consultants to publish this paper is gratefully acknowledged.

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LANDSLIDES AND ROADING: EXAMPLES FROM AUCKLAND, THE TAUPO VOLCANIC ZONE, RANGITIKEI VALLEY AND THE SOUTHERN ALPS

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SUMMARY

A variety of rock masses and soil deposits demonstrate properties which lead to slope instability. These properties include:- sensitivity as a result of a fragile microfabric; extremely weak and highly continuous defects such as bedding-parallel clay seams; very thick residual soils and regolith as a result of weathering; potentially unstable andesite volcanoes; hydrothermally altered and weakened volcanoes; slopes loaded by rapid deposition; and, extreme dilation leading to slope failure of ridge crests and slopes in greywackes.

These geotechnical conditions have either been the cause of slope failure across major roads in New Zealand or they present hazards to existing roads and potential new roading corridors.

1. INTRODUCTION

Some examples are presented of slope failures in a range of engineering geologic terrains, with particular reference to those failures which have affected existing roads or threaten potential new routes and new alignments. The selection discussed is by no means comprehensive but is chosen to highlight geotechnical conditions which are common, rather than unusual. These conditions are at least sufficiently widespread, or individually occupy large enough areas, to be regarded as characteristic of a particular geologic terrain and not "one-off" peculiarities.

In each case, a considerable depth of weathering, or a considerable extent of hydrothermal alteration or the presence of pervasive defects are seen to be important factors. In others there appears to be potentially unstable geomorphic development, as a consequence of oversteepened slopes.

Some notably unstable terrains are not given the detailed assessment they merit, for instance the schist rockmasses of Otago and Westland, the granitic and other rockmasses of Fiordland and Nelson, the clay shales of East Coast Deformed Belt and a whole host of Quaternary deposits.

The intention of this paper is not a comprehensive review but an attempt to highlight engineering geologic conditions of significance to slope failures affecting roads in certain parts of Auckland, the Taupo Volcanic Zone and the Southern Alps (Figure 1).

Preliminary engineering geological models, roughly equivalent to "tentative site models", are put forward in an attempt to explain the instability. These are somewhat general and simplified but may serve as a prompt for identifying geological problems in future investigations. No particular order of importance, magnitude or prevalence is indicated by the order in which these are discussed.

2. CLAY SEAMS IN WEAK ROCK TERRAIN

The "soft" (weak) mudstones, shales, and sandstones of the central eastern and northern North Island are notorious for large block slides and debris slides. Bedding parallel clay seams, crush zones and clay coated fractures have been identified in these rocks as having provided basal rupture surfaces for slope failure. Well known examples which have affected roading include the Utiku slide Rangitikei Valley (Stout 1977, Thompson 1981), Mill Road area Auckland (Kermode, 1991), Howick-Whitford area (Tilsley, 1993), Auckland region generally (Williams, Prebble, Mansergh and Toan 1996).

Similar geotechnical conditions were found at Abbotsford, Dunedin (Coombs and Norris 1981), Hawkes Bay (Pettinga 1987) and have been reported throughout soft weak rock elsewhere in New Zealand (Prebble 1995 (a), Bell and Pettinga, 1988). Fell, Sullivan and MacGregor report similar features elsewhere.

2.1 Auckland Region

Extremely weak clay seams, crushzones and clay coated fractures have been observed in the Southern Landslide Zone, Auckland (Prebble, 1995 (a)) beneath potential slide blocks on which there has been, at most, minimal translation) Clay seams have also been found beneath slide blocks which have moved several metres and beneath debris which indicates several tens of metres of sliding and disruption. Subsurface data and surface mapping has provided tight control on the bedrock structure and stratigraphy and the stratigraphy of the slope deposits. Continuity, waviness, stepping and thickness of the seams and zones has been determined.

Unweathered to slightly weathered fractured rock contains the seams and crush zones. Their detailed structure (e.g. diverging and rejoining splays) also points to a flexural slip origin. Broad open folding with shearing of the contact between sandstones and mudstones is thought to have generated the bedding parallel clays seams and crush zones. Differences in strain characteristics between the adjacent different beds, under high stresses, creates slip which crushes the rock. The end results are crush zones and clay seams (Hutchinson 1988 and 1995).

Lateral freedom, (resulting in side shears) of the landslide blocks is provided by sets of steeply dipping fractures and faults. Wedge failures are also found, bound by intersecting, moderately dipping fractures and faults.

Much of the Southern Landslide Zone is affected by block sliding, wedge slides and debris slides. In some areas up to 70% of the land is affected, to a depth 8 to 20 m. Even in moderately to highly weathered mudrock of this zone, defects are observed to form the rupture surfaces and boundaries of slides. Hence there is a primary geometric control by defects. There is also a considerable depth of weathering, and hence residual soil (15 m). The result is an equally thick mantle of slope debris where sliding has progressed to the point of disruption. Considerable dilation of fractures, on slopes and in the relatively undisturbed residual spines of ridges, allows deep penetration of groundwater. This provides unconfined, cascading fractured rock aquifers perched on top of clay seams. In other situations, fractured rock aquifers are confined beneath clay seams. The defects provide steeply oriented pathways for groundwater, connecting to the gently dipping fractured sandstones which are also effective aquifers.

During earthquake, such as the 2000 year return period earthquake for Auckland, the Southern Landslide Zone has a high hazard potential for slope failure, with some 20% or more of the slopes expected to fail (Williams, Prebble and Toan 1997). Rain induced slope failures in the Southern Landslide Zone (for a 100-year Cyclone, or similarly intense prolonged winter rainfall) are expected to be much fewer than earthquake induced landslides for the steeper slopes of the zone (5 to 20% fail). The somewhat less steep slopes are expected to experience very much less instability (0.5 to 5% fail). It would appear that a significant earthquake is required to induce widespread landsliding in the zone. However, excavation for roads or other earthworks could either induce new landsliding or reactivate existing ones.

Upwards injection of very soft, soupy, near-liquid clay is observed in fractured rock sitting directly above clay seams. This is interpreted as a "pumping" effect of basal rupture clay seam material into fractured slide blocks above, during earthquake. This phenomenon is noted in several places in the Waitemata Group weak rock in Auckland.

3. REGOLITH SLIDES

Up to 30 m of residual soil and slope deposits overly greywacke rock masses in the Auckland region and up to 20 m of residual soil and slope deposits overly Waitemata Group weak rock and its Volcanogenic equivalents. The residual soils span to CW - HW (Completely to Highly weathered) grades and are dominantly clays, or clayey silts sands mixtures. (Williams, Prebble, Mansergh and Toan 1996).

On steep slopes in these soilmasses regolith slides have been observed overlying greywacke at Kawakawa Bay and in the Hunua Ranges. In the Waitekeres they overly andesitic volcanics and residual clays persist throughout the Waitemata Group weak rock terrain.

4. SENSITIVE RHYOLITIC SILTS

Highly sensitive white silts, of rhyolitic composition, are found in Auckland, Bay of Plenty, Waikato and the Rotorua-Taupo region. The silts can be either primary volcanic deposits (tephra) or reworked material which has been deposited by water.

The silts are within terrace deposits of Quaternary age and so form part of thicker, layered soil masses in which there are permeable sand lenses, channel-fill sands, valley-fill ignimbritic deposits, tephra and organic layers. The complex superposition of these materials results in a sequence of permeable leads referred to as ribbon aquifers and aquifer tongues (Prebble, 1986), perched on top of the sensitive rhyolitic silts. The Ruahihi Canal failure (Hatrick, Howarth, Galloway and Ramsay 1982) showed that these soil masses are prone to collapse and rapid slide-flow failure.

Examples of slope failures in road cuts in rhyolitic silts have been seen in the northern Waikato, the Hunua Ranges and Auckland's eastern suburbs.

It is possible that weathering of the silts results in a delicate, fine grained, granular to skeletal microfabric and that this results in the sensitivity, which has been measured by field shear vanes at approximately 30 to 60 in the Ruahihi area (Prebble, 1986).

Several different rhyolitic tephras are now recognised in the Auckland region. Terrace deposits of Late Pleistocene and Holocene age are the geologic and topographic units in which sensitive rhyolitic silts may be anticipated.

5. DEBRIS FLOWS FROM ANDESITE VOLCANOES

Debris flows of two distinct types are observed on andesite volcanoes generally, and on the Tongariro District volcanoes as a specific example. These types are clay-rich debris flows and granular, sandy debris flows.

5.1 Granular, sandy debris flows

Sector slope collapse, pyroclastic flows, crater lake eruptions and andesitic tephra generally are all capable of producing voluminous, fast moving debris flows as a result of the eruption products mixing with water. Commonly referred to as lahars, these debris flows are relatively clay-poor and coarse grained. As they travel away from their source area on the cone, dilution usually transforms the flow to some extent and eventually may result in a hyperconcentrated stream flow. Large debris flows, from a sector collapse such as Mt St Helens 1980 (Schuster 1983) can be sufficiently fast and devastating to be regarded as a large debris avalanche. Recent research possibly indicates that only the larger coarse grained debris flows from andesite volcanoes leave a significant deposit in the ring plain. Hence the best guide to the frequency, magnitude and extent of volcanic debris flows is that of historic observations plus the geological record rather than stratigraphy alone. Ring plain stratigraphy will record only larger debris avalanches and flow deposits.

5.2 Clay-rich debris flows

Slope failure of clay-rich, hydrothermally altered materials on andesite cones gives rise to a very clay rich slug (cohesive mass) of debris which may travel several kilometres out into the ring plain without dilution - and is largely unconfined in its passage and spread.

Examples of these are recorded from Kakaramea Volcano, above the village of Waihi and State Highway 41 at the southwest corner of Lake Taupo (Fig 2). The route corridor of SH 41 crosses a stream gully which has been the main channel for large clay-rich debris flows (possibly debris avalanches) in 1846 and 1910. Continuing instability is evident in the source area for the debris which is the hydrothermally weakened, steep, steaming ground of the Hipaua geothermal field on the Waihi Fault. This locality is marked by arcuate head scarps, slope movement benches, powerful fumaroles, soft clay, intersecting lineaments (older faults) and young NNE striking faults of the Taupo Volcanic Zone. Dilated fractures in rock along the edge of the main fault scarp and landslide headscarp attest to the ongoing instability of this area.

Similar, but very much larger, unconfined cohesive debris flow deposits are seen in the northwest ring plain of Tongariro Volcano. The Te Wharau Formation is a large clay-matrix andesite debris flow or debris avalanche deposit spread over 65 km^2 and is at least 0.35 km^3 in volume. Emplaced about 55,000 to 65,000 years ago, there are other younger deposits of a broadly similar nature and size which were emplaced between 7000 and 100,000 years ago (Palmer, Alloway and Neall, 1991).

An important point is that smaller, channel-confined, but potentially destructive, debris flows and avalanches take place much more frequently than the large ones for which deposits exist in the geologic record. These smaller debris flows are essentially show-string flows (Palmer, Alloway & Neall 1991). The Te Whaiau Formation is considered to have arisen from the northwestern side of Tongariro Volcano in hydrothermally altered and weakened material. Geothermal activity and extremely weak clayey material is probably widespread on Tongariro volcano. The carapace of younger lavas and tephra now mask where the debris avalanche came from. Geomorphology and geothermal activity indicate that the potential for further slope failure and debris avalanches still exists. The Te Whaiau Formation overwhelmed an area which includes several km of State Highway 47 and linking side roads. (Fig. 2).

6. SLOPES LOADED BY RAPID DEPOSITION: VOLCANOES

In urban Auckland many tens of scoria cones and lava fields of basaltic composition have been deposited on top of other materials in the last approximately 100,000 years. The volcanic cones and associated tuff rings grew quickly, it is envisaged, so that existing slopes in residual clayey soils were suddenly overloaded with volcanic tephra.

At Orakei Basin volcano, residual clay developed on Waitemata Group weak rock and was loaded with several m of tephra. The toe of the slope is continuously eroded by tidal currents. A landslide of tuffaceous debris overlying soft clay occupies a whole portion of the slope which is crossed by an important suburban arterial road (Kepa road). Translational sliding in the top few to several metres, in the residual clay, is considered to be the appropriate model.

7. EXTREME DILATION OF RIDGE CRESTS LEADING TO SLOPE FAILURE

Several localities in the Southern Alps show extreme dilation - up to many tens of cms - of pervasive defects such as bedding parallel fractures and dominant joint sets. Defects up to a metre wide and several tens of metres at least of lateral continuity, form a well developed pull apart zone along the crest of the Sealy Range, Mt Cook region. Dip slopes on one side of the openly fractured head trench are failing by complex sliding and toppling. The back slope is failing by deep seated toppling. (Prebble, 1995a). Ridge crest trenches and "rift" zones are widespread in fractured rock of the Southern Alps, but not all are clearly marked. Ridge crest collapse, small scale toppling, and rock fall debris tend to obscure the bedrock, making recognition of the deeper seated failures difficult. However, some localities provide evidence of probably deep seated toppling and sliding (Prebble 1995b). It is postulated that collapse of large topples and slides, on through-going bending surfaces and basal ruptures, leads to large debris avalanches which affect roads downslope.

Paterson (1996) has discussed the potential for toppling to initiate large scale rock avalanches above the zig-zag bends at Arthur's Pass on State Highway 73. He specifically refers to the openly dilated rock mass in the head scarp of the existing rock avalanche deposit and draws attention also to ridge rents (antislope or uphill facing scarps) above the Otira Gorge section of the highway. Whilst the gravity-faulting model of Beck (1968) and the sagging model (Hutchinson 1988) are put forward as possible explanations for the ridge-vents, toppling is suggested here as the most likely mechanism to have created these uphill facing scarps and numerous others elsewhere in the region and throughout the Southern Alps.

Toppling can lead to through-going bending surfaces (Cruden 1989, McAffee and Cruden 1996) and can affect both dip slopes and scarp slopes (Cruden 1989, Cruden and Hu 1994, Prebble 1995 (a) and 1995 (b).

Recent research indicates that large debris aprons have formed downslope of toppling and sliding ridge crests and upper slopes on the Kelly Range and Bald Range, along State Highway 73. Extensive ridge-rents, scarps, benches and spreading of these ranges appears to lead to rock avalanching generated by toppling and sliding. Comparative geomorphology suggests the process is widespread.

8. SYNTHESIS: MODELS FOR VARIOUS GEOLOGIC TERRAINS

In soft weak sedimentary rock, generally of Late Tertiary age, bedding parallel clay seams and crush zones give rise to block and debris slides. Originally known mainly from events such as the Abbotsford Slide (Coombs and Norris 1981) where they were once considered to be "unique" (Gallen, Beca, McCraw and Roberts 1980), these clay seams have been identified as basal rupture surfaces to landsliding in many other localities such as Rangitikei Valley (Stout, 1977, Thompson 1981), Hawkes Bay (Pettinga 1987) and Auckland (Prebble 1995(a)).

Depth of weathering is also a major contributor to sliding. Tectonic deformation (crushing and shearing) has also created extremely weak mixtures of rock fragments and fine soil which form zones of debris prone to landsliding. Examples are seen in north Auckland (Kermode 1991, Prebble 1995(a)) and the East Coast Deformed Belt (Pettinga 1987, Prebble 1995(a)).

Whilst block sliding is the dominant model in regularly bedded rock with clay seams, the crush zones and mixed tectonic debris gives rise to flows, slumps and slides.

The model best suited to active and young andesitic volcanoes is one of a massif made up of intertwining tongues and sheets of gravels and debris, threaded through with highly fractured rock tongues (lava flows). It is not realistic to envisage the volcanoes as being rock masses, rather coarse soil masses, with fine layers included and intercalated in a complex manner. Superposed valley-fill deposits and broader tongues of material are the normal internal geometry.

A major variant of this model is the existence of large masses of clay, with steam and hot water, as a result of geothermal activity.

Slope failure models are usually described as debris avalanche and debris flow. These can be very deep seated (tens of metres to hundreds of metres) as Mt St Helens showed (Schuster

1983).

Volcanic soil masses of rhyolitic composition, when weathered, develop a fragile microfabric which is prone to collapse and rapid slide-flow. Basal ruptures, at least in some instances, can be modelled as sensitive clay-silt layers controlled by palaeotopography, such as buried channels and valleys. It is considered that the Ruahihi Canal collapse demonstrates this type of failure (Prebble 1986).

In the greywackes of the Southern Alps and similarly strong, but highly fractured rock in other mountainous regions, toppling and sliding are widespread failure modes. Toppling is now recognised on both dipslopes and scarp slopes (Cruden and Hu 1994, Prebble 1995(a)) and can be a precursor to sliding, where toppling develops suitably oriented bending surfaces of sufficient continuity. These failures generate large rock avalanches for which the regions are noted (Adams 1981, Whitehouse and Griffiths 1983, Paterson 1996).

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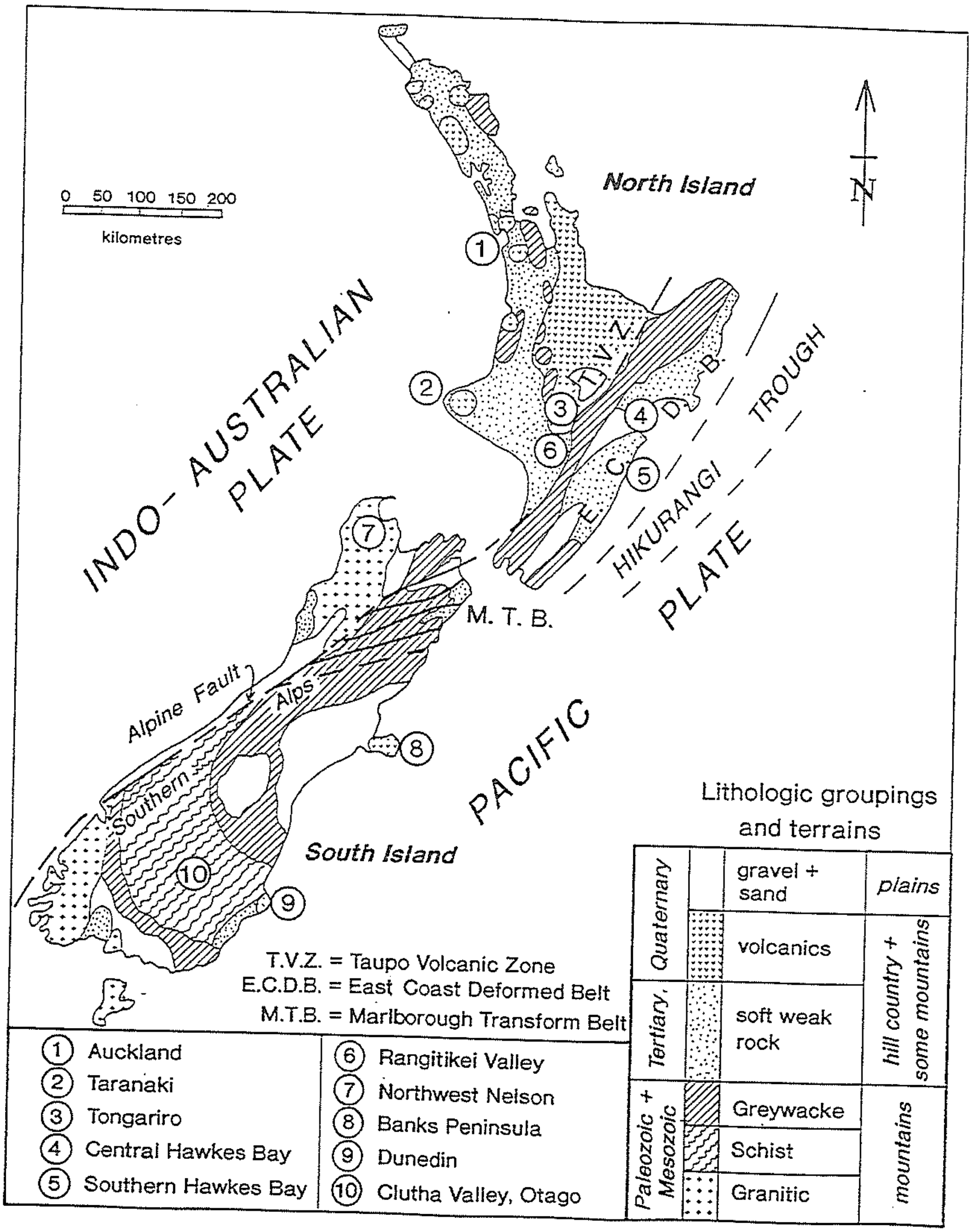

Figure 1 Outline map of engineering geologic terrains in New Zealand.

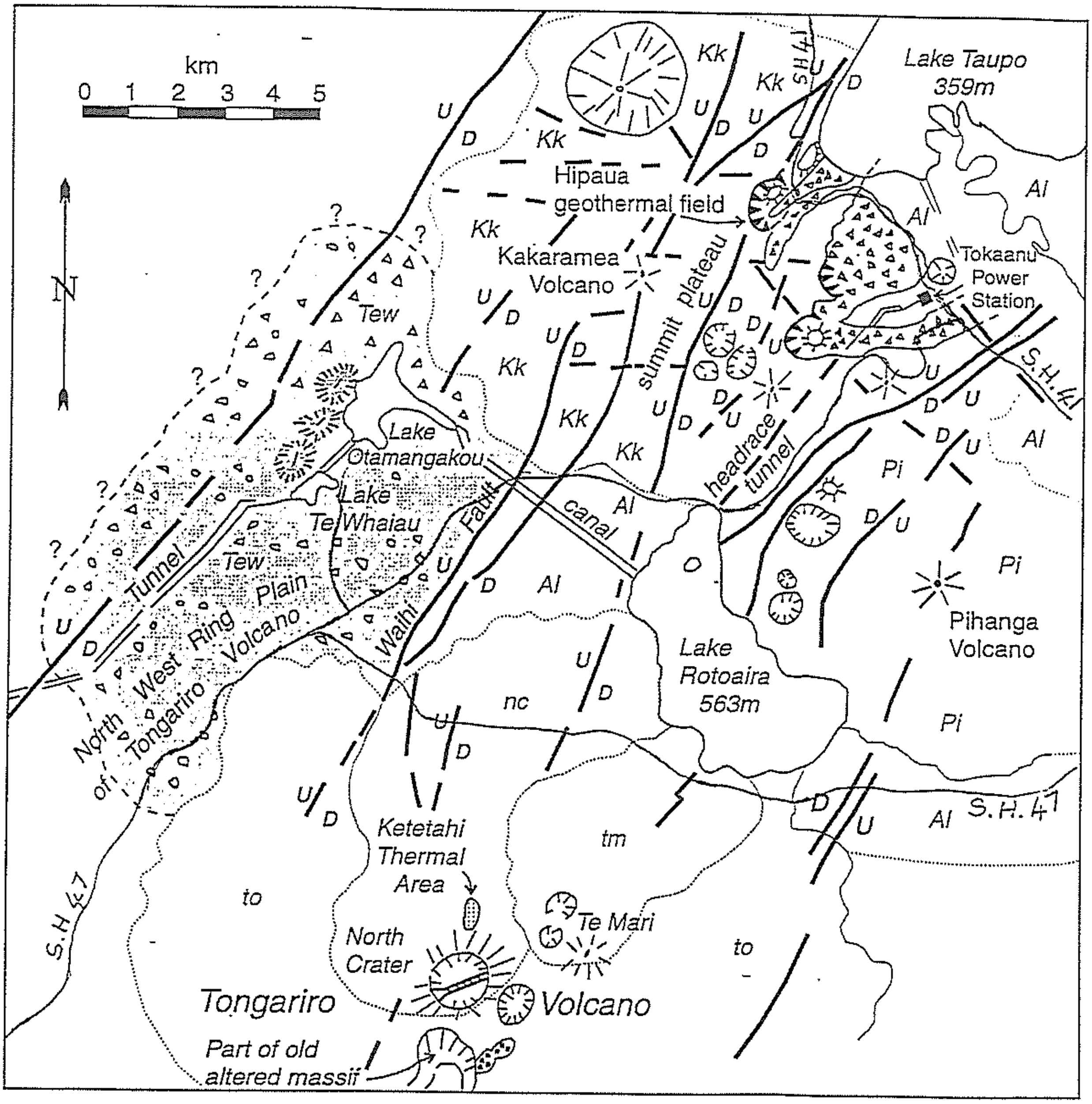

figure 2 Tongariro Volcano, N.W. ringplain and the Tokaanu area. Taupo Volcanic zone.

Legend

A : Slope movement features and deposits

	headscarp to landslide / debris flow
	slide block
	debris mound
	debris field, debris fan, tongue
	trench, pull-away zone (E.N.E. only, on Tongariro North Crater)
Tew	Te Whalau Formation (avalanche debris)
Al	alluvium of Rotoaira and Taupo basin

B : Faults and Volcanic Craters

U/D	Fault, (U) Upthrown (D) Downthrown (N.E. set)
	Faults, inferred (NE, SW + E-W sets)
	Summit crater
	Large basin-shaped crater (? explosion crater or lava filled crater)
	rhyolitic dome

C : Lava flow and scoria fields, originating from the following volcanoes:-

tm	Te Mari Crater	Tongariro volcano
nc	North Crater	
to	Tongariro volcano, older massif	
Pi	Pihanga volcano	
Kk	Kakaramea volcano	

ACTIVE PROCESSES AND THE MANAGEMENT OF THE ROAD NETWORK

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SUMMARY

The New Zealand road network is particularly vulnerable to the effects of active processes such as landslip, erosion, earthquake, river migration and other hazards. Any inspection of Transit NZ and Transfund annual reports provides evidence of the unpredictable nature of budget forecasting for work of this type. Most emergency events and incidents are managed by strategies involving single laning, temporary road closure, and temporary deviation. Occasionally, the adverse effects of active processes are so severe that permanent deviation is called for and, where appropriate, monitoring is increasingly being used as a management strategy to delay capital expenditure without compromising road user safety and asset security.

1. INTRODUCTION

Due to New Zealand's highly varied geology, its high level of seismicity, the steepness of the terrain, and extremes of climate, few areas of the country are free from the effects of active processes. Hazards from a wide range of active processes pose significant problems to the roading network. The active processes include landslip, erosion, alluviation, earthquake, river channel migration, floods, and snow avalanche. Initiating causes include not only extreme events such as high intensity rain ,snow storms and earthquakes, but also geological processes associated with slope development and valley formation, and climatic processes such as freeze and thaw.

The effects of active processes on the road network result in the expenditure of millions of dollars due to planned and unplanned maintenance, disruption to service, and injury or even death to road users. Transit NZ has developed various management practices to optimise this expenditure. As well, thorough investigation of active processes can enable management strategies to be implemented that reduce reinstatement costs.

2. COST OF ACTIVE PROCESSES

2.1 Recent Trends

Annual reports from Transit New Zealand provide testimony to the effect of active processes on the New Zealand roading network. The 1993/94 annual report states that "the Canterbury region was subjected to an unusual number of natural disasters during the year, with three floods and an earthquake adding $1.7 million worth of repair work to the planned workload", while for the same year the Dunedin region experienced major floods which "caused severe damage on State Highway 6 with bridges being washed out and debris blocking the road from Haast Pass to both Wanaka and Westland".

Also in the Canterbury region, Transit NZ's 1994/95 annual report indicates that "a very severe winter along with two earthquakes resulted in high maintenance costs for the alpine passes between Canterbury and the West Coast. Particularly hard hit was Arthur's Pass on SH73 which experienced an unusually high level of rock falls. For motorist's safety, restrictions and closure were necessary at times. A large mudflow on SH7 near the Lewis Pass threatened to close the highway on a number of occasions. Relocation of approximately 1.5 kilometres of highway is needed to bypass this problem". The cost of SH7 reinstatement amounted to $3.2 million (Section 3.3a).

More recently, Transit NZ's 1996/97 annual report demonstrates the unpredictability of budget forecasting for this type of work when it stated that "the number of emergency events causing damage and requiring funding exceeded target by approximately 2.5 times. There were 295 emergency events on local roads and 73 on state highways ".

2.2 Funding Framework

From 1997/98, the cost of repairing the effects of active processes are funded under Output Classes 1 and 2, referred to as the *Roading Maintenance* output, under Transfund's criteria for local road and state highway funding. The *Roading Maintenance* output is for the provision of maintenance projects on all public roads and includes three work categories for which funds are provided, namely *routine maintenance, preventative work,* and *emergency work* as well as other maintenance activities.

The *routine maintenance* category includes funding for the normal care and attention of the roadway to maintain its structural integrity, and which might be routinely expected in any one year. It provides, amongst other items, for the routine maintenance and repair of surface water channels and subsoil drainage, stream clearing and debris removal to maintain water courses and culverts, and snow clearing and ice control.

The *preventative work* category provides for non-routine work required to protect the serviceability of the road network and to minimise the threat of road closure. The work covered by this category includes physical works which protect existing roads from sea or river damage, drainage of landslips, buttressing of landslips, protection planting, and physical work required to overcome the effects of river channel migration but which is not attributable to one climatic event.

The *emergency work* category is for the funding of unforeseen significant expenditure which arises from a defined, major, short duration natural event. It allows for roads and road structures to be reinstated to a condition no better than that which existed prior to the damage occurring. This category of funding does not include minor scour in water channels, landslips that do not require restriction of a traffic lane, the effects of active processes which have accumulated over time, and any other deficiency which has developed from events over a period greater than one month. In the case where it is clear that an improvement to the road or road structures is desirable, the improvement work is required to be justified under Transfund's Project Evaluation Manual.

2.3 Repair Costs

Costs incurred under the *emergency works* and *routine maintenance* categories totalled for both local roads and state highways are itemised in Transfund's annual report for 1996/97, and summarised in Table 1.

TABLE 1

Summary of Transfund Expenditure on Emergency Works and Routine Maintenance, All Public Roads, 1994 - 97

	1994/95 $M	1995/96 $M	1996/97 $M
Emergency works	28.28	22.95	20.34
Routine maintenance	217.50	231.26	252.15
Total expenditure, Road maintenance	372.32	380.59	421.64
Emergency works as % of total expenditure, Road maintenance	8	6	5

Costs provided for under the *emergency work* category in Table 1, which comprise 5 - 8% of the total road maintenance expenditure, can almost entirely be accounted for by the effects of active processes. Costs under the *routine maintenance* category in Table 1 also includes *preventative maintenance* costs. Costs related to the effects of active processes under the *routine maintenance* category are difficult to establish, as this category includes expenditure for a range of other items covered under the general maintenance budget. Therefore the total cost due to the effects of active processes on the road network exceeds the percentage figures given under the *emergency work* expenditure in Table 1.

3. EVENT MANAGEMENT PROCEDURES

3.1 Emergency Policies and Procedures

The Transit NZ policy relating to emergencies is included within Section 8.6 Temporary Closures of State Highways as contained in TNZ : State Highway Control Manual, Issue 1, July 1994.

State Highway emergency procedure strategies have been developed at the local level by each of Transit NZ's regional offices to bring together under a single document all of the requirements for responding to emergency events and incidents. The Canterbury office responsible for the management of highway regions 11 and 12 was one of the first Transit NZ offices to document all the procedures and strategies for emergency response, which describes the principal local road conditions for which possible emergency procedures may need to be implemented.

In addition to providing Transit policies and responsibilities on temporary road closures, public advice on temporary road closure, civil defence operating procedures for road access, and information on local road conditions such as detour information, sign posting and road restrictions, the emergency procedure strategy document sets out the requirements for management of emergency events and incidents.

The responsibility for managing emergency work lies with Transit NZ's Network Maintenance Management Consultants who operate under Transit NZ Professional Services Contracts within each of the highway regions. Under these Contracts, the Network Consultant is responsible for managing emergency work which includes frost and ice gritting, snow clearances, small slips, bleeding, flooding, physical damage to the highway, emergencies due to wind, and other situations which Transit NZ considers should be treated as emergencies.

Transit NZ's Network Maintenance Contractors for each highway region are responsible for the implementation of physical works in respect of emergency events, under separate Pavement and Drainage Maintenance and Emergency Callout Contracts. These specify minimum response times for emergency events, and provide the specifications such as TNZ C9: 1993 for the execution of work in respect of storm damage, debris and slips.

Where the effects of active processes on the road network do not require immediate response but are ongoing and complex, Transit NZ provides for consultants other than the Network Consultant to carry out remedial works under separate investigation, design and construction supervision Professional Services Contracts, where appropriate.

3.2 Emergency Management Strategies

Most emergency events and incidents attributable to the effects of active processes are managed by means of strategies involving single laning, temporary road closure, and temporary detour. One of the criteria on which Transit NZ measures its performance is the percentage of occasions when the highway can be opened to a single lane within 12 hours of an emergency event.

Less commonly, the effects arising from active processes require more drastic management responses, such as permanent deviation of the road network to bypass the hazard. Due to the large costs involved, such responses are not taken without significant investigation of all alternatives including mitigation measures to stabilise the affected area.

In circumstances where the effects of active processes are not immediate, but where there is potential significant medium to longer impact on the road network, monitoring is a strategy which allows informed decisions to be made regarding the timing of any physical works which might need to be implemented. Such a strategy allows capital expenditure to be deferred without compromising the safety and integrity of road users and the road asset.

3.3 Examples Of Specific Event Response

a) Poplars Straight Debris Flow, SH7 Lewis Pass

A debris flow, due to rapid erosion of a steep slope above SH7 forming part of the true left side of the Hope Valley, Lewis Pass, broke out on 8 November 1994, initially triggered by a heavy rainfall event [1]. At that time, large volumes of material became mobilised from an old outwash terrace and debris fan surface at an elevation some 155 m above the highway, which resulted in blockage of the road. Subsequently, over a period of approximately 18 months, instability of the slope continued in response to rainfall and snow melt events, resulting in the development of a major eroded gully system approximately 70 m deep and 400 m long. Some 400 000 m^3 of material was eroded from the gully as numerous small (1 500 m^3) and infrequent large (up to 30 000 m^3) debris flows which were mobilised in the floor of the gully as water-saturated sediment-laden surges. The material which discharged from the mouth of the eroded gully was initially prevented from engulfing the highway by a continuous 24 hour earthworks operation to remove the debris, followed by the construction of debris flow control channels and debris holding areas, by stockpiling of debris, and bridging of the highway by means of a temporary one lane bridge. The high impact forces accompanying individual debris surges resulted in significant downcutting of the valley floor and erosion of the gully side slopes, creating additional gully instability due to oversteepening.

It was predicted that there was potential for a significant alluvial fan to develop from the mouth of the gully, which could extend up to 300 m engulfing the highway and the river terraces between the highway and the Hope River. Accordingly, a number of mitigation measures were investigated to evaluate whether SH7 at this location could be retained on or close to its original alignment. Options investigated included surface and subsurface drainage, removal of potentially erodible material adjoining the gully to remove the supply of sediment, and bridging of the fan. However, it was considered that none of these measures could be implemented at a reasonable cost because of the continuing erosive activity.

The recommended mitigation work involved 1.6 km of highway deviation on a new alignment up to 400 m from the mouth of the gully, which was predicted to be outside of the influence of fan development if erosion of the gully continued at its initial rate. Deviation of the highway was carried out in 1996. The gully has continued to erode although at a much reduced rate which is partly due to the original earthworks and partly because the gully appears to have reached a natural equilibrium at this stage. A modest monitoring programme has been continued to evaluate erosion trends, including surveyed cross sections and rainfall monitoring.

The onset of major erosion at Poplars Straight resulted in a significant cost to Transit New Zealand. This included the cost of earthworks operations to keep the highway open, land purchase, and deviation of the highway totalling some $3.2 million. Initiation of erosion was largely without warning and could not have been predicted. The ultimate magnitude and severity of the hazard was not fully appreciated at its onset, and also could not have been predicted.

b) Newman's Lookout, Upper Buller Gorge, SH6

Major slope instability at Newman's Lookout in the upper Buller Gorge has been threatening SH6 for a number of years. In order to maintain the integrity of the road, Transit NZ has been forced to reconstruct the highway several times at considerable cost by excavating into the hillside which contains the road on a platform 100 m above the left bank of the river [2].

Undercutting of the Buller River is the main cause of the instability, which is in response to large amounts of bedload being added to the river from a large landslip upstream of Newman's Lookout caused by the 1968 Inangahua earthquake. River training works have been constructed to redirect the flow away from the toe of the undercut slope, and a monitoring programme put in place to test its effectiveness for further reducing toe erosion.

The outer edge of the highway continues to be affected by local instability and it is very likely that groundwater in the slope at highway level is a significant contributing factor. For this reason, drainage drilling is presently being carried out to improve local slope stability at highway level.

The monitoring strategy which has been adopted at this site allows long term erosion trends to be identified well in advance of any immediate influence at highway level. This combined with drainage drilling to improve local instability at highway level provides the opportunity for delaying more excavation in to the hillside, thereby deferring capital expenditure as long as possible.

4. FUTURE POSSIBILITIES FOR RISK MANAGEMENT

To date, costs associated with litigation arising out of injury and or death to road users from active processes have not occurred to any significance in New Zealand, but overseas trends in this area [3] and the increasing litigious climate within New Zealand, suggests that this type of cost can be expected in the future.

In order to determine the loss of business cost due to disruption to the road network arising from unplanned events and incidents including active processes, Transit NZ has recently completed a study of the security of the road network throughout the country [4]. This involved consideration of the level of importance of each road network, the risk and expected duration of disruption, and the impact that severance would have on the operation of the road network. These factors were used to rank individual road networks in terms of their assessed vulnerability. Those road networks judged to be most vulnerable included a mix of both urban and rural roads in both North and South Islands.

A number of north american roading agencies have implemented preventative maintenance strategies to manage the effects of active processes and to minimise costs. These include the British Columbia Ministry of Transport and Highways (MOTH), the Oregon State Highway Division and the Washington State Department of Transport [5,6]. These include rating systems based on a range of factors including geotechnical controls, terrain characteristics, hazard history, vehicle and road conditions, and climatic factors.

More rigorous preventative maintenance programmes have also been proposed which allow uncertainty in terms of the frequency of hazard occurrence, and also the cost, risk and consequence of mitigation, to be modelled using probabilistic techniques such as that available using @RISK software [7].

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AUCKLAND ENGINEERING LIFELINES PROJECT: RAIN AND EARTHQUAKE-INDUCED SLOPE INSTABILITY HAZARD

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SUMMARY

The Auckland Engineering Lifelines Project (AELP) was initiated by Regional Government in 1996, with the primary objective of reducing the impact of known hazards on the lifeline services of the Auckland Region. Auckland's road network is one such critical (lifeline) utility. The study identifies areas of the Auckand Region susceptible to slope instability and liquefaction associated with a 2000 year scenario earthquake and 100 year scenario cyclone. Earthquake-Induced Damage Matrices were derived to provide a qualitative assessment of the vulnerability of services for planning purposes.

1. INTRODUCTION

The Auckland Region has a land area of 5,024 km^2 and is home to a third of New Zealand's population (Figure 1). The hazards associated with a 2000 year earthquake and 100 year cyclone in the Auckland Region were assessed as part of a broader study of the risks and impacts of all known hazards to Auckland's lifelines. This work is intended to provide a basis for development of emergency response planning for any critical facility by utility owners. This paper identifies areas in which damage to roads, embankments and bridges is likely in a scenario earthquake and cyclone event.

1.1 Geological Setting

Much of Auckland is built on Miocene to Quaternary sedimentary rocks and soils. Late Quaternary basaltic volcanoes have erupted through these sediments to produce a wide range of volcanic deposits which rest upon thick residual and transported soil masses. Critically steep slopes are susceptible to failure during earthquake along the soil–rock boundary and within regolith. Gentle slopes are prone to failure through bedrock on low angle clay seams parallel to bedding. Quaternary deposits include extremely weak sensitive pumiceous silts and local areas of loose sands which are susceptible to liquefaction.

1.2 Historical Earthquakes

Historically (last 150 years) the Auckland Region is one of the least seismically active regions of New Zealand. Only three historic earthquakes are considered to have produced intensities of Richter magnitude greater than 5 (Table 1).

Table 1. Historical Earthquakes of Magnitude > 5

Date	Distance*	Richter Magnitude
1834/1835	35km	5.5 – 5.6
23.06.1891	60km	5.7 – 5.9
28.02.1974	>200km	5.9

*distance of epicentre from central Auckland

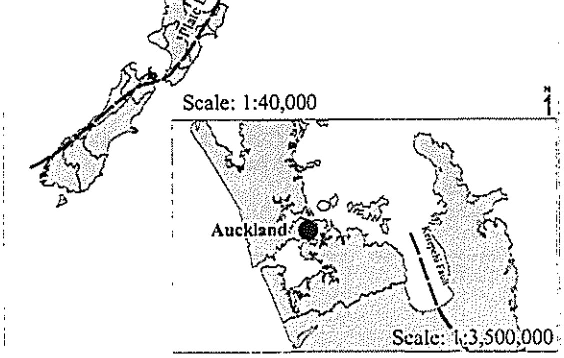

Figure 1. The Auckland Region

1.3 Historical Cyclone Events

Records identify 5 tropical cyclones passing within 220km of Auckland City since 1970 (Table 2), suggesting a return period in the order of one in 6 or 7 years.

Table 2. Historical Cyclones Recorded in the Auckland Region Since 1970

Storm Event	Date	Accumulated Rainfall in 2 days* (mm)	Maximum Wind Gust (km/hr)	Direction Day 2
Watorea	30.04.76	50	82	W
Sina	15.03.80	100	106	S
Bola	08.03.88	126 in 3 days (77 in 24hrs)	130	SE
Fergus	30.12.96	52	-	-
Drena	10.01.97	21 in 24hrs	93	-

* unless otherwise stated

2. 2000 YEAR EARTHQUAKE FOR AUCKLAND

2.1 Kerepehi Fault

In the past, earthquake scenarios for engineering projects in the Auckland area have modelled ground shaking derived from an earthquake along the Kerepehi fault, 35km east of Auckland (Figure 1). The Kerepehi fault is known to be active on land (multiple movements within the last 125,000 years) (Hull et al 1995) and is considered to also be active offshore. Fault rupture length and single-event displacements indicate that a magnitude 7 earthquake on the offshore segment, would generate PGAs (Peak Ground Accelerations) of 0.15g in Auckland and have an expected return period of about 5000 years (McVerry 1997).

2.2 Uniform Hazard Model

A "uniform hazard" model was developed to estimate the ground motions expected in Auckland from a 2000 year return period earthquake based on the historical record of earthquakes. This model estimates a PGA of 0.17g to 0.27g (depending on soil type) for the region and was selected for the AELP because it generates higher levels of PGA than a similar return period event on the Kerepehi fault. Application of this model uniformly to the whole of the Auckland region allows an assessment of earthquake-induced hazards within 20km of any potential epicentre.

A single earthquake event with a selected epicentre was also modelled to assist utility owners in planning for damage to a part of a specific utility and overall services network. The scenario adopted was a point source magnitude 6 earthquake at 10km depth, with an epicentre 20km east of central Auckland. This location was selected so that the higher PGA values for a 2000 year earthquake would be distributed over central Auckland.

3. 100 YEAR CYCLONE FOR AUCKLAND

The cyclone scenario developed for the purposes of the AELP has an accumulated rainfall of 230mm to 415mm distributed over the Auckland Region in 4 days. Scenario rainfalls are given in Table 3.

Table 3. 100 Year Cyclone Scenario

Average 24 hour Rainfall (mm) [Range of 24hr Rainfall in Different Parts of the Region]				Accumulated Rainfall (mm)
Day 1	Day 2	Day 3	Day 4	
74 [30-115]	124 [65-185]	93 [55-120]	24 [10-50]	325 [230-415]

4. SOIL / ROCK MASS GROUPINGS

The soils and rocks were grouped according to their expected response to shaking from earthquakes and susceptibility to high rainfall, based on characteristic physical properties and observed behaviour:
Group 1: Pre-Pleistocene rock-masses. Weathering has produced a layer of soil and slope debris up to 20m thick over most of the older rocks in the region. The behaviour of the residual soils, slope debris and rock defects dominates the response of these slopes to ground shaking. The Pre-Pleistocene rock masses and their residual cover tend to fail along fractures, faults, bedding planes, clay seams, the soil/rock interface, and within the residual soil mantle.
Group 2: Pleistocene sediments. Pleistocene age sediments are generally occur in lower slopes than the older soils and rocks. In steeper slopes, these materials have been subject to creep or shallow sliding. This group includes Quaternary volcanics whose behaviour in slopes is dominated by the underlying sediments.
Group 3: Holocene coastal sands and man-made fill overlying Group 1 and 2 deposits. The high permeability and friction angle allows the sands to stand in relatively steep slopes with a low margin of stability close to the slope face.
Group 4: Holocene estuarine clay-silt and man-made fills overlying Group 3 and 4 deposits. Because of their low strength, and location in low lying areas adjacent to swamps and water courses, the soft estuarine sediments tend to be saturated, of low permeability and occur in relatively modest slopes.

5. SLOPE INSTABILITY HAZARD

5.1 Methodology

The stability of a series of slopes of different height, angle and soil/rock mass composition were back-analysed for differing levels of earthquake acceleration and groundwater conditions. The sensitivity of the slope to short-term high rainfall events (24 hours) was simulated by modelling a water-filled tension crack, and the sensitivity to longer term high rainfall (more than 3 days) was modelled by raising the groundwater level by different amounts for the different slopes analysed. This provided an assessment of conditions under which the margin of stability is reduced during earthquake and heavy rainfall respectively, and indicated the relative importance of each factor to that margin of stability. Classes were developed for each condition, and then weighted according to their influence on slope stability, as indicated by the back-analyses. These data were presented as factor maps and superposed using the GIS based ArcInfo system, to produce hazard maps for the Auckland Region.

5.2 Soil/Rock Mass Classification

Scores were allocated to each soil/rock mass group according to its engineering geological characteristics and the contribution to the margin of stability (Table 4). Scores indicate the relative susceptibility to earthquake or cyclone. Table 4 shows that Holocene estuarine deposits and fill (Class D) are considered to be more susceptible to earthquake-induced instability than the other soil and rock types.

Table 4. Soil Category Contribution to Stability

Soil / Rock Mass*	Class	Score	
		EQ	Cyclone
Group 1	A	0.7	0.5
Group 2	B	0.4	0.4
Group 3	C	0.6	0.8
Group 4	D	0.25	0.25

* Refer Section 4.

Table 5. Slope Grade Contribution to Stability

Slope Grade	Class	Score	
		EQ	Cyclone
Gentle (0-7°)	A	1.0	1.0
Moderately steep (8-15°)	B	0.8	0.8
Steep (16-20°)	C	0.5	0.5
Very steep (>20°)	D	0.25	0.25

5.3 Slope Grade Classification

Slopes were grouped into the four classes shown in Table 5. Relative scores were assigned, based on the back-analysed behaviour of the soil classes within each slope range. Scores indicate the relative reduction in stability compared to a slope of 0-7°. For example, for the soil types considered, a very steep slope (Class D), has a margin of stability four times lower than a gentle slope (Class A).

5.4 Earthquake and Cyclone Scenarios

Scores were allocated to the ranges of PGA and rainfall in a specified period, according to the predicted contribution of these effects to reduction of the margin of slope stability (Tables 6 and 7). Higher levels of PGA or rainfall increase the likelihood of slope failure. For example, the risk of failure for slopes experiencing Class D shaking is five times more than when experiencing Class A shaking (Table 6).

Table 6. 2000-Year Earthquake, PGA effect

PGA (g)	Class	Score
≤ 0.05	A	1.00
>0.05 – 0.10	B	0.70
>0.10 – 0.15	C	0.35
>0.15 – 0.20	D	0.20
> 0.20	E	0.10

Table 7. 100-year Cyclone Contribution

Rainfall (mm)		Class	Score
24 hour	3 day		
< 50	< 100	A	1.0
50 - 70	100 - 250	B	0.8
70 - 100	250 - 400	C	0.5
> 100	> 400	D	0.4

5.5 Slope Instability Hazard Assessment

The combined scores from Tables 3, 4 and 5 were totalled using the GIS Arc-Info system to produce Earthquake Hazard Scores for the Auckland Region, and Hazard Maps (eg Figure 2) which identify zones of low, moderate, moderately high and high hazard, as described in Table 8. A similar process was carried out using scores from Tables 3, 4 and 6 to assess rain-induced slope instability hazard.

Table 8. Interpretation of Hazard Score

Hazard Class	Factor of Safety*	Interpretation	Score
A	≥ 1.5	Low Hazard: 0.5% of slopes fail	> 0.2
B	≥ 1.2 - 1.5	Moderate Hazard: 0.5% to 5% of slopes fail	> 0.10 - 0.20
C	≥ 1.1 - 1.2	Moderately High Hazard: 5% to 20% of slopes fail	≥ 0.05 - 0.10
D	1.0 - 1.1	High Hazard: 20% or more of slopes fail	≤ 0.05

* during or immediately following earthquake or cyclone as appropriate.

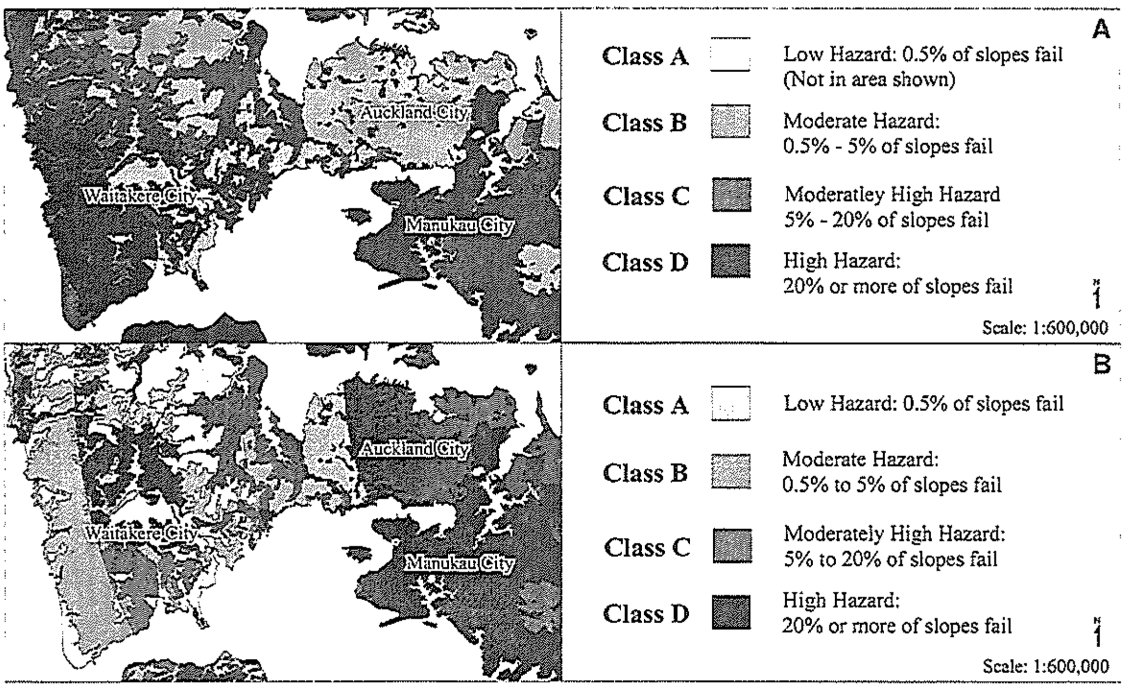

Figure 2. Extracts from Earthquake-Induced Slope Instability Hazard maps. A: Uniform hazard. B: Specific scenario (epicentre 2.5km northeast of area shown).

Figure 2A illustrates the earthquake-induced slope instability hazard for the Auckland Region assuming a uniform level of PGA of 0.17g to 0.27g depending on the soil/rock type, that is, the level expected within 20km of a 2000 year return period earthquake epicentre.
Figure 2B shows levels of hazard for the 2000 year earthquake specific scenario. These zones will shift for different epicentres. The adopted scenario would result in failure of 20% or more of slopes in the vicinity of many of Auckland's essential lifelines corridors.
In most areas, the 100-year cyclone scenario considered would result in only a low or moderate slope instability hazard, that is, rainfall is considered likely to cause in the order of 0.5% to 5% of slopes in the region to fail. Moderately high and high hazard areas occur where slopes are steep (for example valley and coastal slopes) and predicted rainfall is greatest.

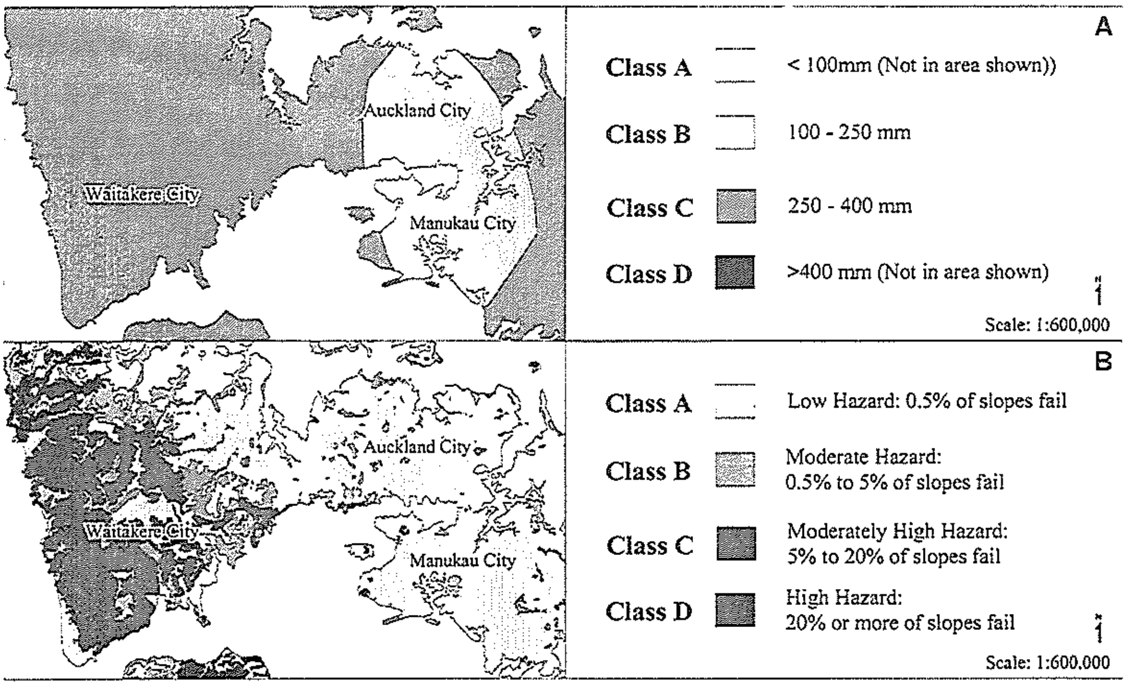

Figure 3. Extracts from 100-year cyclone scenario map, A; and rain-induced slope instability hazard map, B.

6. LIQUEFACTION HAZARD

Liquefaction occurs in saturated cohesionless soils, in response to repeated shaking during earthquakes resulting in liquefaction-induced slope instability, lateral spreading and subsidence. As for earthquake-induced slope instability, liquefaction was assessed both in terms of a uniform hazard model and a scenario event caused by a single earthquake within 20km of Auckland City. Parameters used to assess response to earthquake shaking include the density and grading of the soils, depth and thickness of sand layers, saturation, and the duration and strength of ground shaking. A methodolgy similar to that developed for earthquake induced instability was used.
All potentially liquefiable soils within 20km of a 2000 year return period earthquake in the Auckland Region are expected to experience PGA's of >0.2g. For the uniform hazard model, the likelihood that these soils will liquefy is high, that is, in the order of 30 - 90%. The specific scenario event suggests that on average, 0.5 - 10% of such soils in the northernmost part of the region may liquefy.

7. DAMAGE MATRICES

An attempt has been made to provide a qualitative assessment of the likely type and degree of damage to roading infrastructure which might result from ground shaking during earthquake. A matrix approach (Tables 9 and 10) was used to assess earthquake-induced damage in broad terms based on:

1. The likely design factors of safety for each type of structure;
2. Earthquake hazard models developed for recent engineering projects in Auckland City;
3. Observed and reported performance of similar structures in other recent earthquakes (Edgecumbe, New Zealand 1987, Loma Prieta, San Francisco 1989, Northridge, Los Angeles 1994, Great Hanshin earthquake, Kobe, Japan 1995); and
4. Modified Mercalli intensity scale (NZ Society for Earthquake Engineering 1992).

The probability ratings anticipated (negligible, low, medium and high) for each type of structure and the nominated ranges of PGA (Matrix 1, Table 9) can be used in conjunction with Matrix 2 (Table 10) to estimate the nature and severity of damage which might be sustained. This provides a basis on which to develop earthquake response planning.

A similar matrix evaluation of potential rain-induced slope instability damage could be prepared.

Table 9. Earthquake-Induced Damage to Road Structures Matrix

Structure	Range of ground acceleration (PGA) and modified Mercalli earthquake intensities (MMI)							
	MMI ≤ VI 0.05 - 0.10g	MMI VI - VII 0.1 - 0.15g		MMI VII - VIII 0.15 - 0.20g		MMI > VIII > 0.2g		
	No risk of Liquefaction	Minor Risk of Liquefaction		Some Risk of Liquefaction		Non-Liquefiable Soils		Liquefiable Soils
		Slopes A/B	Slopes C/D	Slopes A/B	Slopes C/D	Slopes A/B	Slopes C/D	
Roads, Rail	Negligible	Negligible	Low	Negligible	Moderate	Low	High	High
Embankments	Negligible	Low	Moderate	Low to moderate*	Moderate to high*	Moderate	High	High
Bridges†:								
Modern	Negligible	Negligible	Negligible	Low	Low	Low to moderate	Low to moderate	Moderate to high
Old	Low	Low to moderate	Low to moderate	Moderate	Moderate to high	High	High	High

* probability dependent on embankment height: approximately <6m vs >6m

† "modern" buildings and bridges: those constructed since c.1970 for concrete and c.1980 for other materials

Table 10. Earthquake-Induced Damage Type Matrix

Structure	Negligible	Low Probability	Moderate Probability	High Probability Non-liquefiable Soils	Liquefiable Soils
Roads, Rail and Embankments	Minor distortion (rail only)	Movement in a downhill direction; visible distortion or cracking	Distortion to rails and cracking or scarp displacement of roads	Buckling of rails or loss of support; impassable scarps in roads	Loss of support
Bridges†: Modern	Minor cracking	Minor cracking	Spalling, abutment damage	Lateral movement, loss of alignment	Loss of span, loss of approach ramp, foundation damage, rotation
Old	- Minor spalling	Spalling, loss of support	Spalling, loss of span	Damage, loss of span, collapse	Collapse

8. CONCLUSIONS

Ground shaking associated with a 2000 year earthquake in the Auckland region is likely to cause widespread failure of slopes and liquefaction of susceptible soils within 20km of the earthquake epicentre and beyond. Such an event will have a significant impact on our road network. A 100 year cyclone event will have a lesser effect, primarily influencing structures located in areas where steeper slopes occur, such as valley sides and coastal areas. The evaluation of the potential for slope failure and liquefaction and the assessed likely impacts on infrastructure provide a first step basis on which utility owners can assess the risk and vulnerability of essential services and begin to develop initiatives for emergency response planning.

Acknowledgments. The author wishes to thank Michele Daly of the Auckland Regional Council (AELP leader and coordinator) and members of the hazard task groups.

ALPURT NORTHERN MOTORWAY EXTENSION - SECTOR A ENGINEERING GEOLOGY AND GEOTECHNICAL INVESTIGATION

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SUMMARY

This paper reviews some of the results of the engineering geological and geotechnical investigation undertaken for the 15 km long ALPURT Sector A extension of the Auckland Northern Motorway from Albany to Silverdale. The total earthworks volume is approximately 3 Mm^3. Ten bridge structures are to be built. The total cost of the investigation was 1% of the $80M project. Of this, 20% was spent on investigations specifically for structures. The investigation findings include:

- the initial field investigation and lab testing followed a recipe set by the client. The program for subsequent design investigations was determined by Opus in consultation with the peer reviewer and Serco, the project management consultancy appointed by TNZ.
- the geology is dominated by Northland Allochthon mudstone in the northern Sector A2 and Waitemata Group sandy and silty soils in the southern Sector A1. Tauranga Group alluvium occurs beneath river flats up to 15m depth.
- the Northland Allochthon unit is typically unstable however no natural slope slip features have been identified. Terrain evaluation has been used to determine a "safe" long term cut slope inclination of 4H:1V. Localized slips may even occur at this flat slope - gabion supported rock buttresses will be used to stabilize such features.
- the Waitemata Group terrain is typically stable, however earthslides do occur in the steeper slopes north of Lonely Track Road. Cut slopes in this terrain are inclined at 2½H:1V with local flattening to 3H:1V in critical areas.
- short term geogrid reinforcement has been used to permit construction of 5m high embankments over thick alluvium. Deep 6m undercuts, high strength geotextile reinforcement, wick drains and stabilizing berms have been utilized at three sites to permit quick construction of higher embankments and therefore minimize post-construction settlements.
- Northland Allochthon mudstone requires wetting up and use of very heavy compactive effort to achieve a low void fill of adequate strength. Clegg hammer is used to determine the strength of the compacted mudstone.
- Waitemata soils require drying to very close to optimum water content in order to achieve a bulk fill strength adequate for long term stability of the 2H:1V inclined up to 40m high slopes. Control is by shear vane and air voids.
- pressuremeter limit pressure and SPT N values have been used to determine the bearing capacity of the Northland Allochthon mudstone for the bored bridge piles. UCS have been used to assess the capacity of Waitemata rocks.

1 INTRODUCTION

The SH1 Albany to Puhoi Realignment (ALPURT) Sector A is part of Auckland's single largest roading project. The 15 kilometres of new highway in Sector A from the Silverdale Interchange in the north to Greville Road in the south is divided into two construction contracts - Sector A1 from Awanohi Road to Greville Road and Sector A2 from SH1 Silverdale to Awanohi Road (Figure 1). An enabling contract was also let to permit an early start to the earthworks program. Sector A includes approximately 3.0 Mm^3 of earthworks; 0.35 Mm^2 of pavement; four major interchanges; six overbridges and five major culverts.

Four main geotechnical design issues were the focus of the engineering geological and geotechnical investigations:

- cut slope stability
- earthworks properties
- fill embankment stability and settlement
- deep pile foundations for bridge structures

Initial field investigations were let out as a separate short term contract by Serco Consultancy on behalf of Transit NZ, with the investigation following a programme of tests set by the client. Subsequent design field investigations for the alignment and the structures were based on the findings of the initial geotechnical investigation, a terrain evaluation of cut and natural slopes in the project area, and recommendations of the peer reviewer.

There were a total of some 105 boreholes and associated field and laboratory tests. Helicopter access for the rigs was required in steeper inaccessible country. At least 134 machine excavated pits, and a number of CPT and DCP tests were undertaken.

2 GEOLOGY

2.1 Stratigraphy

Three major geological units occur in the Sector A realignment. Northland Allochthon (commonly referred to as Onerahi Chaos Breccia or "chaos") occurs from Silverdale to Awanohi Road; Waitemata Group occurs from Wright Road to Greville Road; and Tauranga Group alluvium overlies both Northland Allochthon and Waitemata Group units in the low lying areas.

The stratigraphic relationship between the Northland Allochthon and the Waitemata Group, the contact of which lies buried beneath the alluvium between Awanohi Road and Wright Road remains unclear. It would appear from borehole investigations that the contact is abrupt, possibly a steeply inclined fault although it is possible that the Northland Allochthon rocks may underlie and/or interfinger and lens with Waitemata Group rocks at depth beneath Wright Road and areas further south.

2.2 Lithologic Descriptions

2.2.1 Northland Allochthon

The Northland Allochthon rocks are mainly extremely weak to very weak siltstone / claystone with very closely spaced polished fractures. The rock is calcareous (18-25% $CaCO_3$) and smectite - rich (clay index 14 to 22). The soil overburden, consisting of firm, occasionally soft silty clay, is typically about 2m to 4m thick, with the transition between soil and rock usually rapid.

2.2.2 Waitemata Group

The Waitemata Group rocks are very weak to weak sandstone with subordinate very weak mudstone layers. The rocks typically have very widely spaced fractures. The soil overburden ranges from 2m to 12m depth in the Wright Road to Lonely Track Road section, and increases to 20m to 30m in the McClymonts Road and Spencer Road areas. The contact between soil and rock is usually transitional and occurs over many metres.

2.2.3 Tauranga Group Alluvium

Tauranga Group alluvium usually occurs in low lying areas. The alluvium is often 2m to 6m thick, and reaches up to 15m thickness south of Wright Road. Tauranga Group alluvium also infills paleovalleys which have no topographic expression and which can occur in more elevated areas. The alluvium consists of soft to firm clays and silts, interlayered with clayey sands. The unit includes Pleistocene alluvial deposits and Recent (Holocene) undifferentiated alluvial deposits along watercourses.

2.3 Hydrogeology

The hydrogeological regimes of the Northland Allochthon and the Waitemata Group terrain are different. Multi-level piezometers in the Northland Allochthon show variable piezometric levels with depth. Increasing piezometric heads with depth were encountered in some of the deeper excavations, while artesian heads occurred at topographically low and high sites.

Piezometers in the Waitemata Group always exhibit decreasing piezometric head with respect to the ground surface indicating a series of perched but leaky water tables within the Waitemata Group soils and rock. The perched water tables are due to the interbedding of sand and clay layers of varying permeability. This groundwater head profile with depth is typical of recharge areas within the Waitemata Group terrain, where vertical infiltration is the main recharge mechanism.

3 SLOPE STABILITY

3.1 Northland Allochthon Terrain

3.1.1 Natural Slope Stability

The Northland Allochthon terrain is gently undulating with slopes in the order of 10H:1V to 15H:1V. Locally the inclination of the slopes increase to approximately 3H:1V. Whilst this geologic unit is known for its slope stability problems, there is no evidence of natural slope failures of significant size within Sector A2. The dominating slope process appears to have been soil creep associated with seasonal wet and dry volume changes within the clay soil overburden and the uppermost degraded mudstone material. Soil creep debris eventually becomes incorporated within the Tauranga Group alluvium. The alluvium has aggraded the stream valleys resulting in no significant down cutting or slope flattening processes occurring at the present time. The natural slopes are now effectively at equilibrium.

3.1.2 Cut Slope Stability and Design

Three approaches were taken to assess the stability of cut slopes in Northland Allochthon rock:

- Site visit to Redvale quarry/landfill to examine cut slope designs and performance in Northland Allochthon material.
- Inspection of the stability of cut slopes along East Coast Road.
- Inspection of a trial cut 300m north of Wilks Road where three cut slopes were trialed: 2H:1V, 3H:1V, and 4H:1V.

The Northland Allochthon is a heterogeneous soil/rock which apart from limestone units, is prone to instability at relatively low cut slope inclinations. Failures in the Northland Allochthon are relatively shallow translational failures commencing at the toe with a ravelling action as the rock progressively "explodes" due to stress release. The slip often regresses up the slope, aided by typically near surface piezometric heads.

Experience of highway cuts and Redvale Landfill slopes shows that the time to failure following excavation is inversely proportional to the steepness of the cut slope i.e. 2H:1V cuts fail within several months of excavation but most 3H:1V cut slopes that fail do so several years after excavation. Observations at the 15m high Wilks Road trial cut suggest that the rock at this location may be more competent than is typical, resulting in the 2H:1V inclined cut slopes remaining stable during the 1997 winter. Both the 2H:1V and 3H:1V slopes developed cracks and bulged in late October early November i.e. seven months following excavation. The trial cut has very recently been re-contoured to the 4H:1V inclination.

The processes leading to slope movement cannot meaningfully be assessed using conventional limit equilibrium stability analyses but can be likened to unloading a material with a horizontal earth pressure coefficient in excess of 1 which is consistent with the original emplacement of the Northland Allochthon by means of large gravity slide. The high geological horizontal earth pressures, rock dilation effects and microstructural defect orientation are difficult to include in conventional analyses. This is why terrain evaluation has been adopted as the basis for cut slope design in Northland Allochthon materials. The 4H:1V inclined cut slopes may have an average margin of safety against whole slope failure of about 1.1 to 1.3, given typical Northland Allochthon rock and groundwater conditions. The proposed 4H:1V inclined cut slopes should have a safety factor in excess of 1.3 with respect to the conditions of the trial cut slope 1 year after excavation. Some failures are expected at localized parts of the cut slopes where either rock and/or groundwater conditions are less favourable than at sites that have been investigated. Earthslides that do occur will be remediated using contingency remedial measures of a gabion supported rock buttress, possibly incorporating bored horizontal drains where site conditions indicate that drainage may be appropriate.

Local steepening of the cut slopes to 2½H:1V and 3H:1V are based on terrain evaluation of slopes and cuts in the immediate vicinity of the proposed excavation. These steeper slopes have been utilised to limit the requirement for long sidling excavations within rising topography.

3.2 Waitemata Group Terrain

The Waitemata Group geology of A1 typically has a thicker than normal weathered soil profile. The topography is steep to undulating. Cut batters up to 25m high, are typically inclined at 2.5:1, being reduced to 3H:1V in some critical areas. Cut slope design is based on triaxial test results (Figure 2) and groundwater monitoring. Bored horizontal drains have been used in many of the deeper cuts to control the water table and reduce the probability of slope instability.

4 FILL EMBANKMENTS

Fill embankments in Sector A2 range up to 10m height and the side slopes are inclined at 2H:1V. The typical subsurface profile in this sector consists of about 2m thickness of soft / firm Northland Allochthon silty clay and/or Tauranga Group clay alluvium, overlying Northland Allochthon rock. These soils typically have been "cut to waste" because of their wet and soft nature, and replaced by an engineered soil fill.

Fill embankments in Sector A1 reach 30m height with fill slopes reaching nearly 40m vertical height from crest to toe. The inclination of the side slopes are typically 2H:1V but locally some steepening is required to maintain the fills within the designation boundary. Typically foundation treatment involved undercutting any soft mullock. The high sidling fills north of Lonely Track Road require undercutting of about 2.5m of Waitemata Group soils to establish a competent foundation and placement of a 300mm thick drainage layer to alleviate groundwater rising into the embankment. The depth of undercut reaches about 5m at Stn 22660 in order to remove slip material prior to placement of the sidling fill on a 3H:1V inclined slope.

4.1 Fill Embankments over thick alluvium

Site specific foundation treatments have been undertaken where fill embankments are placed on soft compressible alluvium of thickness greater than 3m. The foundation assessment for the Okura Bridge approach fill and the fill embankments on alluvium in Sector A1 south of Wright Road, were based on CPT Piezocone and Geonor shear vane field test results, laboratory triaxial and oedometer tests. Correlation between CPT and vane strength is poor at most sites (Figure 3).

The Geonor shear vane results have been used for the stability design. Total stress triaxial tests have been used to confirm the conservative nature of the strength increases in the alluvium assumed to occur during consolidation under the fill loads for the various stability scenarios analysed. (Figure 4). The slope stability of embankments at seven sites were analysed using SLOPE/W. The various scenarios analysed were a full height fill with and without geogrid / geotextile reinforcement, immediately after construction (one month) and following 50% and 100% consolidation of the soil with and without wick drains. Settlements were calculated using SIGMA/W.

The acceptance stability criteria adopted for design are as follows:

- SF>1.2 immediately after construction. Construction period assumed to be one month for geogrid strength reduction and soil consolidation purposes.
- SF>1.5 after complete consolidation of the alluvial foundation soils with appropriate strength reductions due to creep in the geogrid (100%) and geotextile reinforcements.
- SF>1.0 after complete consolidation of the alluvial foundation soils and seismic coefficient of 0.15g.

Consideration was given to accelerating the rate of settlement by installation of wick drains. Because of the high cost of installation the client decided to tolerate up to 200mm of post-construction settlement (i.e. time period after pavement construction which will be about 18 months after the fill construction), and up to 100mm of differential settlement across the embankment, with local remediation by pavement shape corrections at 10 year intervals. The exceptions are the up to 9m high fill north of Okura where wick drains at 2m centres have been utilized to accelerate settlement in the order of 600mm and the 9m high fill south of Bawden Road where total post-construction settlement will be in the order of 400mm.

The types of specific foundation treatments undertaken at specific sites are tabulated below. The 200kN polypropylene geogrid was used for improvement of the short term stability condition only because of the strength reduction with creep. Adequate long term stability is achieved by consolidation settlement of the alluvium, with the exception of the Okura fill embankment where some long term reinforcement is still required to meet design stability conditions.

Extent of Fill	Maximum Fill Height	Max. Depth of Alluvium	Foundation Treatment	Calculated Post Construction Settlement
13520-13880m	5m	5m (stiff at 9m)	None	163mm (90% in 12.4 yrs)
16540-16860m	5m	5m	Geogrid (one layer of Tensar ER200)	160mm
18910-19180 Sth Bawden Rd	9m	9m	4.5m high, 20m wide side berms	440mm
20260-20460m North Okura	9m	12m	Wick drains spaced 2m in triangular grid. High strength polyester geotextile with tensile strength of 600 kN/m	91mm
21000-21500m Sth Wright Rd	5m	14m	Geogrid (one layer of Tensar ER200)	202mm (90% in 6.5yrs)
21620-21900m	5m	6m	Geogrid (one layer of Tensar ER200)	129mm
22400-22460m	15m	6m	Undercut and replace with engineered fill for stability.	-

5 COMPACTION PROPERTIES OF THE SOIL

5.1 Sector A1

Waitemata Group soils require an average 14% drying from field to optimum compacted water content. To enhance the rate of the bulk fill earthworks, consideration was given to compacting soils wet of optimum. A minimum undrained shear strength of 130 kPa for the compacted bulk fill was found to be required to achieve adequate long term stability of the higher 2H:1V fill slopes. This is based on effective stress triaxial and oedometer tests undertaken on samples compacted at and slightly wet of optimum. Most Waitemata soils will need to be dried back to within 1% to 2% of optimum to achieve 130 kPa strength. Bulk fill placed in fills less than 6m height require only 100 kPa which usually can be achieved at 3% to 5% wet of optimum.

5.2 Sector A2

Northland Allochthon (Onerahi Chaos) mudstone requires only light ripping on account of its fractured nature. The rock is typically 2% to 5% dry of optimum and requires wetting up and compaction using heavy rollers to achieve sufficiently low voids (≤8%) for permanence under wet conditions. The Clegg hammer is used for determination of compacted strength because of the granular nature of the broken up rock. A CIV ≥7 and ≥10 is required for <6m and >6m high fills respectively. Laboratory tests show a relatively poor correlation between CBR and CIV (Figure 5), particularly in the lower range of compacted strengths.

6 STRUCTURES

6.1 Retaining Walls

Proprietary soil nailed walls with grass panel facings have been used to retain a 6m high slope south of the Greville Road Interchange and a 8m high slope north of Lonely Track Road. This wall type was chosen because of the minimal amount of excavation required behind the face of the wall, the aesthetic look of the grassed face, and because of the minimal drainage requirements behind the wall (at the Greville site drainage may have intercepted leachate from the adjacent landfill; at the Lonely Track site drainage may have had a negative impact on established trees up slope of the wall). Concrete panel and Keystone block walls with steel ladder and geogrid soil reinforcing have been utilised for bridge abutments and culvert ramps.

6.2 Bridges

Bored friction / end bearing piles, reinforced earth / MSE (mechanically stabilised earth) abutments and pad footings have been used to found the bridges. The investigation for the bridge structures involved rig bores with SPT's at 1.5m intervals. In the Waitemata Group terrain UCS tests were performed on rock samples to assess bearing capacity, while in the Northland Allochthon terrain, a number of pressuremeter tests were undertaken (by Geotechnics Ltd) to qualify the use of SPT for the determination of bearing capacity. The fractured nature of the Northland Allochthon rock and its propensity to dilate make bearing capacity determination from laboratory tests difficult.

An allowable end bearing pressure of 2.0 MPa was adopted for bridge piles founded in Northland Allochthon rock. The required insitu strength for founding was assumed to be either the pressuremeter limit pressure P_l>2.0 MPa or SPT N>50. The relation between P_l and SPT is shown in Figure 6.

Allowable end bearing pressures for bridge piles in Waitemata Group rock ranged from 1.2 to 2.7 MPA. Longer piles had a significant skin friction component.

7 ACKNOWLEDGEMENTS

The authors wish to thank P Kelsey (Earthtech) who peer reviewed all stages of the ALPURT Sector A geotechnical investigation, and the client Transit New Zealand for approval to present this paper.

FIGURE 1
LOCALITY & GEOLOGY PLAN

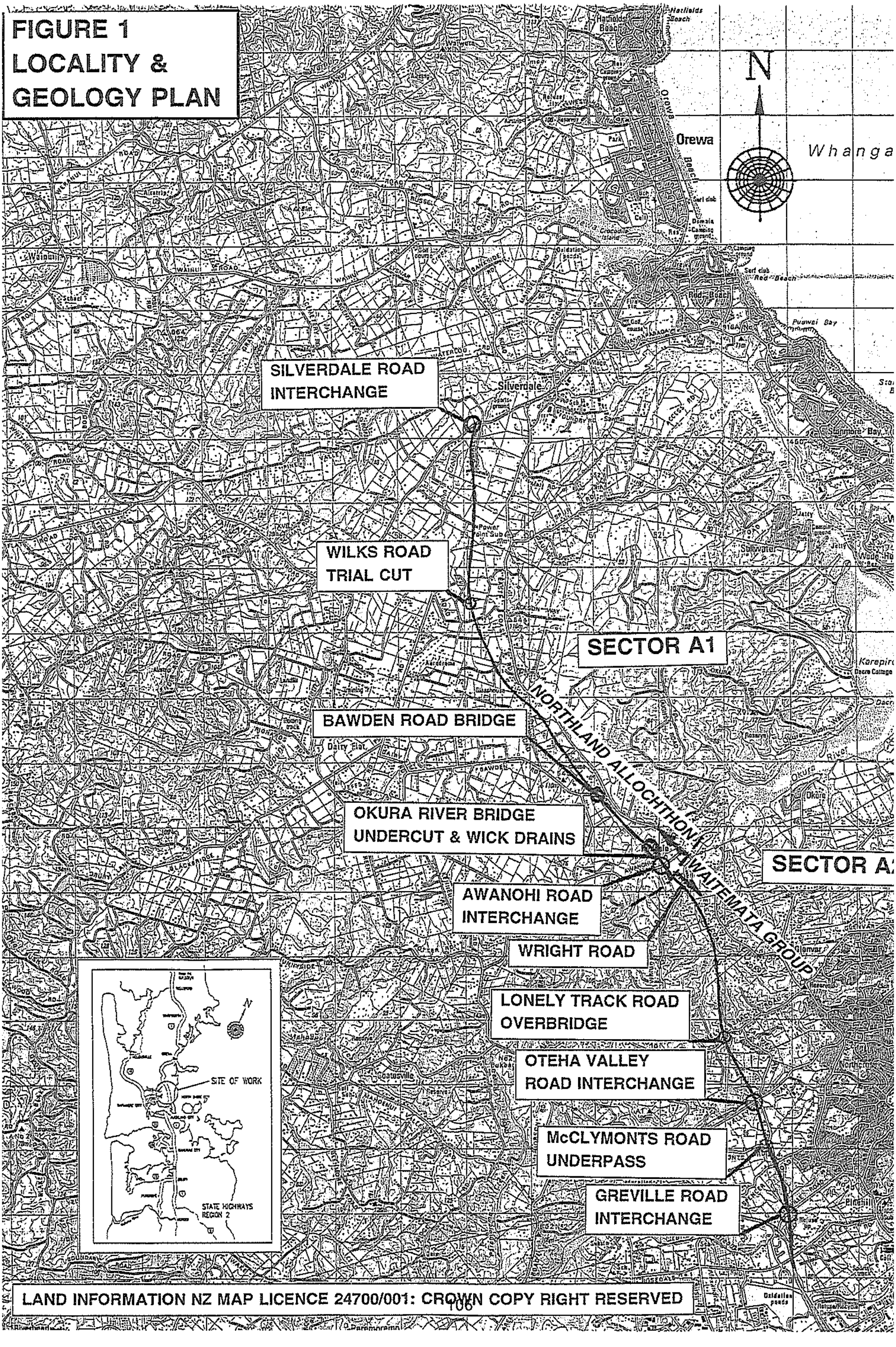


Figure 2 Effective Stress Strength Parameters for Waitemata Group and Northland Allochthon Soils

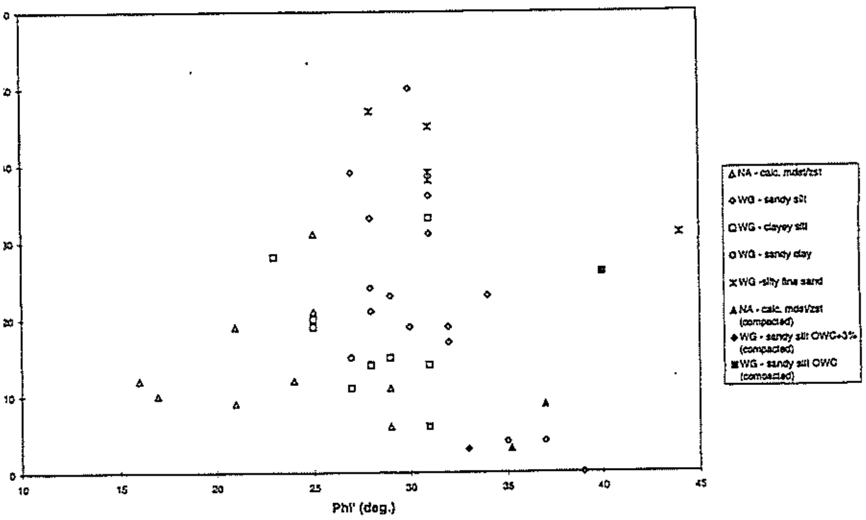


Figure 3 - Relationship between CPT Qc and Geonor Cu

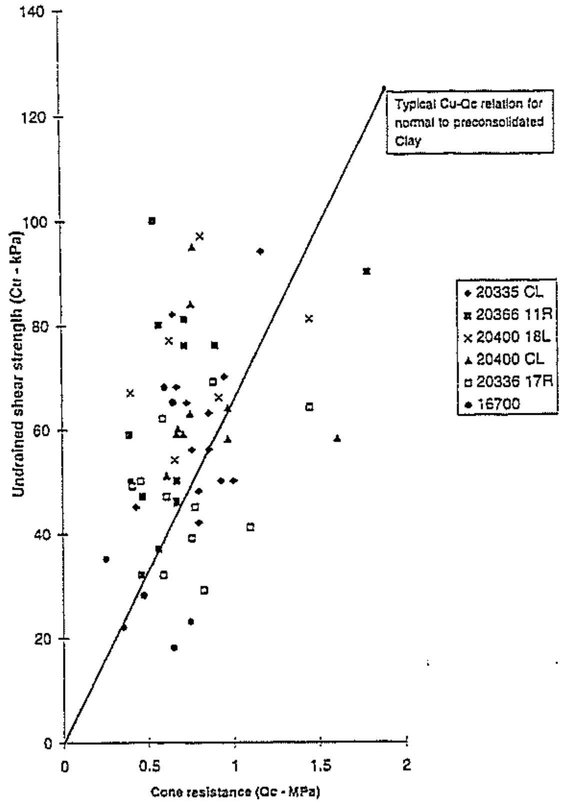


Figure 4 Relationship between Shear Strength and Consolidation Pressure from Fill Loading

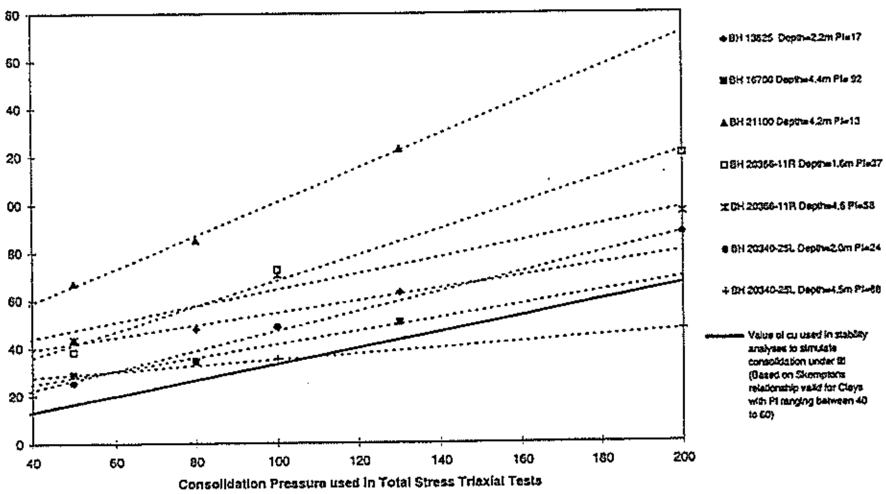


Figure 5 CBR VS CLEGG IMPACT VALUE CHART

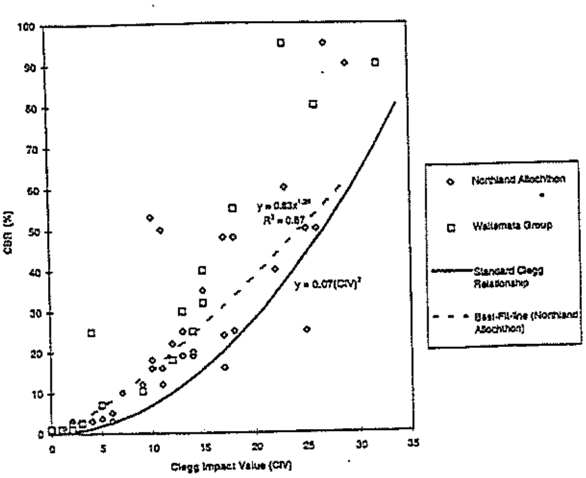
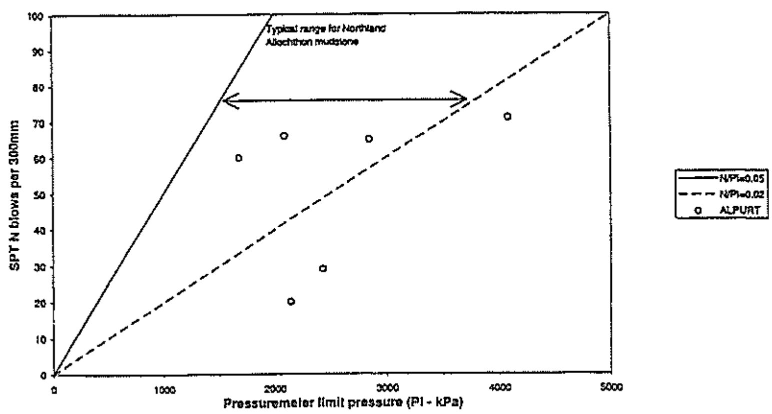


Figure 6. SPT - Pressuremeter limit pressure relation



ALPURT MOTORWAY - SECTOR A: PAVEMENTS DESIGN AND CONSIDERATION

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SUMMARY

This paper summarises the pavement design procedures, pavement alternatives, and considerations of the pavement for the 15 km ALPURT extension to the Auckland Northern Motorway. Two contracts were let in late 1997, Sector A1 the southern 7 km, and Sector A2 the northern 7km. At this time the motorway is under construction but the pavement construction has not commenced and the details of the methodology are still under review.

ALPURT is the first major highway to be designed under the AUSTROADS Pavement Design introduced by TNZ in 1996 to replace the 1988 NRB State Highway Design and Rehabilitation Manual. The design brief called for a conventional flexible unbound aggregate pavement on a stabilised subgrade with a chip/friction mix seal. With a design traffic design of 1.7 x 10^7 ESA's the pavement designed under AUSTROADS is significantly thinner than would be the case under the NRB design. This emphasises the risk of using low grade materials and poor construction techniques. All previous experience on the current Auckland motorways has been with the thicker NRB pavements and while pavement performance has been adequate, it has not been shown to be conservative.

The pavement construction for the motorway and ramps totals some 320,000m^2 and the cost is a significant proportion of the total construction cost of the project. The quantity of aggregate required (100,000m^3) is unprecedented for Auckland roading projects to date. To minimise the aggregate requirement, and to qualify the costs of more robust materials, alternative pavement systems were investigated. Alternative options reviewed include structural Asphalitic Concrete, unbound over Lean-Mix concrete (upside down pavement), and a Portland Cement rigid concrete pavement.

The paper will outline the traffic design procedures, the choice of design parameters for mechanistic design, and compare the Present Day Worth Costs of the pavement options. The results of repetitive load triaxial tests on representative high grade and marginal aggregates will be shown. The results will indicate that the use of a stabilised local marginal aggregate for subbase in place of a higher grade aggregate is promising. Thoroughout the paper AUSTROADS limitations will be reviewed relative to practical considerations and design philosophy.

1 BACKGROUND

The pavement construction for the 15km of motorway and ramps for ALPURT Sector A totals some 320,000m^2 and the cost is a significant proportion ($13M or 25%) of the total construction cost of the project.

- The design brief called for a conventional flexible unbound aggregate pavement on a stabilised subgrade with a chip/friction mix seal designed to AUSTROADS. This unbound pavement would use some 100,000m^3 (130,000m^3 loose) of aggregate. This quantity is unprecedented for Auckland roading projects to date.

- The granular pavement is not without risk as the AUSTROAD design has a thickness considerably less than the superseded 1989 NRB design for a highly trafficked motorway (2 x10^7 ESA's for 20 years design life). This motorway would be the first test of the AUSTROADS design in NZ under high traffic loadings

- Although only low risk premium aggregate was considered for both base and subbase, consideration was given to subbases constructed from appropriate marginal aggregates with a suitable stabiliser additive, provided they could be shown to have a similar performance to premium aggregate over the design life.

- To minimise the aggregate requirement and reduce the design risk, practical, low risk, alternative pavement systems were investigated that could offer long term cost savings and benefits.

2 SUBGRADE EVALUATION

The undulating to steep topography and design vertical alignment means that some 50% of the subgrade will be compacted for embankment or sideling fills and 50% will be within cut at a natural or recompacted state. The terrain of this project is such that the lengths between cut and fill are very short, generally in the range 100m to 300m with a mean of 210m. Fills and cuts are up to 20 metres in height. For practical and economic reasons the subgrade in the cut areas will not be undercut and recompacted unless significantly disturbed during the formation of the subgrade. The insitu soils/rock is assumed to be fully saturated and at a permanent strength.

- The geology and corresponding subgrade conditions of Sector A1, the southern 7 km length, consists of firm weathered cohesive soils over the parent Waitemata Group sandstone/siltstone "bedrock". The "bedrock" is at some depth and, in general, the firm weathered soils, consisting of clays and silts (PI 21 to 39), forms the subgrade in either a natural or recompacted state.

- The geology and corresponding predominant subgrade of the 8km northern length consists of Northland Allochthon (Onerahi Chaos) geology, a very fractured weak siltstone with a thin clay matrix. The firm to stiff shale like material will form the subgrade in either a natural or recompacted state.

The strength of the insitu cut subgrade CBR was evaluated during the geotechnical investigation with the Scala (DCP), Pilcon Shear Vane, and Dutch Cone (CPT) using conventional correlations. The permanent long term strength of the recompacted fill subgrade was assessed using the soaked CBR value of representative samples compacted to NZS4402 Standard Compaction. Reduced CBR values are summarised in Table 1, as would be expected, some variability exists.

Subgrade Conditions	Sector A1- Waitemata Group Soils	Sector A2 - Northland Allochthon Soil/Rock
Insitu CBR	2 to 13 ($\bar{x}$ 10)	3 to 11 ($\bar{x}$ 10)
Unsoaked Compacted CBR (NZS Max DWD/OWC)	10 to 18	11
Soaked NZS Compacted CBR (NZS Max DWD/OWC)	3 to 10 ($\bar{x}$ 5)	3 to 8 ($\bar{x}$ 5)
Design CBR (10% Percentile of Insitu and Soaked)	4	4
Stabilised with 3% hydrated lime, Soaked CBR	>20	>30

Table 1 Summary of Subgrade Testing and Evaluation

With the short intervals between cut and fill and variations in CBR values over short lengths of cut it was considered impractical to vary the pavement design, and more specifically the pavement drainage, to accommodate a higher CBR of selected borrow soil in the engineered fills and the deeper cut areas. Also it was not considered practical or economic to undercut and recompact the subgrade. On this basis the permanent CBR value for the subgrade was set at 4 (10% percentile of the assessed values) for design with local undercutting of weaker materials.

Both materials responded well to stabilisation and soaked CBR values of at least 20 were obtained using 3% lime

3 ENVIRONMENT - WATER AND PAVEMENT PROTECTION

- Sector A1: The Waitemata Group soils have a high water table but as this is generated by local infiltration a downward flow to the regional water table should prevail. Experience has indicated that subsoil/transition drains under the shoulders and median will control, and prevent, water ingress into the upper pavement layers.

- Sector A2: The subsurface conditions are such that the water table is near the surface and of high significance is the upward flow artesian effects in the Onerahi Chaos geology. This needed specific attention in the cut areas. The measured permeability of the material is low and the conventional granular subbase,in addition to the one metre deep subsoil/transition drains under the shoulders and median, will control the flow and pressures. A full depth bound pavement will require a specific drainage blanket between the subgrade and the bound pavement in the cut areas to alleviate pressure heads under the pavement.

4 DESIGN LOADINGS

The current NZ supplement included the “weigh in motion” results from Sulphur Point (Northern Motorway). These results in conjunction with the projected AADT and the predicted 15% HCV of traffic growth have been used to determine the design loading. All design loadings are in terms of Equivalent Standard Axle (ESA) and Commercial

Vehicle Axle Groups (CV) as specified by AUSTROADS. Unfactored ESA values of 1.75×10^7 and 1.35×10^7 were determined for South and North of Oteha Valley Rd respectively for a 20 year design life. Where appropriate these values have been modified for flexible bound pavements as required by AUSTROADS. The attachment summarises the design loading for unbound, and bound pavements (20 years design life) and a long life Rigid Concrete pavement (No. of HCV over 40 year design life). A later addition is the ESA value 5×10^7 for a long life (40 year) full depth Asphaltic Concrete alternative pavement.

5 PAVEMENT OPTIONS

The basic design of the conventional flexible unbound aggregate pavement uses a high grade aggregate subbase (AP65) and a TNZ M/4 base aggregate topped with a non structural chip seal. At 2 t o 3 years a 25mm friction course will be added. The subgrade is made up of a design CBR of 4 overlain by 300mm lime stabilised subgrade improvement layer with a CBR in excess of 20.

Comparisons have been made with alternative pavement systems that could offer long term cost savings and benefits. The alternatives reviewed for the project include structural Asphaltic Concrete, unbound over Lean-Mix concrete (CTB) ("upside down pavement") for the traditional 20 year pavements. In addition long life rigid Portland Cement concrete pavement was reviewed for a 40 years design life. A further alternative pavement was added when the contractors submitted a long life (40 year) full depth Asphalitic Concrete alternative. All are to be constructed with premium aggregates. Pavement with modified (stabilised) marginal aggregates are a separate alternative to those above.

6 PAVEMENT DESIGN

Pavement design to determine dimensions has been carried out in accordance with AUSTROADS standards with the exception of pavements containing structural Asphalitic Concrete for which the Shell Manual and supplements has been used [6,7]. The Queensland Transport Pavement Design Manual [5} has been used for the philosophy and material requirement of lean-mix and the "upside down pavement".

Mechanistic pavement design analyses (Circly) have been carried out as per AUSTROADS [1] and TNZ New Zealand Supplement [2] with a tyre pressure of 750kPa. The moduli utilised and the methodology largely conformed to AUSTROADS. The AUSTROADS Rigid Concrete pavement design procedure has a proforma analytical method based on the tensile strength of the concrete and detailed traffic loading. The following considerations have been made:-

- Although AUSTROADS contains the nominal design for TNZ pavements, the Shell Manual and Supplements has been utilised as a basis for the alternative structural asphaltic pavements. Shell addresses the difference between "on the road" fatigue characteristics to those determined in laboratory testing. For this reason there is a multiple of 15 to the fatigue life (critical tensile strain) of Asphalitic Concrete for bitumen typically used in NZ [3] using the Shell methodology over that of AUSTROADS (The original 1989 NRB/TNZ manual used a multiple of 10). AUSTROADS uses the fatigue characteristics of the laboratory tests results without modification even though the guide is modelled from Shell technology. For given conditions there is a requirement for a thicker asphalt layer using the AUSTROADS Guide in place of the Shell Manual

- The Queensland Transport Pavement Design Manual has been used for the philosophy and material requirement of the alternative lean-mix "upside down" pavement and attention should be drawn to the greater potential for cracking in the event that a low grade aggregate is utilised.

- Mechanistic analyses make no allowance for the practicality of construction practices. For this reason the pavement thickness required from the analyses have an additional 20mm added to the subbase in all options, apart from the rigid pavement, to act as safety factor against construction subgrade punching and similar. For similar reasons the pavements have been designed with a 200mm subgrade improvement layer in place of the contract thickness of 300mm. In addition the modulus of the subgrade improvement layer was reduced from a theoretical value of 200MPa (partially cemented) to 100MPa (modified) to compensate for the possibility of the tensile stresses and/or strains exceeding that permissible at the base of the layer.

6.1 Unbound Aggregate Pavement

The conforming unbound AUSTROADS design with a 200mm lime stabilised upper subgrade (subgrade improvement), over the subgrade proper resulted in 165mm/170mm of base and 175/180mm of subbase. These result from an ESA value of 1.35×10^7 increasing to 1.7×10^7 between Oteha Valley Road and Greville Road. There are lengths of non arterial off ramps for which a lesser thickness has been prescibed. The subgrade used for design is a CBR of 4 (modulus 40 MPa) overlain by a 200 mm insitu stabilised layer (reduced from 300mm for design) of CBR 20 (Design modulus of 100MPa). To minimise construction tolerance risks 20mm has been added to the subbase thickness as descibed above. A summary of the results is given below.

Subgrade For Design: 200mm Modulus 100 MPa over Modulus 40MPa (CBR 4)

Position	Loading	Subbase (mm)	Base(mm)	Total (mm)
North of Oteha Valley Rd	ESA 1.35×10^7	175+20	165	360
South of Oteha Valley Rd	ESA 1.75×10^7	180+20	170	370

6.11 Technical Reservations of the Unbound Aggregate Pavement using the AUSTROADS Design

- The 16km of ALPURT motorway is the first major highway (ESA 1.75×10^7) to be constructed under the AUSTROADS design imposed by TNZ. As such, the unbound granular pavement is significantly thinner than would be the case if the pavement was designed prior to 1996 when the 1988 NRB State Highway Design and Rehabilitation Manual would have been utilised. In this case the pavement is a very significant 150mm thinner (30%) than would have been the case if the 1988 NRB manual design had been utilised (370mm against 520mm). Further, if the AUSTROADS pavement thickness of 370mm was applied to the 1988 NRB design the maximum ESA is equivalent to 2×10^6 or 10% of the required traffic loading. This puts into perspective the risk of using low grade materials and poor construction techniques. All previous experience on the current Auckland motorways has been with the thicker 1988 NRB pavements and while pavement performance has been adequate, it has not been shown to be conservative. The equivalent traffic loadings that will be generated on this AUSTROADS designed pavement are unprecedented in New Zealand. All recent highly trafficked pavements (ESA $>10^7$), ie SH20 and SH1 Bombay, have been designed using 1988 NRB designs.

6.2 Alternative Pavements

6.2.1 Flexible Asphalitic Concrete Pavement (20 year life)

The provisional design for the Asphalitic Concrete option over the main thoroughfare has Mix 20 of modulus 2000 MPa over quality subbase capped with a non structural chip seal. At a later stage a 25mm Friction Course may be added.

Subgrade For Design:200mm Modulus 100 MPa over Modulus 40MPa (CBR 4). AC Modulus 2000MPa

Position	Loading	Subbase (mm)	Asphalitic Concrete (mm)	Total (mm)
North of Oteha Valley Rd	ESA 1.48×10^7	170+20	110	300
South of Oteha Valley Rd	ESA 1.92×10^7	180+20	110	310

6.2.2 "Upside Down" Lean Mix Pavement

The design was based on a micro-cracked "upside down" lean-mix option. 125mm of premium M/4 base unbound aggregate is constructed over lean-mix manufactured from quality aggregate (preferably mixed in a pug mill).

Subgrade: 200mm Modulus 100MPa over 40MPa (CBR 4), under Micro-cracked Lean-Mix Modulus 600MPa - Subgrade Strain Criteria (Permanent Deformation in Subgrade.)

Position	Loading	Lean-Mix mm	Base Aggregate mm	Total mm
North of Oteha Valley Rd	ESA 1.48×10^7	180+20	125	325
South of Oteha Valley Rd	ESA 1.92×10^7	190+20	125	335

- The 125mm thickness of premium base is the recommended minimum requirement to alleviate shrinkage and structural cracks reflecting to the surface [5,7].

- Like all unreinforced concrete materials lean-mix will fatigue crack if the loading results in tensile stresses in excess of 50% of the failure stress and this would occur after some 5.0×10^5 to 10^6 load repetitions. At this fatigue point the lean-mix is assumed to be a dense unbound base and the design criteria changes from tensile strength in the lean-mix to strain in the subgrade. In general a cracked Lean-Mix is an excellent subbase with a 600 MPa moduli, providing the pavement is appropriately designed. The Queensland Transport Pavement Design Manual has a number of satisfactory designs based on leanmix cracking within the design loading period. In the pavement analyses studies for this project both an uncracked (E=10000 to 13,500MPa, compressive strength range of 4 to 7 MPa) and a thinner cracked (E=600MPa) lean-mix subbase layer were considered.

- The moduli of E=600MPa for cracked Lean-Mix is significant to the design. The value has been sourced from a number of references [4, 9].

6.2.3 Plain Rigid Concrete Pavement PCP (Long Life Pavement - 40 year design)

- The provisional pavement considered was a jointed unreinforced Concrete pavement (preferably with a 28 day design characteristic flexural strength in excess of 4.25MPa - AUSTROADS Section 6.5.3) over low shrinkage lean-mix (7MPa). Saw cuts would be required at a nominal 5 metre interval to control shrinkage cracking and tie bars as prescribed by AUSTROADS Pavement Design for longitudinal joints. Shoulders have been included (0.6m extended slab) in the design to invoke a lesser thickness of concrete and optimise the material quantities.

- The subgrade is made up of a CBR of 4 without a stabilised subgrade improvement layer

- No initial chip seal running course will be required provided it can be shown that the skid resistance, roughness count, and noise generation of the proposed concrete surface complies with TNZ and TA requirements. Slip form construction is considered essential to obtain an adequate surface.

- The AUSTROADS design method has been used for the assessment with the design loadings given in the attachment and the axle proportions based on the NZ Supplement. An appropriate load safety factor of 1.2 has been utilised .

- Roller compacted construction is a possible alternative but the concrete would almost certainly need to be overlain by 25 mm of friction mix to control roughness and obtain the necessary skid resistance,

Subgrade: (CBR 4)

Position	Loading (40 years) (No of CV axle groups)	Lean-mix mm	Concrete mm	Total mm
North of Oteha Valley Rd	10.5×10^7	100	240	340
South of Oteha Valley Rd	13.1×10^7	100	240	340

The 100mm thickness of lean-mix subbase could be substituted by 150mm of bound (5% cement) subbase aggregate.

7 MATERIALS

7.1 Subgrade improvement layer:

- In all designs apart from Rigid concrete, a subgrade improvement layer will be constructed by stabilising insitu the upper 300mm of the formed subgrade. The stabilisation considered is 3% by weight of hydrated lime (or quicklime). Both Sectors A1 and A2 have laboratory compacted CBR values in excess of 20 with this percentage of lime.

- A review of this CBR value in analyses indicates that the tensile stress and/or strains that develop at the base of this layer may exceed the allowable for a cemented material when compared to the requirement outlined in AUSTROADS and the New Zealand Supplement. For this reason the lime stabilised layer has been considered a modified layer with a conservative effective modulus of 100MPa.

- The structural requirements of the subgrade improvement layer is significant. If the stabilisation fails the pavement will fail prematurely. For this reason quality assurance of the insitu lime stabilised layer is essential prior to placement of the subbase. At least 4 days has been specified to allow the lime to cure with the soil. If adverse cold and wet weather is prevalent at the time of subgrade preparation lime stabilisation will be unreliable and a granular subgrade improvement layer will be required.

7.2 Granular Layers

- High grade aggregate conforming to TNZ M/4 has been specified for both base and subbase for the conforming unbound granular pavement and in the alternative pavements. To supply ALPURT a minimum haulage distance of some 40 to 60km will be required.

7.3 Alternative Modified Marginal Aggregates and Repetitive Load Triaxial Testing

- Stabilised lower grade durable aggregates were considered as an alternative to a premium aggregate in the subbase layer. Such aggregate must have reasonable strength (crushing resistance) but can contain a moderate amount of active fines. The local Flat Top Quarry aggregate with a haulage distance of some 20 km was considered a likely candidate.

- The Flat Top Quarry aggregate was tested with a range of stabilisers including lime, cement and KOBM - a New Zealand Steel slag by-product. In addition to the standard soaked CBR testing, repetitive load triaxial testing was carried out on the modified aggregate. Comparisons between unstabilised and stabilised were significant with all stabilisers. Under relatively harsh conditions of 92% relative density and full saturation the modified aggregate performed as a high grade aggregate but unstabilised the material strained to failure at low repetitions. The results are attached together with the results of a TNZ M/4 aggregate from the Wharehine Quarry near Warkworth.

7.4 Lean-Mix Aggregates

- The "upside down" and Rigid Concrete pavements have a cemented aggregate in place of the subbase. This is a layer where some 4.5% of cement is added to the aggregate, with the minimum of water, to produce a lean concrete of 5 to 7 MPa compressive strength (7 day). The material would preferably be paver laid from a batching plant source.

- As with all concrete products Lean-Mix can shrink and crack during the curing process. In a controlled environment this cracking can be alleviated but in road construction this is difficult and, in general, cracking is designed against or controlled. In the case of the "upside down" pavement any cracking must penetrate the overlying base aggregate before in it is detrimental to the pavement. The Queensland Transport Pavement Design Manual [5] has been used for the philosophy and material requirement of Lean-Mix and the "upside down pavement". The most relevant recommendations from this manual are as follows:-

    - The minimum thickness of quality unbound aggregate base cover, above the Lean-Mix, to alleviate crack reflection is 125mm.

    - To minimise shrinkage the Lean-Mix aggregate should be have a low percentage of plastic fines. (PI<4).

8 RUNNING SURFACE LAYER

For vehicle noise control a 25mm non structural friction mix running course is recommended although for the first 2 to 3 years a less costly chip seal is considered more appropriate. Such a running course will allow the unbound pavement to "settle down" prior to an overlay with the friction course.

9 CONSTRUCTION MONITORING

Construction confirmation monitoring will be carried out as the pavement construction develops. The implementation of pavement design includes close liaison between construction and design groups to gain optimum utilisation of materials and identify variations in subgrade conditions. The following has been included as part of the contracts:-

- A Benkelman Beam will be used at subbase and base level for displacement and curvature to assess uniformity and confirm design.

- FWD testing will be undertaken on the finished pavement to produce an "As Built" profile.

10 CONSTRUCTION COST COMPARISONS AND RISK

The assessed initial construction costs of the conforming pavement and alternative design options are summarised below. It must be emphasised that the unit rates of the materials have been obtained from recent contracts and are realistic but are for comparitive purposes only. They are not necessarily representative of a contract price for this project. The costs of the flexible pavements include a two coat chip seal but exclude the friction mix. A 25mm friction mix overlay, constructed largely for noise control, will be included as an maintenance item at 3 years.

Table 2 Assessed Initial Costs

Pavement Options	Total Thickness mm	Cost $/sq-m	* Total Cost $M	Risk	Reason
Unbound	360 -370	33.25-33.89	$13.8M	Low	Readily Maintained
Asphalitic Concrete	290- 300	56.25- 56.82	$22.9M	Low	Readily Maintained
"Upside Down" Lean-Mix	325 -335	31.38 - 32.09	$13.1M	Medium	Possible Crack Reflection
Rigid Concrete	340	51.06	$20.9M	Medium	Unfamiliar Technology

* Based on total main carriageway area of 381,359 m^2 inc. kerb/edge strips for both sides over the full pavement length.

11 ECONOMIC COMPARISON OF DESIGNS

An enonomic assessment of the design alternatives was compared to the conforming Unbound Granular pavement using a present worth of costs technique. The maintenance intervals, costs and the salvage costs have been reviewed for each alternative and along with the evaluated initial construction cost have been used in the assessment The results are summarised below for 20 and 40 years with discount values of 5% and 10%. Apart from Rigid Concrete the 40 year period assumed a complete reconstruction or overlay at 20 years.

Table 3: Comparison of Present Worth of Costs (PWOC) for 20 and 40 year Design Life

Pavement Type	Estimated Initial Construction Cost ($)	Present Worth Of Costs (PWOC) ($) * Discount rate			
		20 year analysis period		40 year analysis period	
		5%*	10%*	5%*	10%*
Unbound Granular	$13.8M	$17.6M	$18.0M	$24.3M	$21.8M
Asphalitic Concrete	$22.9M	$24.9M	$25.8M	$33.7M	$30.8M
Lean-Mix	$13.1M	$19.8M	$18.3M	$25.5M	$22.3M
Port. Cem. concrete	$20.9M	$19.7M	$22.8M	$28.5M	$26.8M

- Of the four pavement alternatives the most economical considering initial construction costs are the Unbound and Lean-Mix pavement options. Based on the assumptions used in the costing predictions the rigid Portland Cement and Asphalitic Concrete pavements would cost some $7M to $9M or 50% more than the Unbound Granular and "Upside Down" Lean-Mix pavements. The exact values should be treated with some caution as they are not necessarily representative of a contract price for this project.

- Performing a design comparison on the four options over the 20 year analysis periods - assuming discount rates of 5% or 10% - highlights a present worth cost increase for Unbound and Lean-Mix options and a decrease for the Asphalitic Concrete and rigid Portland Cement options. This is largely due to the lower maintenance cost of Asphalitic Concrete and high salvage cost of Portland Cement (fully recoverable at 20 years). Again the exact values should be treated with some caution and perhaps only the trend should be noted. A similar trend can be observed for the 40 year analysis periods. The accuracy of the input data is such that, perhaps with the exception of the 20 year design life Asphalitic Concrete, the results of the comparisons could be considered close. The suitability of the alternative pavements are more significant when the benefits, especially those of rigid concrete, are considered.

12 TENDER DOCUMENTATION AND EVALUATION

Both the A1 and A2 contracts specified and scheduled the Unbound Granular pavement as the conforming pavement but, in addition, allowed the submission of an alternative pavement under a set of prescribed conditions and the supply of design verification documentation.

- Both the successful contractors submitted as an alternative a rigid Concrete Pavement (both plain and reinforced) and a long life (40 year) full depth Asphalitic Concrete pavement. This latter pavement had not been evaluated in the design process and consisted of a 200 mm thickness of composite structural asphalt layers with properties designed to optimise the fatigue performance of the pavement. In addition one of the contractors submitted a modified subbase based on a KOBM stabilised marginal aggregate from their own quarry. Repetitive load triaxial testing has shown this modified subbase to be promising.

- The verification documentation of both the concrete and asphalt pavements had been submitted to both contractors through the respective industries. The data received from both industries was considered incomplete but an evaluation of what was presented indicated that both the plain concrete and the full depth asphalt pavements were very competitive when a PWOC analyses were carried out. The tendered unit rates for the asphalt pavement was significantly lower than that used in the design assessment.

- While an additional intial construction cost of some $7.5M would be require for the long life pavements the discounted PWOC was not significantly higher than the conforming Unbound Granular pavement and the benefits over the same are significant. Based on a combination of reduction in long term roughness, fuel saving, the reduction in aggregate usage, and the decrease in construction time, both the concrete and asphalt long life pavements had benefit/cost ratios that were promising. At this stage neither the concrete or asphalt could not be distinguished as a preferred alternative.

13 WHERE TO FROM HERE

At this point in time a long life pavements specification has been submitted to the contractors inviting alternative tenders for both of the alternatives for their respective contracts. The responsibilty for design has been placed on the contractors with the proviso that specific performance criteria outlined in the supplied design specifications are achieved.

14 REFERENCES

1 AUSTROADS Pavement Design, A Guide to the Structural Design of Road Pavements, 1992

2 AUSTROADS Pavement Design, A Guide to the Structural Design of Road Pavements, New Zealand Supplement, July 1997

3 Gerritsen A.H. and Koole R.C., Seven Years Experience with the Structural Aspects of the Shell Pavement Design Manual, 1985

4 National Roads Board, State Highway Pavement Design Manual, 1989

5 Queensland Department of Transport, Pavement Design Manual, 1990

6 Shell Pavement Design Manual, Asphalt Pavements and Overlays for Road Traffic, 1978

7 Tait J.B., Cement Stabilisation for New Zealand Roads, RRU Technical Recommendation TR5 , 1981

8 Ullidtz Per, Pavement Analysis, 1987

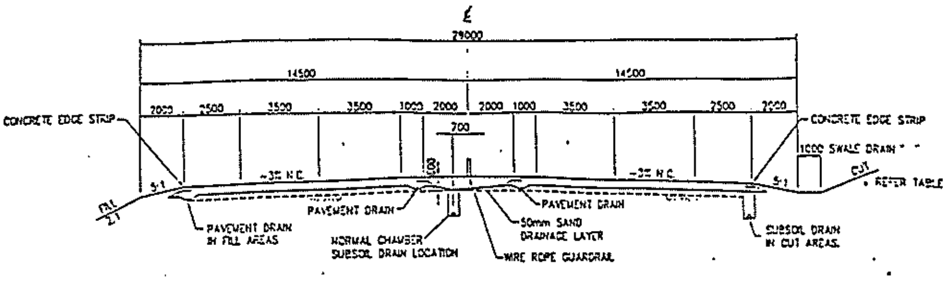
9 Vroombout F., Monteith R. And Sharp K.G., The Use of Interlocking Concrete Blocks on an Aircraft Pavement in Australia, Pave New Zealand, Proceedings Volume 2, pp 217, 1992

ACKNOWLEDGEMENTS

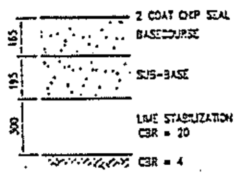
The author wishes to thank G Higgins (now of Higgins Contracting) who assisted in the design assessments, and D Hutchison who peer reviewed the reports, M Smith of TNZ Auckland for foresight, and Transit New Zealand for approval to present this paper.

ALPURT SECTOR A

TYPICAL SECTION



FOUR LANE TYPICAL CROSS SECTION



STANDARD CARRIAGEWAY A2, SILVERDALE INTERCHANGE AND SOUTH FACING RAMPS

PAVEMENT DESIGN PARAMETERS

12/05/97

Pavement Loading

Tyre Radius	92.93 mm
Tyre Stress	750 kPa

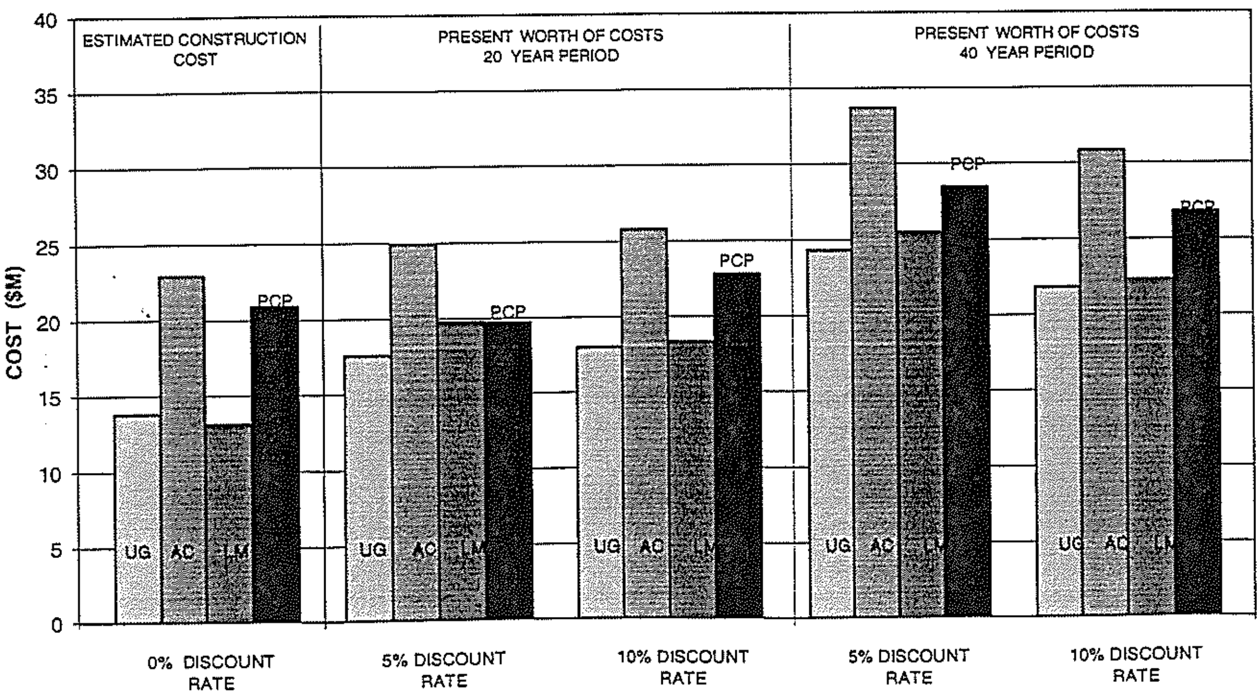
Design Traffic

Pavement Material Layer	Design Life Years	Design Traffic (N*) x 10^7 ESA Nth Oteha Valley Rd	Sth Oteha Valley Rd	Low Loading Ramps	Design Traffic Calc Method	Pavement Analysis Method
Unbound Granular Layer (Ne)	20	1.35	1.75	0.31	AUSTROADS	AUSTROADS
Asphaltic Concrete Layer (1.1Ne)	20	1.48	1.92	0.34	AUSTROADS	Shell 1978
Lean Mix Concrete Layer (20Ne)	20	26.90	34.90	6.20	AUSTROADS	AUSTROADS
Cemented Layer (20Ne)	20	26.90	34.90	6.20	AUSTROADS	AUSTROADS
Subgrade Layer (1.1Ne)	20	1.48	1.92	0.34	AUSTROADS	AUSTROADS
Portland Cement Concrete Layer (No of CV Axles)	40	10.1 (CV's)	13.1 (CV's)	2.5 (CV's)	AUSTROADS	AUSTROADS

Material Properties

Pavement Material	Modulus (MPa) * Varies	Poissons Ratio	Anisotropic	Interface
Basecourse (UG)	500	0.35	Yes	Rough
Asphaltic Concrete (AC)	2000	0.40	No	Rough
Portland Cement Concrete (PCP)	23500	0.20	No	Smooth
Lean Mix Concrete (LM)	13554	0.20	No	Rough
Cracked Lean Mix Concrete (Cr. LM)	600	0.35	No	Rough
Subbase (SB)	500/200*	0.35	Yes	Rough
Stabilised Subgrade (St.SG)	100	0.20	Yes	Rough
Subgrade (SG)	40	0.45	Yes	Rough

COMPARISON OF CONSTRUCTION ESTIMATE COSTS AND PRESENT WORTH COSTS



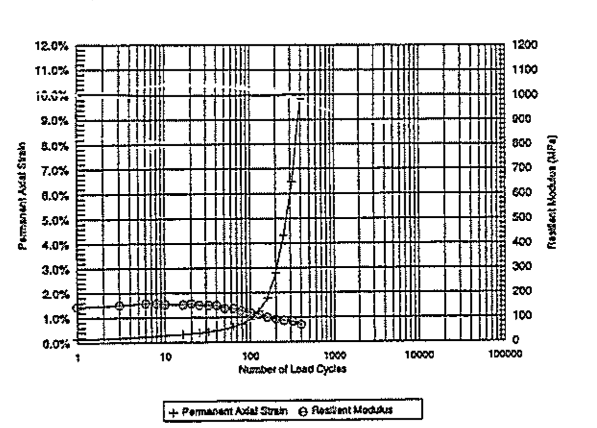
REPEATED LOAD TRIAXIAL TEST
Permanent Deformation

OPUS INTERNATIONAL CONSULTANTS

Project Name:	Alpurt Aggregate Testing	Test No:	2-97/100F
Client:	Opus International Consultants, Auckland	Test date:	10.10.97
Client ref:		Project No:	133343.10

Sampled by:	unknown	Date sampled:	unknown
Source:	Winstone Flattop	Sampling method:	unknown
Description:	Sub base Natural		

Sample preparation:	-19mm fraction	Sample diameter:	102.3mm
		Sample height:	200.0mm
Compaction method:	Vibrating hammer, 4 layers	Date compacted:	9.10.97
Comp moisture content:	9.0%		
Comp Dry Density:	2.042t/m³ (92% MDD)	Confining pressure	26kPa
Test conditions:	BP saturated, consolidated, undrained, ppm	Deviator stress:	349kPa



Test method in accordance with AS 1289.6.8.1: 1995

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Formerly Works Consultancy Services Limited

REPEATED LOAD TRIAXIAL TEST
Permanent Deformation

OPUS INTERNATIONAL CONSULTANTS

Project Name:	Alpurt Aggregate Testing	Test No:	2-97/100A
Client:	Opus International Consultants, Auckland	Test date:	23.09.97
Client ref:		Project No:	133343.10

Sampled by:	unknown	Date sampled:	unknown
Source:	Winstone Flattop	Sampling method:	unknown
Description:	Sub base GAP 65 + 2.5% Lime		

Sample preparation:	-19mm fraction	Sample diameter:	102.3mm
		Sample height:	200.0mm
Compaction method:	Vibrating hammer, 4 layers	Date compacted:	15.09.97
Comp moisture content:	11%		
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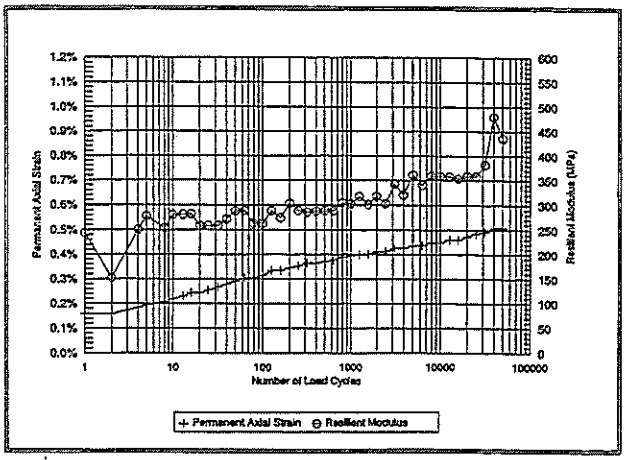
Test conditions:	BP saturated, consolidated, undrained, ppm	Deviator stress:	349kPa



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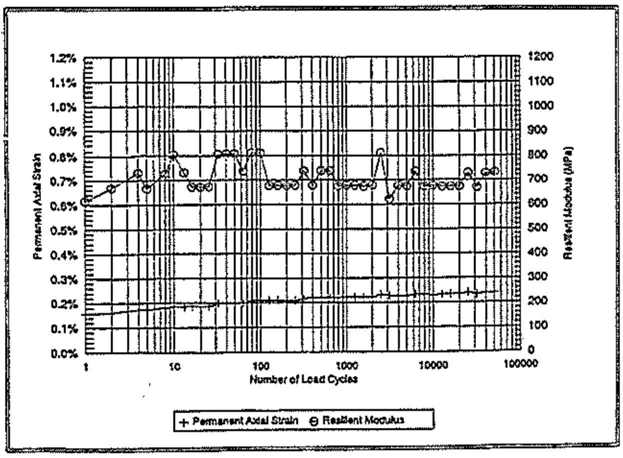
REPEATED LOAD TRIAXIAL TEST
Permanent Deformation

OPUS INTERNATIONAL CONSULTANTS

Project Name:	Alpurt Aggregate Testing	Test No:	2-97/99B
Client:	Opus International Consultants, Auckland	Test date:	19.09.97
Client ref:		Project No:	133343.10

Sampled by:	unknown	Date sampled:	unknown
Source:	Wharehine, Omaha	Sampling method:	unknown
Description:	AP40		

Sample preparation:	-19mm fraction	Sample diameter:	102.3mm
		Sample height:	200.0mm
Compaction method:	Vibrating hammer, 4 layers	Date compacted:	18.09.97
Comp moisture content:	5%		
Comp Dry Density:	2.15t/m³ (95% MDD)	Confining stress:	212kPa
Test conditions:	BP saturated, consolidated, undrained, ppm	Deviator stress:	493kPa



Test method in accordance with AS 1289.6.8.1: 1995

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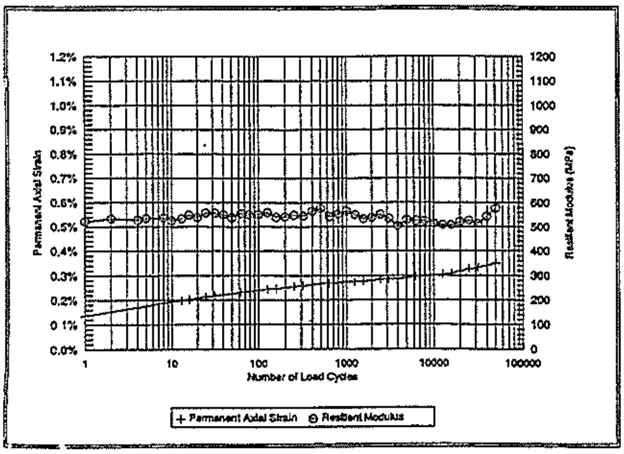
REPEATED LOAD TRIAXIAL TEST
Permanent Deformation

OPUS INTERNATIONAL CONSULTANTS

Project Name:	Alpurt Aggregate Testing	Test No:	2-97/100C
Client:	Opus International Consultants, Auckland	Test date:	30.09.97
Client ref:		Project No:	133343.10

Sampled by:	unknown	Date sampled:	unknown
Source:	Winstone Flattop	Sampling method:	unknown
Description:	Basecourse GAP 65 + 2.5% lime		

Sample preparation:	-19mm fraction	Sample diameter:	102.3mm
		Sample height:	200.0mm
Compaction method:	Vibrating hammer, 4 layers	Date compacted:	23.09.97
Comp moisture content:	11.5%		
Comp Dry Density:	2.109t/m³ (95% MDD)	Confining pressure	212kPa
Test conditions:	BP saturated, consolidated, undrained, ppm	Deviator stress:	493kPa



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THE DIRT ON MANGERE MOTORWAY (SH20)
Geotechnical Aspects of Design and Construction

Stephen Crawford
Tonkin & Taylor Ltd

SUMMARY

The State Highway 20 Mangere Motorway Extension project completed the link to Auckland International Airport and Manukau City and was the first significant design-build roading contract let in New Zealand by Transit NZ. The $40-million project involved the design and construction of 7 km of four lane highway, nine vehicular bridges and four footbridges, fill embankments up to 10 m high over soft ground and 3 main cuts typically 6 m up to 10m deep. In addition, some 200 m of tied back pre-cast walls and 800 m of cantilevered walls were required to retain areas within the designated urban motorway corridor.

This paper outlines the geotechnical aspects of design and construction including the results of settlement monitoring using a horizontal profilometer, lateral load testing of piles, pile driving, earthworks control, the stability of cuts and an outline of pavement conditions.

BACKGROUND

Auckland's south-western motorway had been planned in the 1960's when the route from Mt Roskill to Wiri was defined by the National Roads Board. The Mangere section was then largely pasture or market gardens. In the following 25 years significant urban development, including six schools and three large reserves, took place next to the motorway corridor. As a result five grade-separated interchanges and pedestrian/cycleway routes with four footbridges were incorporated in the landscaped motorway design (see Figure 1).

Five design-build consortiums bid for the project. A geotechnical investigation was carried out as a combined effort by the various consultants in the bidding consortiums prior to tender. Transit NZ awarded the contract to the Fulton Hogan Ltd / Albert Smith Industries Ltd / Tonkin & Taylor Ltd team for an alternative tender design.

Construction began in late 1993 with an unusually wet earthworks season. Variations during construction comprised an extra bridge, an off-ramp, a fire station on-ramp, 800 m of tied back and cantilever walls and two bridge crossings over the main sewer feeding the Mangere Sewage Treatment Plant. Despite consequent delays in an already tight programme, the project was completed on time in March 1997.

GEOTECHNICAL CONDITIONS

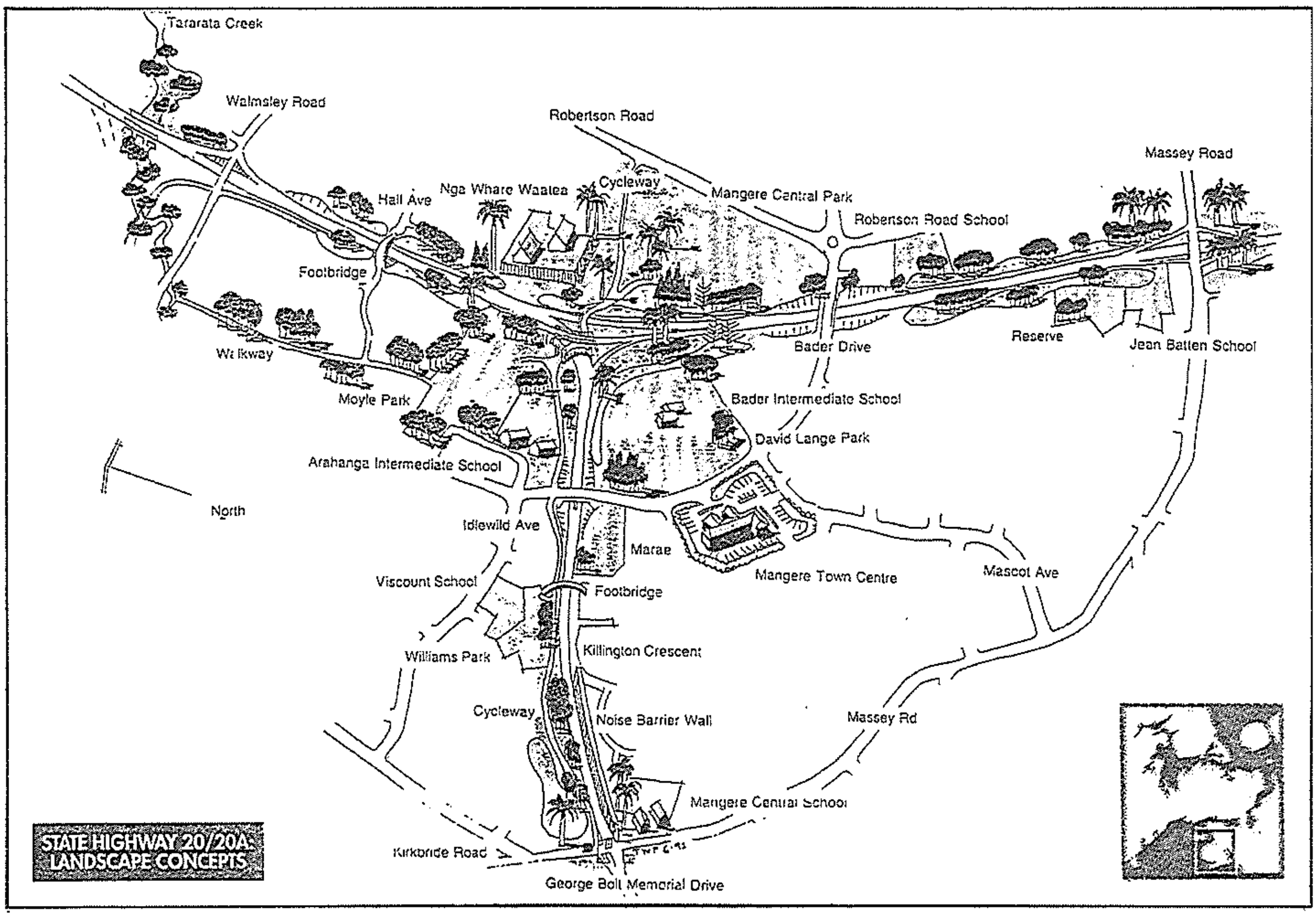
The geotechnical investigations prior to tender comprised some 150 Nº machine drilled boreholes, cone penetrometer tests (CPT's) hand augers and testpits. A further 30Nº boreholes and CPT's were required to investigate the additional bridges, ramps and walls.

The SH20 project site is underlain by three main geological units:

- A mantle of weathered volcanic ash (Auckland Volcanic Group) up to approximately 4 m thick
- Up to approximately 30 m thickness of sub-horizontally interbedded silty sand, silty clay and clayey peat, all of variable strength (Tauranga Group)
- Very dense uncemented sand of more than 10 m thickness (interpreted as the Kaawa Formation), underlying the Tauranga Group sediments. The top of the Kaawa sands was generally encountered at between RL – 15 m to RL – 20 m. The Upper Tertiary Kaawa sands overlie the Waitemata Group (Kermode, 1992).

The site located on the Manukau lowlands with topographic relief generally varying between RL 5 m and RL 20 m.

FIGURE 1: LAYOUT OF THE MANGERE MOTORWAY (SH20) EXTENSION PROJECT



EARTHWORKS

Materials and Compaction

The project earthworks involved cuts and fills of up to 10 m with some 0.5 million m^3 of soil being shifted to form the SH20 alignment. The overlying volcanic ashes generally proved to be easy to handle. However in the deeper cuts the Tauranga Group soils required significant drying to achieve compaction standards. The specification required bulk fill to be compacted to 95% maximum dry density (NZ Std Compaction), $\overline{C}_u \geq 110$ kPa and a maximum air voids of 10%. The fill materials varied depending on where they were obtained in the cut and the degree of mixing that occurred as a result of the method of excavation used. Determination of the appropriate maximum dry density was often very difficult. However, shear strength and air voids requirements were always met and these earthworks criteria are the basic requirements for the Alternative Compaction Specification (Pickens, 1980). The SH20 experience has shown this latter specification control to be more suitable than the "95%" approach for sites where fill materials are variable.

A wet earthworks season in the early stages of construction complicated the programme by ensuring that the settlement period for the larger fills extended well into 1995/1996 season, particularly in the swampy interchange (central) area of the site. Also the construction method for the 4 underpass bridges limited access to significant volumes of cut materials until nearer the middle of the project programme.

Settlement of Main Embankment

The interchange area is underlain by some 8 m of soft to firm silts and clays. The anticipated settlement beneath the 10 m high abutment fills for the 130 m long Interchange overbridge was 300 to 400 mm with a consolidation period of 12 to 18 months. This was analysed using 2-D elastic methods using the software FLEA(1991) and FLAC (1991), which allowed for assessment of multi-layered soil models and variable surface loads. Average correlations of cone penetrometer resistance (q_c) to drained Young's moduli (E') of $E' \approx 4.5 q_c$ for silt/clays and $E' \approx 2.5 q_c$ for sands were adopted. Sensitivity analyses were performed for the silts/clays using $E'=2$ to 6 q_c. These correlations were based on the 1-D consolidation test results and associated CPT's for the project, and typical ranges of deformation parameters (refs 6-8). We understand the five bidding parties at tender estimated the maximum embankment settlement at about 180 mm, 350 mm, 500 mm, 600 mm and 1 m.

Settlement Monitoring/ Profilometer

Actual settlements were measured with a *horizontal profilometer* beneath the fill which allowed earthmoving machinery to work unimpeded. The Tonkin & Taylor profilometer instrument uses a transducer to measure the change in pressure along access pipes. These pipes were laid in the foundation soils prior to filling. The sealed instrument is pulled through the empty pipe to determine changes in level along the pipe alignment, relative to survey control points at either end (see Figure 2). The accuracy of the measurements is typically ±10 mm over the pipe lengths of 80 m to 100 m.

Additional surface pins were monitored once the finished embankment profiles were achieved. Actual maximum settlements measured for the south and north abutments of the Interchange Bridge were 480 mm and 520 mm respectively. Results for the south abutment are presented in Figs 3 and 4.

Stability of Embankments and Cuts

Slope assessment was carried out using GALENA (1993). Soil strength parameters were based on laboratory tests (refer Table 1).

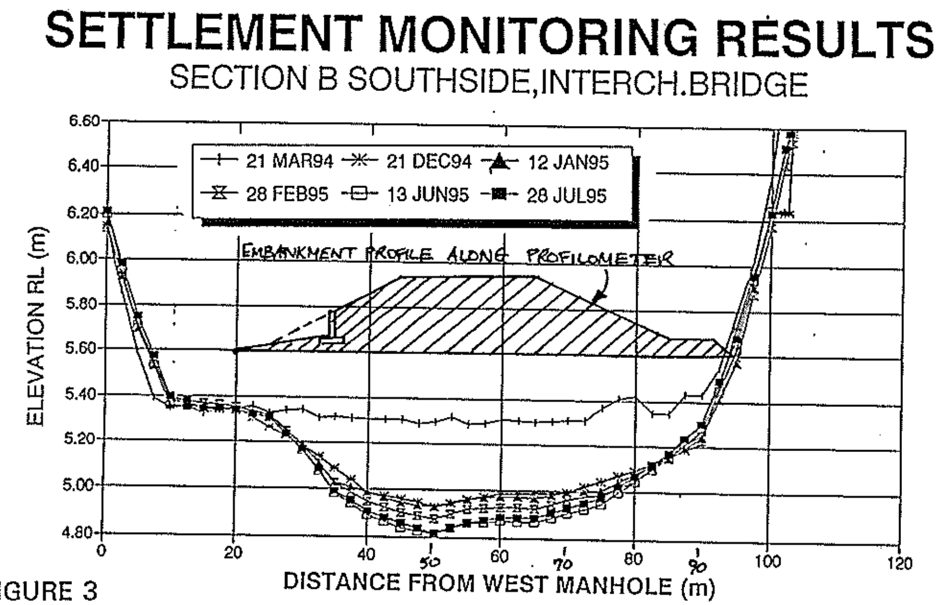


FIGURE 3

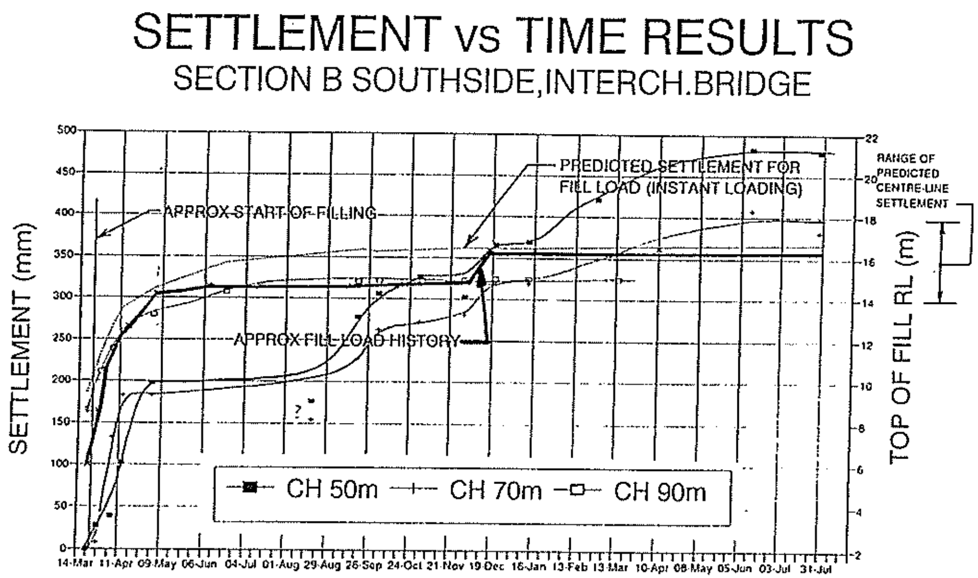


FIGURE 4

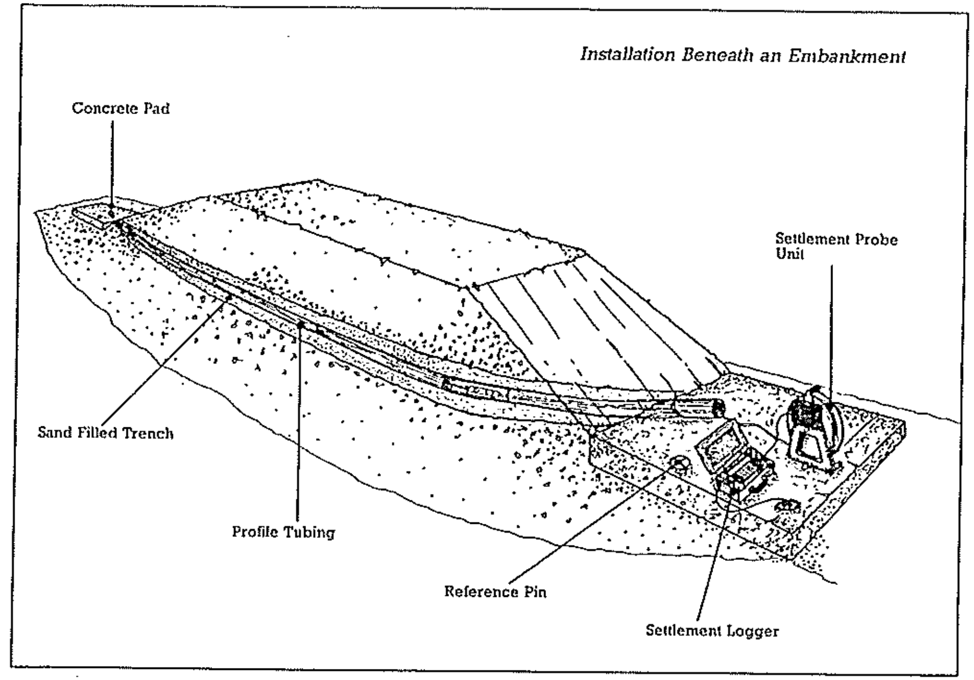


FIGURE 2: SCHEMATIC LAYOUT OF PROFILOMETER

Buttressing of the main embankments was carried out to maintain short term stability of batters over a swampy foundation. For the 5 m high fill abutments over the Tarata Creek/estuary crossing, the soft tidal mud was excavated to approximately 4 m depth. Basalt boulders were used as backfill and were founded on stiff to very stiff silts. This approach minimised stability and post construction settlement problems as well as construction delays.

For cut slopes, drainage was important in maintaining adequate factors of safety, particularly in the long term. Finished cut slopes were typically formed at 2:1 (H:V). The central part of the main cut areas was drained in advance of general earthworks by excavating a deep open drain along the centre of the motorway corridor. This also served to improve trafficking of earth moving plant over saturated silts and fine sands, and assisted in primary sedimentation for site run-off during earthworks. Horizontal drains were installed within the deeper cuts to assist in drainage of groundwater to improve stability. Groundwater levels were monitored (using standpipes) in the cut areas and measurements confirmed that these levels were eventually low enough to meet design requirements. No signs of significant slope instability have been observed within the permanent slopes.

TABLE 1: SH20 PROJECT – Soil Strength Parameters – Triaxial Testing (CUP)

Unit*	BH/TP	Depth (m)	Description	γ (kN/m³)	c′ (kPa)	ø′ (°)	W/C (%)
AV	M68	1.6	clayey SILT, firm, light brown, some medium sand	13.2	5	38°	133
AV	M19	1.7	silty CLAY, very stiff, dark yellow brown minor roots	18.5	24	31°	38
AV	TP69	1.8	clayey SILT, dark orange brown	14.4	24	34°	111
UTG	M11	6.0	sandy SILT, firm, grey with black staining	18.0	14	22°	49
UTG	M46	3.5	silty CLAY, firm, light grey	18.4	10	24°	40
UTG	M79	3.4	silty CLAY, firm,-stiff, light grey with orange stain	17.9	6	32°	40
UTG	M1	3.7	sandy SILT, stiff, highly plastic, dark grey	19.9	30	25°	28
UTG	M17	5.0	clayey SILT, soft firm, light grey brown	16.6	13	19°	58
UTG	M64	4.7	silty CLAY, soft, light brown, sensitive	14.5	20	24°	83
UTG	M127	2.3	sandy SILT, stiff, light grey and yellow brown	18.0	7	31°	38
TG	M14	8.8	sandy SILT, firm-stiff, light brown grey, some organics	17.8	12	33°	36
TG	M95	6.1	clayey SILT, very stiff, grey some organics	15.2	9	31°	107
TG	M95	8.7	slightly sandy SILT, very stiff, light grey	16.4	(C_u = 110 kPa)		45
TG	M16	6.1	clayey SILT, soft-firm, blue grey, organic pockets	17.1	25	21°	35

* UNIT KEY: AV =Auckland Volcanics UTG = Tauranga Group (upper) TG = Tauranga Group (undifferentiated)

BRIDGE PILE FOUNDATIONS

The bridge designers, Holmes Consulting Group Ltd, developed a U-beam approach with greatly improved weight to strength ratios for spans of 12 m to 30 m for bridges ranging in length from 37 m to 130 m. Pile working loads were up to 2MN.

<u>Pile Driving</u>

Piled foundations comprising 500 mm octagonal pre-cast concrete piles were driven to depths below ground of between 25 m and 35 m to the underlying dense to very dense Kaawa Sands. Driving was initially carried out using an eight tonne hammer. For the first bridge at Bader Drive/SH20, some difficulty was encountered at about 10 m to 13 m depth driving through medium dense to dense sands within the Tauranga Group. Insufficient capacity, however, could be developed in these upper sands because of the variable thickness and nature of this sand layer. Proof drilling would have been required to confirm there was an adequate thickness of sand beneath the base of the piles. The bridging sub-contractor chose to drive through the layer.

At this stage, spalling of the recently cast piles occurred during the hard driving. This was despite the specified concrete strength being achieved. A discussion ensued amongst the various design and review engineers as well as contractors on the minimum age of piles and associated concrete stiffness before driving could proceed. Opinions varied from 3 to 28 days, with a minimum age of 10 days eventually being adopted. This significantly reduced the spalling of piles.

Greater driving efficiency was later achieved by using a 12 tonne hammer. The upper 10 m of soil was pre-augered to 500 mm diameter at each pile location (the effective diameter of the octagonal piles was 520 mm). This allowed pitching and driving of the pile without leaders and so sped up construction.

Piles were driven to full depth and were found to pull up quickly on encountering the Kaawa sands. Actual pile tip levels generally agreed very well with the predicted founding levels of RL – 16 m to RL – 23 m for the various bridges within the site. Ultimate pile capacities of more than 6,000 kN were measured during construction using the Hiley formula. Required sets ranged from 3 mm to 10 mm for a 1.4 m drop of the heavier hammer.

Bitumen Coated Piles / Downdrag

Where piles were driven adjacent to recently constructed earth embankments, the pre-cast piles were coated with a 'Shell SL Bitumen' compound, to minimise the downdrag due to settlement of the underlying compressible soils. This compound was designed to "creep" under slowly applied downdrag loads but withstand relatively high frictional (short term) driving loads. As added insurance against peeling, an epoxy collar (about 10 mm thick) was applied at the base of the bitumen-coated length of the pile prior to driving. No significant peeling of the bitumen compound from the pile was observed during driving.

The bitumen was sprayed on by a specialist applicator and the pile was generally driven within a day or two of spraying to minimise adverse flow/creep effects of the bitumen. The thickness of the coating was checked prior to driving and was typically measured at around 3 to 5 mm.

Lateral Seismic Loading of Piles

All but one of the vehicular bridges were designed as "locked in structures with stiff filling" in accordance with the Transit NZ Bridge Manual (1991). The abutments were designed to resist lateral (longitudinal and transverse) seismic loads with a minor component being provided by the lateral resistance of the piles. Half the lateral pile capacity was relied on for ultimate conditions. The design horizontal subgrade coefficient (k_h) for the soil/pile system was assumed at typically 14 to 18 MPa/m with reduction of this coefficient by generally 50% to allow for near surface soil disturbance.

Lateral Load Test on Piles

An on-site lateral load test was carried out between two driven piles to confirm that the design lateral stiffness of the soil-pile system could still be achieved despite pre-augering of the pile locations. The results of the (70kN load) test indicated the back analysed horizontal subgrade coefficients were an order of magnitude greater than the design assumptions based on Terzaghi (1955) and Horide (1990).

RETAINING WALLS

The major retention works for the project comprise the Massey Road tied back pre-cast walls which support cuts of up to 8 m in height. The walls were designed for a 0.1 g earthquake and limited deflection soil conditions -half (k_o + k_a). Analyses were carried out using WALLAP(1993) and GALENA(1993). The wall system comprised a pre-cast T-column with pre-cast panels, tied back at the top to a buried deadman. The wall foundation system comprised a 900 mm ϕ cast-in-situ pile. The foundation pile was sized to allow embedment of the T-column.

Pile / Column Connection

However, during construction, at the request of subcontractor Albert Smith Industries Ltd, this embedded connection was changed to an epoxied drossback connection. This drossback allowed for better positioning of the T-column after the foundation pile had set. This enabled more efficient construction and crucial accuracy in setting out columns and panels for the specified architectural finish.

Corrosion Protection of Anchors

The wall was required by Transit NZ to have a 100-year design life. This necessitated triple corrosion protection for the permanent tie-backs. This protection comprised an epoxy coating on the tie-back rod which was then inserted into a corrugated plastic sleeve and grouted. This tie-back was then inserted into a 100 mm ϕ duct before nominal tensioning and QA testing.

PAVEMENTS

Pavement Foundation Materials

The pavements for the SH20 Extension were developed over typically wet cut areas or low level at-grade alignments. Subgrade CBR's were generally in the 2% to 7% range within the non-filled areas. Laboratory testing of stabilised materials indicated an increase in subgrade stiffness of 3 to 5 times could be achieved by stabilising with cement or lime. This improvement was generally achieved with relative ease during construction.

Subgrade improvement measures typically comprised undercut or cement stabilisation. Lime stabilisation was occasionally used in some fill areas to condition fills and control fill moisture contents where drying of soils proved very slow and impacted on construction programme.

Mechanistic Design & Construction Control

Pavement design was carried out using new mechanistic design methods with controls on subgrade compressive strain, and tensile stress for stabilised layers. A simple set of pavement designs was developed for the three specified traffic loadings of 5, 9 and 15 million EDA. The Contractor then tested the subgrade in-situ CBR and adopted the conforming design which suited his construction operations.

Basecourse and sub-basecourse specifications were based on TNZ M/4 with very minor modifications. Construction of the pavement layers was specified generally in accordance with TNZ B/2. Basecourse and subbase were required to be compacted to a dry density at least 85% of the material *solid* density. On occasions when very poor subgrade conditions were encountered (CBR <2), or when deflections during construction of the intermediate pavement layers were higher than expected, cement stabilisation of the lower sub-base was carried out. The mechanistic design method allows for such construction-driven changes to be easily accommodated within the design to achieve the desired finished levels and highway geometry.

CONCLUSIONS

- The SH20 project was the first major design-build contract let by Transit NZ. The project was completed on time and accommodated some significant changes during the course of the Works.
- Earthwork specifications based on the minimum shear strength and maximum air voids provide better control than the "95% of maximum dry density" approach where fill materials are variable.
- Settlement magnitudes and times for consolidation were predicted with reasonable accuracy using 2-D elastic analyses. The horizontal profilometer can be used to measure settlement profiles with reasonable accuracy and allows for earthwork construction to proceed without the hindrance of other normally adopted monitoring methods.
- Cut and fill batters formed at 2:1 (H:V) meet design stability requirements providing adequate drainage is maintained. Monitoring of groundwater levels has confirmed design expectations.
- The bridge piled foundations with working loads of up to 2 MN were taken down 25 m to 35 m to the Kaawa Sands. Bitumen coated piles were needed to minimise downdrag caused by settlement of abutment fills. Lateral load testing confirmed that back-analysed subgrade reaction coefficients were significantly higher than those obtained from literature correlations.
- Precast concrete walls with tiebacks as well as cantilevered walls were used to contain the motorway in the planned urban corridor. A drossback connection between the in-situ foundation piles and the precast columns allowed for quicker construction and a higher accuracy for the specified architectural finish. Triple corrosion protection was required for the permanent anchors.
- Mechanistic pavement design was successfully used for the SH20 project. This approach allows for construction driven changes to pavement designs to be easily accommodated.

ACKNOWLEDGEMENT

The author wishes to acknowledge Fulton Hogan Ltd and Transit NZ for granting approval to make this presentation.

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BUILDING A ROAD OVER A SEWER THE STATE HIGHWAY 20 EXPERIENCE

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SUMMARY

Stringent settlement criteria were set by WaterCare Services for the design of SH 20's two highway crossings over their 3m diameter Eastern Interceptor Sewer in Mangere, Auckland. The two road crossings of the sewer, which were a part of Fulton Hogan Ltd's SH20 design build contract, required fill embankments of 2.5 and 5 m height. The sewer was located at ground surface, underlain by 15m of firm clayey silt and medium dense sand layers. The three crossing options considered were light weight fill (pumice and expanded polystyrene) fill, a buried pile-concrete beam bridge and a conventional bridge. Finite element analyses were carried out using settlement data from laboratory testing and existing monitored fills. The conventional bridge was the lowest cost option. Monitoring during and after crossing construction showed settlements were within the design criteria.

1. INTRODUCTION

1.1 Options

Two crossings of an existing 3m diameter sewer line (called the Eastern Interceptor Sewer, EIS) were required for the construction of SH 20, in Mangere, Auckland. The two crossings comprised the 'North Grade Line' and the combined 'South Grade Line' and 'State highway 20A', (Figure 1).

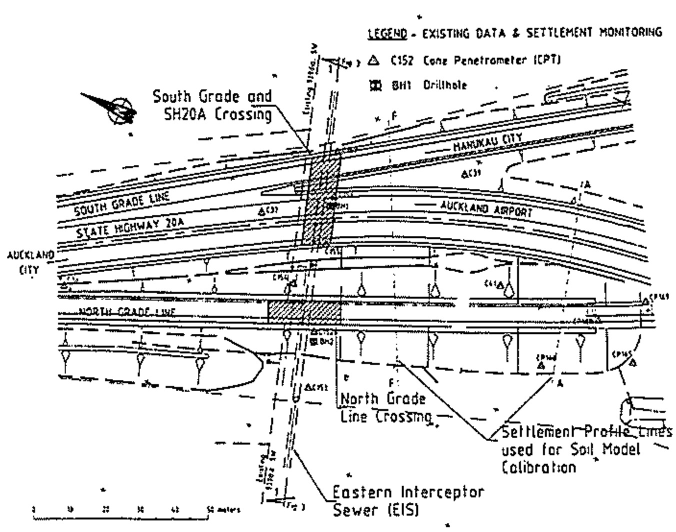
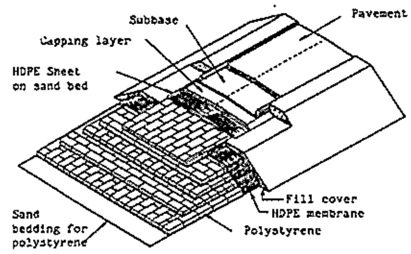


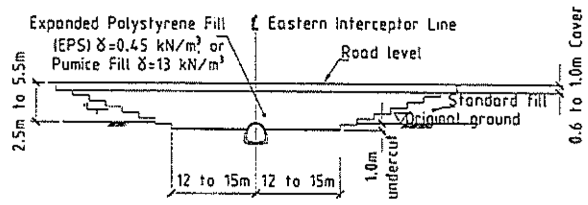
Figure 1 : Location Plan

Work carried out by WaterCare on the crossings indicated that the large sewer was sensitive to settlements induced by large embankment loads. Three options for reducing settlements were selected for detailed assessment. These were:

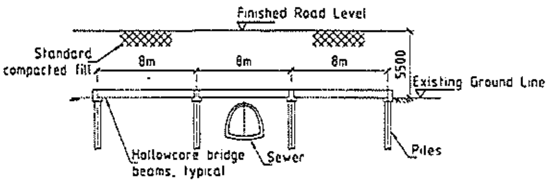
a) light weight fill (combinations of pumice and/or expanded polystyrene (EPS))
b) underground bridge
c) conventional bridge

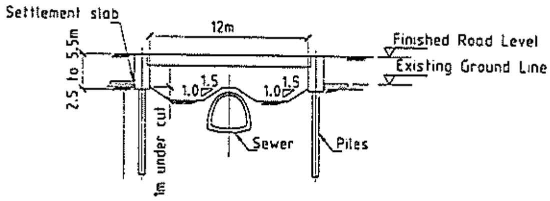
Figure 2 shows schematic views of each option.





Expanded Polystyrene Fill Isometric **Lightweight Fill Options**





Underground Bridge Option **Conventional Bridge Option**

Figure 2 : Crossing Options

1.2 Eastern Interceptor Sewer (EIS) Design Criteria

The "failure" criteria set by WaterCare were:

a) vertical loading above the crest of the sewer not greater than 54 kPa (ie. equivalent to some 3m of conventional fill).
b) longitudinal vertical deflection minimum radii of curvature in the negative direction of 4000m and 7800m in the positive direction. This allowed a maximum differential angular distortion of some 16mm in 10m.
c) maximum total settlement for operational (hydraulic) reasons of 100mm

Given the inherent difficulty in predicting settlement and the severe consequences and liabilities if the sewer was damaged, conservative target design settlement criteria of half the WaterCare "failure" criteria were used for the different crossing options, ie. 50mm total settlement and 8mm in 10m differential settlement.

Figure 3 shows a cross section of the proposed embankments and a simplified soil stratigraphy.

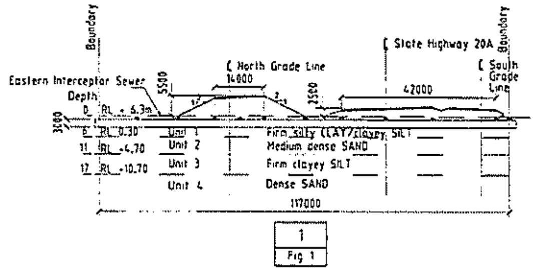


Figure 3 : Cross Section at Interceptor Sewer

2. GEOTECHNICAL MODEL

Extensive investigations were carried both along the line of the sewer and in the general vicinity. Figure 4 shows the avearge cone bearing profile for the six static Cone Penetration Tests and logs for the two drillholes along the line of the sewer (see Figure 1 for investigation locations). The results showed reasonably uniform, well defined soil layers of;

a) 5 to 6 meters of firm clayey silt/silty clay, over
b) 4 to 6 metres of medium dense sand, over
c) 5 to 7 metres of firm clayey silt, over
d) dense to very dense sand to unknown depth.

Five one dimensional consolidation tests were also carried out on undisturbed samples from the drillholes at depths shown on Figure 4.

This data formed the basis of the geotechnical model used for predicting settlements beneath the sewer.

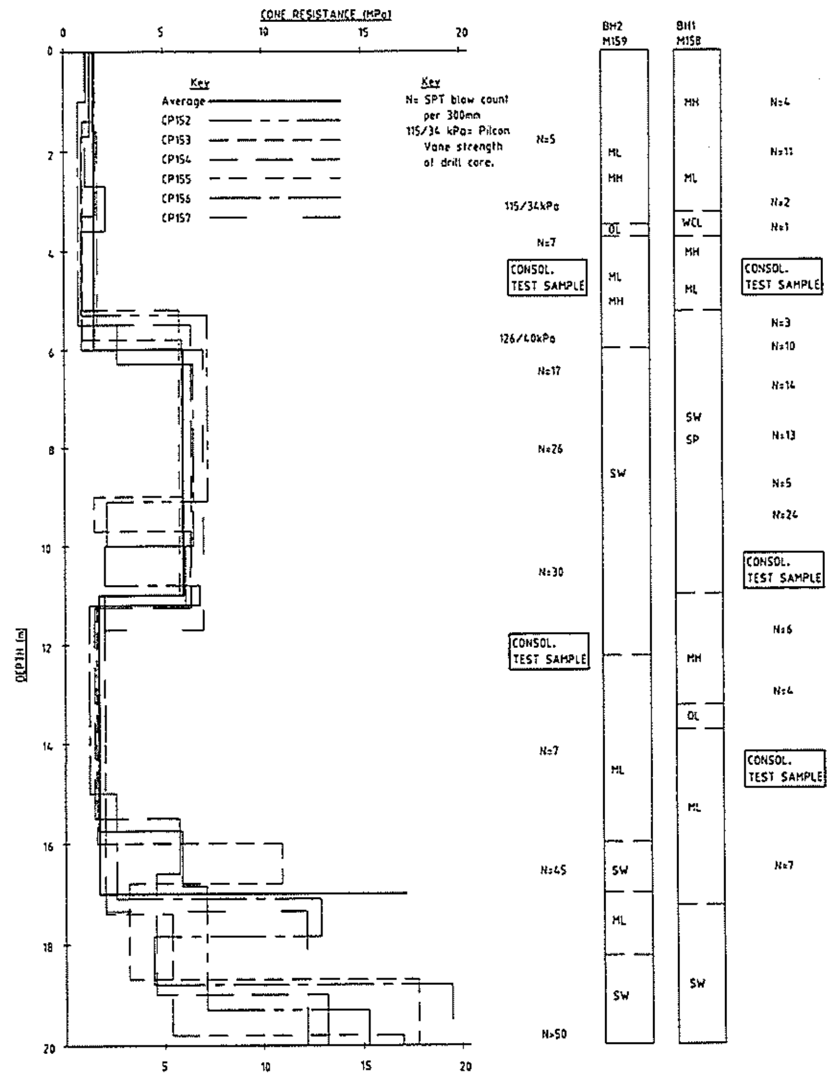


Figure 4 : Geotechnical Model

3. ANALYSIS METHODOLOGY

Stresses below each embankment were analysed using both a finite element program (SAP90) and the Boussinesq theory for vertical stresses. Consolidation settlements were calculated using the laboratory oedometer test data and one dimensional consolidation theory.

Measured settlements for the existing approach fills were used to "calibrate" the soil model parameters (shown on Figure 1 as lines AA and FF). Equivalent drained elastic moduli were also back analysed from the observed settlements using the finite element model and used to check settlement values.

Total and differential settlements were estimated for each of the proposed fill and bridge options.
Differential settlements were found to be the critical criteria and greatest in the vicinity of the embankment fill toe. The differential settlements were calculated from the crest to the toe of the embankment, and from the toe to a distance of six metres beyond the toe using the stress distribution given by the SAP90 programme. Total settlements were calculated using the stress distribution under the embankment centreline.

Total and differential settlements along the sewer line were three dimensional. The settlement at any point along the sewer line was influenced by the light weight fill embankment cross section geometry along the sewer as well as the influence of the 'standard' fill embankment located at some distance away from the sewer line (ie. beyond the limits of the lightweight fill). Two dimensional settlement analyses were carried out both along the sewer line and along the road centreline. The resulting settlements were added to give the predicted total and differential settlements.

4. GEOTECHNICAL MODEL CALIBRATION

Existing fills in the SH20 interchange area had already been placed and settlements measured by profilometers under the fills (Settlement profile line AA and FF on Figure 1). These measured settlements were used for calibration of the geotechnical model. The subsoils were similar to those at the EIS.

The laboratory oedometer tests carried out on the representative soil layers gave the following consolidation recompression indices (Cr, the slope of the e-log p curve up to the preconsolidation pressure) of:

- Unit 1, firm clayey silt/silty clay, Cr values of 0.10 and 0.17 (two samples)
- Unit 2, medium dense sand, Cr value of 0.04
- Unit 3, firm clayey silt, Cr values of 0.19 and 0.21 (two samples)

Using the higher consolidation recompression indices of 0.17 and 0.21 gave suprisingly good agreement between measured and calculated settlements for the two embankment geometries selected for model calibration, as summarised in Table 1:

Table 1: Geotechnical Model Calibration Settlements

Embankment Geometries	Measured Settlements (mm)	Calculated Settlements from Cr values (mm)
4m height, 30m crest, 2H:1V side slopes	260	258
1.7m height, 5m crest, 13H:1V side slopes	80	81

For the 4m high embankment, a maximum settlement of 257mm was computed using equivalent drained moduli of 2MPa for Unit 1, 16 MPa for Unit 2, and 3 MPa for Unit 3.

5. ASSESSMENT

5.1 South Grade/SH20A

Assessed total settlements, differential settlements, and estimated capital costs for each crossing option are summarised for the South Grade/SH20A line in Table 2 below.

Table 2: South Grade/SH20A Crossing Options Summary

South Grade/SH20A Crossing Option	Total Settlement (mm)	Differential Settlement (mm in 10m)	Estimated Cost (excl. GST)
Conventional Fill (density 17 kN/m^3) (For comparison purposes only)	127		
Light Weight Fill			
Pumice, 2.5m undercut (density 13 kN/m^3)	93	35	$390,000
Pumice/EPS combination, 1m undercut	76	27	**
EPS, 1m undercut (density 0.45 kN/m^3)	25	12	$765,000 *($660,000)
Underground Bridge (NA)			
Conventional Bridge			
12m clear span, 1m undercut	25	10	$776,000

* estimated cost with partial use of lower grade (HD) polystyrene instead of the extra high density (EHD), a reduction in volume of EPS due to refinement of undercut depth from 1.0m to 0.7m, and reduction of reinforced concrete cap thickness from 150mm to 100mm.

**EPS/pumice combination used EPS along the outer thirds of the embankment to minimise the critical differential settlements and pumice in the centre for the not so critical total settlements. This sub-option was not costed because differential settlements were greater than design criteria.

5.2 North Grade Line

Assessed total settlements, differential settlements, and estimated capital costs for each crossing option are summarised for the North Grade Line in Table 3 below.

Table 3: North Grade Line Crossing Options Summary

North Grade Line Crossing Option	Total Settlement (mm)	Differential Settlement (mm in 10m)	Estimated Cost (excl. GST)
Light Weight Fill			
EPS, 1m undercut (density 0.45 kN/m^3)	16 to 25	12 to 14	$960,000 *($870,000)
Underground Bridge	30	8	$1,020,000
Conventional Bridge			
27m clear span	25	10	$535,000

* estimated cost with partial use of lower grade (HD) polystyrene instead of the extra high density (EHD), a reduction in volume of EPS due to refinement of undercut depth from 1.0m to 0.7m, and reduction of reinforced concrete cap thickness from 150mm to 100mm.

6. RECOMMENDATIONS

While the EPS light weight fill and conventional bridge options were of similar cost for the South Grade/SH20A Line (with possible savings of up to $104,000 in the EPS option with further refinement) the conventional bridge option was chosen. This selection was based on the lower predicted differential settlements and the lower risk of a conventional bridge construction compared to the other options.

The conventional bridge option was also chosen for the North Grade Line due to the cheaper cost.

7. CONSTRUCTION MONITORING

Eleven settlement monitoring rods were fixed along the top of the concrete sewer and readings taken during pile driving and abutment fill construction. Settlements were monitored for 5 months after construction.

The most interesting data that came out of the monitoring was that critical differential "settlements" occured as "heave" during the State Highway 20A/South Grade Line pile driving stage!

Pile driving comprised pre-drilling a hole to 10m depth, placement of the 25 m long pile in the hole and then driving the remaining 15m. The State Highway 20A/South Grade Line piles were at 4.5m centres, 6.5m away from the centre line of the sewer. Maximum heaves of 20mm were measured in the centre third portion of the State Highway 20A/South Grade Line crossing during pile driving. Measured maximum differential heaves were 8 to 12 mm in 10m.

The sewer then settled 14mm over a period of 2 months during the completion of the abutment approaches resulting in a net total "heave" of 4 to 8 mm.

No significant heave was measured at the North Grade Line crossing during pile driving. At this crossing the piles were at 4m centres, 14m away from the sewer centreline. Maximum settlements of 10mm were measured at the North Grade Line crossing due to construction of the bridge approaches. Maximum measured differential settlements were 2mm in 10m.

ACKNOWLEDGEMENTS

Permission from Fulton Hogan Ltd and Transit NZ to publish this paper is gratefully acknowledged.

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SOIL NAILING FOR STATE HIGHWAY 2 WIDENING AT SILVERSTREAM, HUTT VALLEY

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SUMMARY

A short section of State Highway 2 near Silverstream in the Hutt Valley north of Wellington was widened to 4 lanes in 1997. The highway is located between steep hillside slopes and a railway line. The widening had to be carried out without affecting the railway line and also had to minimise the impact on a scenic reserve located on the hillside above the highway. This was achieved by using soil nailing to stabilise cut slopes into the hillside, with a mechanically stabilised earth retaining wall between the highway and the railway line.

Soil nailed stabilisation of cut slopes of up to 8 m in height was required in alluvial debris fan material located at the toe of steep greywacke sandstone hillside slopes. A total length of 200 m of cut slope was soil nailed, covering 800 m^2. The works are located within 300 m of the Wellington Fault zone, and the design allows for limited lateral displacement of the toe of the stabilised slope under the design earthquake (peak ground acceleration of 0.4g).

The 50° face of the cut slope was covered in wire rock fall mesh attached to the soil nails to provide support for the materials on the face of the slope between nails. A small amount of shotcrete was placed at the head of each nail to provide additional load spread and at the same time allowing revegetation of the slope.

Ponga logs were attached to the rock fall mesh to provide a natural finish that blends in with the dense bush of the scenic reserve above. Topsoil and forest floor detritus was gathered from the site and was retained in hessian bags beneath the logs to maximise regeneration of vegetation.

Construction was successfully carried out in 1997 with the soil nailing as a design build package. The soil nailed slope has performed satisfactorily and has a very natural appearance. Pull-out tests were carried out as part of the construction to confirm design assumptions and additional bars have been installed to enable future pull out tests to be carried out to verify the long term capacity of the nails.

1. INTRODUCTION

It was proposed to widen a 2 lane section of State Highway 2 between Silverstream Bridge and Manor Park (approximately 25 km north of Wellington) to four lanes.

The proposed widening was to be accommodated by cutting back into the existing bush covered hillside (part of the Keith George Memorial Park) on the western edge of the existing SH2 and by supporting the new road with a retaining wall along the Wellington - Masterton Railway boundary.

Preliminary alignment proposals considered cut profiles of up to 70° in the existing slopes above the highway to minimise the intrusion into Keith George Memorial Park. This was later revised to 50° to satisfy a requirement of the planning consent to have a wall facing which minimised visual impact. This enabled the face of the soil nailed wall to be finished with ponga logs which provide a growth media, promote regrowth and reduce the visual impact of the cut face.

Site investigations had been carried out previously at the site in 1986 as part of an earlier scheme, and these were supplemented by additional investigations by Opus International Consultants Ltd in 1991 and 1995.

2. GEOLOGY

The site is situated on the south-east facing slope of an eroded back scarp, located on the up thrown side of the Wellington Fault which is a Class I active fault. The Wellington Fault zone is considered to be up to 300 metres wide and the site is within the fault zone.

The slopes at the northern end of the site consist of highly weathered greywacke sandstone overlain by a layer of colluvium. The colluvium is between 6 and 10 metres thick at the over steepened toe of the slope, and comprises loose angular gravels with minor rare silt, inter-layered with moderately dense silty sandy angular gravels. Some small seepages were observed. Natural hillside slopes of 35° to 40° were observed in these materials. Some existing cut faces in the toe of the colluvium were standing up to 5 metres high at angles between 60° to 70° and have remained stable for 25 years.

At the southern end of the site, fan deposits comprising moderately dense silty sandy gravels were encountered in a borehole drilled in 1995 to a depth of 13.5 metres. The presence of unsupported cut faces at the toe of the fan at angles of between 60° to 70° suggests that this material exhibits some cohesion.

3. STABILITY ISSUES

The existing relatively steep road cuttings at the toe of the colluvium and fan deposits are unsupported. Support for the cut face was inferred to be provided by the natural vegetation and other positive effects, such as the apparent cohesion due to negative pore water pressures in the finer matrix materials and the interlocking effects of angular fragments. These slopes were inferred to be marginally stable in static loading with a clear potential for slope instability in large seismic events.

4. SEISMIC DESIGN

It was realised early in the project that while the final proposed cut slopes would require some form of retention system to aid stability. It would not be economic to design a system which was capable of providing sufficient seismic resistance without some deformation occurring, in either the design earthquake (0.4g) or a larger regional event (0.8g). Therefore, it was decided to adopt a retention system that was tolerant of deformation and to accept that a certain level of deformation may occur in large earthquakes. Normally acceptable factors of safety for overturning and bearing capacity would be achieved by the retention system, but a factor of safety of 1.0 or lower would have to be accepted on sliding and gross stability of the slope above under earthquake loading.

5. SOIL NAIL CONCEPT

Soil nailing provides a method of reinforcing in situ soils by steel rods or "nails" drilled and grouted in place, usually as excavation proceeds from the top down. The soil nails act to hold the soil mass together and form a reinforced block which acts as a gravity wall. Reinforcement forces the critical slope failure surfaces deeper into the slope. A concrete pad is often provided at the head on the nails to distribute loads from the adjacent material, though these loads are generally small. Shotcrete, reinforced with steel mesh, is usually provided to provide face stability and additional corrosion protection for the nail heads. The top down construction method involves sequential excavation of benches, insertion of the nails and shotcreting of the face in lifts of about 2.0 metres (depending on the short-term stability of the soil).

Soil nailing is a relatively new technology in New Zealand, although this has become an acceptable method in many overseas projects.

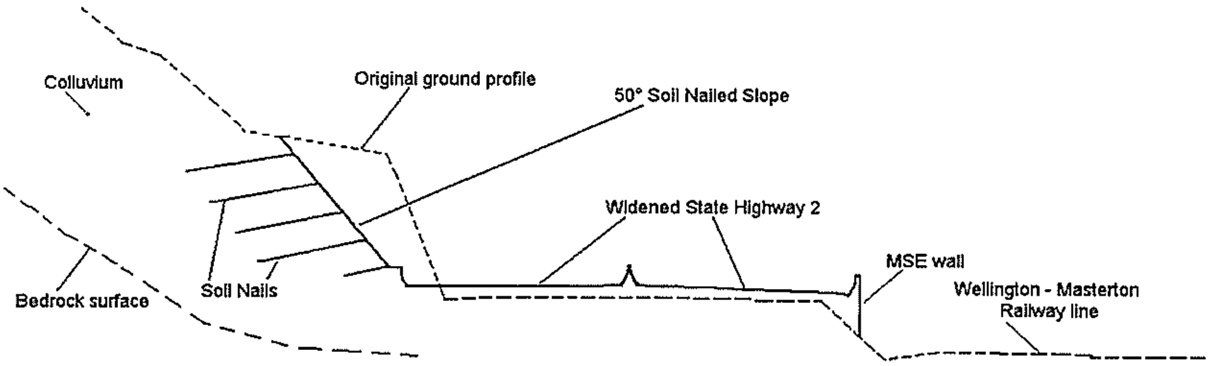

Figure 1 - Typical Cross Section

6. ANALYSIS

6.1 Overall Stability

Conventional slope stability analysis was used to prepare preliminary design details and to check the external stability of the soil nailed block. Models for stability analysis were developed based on the ground and groundwater data from the site investigations. Dry groundwater conditions were assumed for the analyses based on investigation results which indicated that groundwater levels were generally below the level of the toe of the proposed 50° cut face. However, perched water was observed in part of the slope and it was assumed that sub-horizontal drain holes would be installed to control groundwater levels.

Analysis of the models was carried out using the PC-based SLOPE/W package. A Morgenstern and Price algorithm was utilised with a pseudo-static inertia load included to model seismic loading. A design earthquake acceleration of 0.4g for a Class 1 structure, in accordance with NZS 4203:1992, was assumed for the seismic case.

Material properties for the stability analyses were estimated from back analysis of the over steepened toe of the existing cut slopes assuming a factor of safety of unity. The computer models for back analysis were developed from the proposed alignment design cross-sections and existing geological information. The back analysis suggested the following material strength parameters, which were used in the later stability analyses.

$c' = 6\ \mathrm{kNm^{-2}}$
$\phi' = 35°$
$\gamma_b = 20\ \mathrm{kNm^{-3}}$

These parameters appeared consistent with the soil descriptions and in situ SPT results.

After initial analysis to determine the material parameters by back analysis, the stability of the whole slope was analysed to assess the factor of safety for the existing slope. The factor of safety obtained was then compared with the value estimated for the slope soil nailed slope. The results obtained from the stability analyses are summarised in Table 1 below.

The results indicate that under static conditions, the overall stability of the northern slope improves slightly following the proposed cutting back of the slope to 50°. While less than the ideal figure of 1.5, the obtained factor of safety (1.24) was considered acceptable given the nature of the slope and the method of analysis used. There is a significant decrease in the factor of safety for the southern slope because construction removed a natural bench that was buttressing the slope. However, the resulting factor of safety of 1.53 suggests it will be sufficiently stable. In both sectors, the factor of safety under seismic loading for the existing slopes is less than 1 and deformation of the soil nailed slope and hillsides above would occur under seismic conditions.

6.2 Slope Deformations

The possible slope deformations resulting from a large earthquake were assessed for both the existing slopes and the final (proposed) geometry. Two possible earthquake scenarios were considered:

- The "design" earthquake with a ground acceleration of 0.4g in accordance with NZS 4203:1992.
- A large magnitude, low probability event such as may be associated with a rupture of the nearby Wellington Fault. An earthquake of magnitude M = 7.5 on the Wellington Fault would have an average recurrence interval of 600 years is predicted to give a peak ground acceleration at the site of 0.6g. This event would have a 10% probability of occurrence within the next 50 years.

The critical acceleration, K_c, was determined for failure at the two sections and deformation in the design event was predicted (see Table 1) using Ambraseys and Menu (1988). Lateral deformation of less than 250 mm is expected to occur in the design earthquake.

Deformations of up to 1 m are predicted in a large regional earthquake on the Wellington Fault. There is a 1.5 m wide cycle lane located beside the highway, and hence a lateral deflection of the slope of about 1 m was considered acceptable, as this would not block the highway. Other more severe roading damage is likely to occur if the fault ruptures in this area.

	Static Factor of Safety		Seismic Factor of Safety (Design earthquake 0.4g)		
	Existing Unsupported Cuts	Proposed 50° Soil Nailed Slope	Proposed 50° Soil Nailed Slope	K_c	Deformation
Northern Slope	1.21	1.24	0.73	0.14g	< 250 mm
Southern Slope	2.07	1.53	0.78	0.26g	< 50 mm

Table 1 - Summary of Stability Analysis Results

6.3 Design of Reinforcement

Design of the reinforcement for the soil nailed slope was carried out on a design/build basis for the construction contract and was checked by Opus using the ReSLOPE software package. The design employed bar lengths of between 3 m to 9 m at 10° inclination and nominal 2 m centres horizontally and vertically.

7. CONSTRUCTION

7.1 Soil Nailing

Construction was completed in August 1997. The slope was constructed using galvanised rockfall mesh over the face between the soil nails with a minimum area of shotcrete around each nail head to provide the required load spread and corrosion protection for the nail heads. Galvanised 16 mm diameter 500 MPa yield strength steel bars were grouted into 50 mm diameter holes. Drilling was carried out using an Atlas Copco 1036HB hydraulic drill mounted on a truck Hiab. The bars were rotated during grouting to minimise air entrainment in the grout.

Sub-horizontal drain holes (40 mm diameter, minimum length 3.5 m) were installed in a wet section of the slope.

Pullout tests were carried out by the designer on trial nails of 2.0 m anchor length and bar yield or coupler failure occurred at about 100 kN which was approximately 2.5 times the required design loading. Additional soil nails were installed to allow pullout tests to be carried out in the future to confirm the long-term nail capacity.

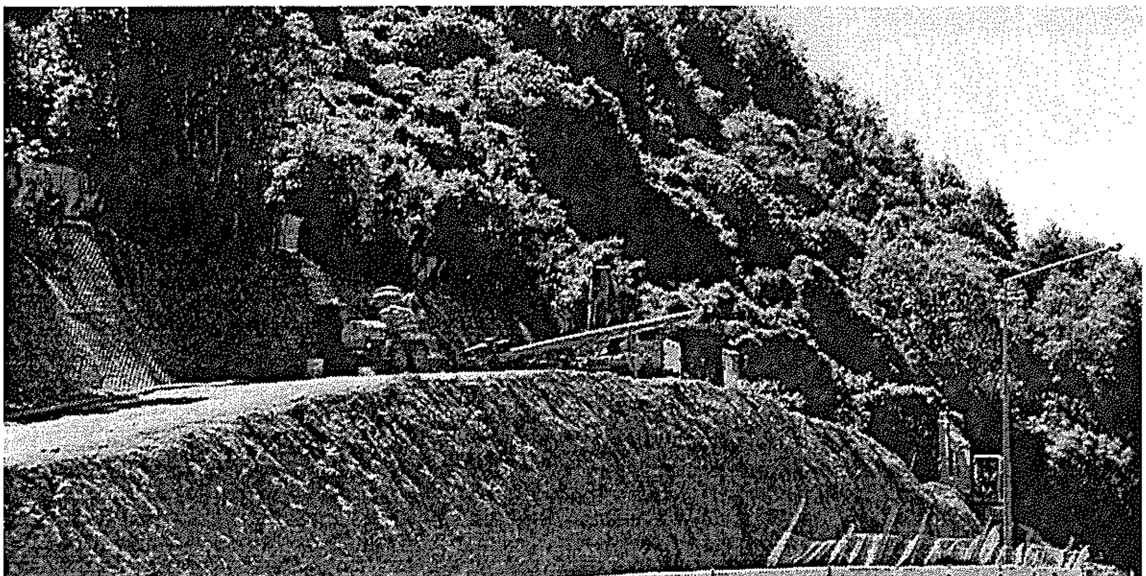

Figure 2 - Drilling rig installing soil nails and wire mesh on face.

7.2 Ponga Log Facing

The ponga logs are attached to the wall by lacing them to the mesh with plastic coated, galvanised wire. This will retain the ponga logs until vegetation is established. Topsoil and forest floor detritus were gathered from the site and retained in hessian bags between the soil nails and underneath the logs to maximise regrowth.

Figure 3 - Placing ponga logs on soil nailed face

8. CONCLUSIONS

The design concept has been developed into a practical solution for the proposed application. Construction of the soil nailed wall was carried out without any significant difficulties. The ponga facing provides an environmentally friendly finish to the slope and appears to be standing up well to service with significant regeneration of vegetation.

9. ACKNOWLEDGEMENTS

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GEOTECHNICAL DESIGN OF THE GOODWOOD REALIGNMENT, SH1

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SUMMARY

The Goodwood realignment project has required the construction of a large fill embankment over a soft foundation. Difficulties during the investigation limited the amount of information obtained for the foundation area and conservative assumptions had to be made during the design. Large settlements taking many years to occur were predicted and differential settlement at the piled bridge abutments had to be controlled. A cost effective solution was adopted of using wick drains to allow consolidation at the bridge abutments to be largely complete by the end of construction but accepting longer term settlement over the bulk of the embankment. The vertical alignment on the embankment was to be corrected by way of granular overlays.

Additional investigation at the start of construction and the use of monitoring equipment has changed the understanding of the foundation soils. Settlement of the embankment will be as high or even higher than predicted, but drainage of the soft soils by permeable layers not previously identified indicates that the consolidation will occur very much faster and may well be largely complete by the end of construction. The rapid drainage of the soil also means that an acceleration of the construction is likely.

1. INTRODUCTION

The Goodwood Hill on State Highway 1 is approximately 50km north of Dunedin. The existing alignment of tight horizontal curves combined with short vertical curves and narrow carriageway is being replaced with a new alignment of a single 1800m radius curve with a 350m long cut of up to 16m depth through the hill and a 600m long fill embankment up to 12m height over Watkins Creek. Waterway requirements at Watkins Creek required a bridge crossing with 6m fill height at the abutments. The embankment crosses an area of soft compressible soils. The large settlements predicted, the interface with the bridge and potential fill instability presented geotechnical design challenges.

2. GEOTECHNICAL INVESTIGATIONS

The site investigation was carried out in a number of stages. The initial site investigation was carried out in March 1996 over a total length of road of 4 km, with 5 rotary wash boreholes to between 10 and 22m depth and nine test pits to between 2 and 4.9m depth. A change in the proposed alignment necessitated the drilling of a further three boreholes to between 15 and 20m depth in June 1996 and a further four boreholes to between 8.3 and 16.2m depth at the end of August.

Part of this additional drilling was to further explore the foundations for the deep fill which had been identified as a potential problem. However very soft ground conditions restricted access to the Watkins Creek fill area, and only one borehole was able to be drilled, well off the centreline Two test pits to 4 .4 and 6m depths were subsequently excavated here in December 1996 to supplement the boreholes. Because of the limited data for this area, at the start of construction when the first fill layers allowed access on the centreline of the embankment, three static cone penetration tests and two boreholes were made to check the assumptions made for this area. These tests changed the understanding of the foundation behaviour, as discussed below.

3. GEOTECHNICAL PROFILE

The area is underlain with Tertiary age sedimentary soft rocks, principally an impure limestone and interbedded mudstone of the Goodwood Formation. The mudstone has been eroded into a now buried topography which appears to be only partially related to the current topography. The top of the mudstone lies just above the road level through the hill but dips steeply to be at 7 to 11m depth under the Watkins Creek Flat. These soft rocks were covered with alluvium gravels during the glacial period. The Goodwood ridge was formed by the subsequent erosion of the gravels and mudstone from each side and the later deposition of the wind blown loess which mantles the older soils.

This results in a typical sequence of soils through the Goodwood hill of topsoil over 2 to 7m thickness of loess, a thin layer of gravel, 0.8 to 4m of clay and clayey silt, 2 to 7m of gravels overlying the mudstone. The water table as ascertained from the boreholes and spring features appears to be at the top of the mudstone, near the base of the gravels.

The loess is firm with SPT results of 16 to 30. The material is strong when dry but is erodable. The gravels as exposed on the road batters are in the main part clay bound but as is typical of alluvial deposits exhibit considerable variation in grading through the layer. SPT values range from about 20 to refusal. The mudstone is hard with SPT tests meeting refusal in most instances. It is finely bedded horizontally, with subvertical joints and weathers quickly on exposure. The gravels precluded the use of steep batters, and the erodible loess is best cut flat enough to topsoil. As a result the cut section is to have 2:1 batter slopes.

The lower valley bottoms consist of Holocene flood plain, lake and estuarine deposits. Typically there are clayey silts with some sand and fine gravel layers overlying mudstone at between 7 and 11 m depth. Most of these Holocene soils are soft to very soft with SPT values of less than one. At the design stage the limited site information indicated a thick layer of silt with effective drainage only to the ground surface, and this layer became central to the design considerations. A total of 5 samples from the silts were tested to give the following parameters for the soil:

c_v (m^2/year)	2.2	Plasticity Index	24
m_v(m^2/MN)	0.51	cohesion (kPa)	14
Dry density (t/m^3)	1.25	∅ (degrees)	26
moisture content (%)	43		

The consolidation test results indicate a very compressible soil, with a coefficient of volume compressibility at the lower end of the normal silt range (the mv values given are all for a 200kPa load increment) and a coefficient of consolidation more typical of clays or fine grained organic soils than silts. Values of c and ∅ were obtained from consolidated undrained shear box testing

4. EMBANKMENT DESIGN

The poor foundation conditions for the 12.5m high fill presented problems of embankment settlement and stability. On the basis of the test results it appeared that the compressible silts were not drained by any permeable layers and large settlements of up to 0.7m and particularly long consolidation times of about 60 years for 90% consolidation were estimated. The settlement at the Watkins Bridge abutments was estimated at 0.4m. This presented additional problems of differential settlement at the abutments of the Watkins Creek bridge. The soft silts were also a weak layer under the fill and slope failures were predicted for steep fill batters built directly on the silt.

Numerous options for the settlement of the embankment were considered. These included:

- No foundation treatment. This would have created unacceptable level differences at the bridge as the embankment settled relative to the piled bridge abutment.
- Sand blanket - would reduce the consolidation time to about 10 years by providing drainage under the fill.
- Sand blanket with vertical drains - depending on the drain spacing this could reduce the 90% consolidation time to as little as 18 months.
- Stone columns - These would act as vertical drains to increase the rate of consolidation, significantly reduce the amount of settlement, and increase the overall shear strength.

Options for embankment stability included:

- Geogrid reinforcement to the base of the fill.
- Use of flatter batter slopes.
- Use of stone columns to enhance the foundation strength.
- Staged construction, with and without foundation drainage to speed pore pressure dissipation.

Methods considered for dealing with differential settlement at the bridge included:

- Using shallow pad footings to settle with the fill.
- Delay bridge construction to allow the embankments to settle as much as possible and incorporate settlement slabs to prevent abrupt steps developing.
- Install piles under the fill to carry the embankment or stone columns to minimise the settlement magnitude.
- Reduce the height of fill.
- Use lightweight fill.

Consideration of these methods, the expense of having to import suitable material for drainage layers and stone columns and the related cost implications led to the adoption of the following design approach:

- Place the bulk of the embankment fill directly on the sub-grade without a drainage blanket or other ground improvements, and accept a long term settlement. This will affect the vertical alignment of the road over a considerable time estimated as decades, and periodic shape corrections will probably be needed to maintain the alignment within acceptable limits.
- Batter slopes of 2.5 : 1 for a conservative batter but with a good cut to fill balance.
- Staged construction over three seasons to ensure embankment stability.
- At the bridge abutments, install vertical wick drains and a drainage blanket, to speed consolidation. The wick drains to be placed at increasing centres away from the abutments to form a settlement transition to the remainder of the fill.
- Delay bridge construction for as long as possible to allow consolidation of the abutments to occur prior to pile driving.
- The inclusion of several piezometers and settlement cells to monitor pore pressures in the foundation soils and consolidation magnitudes and patterns

The staged embankment filling is necessary to ensure stability during construction. The low coefficients of consolidation from the testing indicate slow dissipation of pore pressure. The embankment was modelled in Slope W programme with instantaneous fill placement to various heights and no excess pore pressure dissipation. This limited the construction heights to 6.5m in the first season, 9m in the second season and 12m in the third to maintain a minimum short term factor of safety of 1.2.

As the adequate dissipation of increased porewater pressure is critical for short term stability, an array of piezometers was specified to be installed in the silt layer and monitored to allow this dissipation to be observed and the actual performance to be compared with the model. Adjustments to the height of fill placed in any season or the rate of placing may be needed during construction.

5. CONSTRUCTION DEVELOPMENTS

Construction started on site in late February 1998. At the time of writing this paper the fill construction was at about 4 -5m fill depth, with the wick drains and sand blanket at the bridge abutment complete. No difficulties have been encountered to date. The construction contract required the installation of four piezometers and three settlement cells under the fill, and further testing as construction allowed access over the soft soils along the centreline. Three CPT tests were made on the embankment centreline and two boreholes were drilled at the bridge abutments to confim pile lengths.

The addtitional test data and the monitoring results to date indicate conditions are considerably better than assumed at the design stage. The CPT and boreholes show much greater variability in the soil profile, and the relatively uniform soft silt layer assumed is now known to be a compexely interbedded sequence of silts, clayey silts, sands and some gravel layers. Two of the boreholes drilled to install the piezometers penetrated significant thicknesses of sand.

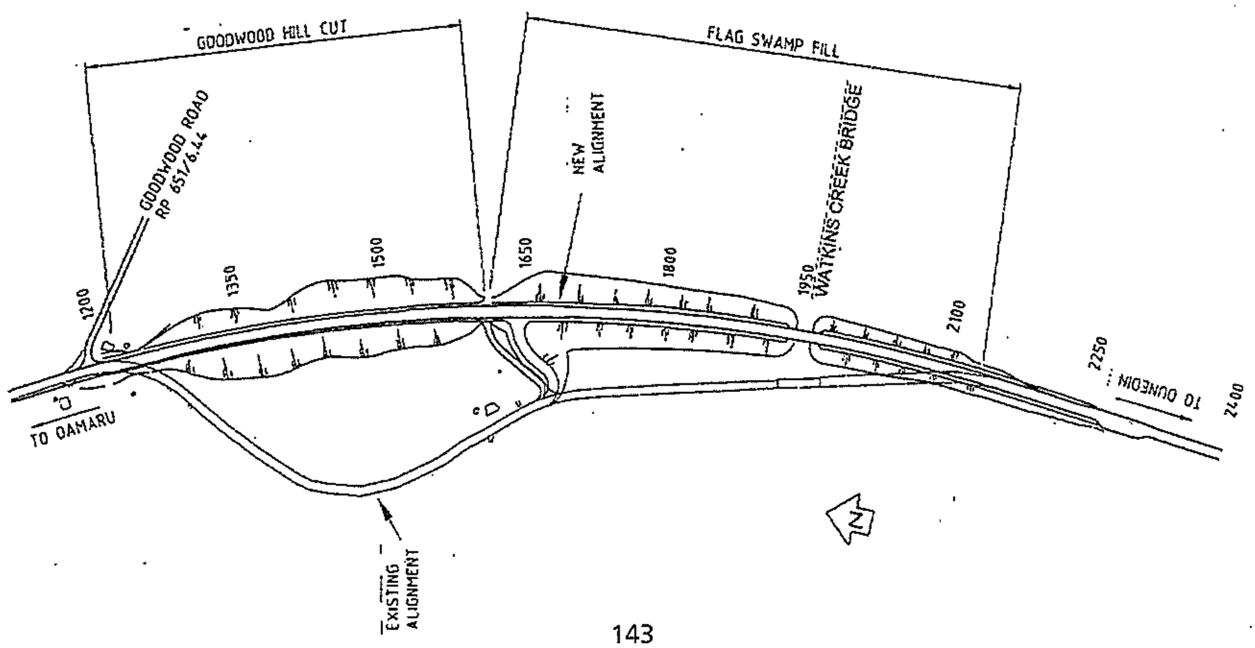
It is now clear that the original consolidation model and its predictions were conservative and that the the soil profile is well drained with permeable layers.

The depth to siltstone is also now known to be deeper and more variable than the assumed relatively uniform 7.5m. The siltstone appears to dip to over 11m depth 80m out from the toe of the hill, rises to about 8.5m depth 150m from the toe, is about 9m at the north bridge abutment and 7.5m at the south abutment. This suggests that the long term settlement over the highest length of the embankment could be 25% higher than estimated during design with a total of about 800mm.

The different soil profile now apparent is also reflected in the monitoring data to date. It was intended that the monitoring equipment would provide an early indication of the accuracy of the performance predictions with a possible shortening of the construction programme if pore pressure dissipation allowed. Unfortunately the equipment was not installed until well after construction had begun and initial readings were further delayed. At the time of writing the piezometer data is of only 3 weeks duration with the first readings made two months after construction began on site. However the piezometers indicate that pore water pressure rise is minimal, implying that drainage is much better than assumed from the laboratory test results and settlement will occur much more rapidly than predicted. The settlement cells have readings spanning 5 weeks. Over the first 4 weeks the fill was increased in height by 2m and the settlement cells registered 230 and 350mm of settlement. There has been very little settlement in the last week recorded, suggesting that consolidation is occurring much faster than anticipated in the design model. However the magnitude of the settlement is much greater than predicted, and a total settlement of about twice that expected could now be possible.

From the new site information and the monitoring results to date, it appears likely that significant settlement of the embankment after completion will not occur. This will reduce if not eliminate the need for shape correction overlays. It also appears likely that the construction period can be shortened by about one year.

Figure 1
Plan of Goodwood Realignment



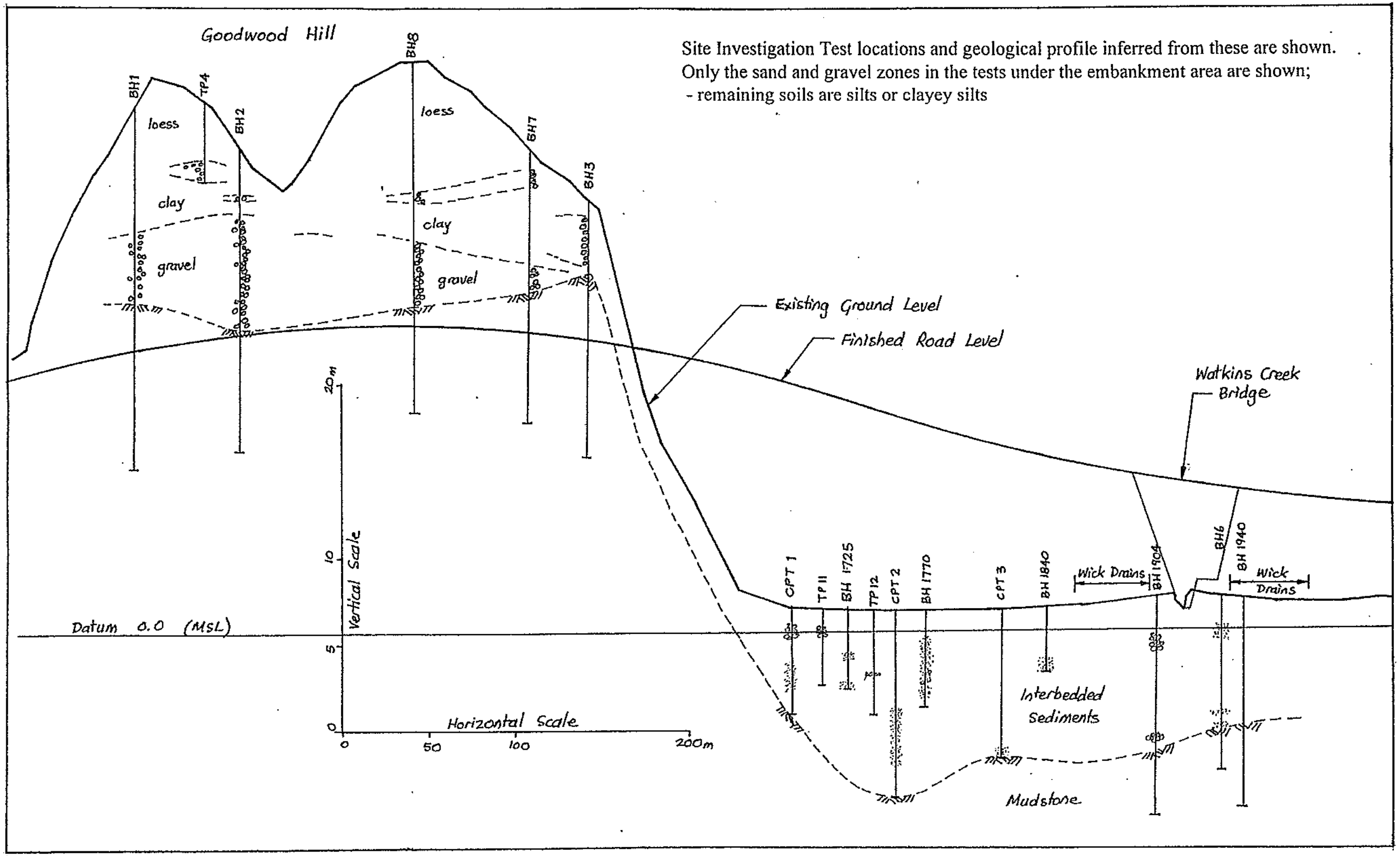


Figure 2
Longitudinal Section Through Goodwood Hill Cut and Embankment

ELASTIC MODULUS OF PAVEMENT MATERIALS

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Summary

Transit New Zealand have adopted the AUSTROADS pavement design procedure. In this procedure a trial pavement is analysed using a multi-layer elastic modeling program CIRCLY and the critical strains occurring under specified loading conditions are established. AUSTROADS predicates a relationship between the elastic strain that occurs in a particular layer and pavement life in terms of accumulated permanent deformation. The designer has to nominate an appropriate value for the resilient modulus for each layer of material as part of an analysis. The variety of methods that can be used to measure the modulus and the selection of the most appropriate value are discussed.

1 INTRODUCTION

In July of 1995 Transit New Zealand adopted the AUSTROADS pavement design procedures as described in the document, *A Guide to the Structural Design of Road Pavements* (AUSTROADS 1992), herein after referred to as the AUSTROADS procedure. The decision is considered to be a sound one as Transit New Zealand is a member of AUSTROADS and a consistency of approach between New Zealand and Australian practitioners makes sense. It means that the roading fraternities of both New Zealand and Australia can readily benefit from continued developments in pavement technology on both sides of the Tasman.

While the overall philosophy used in the AUSTROADS procedure is similar to that used in earlier design procedures there are differences in approach to some aspects. The basic design philosophy involves the acceptance that there is a relationship between the elastic strain that occurs in a particular layer of the pavement and the "life" of the materials in that layer. In the case of the unbound granular pavement with a thin surface seal the critical condition is related to the elastic compressive strain in the subgrade.

One of the most conspicuous differences between the AUSTROADS procedure and the previous NRB/TNZ design methods (commonly referred to as the *orange book*) is the necessity to analyse the pavement using a multi-layer elastic model computer program. This results in a need to input appropriate elastic properties for the materials to be modeled. This paper discusses some of the issues regarding the establishment and the use of the elastic modulus parameter in pavement design.

2 WHAT IS THE ELASTIC MODULUS, AND WHY USE IT?

The elastic modulus (E) of a material is defined as the slope of the stress versus strain plot, i.e.

$$E = \frac{\sigma}{\varepsilon}$$

where σ = applied stress; and,
ϵ = resulting strain.

There are many variants of the modulus parameter depending on the configuration of the applied stresses and the response of the material, e.g. secant modulus, bulk modulus, volumetric modulus, etc. In pavement design the resilient (elastic) modulus (E_R) is the parameter that is used, and it is defined as follows:

$$E_R = \frac{\sigma}{\varepsilon_R}$$

where σ = applied stress; and,
ϵ_R = recoverable strain.

The resilient modulus is defined in diagrammatic form in Figure 1. Figure 1 shows a specimen of material being subjected to an axial load. As a result, the specimen deforms by reducing in length and increasing in diameter. When the load is subsequently removed the specimen rebounds but it may not fully recover to its original dimensions. The resilient modulus is based on the *recoverable* component of the deformation only. The remaining deformation is termed *plastic* or *permanent* strain. In constructing a pavement, one objective is to minimise the occurrence of permanent strains because they manifest as permanent deformation in the form of depressions (ruts) in the wheel paths at the pavement surface.

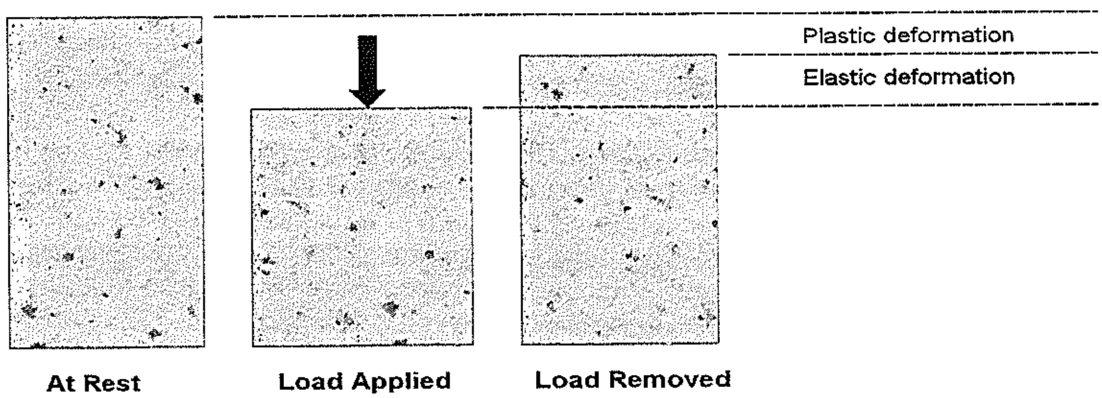

Figure 1 Illustrating plastic and elastic deformation.

Most modern mechanistic design procedures treat pavements as multi-layer elastic structures. The analysis technology has progressed from the original Boussinesq theory for single layered structures to today's sophisticated multi-layer elastic computer programs that use complex numerical methods. Further sophistication in terms of finite element software can also be utilised, however this has remained within the realms of research and academia to date.

The materials in each layer are assumed to be homogeneous and conform to a linear elastic stress - strain response. It is well established that these assumptions are flawed, however, the assumption of linear elasticity is reasonable provided the stresses imposed on the materials are low relative to their ultimate capacity (Thenn De Barros 1964). Note that for linear elastic materials the elastic modulus and the resilient modulus parameters coincide since, by definition, there is no plastic strain.

In the AUSTROADS design procedure a trial pavement is analysed using a multi-layer elastic modeling program (CIRCLY) and the critical strains occurring under specified loading conditions are established. From there, the various pavement components are assigned a service life depending on the magnitude of the critical strains.

The weakness of mechanistic pavement design procedures is that while they exhibit some elements of rational analysis, they are virtually all based on empirical relationships between elastic performance and plastic failure. Two possible "failure" criteria are recognised for flexible pavements, i.e:

- fatigue of the asphalt surfacing i.e. the surfacing cracks; and,
- vertical strain in the subgrade which appears as ruts in the surface of the pavement.

The second criterion had its genesis in the reports of famous AASHO Road Tests in the 1950's and was used in the Shell Design Method (1963). It still is accepted unquestionably by most practitioners to this day even though many of the AASHO Road Test results have been criticised in the technical literature.

An intriguing aspect of pavement design is the fact that there is no known method of applying classical geotechnical theory to estimate pavement life. Elements of load and stress distribution, material anisotropy and elastic analyses all feature but overruling all these is the pavement engineer's conviction that the pavement ultimately fails by some fatigue mechanism. Work being carried out by Professor Ian Collins (Auckland University) into the mathematical analysis of *shakedown theory* holds promise that a more appropriate geotechnical approach to pavement design will be developed.

3 WHERE DID THE ELASTIC MODULUS REQUIREMENT COME FROM?

Many members of the New Zealand roading fraternity regard the requirement to establish the elastic modulus is overly onerous and comment that it wasn't necessary in the old NRB/TNZ design method. While this may be true, the fact is that elastic moduli were always there but they were effectively transparent to the user of the design charts.

The design charts in the old *orange book* were simply the result of numerous multi-layer elastic analyses similar to the CIRCLY analyses performed in the current AUSTROADS procedures. In these analyses the elastic moduli of the layer were governed by the CBR of the subgrade. The subgrade CBR was converted to an elastic modulus using the following relationship:

$$E_{SG} = 8(CBR) \qquad CBR > 13\%$$

$$E_{SG} = 20(CBR)^{0.64} \qquad CBR < 13\%$$

The elastic modulus of the overlying aggregate layer (basecourse and subbase layers combined) was then assigned a value that was dependent on the thickness of the layer, i.e.

$$E_{AGG} = kE_{SG}$$

where $k = 0.2h^{0.45}$

h = combined basecourse and subbase layer thickness (mm).

The advantage of the AUSTROADS procedure is that the designer runs the multi-layer elastic analysis instead of it being already done in the form of a design chart. This provides the opportunity to investigate the influence of the full range of modeling parameters which was previously not possible. In the process the designer gets to more fully appreciate the influence of each layer and where critical features lie.

4 ESTABLISHING THE ELASTIC MODULUS

The non-linearity of most subgrade soils and unbound aggregate courses makes the determination of the elastic modulus parameter something of a moving target. It is well established that the elastic modulus of a cohesive soil is a function of the magnitude of the prevailing deviator stress. As the deviator stress increases, the elastic modulus tends to decrease. On the other hand, the elastic modulus of a non-cohesive material is a function of the prevailing average stress. As the magnitude of the average stress increases, so too does the elastic modulus.

The stress conditions experienced by a given element of material in a pavement vary depending on the elevation in the pavement. Therefore, the elastic moduli of the materials also vary with elevation, even within layers of the same material. In general, the effective elastic modulus of an unbound material tends to decrease with depth while the effective elastic modulus of a cohesive soil tends to increase with depth. If variations in material consistency, water content and density are also considered, it becomes clear that characterising the pavement in the modeling process is far from simple.

There are several procedures available for determining the elastic modulus of pavement materials. The AUSTROADS procedure suggests the following tests are appropriate for unbound aggregates, in order of preference:

- repeated load triaxial test;
- back-analysis from deflection tests; and,
- presumptive values.

For subgrade soils the AUSTROADS procedure suggests the following (in no particular order):

- Scala penetrometer tests;
- in situ CBR tests; and
- laboratory CBR tests.

While the use of repeated load triaxial tests for subgrade soils is not mentioned in the AUSTROADS procedure, the AUSTROADS Pavement Research Group (APRG) offers a suitable protocol in the APRG Report No 8 document.

The simplest method of establishing the elastic modulus parameter for any material is to estimate an appropriate value from the designer's experience with the material in question, or to use a simple index test to suggest an appropriate value. Ranges of elastic moduli are available for various soil classifications (AUSTROADS 1992) as shown in Table 1.

Table 1 Inferred elastic moduli for various soil classifications (after AUSTROADS 1992).

Description of Subgrade		Typical (Inferred) Elastic Moduli (MPa)	
Material	**USC Classification**	**Well Drained**	**Poorly Drained**
Highly Plastic Clay Silt	CH ML	50	20 - 30
Silty Clay Sandy Silt	CL SC	60 - 70	40 - 50
Sand	SW, SP	150 - 200	N/A

Other empirical correlations are available, e.g. Hall and Thompson (19xx) reported that the elastic modulus of a cohesive soil at optimum water content is a function of the percentage of clay, the plasticity index and the percentage of organic carbon, i.e:

$$E_{OPT} = 6.90 + 0.0064(C) + 0.216(PI) - 1.97(OC)$$

where E_{OPT} = subgrade elastic modulus at optimum water content (ksi);
C = percent clay;
PI = plasticity index; and,
OC = percent organic carbon.

As outlined above, a common way of establishing elastic moduli is to use a correlation with CBR. The relationship used in the NRB/TNZ orange book has already been presented but there are several variations. The relationship used in the AUSTROADS procedure is simply:

$$E = 10(CBR)$$

It should be noted that these relationships are only valid for subgrade soils and also that they should be used with great caution. The relationships originated from research by Heukelom & Klomp (1962) who established that the elastic modulus of a soil was proportional to CBR, but the constant in the relationship varied from 5 to 20. The factor 10 in the AUSTROADS procedure was chosen as a reasonable compromise.

In the authors experience relationships between elastic modulus and CBR are fraught with danger. In fact, the Californians stopped using the CBR test more than fifty years ago because of its deficiencies. Brown et al (1990) reported that there is not a simple relationship between elastic modulus and CBR but rather it is influenced by both soil type and the state of stress. Brown (1996) concluded that, "the CBR test was not helpful for determination of the soil stiffness for pavement design". Using a New Zealand example, pumice soils often achieve CBR values of 30 or higher, however, elastic modulus values determined from field deflection tests may be in the vicinity of only 50 to 100 MPa, certainly not 300 MPa as predicted by the relationship presented above.

Sweere (1989) performed a number of CBR and elastic modulus tests and found virtually no correlation between the two sets of data. The reason for this is that the CBR test involves a very high level of strain using a rigid loading platen. This produces extremely high stresses around the edges of the CBR plunger which cause the soil to yield. It is this yielding that dictates the outcome of the test. Therefore, the CBR test is more a measure of the plastic properties of the soil than the elastic properties.

The AUSTROADS procedure also describes the use of the Scala penetrometer for establishing subgrade elastic modulus values. The Scala blow counts are converted to equivalent CBR values, and then to elastic moduli. This suffers from the same problems as the direct conversion from CBR to elastic modulus except there is a further step in the process which itself is subject to error. The use of the Scala penetrometer in fine grained soils is also a point of contention amongst geotechnical and pavement engineers. In the authors view, the Scala penetrometer can be useful for identifying changes in soil stratigraphy however its main purpose is to determine the density of non-cohesive soils by measuring the resistance to penetration.

Elastic modulus values can be determined by analysing the deflection of a pavement under a known loading condition. This procedure is considered to be preferable to the correlations described above, however nonlinear effects in the subgrade can produce inconsistent results. Deflections can be analysed from apparatus such as the Benkelman beam, plate loading test, falling weight deflectometer (FWD) and the Loadman portable FWD. The authors are currently developing a procedure for using the Loadman portable FWD in laboratory tests.

Another method of determining elastic moduli is to use the repeated load triaxial test. While the test procedure and equipment is relatively complex, it provides the benefit of being able to determine the elastic modulus over a range of stress conditions and sample configurations. Test specifications are available from the APRG and the American Association of State Highway and Transportation Officials (AASHTO) organisations. Disadvantages of the triaxial test are its relatively high cost and the influence of sample disturbance.

Measurement of the velocity of shear waves within a soil has been used to determine elastic properties, however this practice seems to have reduced in popularity in recent.

5 APPLYING ELASTIC MODULI IN DESIGN

It has already been described how the elastic modulus of materials in a pavement vary with depth, even within layers of a single material. However, when a trial pavement is modeled using CIRCLY there can only be one elastic modulus assigned to each layer. The use of sublayering in unbound aggregate courses assists in this matter as each sublayer is configured so that its thickness is in the range 50 mm to 150 mm. Typically this means that most aggregate courses are subdivided into two, three or four sublayers and appropriate elastic moduli are assigned to each. As with most geotechnical analyses of layered structures it is reasonable to characterise the physical properties of the materials at the mid-points of each sublayer. Therefore, the associated effects of elevation in the aggregate course and material nonlinearity are minimised.

In the subgrade the situation is not so simple since the thickness of the layer is effectively infinite. Clearly it is impossible to characterise the subgrade at the mid-point of the layer. So, at what elevation should it be characterised? Perhaps it should be at the top where the vertical compressive strain is determined? It is however the most conservative option since the stress conditions are the most demanding at the top of the subgrade, and considering material nonlinearity, that corresponds to the lowest elastic modulus throughout the entire subgrade depth.

Lottman (1976) suggests that a subgrade should be characterised at an elevation corresponding to the *average depth of significant stress* (ADSS). The ADSS is defined as the distance from the ground surface to one third of the depth where the deviator stress is reduced to one tenth of the deviator stress occurring at the top of the subgrade. The stress conditions at the ADSS can be determined for a trial pavement structure using CIRCLY and then laboratory triaxial test data can be used to establish the corresponding elastic modulus. This process is iterated until the stress conditions and elastic modulus data converge.

To obtain a simpler alternative to the ADSS a series of CIRCLY runs were performed by the authors using a two layered structure comprising typical elastic parameters and aggregate layer thicknesses. The following variables were used in the analyses:

	E_{AGG}	: 200, 300, 400 MPa
	E_{SG}	: 30, 50, 70 MPa
	h_{AGG}	: 150, 200, 250, 300, 350 mm
where	E_{AGG}	: elastic modulus of the aggregate (top sublayer);
	E_{SG}	: subgrade elastic modulus; and,
	h_{AGG}	: aggregate layer thickness.

The deviator stress was determined at regular intervals of depth up to 1 m. A plot of deviator stress versus depth into the subgrade is presented in Figure 2. It is postulated that the centroid of the area under the deviator stress versus depth plot (up to 1 m depth) is an appropriate depth at which the subgrade layer can be characterised. Analysis of the shapes of the plots in Figure 2 indicate that the centroids were all in the depth range of 280 mm to approximately 340 mm. Therefore a depth of 300 mm is considered to be reasonable.

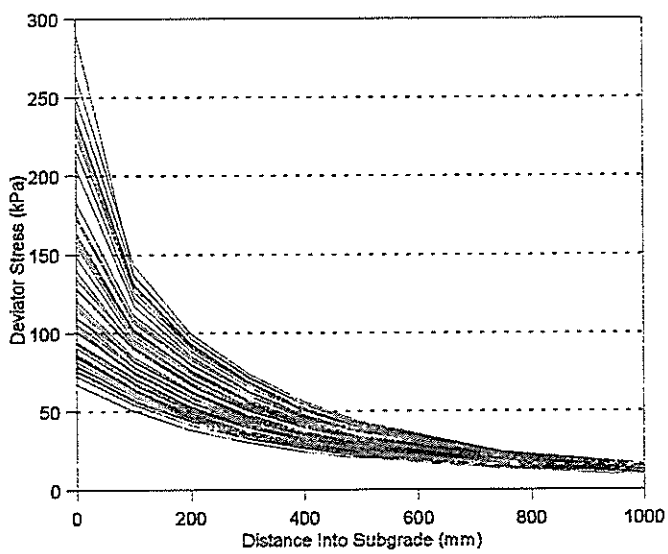

Figure 2 Deviator stress versus depth for typical unbound aggregate pavements.

6 INFLUENCE OF THE ELASTIC MODULUS PARAMETER

The influence that the elastic modulus parameter has on typical pavement designs has been evaluated by performing a number of CIRCLY runs using typical pavement configurations.

Figure 3 shows a plot of pavement design life versus subgrade elastic modulus. The pavement models used to generate the data included an unbound aggregate layer thickness of 300 mm, an aggregate layer elastic modulus of 350 MPa and AUSTROADS sublayering. The Poisson's ratios of the aggregate layer and subgrade were 0.35 and 0.45 respectively.

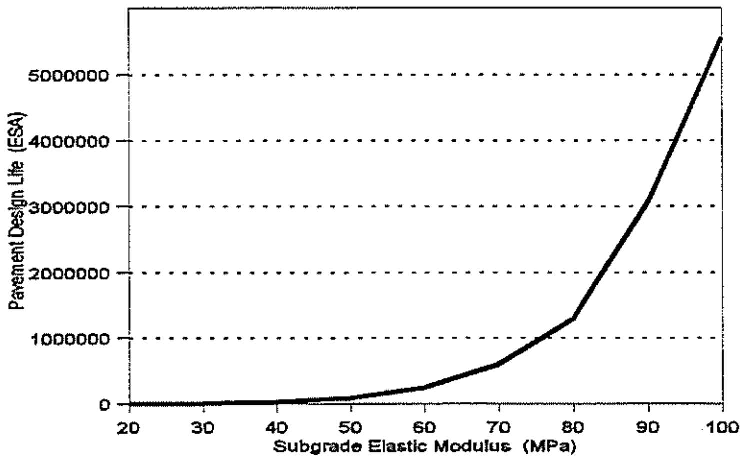

Figure 3 Pavement design life versus subgrade elastic modulus for typical unbound aggregate pavements.

Figure 4 shows a plot of pavement design life versus aggregate layer elastic modulus. The CIRCLY models used the AUSTROADS sublayering scheme and the aggregate layer elastic modulus corresponds to the value for the top sublayer. The pavement models used to generate the data included a subgrade elastic modulus value of 50 Mpa.

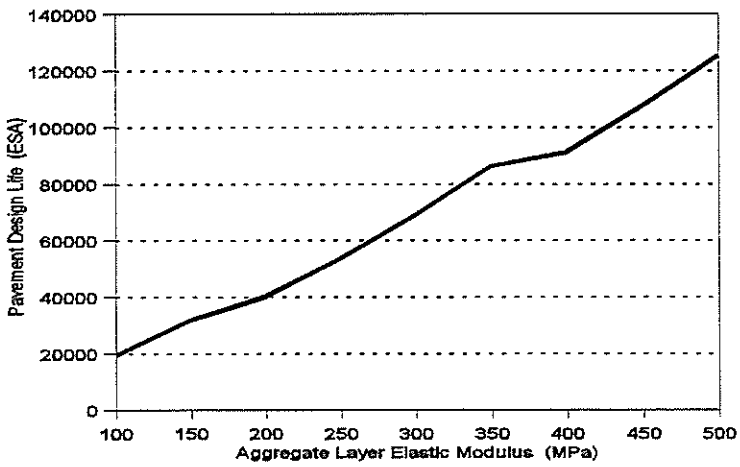

Figure 4 Pavement design life versus aggregate layer elastic modulus for typical unbound aggregate pavements.

It is clear from Figure 3 and 4 that, in terms of pavement design life, the subgrade elastic modulus parameter is significantly more influential than the aggregate layer elastic modulus parameter. Therefore, materials investigations for pavement projects should concentrate on the accurate characterisation of the subgrade, while typical or presumptive elastic moduli may be adequate for the unbound aggregate materials.

7 CONCLUSIONS

The adoption of the AUSTROADS procedure in New Zealand has meant that designers now have to consider the elastic modulus of the materials as an essential part of the analysis of trial pavements. Presumably, they will eventually be interested to know the elastic modulus that is actually achieved during construction and how to monitor it.

The general perception of the AUSTROADS procedure is that is significantly different from the NRB/TNZ procedures of the old orange manual. In fact, the approach is very similar except that the designer now has a great deal more flexibility and is not tied to the pavement conditions that are implicit in a design chart.

It is often perceived that the so-called mechanistic pavement design procedures such as those described in the AUSTROADS procedure are scientifically accurate and robust. Designers must be aware that in spite of the pseudo scientific approach of all modem pavement design methods, there is at the heart a very empirical relationship of doubtful legitimacy.

There are a number of variations of the elastic modulus parameter. However, for the purposes of pavement design AUSTROADS specifies the *resilient modulus* which is the ratio of the applied stress to the recoverable strain.

A number of methods for measuring the resilient modulus are discussed in the AUSTROADS procedure. The method adopted should match the scale of importance of the pavement.

Analysis using typical flexible pavement structures show that the subgrade elastic modulus parameter has a much more significant influence on pavement design than does the elastic modulus of the unbound aggregate layers. Therefore, the majority of the effort should be placed on investigating the subgrade soils and establishing appropriate elastic parameters.

It may be argued that presumptive elastic moduli for unbound aggregate layers are adequate for most low to moderate budget pavement projects. However, in time the aggregate suppliers should be able to supply the results of resilient modulus tests for each of their individual roading products.

The AUSTROADS procedure promotes the use of the CBR test for establishing the elastic modulus of subgrade soils. The authors disagree with this approach as the CBR test has been widely exposed as being appropriate for determining elastic parameters.

When analysing trial pavements in the design procedure, the various layers should be characterised at appropriate locations in the model. For unbound aggregate layers the appropriate location is the mid-height of each sublayer. For the subgrade layer the appropriate location is the average depth of significant stress. This is typically about 300 mm below the top of the subgrade.

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MEASUREMENT OF RESILIENT MODULUS AND PERMANENT STRAIN BEHAVIOUR OF ROADING MATERIALS

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SUMMARY

This paper presents a summary of the testing methodologies for the repeated load triaxial tests. Axial strains were measured on the compacted sample under cyclic loading, using internal transducers clamped to the sample, and an external transducer attached to the triaxial cell. Comparisons were made of the resilient moduli of three material types, as well as the differences in the permanent strain measurements of an aggregate subjected to more that 50,000 cycles of repeated loading.

1. PAVEMENT DESIGN

Road structures have traditionally been designed (i.e. the materials and layer thickness of the different structural layers) using empirical design methods. With time, such methods have been calibrated for practical application by either systematic road tests or observations and back calculations from road performances .

Typical examples of such methods:
Index method (Highway Research Record 337-1945) using soil classification tests.
Californian Bearing Ratio (C.B.R.) a compression test, originating in the USA and used world wide which assesses the elastic and plastic properties of materials.
AASHTO method (1986) based on the AASHO road test performed in the early 1960's.
T.R.R.L. (Powell et al 1984) developed in Great Britian for the structural design of bituminous roads.
Additional empirical methods from Canada and Germany rely strongly on a plate bearing test.

The new design analytical methods or mechanistic design procedures,based on the structural analysis of a multi-layered pavement subject to traffic loading. These methods aim to model the behaviour of the road structures based on the basic mechanical and physical properties of the roading structural materials. The pavements are analysed as layered elastic systems in which each layer (or material type) is described by its elastic modulus and Poisson's ratio. The stress range is limited to ensure that the aggregate deformations caused by repeated loading are almost entirely recoverable. The elastic modulus and permanent deformations of various materials are investigated by the Repeated Load Triaxial Test (RLTT).

Transit New Zealand recently adopted the pavement procedures outlined in the Austroads Pavement Design Guide. The RLTT method adopted was initially based on a procedure published by the Australian Pavement Research Group (APRG Report No. 8 - 1993) and recently adopted by Standards Australia (AS1289.6.8.1 - 1995). The APRG subcommittee took into account the AASHTO Test Designation methods of test from the American Strategic Highway Research Programme (SHRP) Protocol P46 which defined a similar test standard but having a differing loading cycle (havesine) applied for a shorter duration (1 second).

2. TEST DESCRIPTION

The pavement material tested by RLTT methods may consider two strain components in response to the repeated loading: the resilient strain (elastic or recoverable) and the permanent strain (plastic). It is recognised that the triaxial testing does not fully simulate the complete stress path in a pavement basecourse, under a moving wheel i.e. such tests cannot model the rotation of principal stresses as a wheel approaches, passes over and continues from a fixed point.

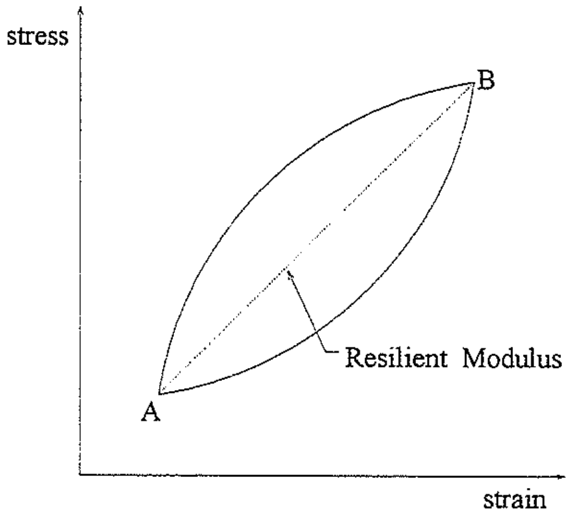

Figure 1. Stress/strain relationship for determining the resilient modulus.

The resilient modulus is represented by the stress/strain relationship as shown in Figure 1.

The test conditions used in this study, hold the cell pressure constant and repeat the axial load (square wave) during each load cycle (1/3 Hz). As most aggregates have a non-linear response to loading, a range of stress levels are chosen (related to the contact stress(P) at the surface of the pavement - typically 550 or 700 kPa), with the resilient modulus measured for a few hundred loading cycles at each stress level, on a single sample, to characterise the material response.

For the permanent strain, the test applies one stress level for many loadings (typically 50,000) and records the permanent strain that develops.

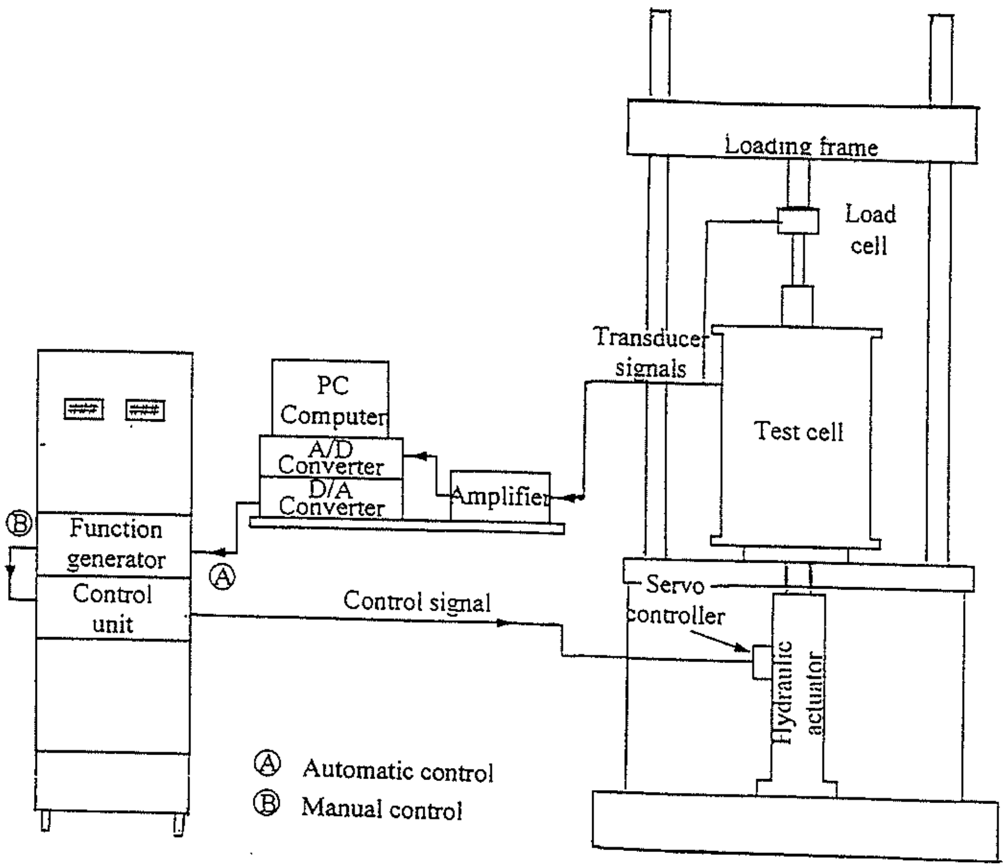

Figure 2. Schematic diagram of the repeated loading triaxial testing equipment.

3. REPEATED LOAD TRIAXIAL EQUIPMENT

The dynamic triaxial tests, consisted of the application of repeated axial loads to a cylindrical compacted sample contained within a triaxial cell. Figure 2 shows the axial loading test system used; an M.T.S. 810 system having a servo-controlled electro-hydraulic actuator in a load frame of 250 kN capacity. Electronic control of the loading was from an on-board function generator controlling the frequency and the pulse shape, and a controller ensuring the dynamic load was of a stress controlled mode.

The instrumentation of the triaxial cell used independent transducer measurements (internal and external to the triaxial cell) connected to a separate Daytronic instrumentation signal conditioning system. Scaled and conditioned data was acquired by a 12 bit computer A/D board. A software package (written in Turbo Pascal) was used to record the analogue outputs simultaneously for the load and displacement transducers for a "batch" of some 8-10 cycles. The computer screen displayed the load/displacement loop as it was recorded, with the data subsequently analysed and presented in tabular form of the stress, strain, cell pressure and resilient modulus of each cycle recorded. For the dynamic testing, the "batch" of data was collected about the 50, 100 and 200 cycle loadings, enabling the average stress and resilient modulus values to be compared for such points of the loading sequence. The 200th cycle average result was normally reported.

4. TRIAXIAL CELL

The axial deformations of the sample are measured using transducers located both inside and outside the test cell. Due to uncertainties induced by the fact that the loading stresses are not evenly distributed near the ends of the specimen (Toan 1975) the internal axial deformations are measured over a length of 100 mm in the centre of the specimen (Figure 3) using two R.D.P. ± 5 mm L.D.V.T. transducers mounted between two perspex calipers, spring clamped to the surface of the membrane enclosed sample. The outputs from the transducers were

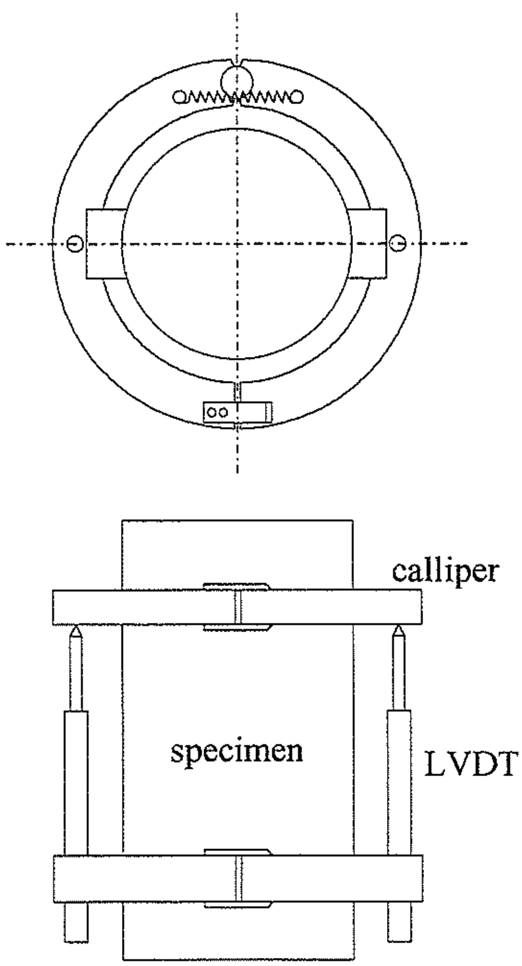

Figure 3. Diagram of internal displacement transducer mounting.

differentially connected to a Daytronics 9132 differential, ac-excited, signal conditioner/demodulator, to give the average axial deformations of the sample. These transducers has been found to operate successfully under a water pressure of 1 MPa. The perspex calipers have the ability to measure the radial deformations of the sample - such measurements were not undertaken in this study.

5. TEST RESULTS

The results of this study of the resilient modulus and permanent strain characteristics is of laboratory testing of soil, aggregate and lightweight foam concrete. Table 1 presents the classification, bulk density and water content of the materials presented in this study. In order to minimise the effect of the loading platen and material interaction, the casting of a dental plaster capping on each end of the sample was undertaken, with small drainage holes drilled through the capping to facilitate sample drainage. Figure 4 shows the combined results of the resilient modulus using both internal and external strain measurements, with the error increasing with an increasing mean principal stress for the aggregate and foam concrete. Figure 5 shows both the permanent strain (%) and the resilient modulus differences for internal and external measurements. The trends of a reduction in resilient modulus for the high cycle numbers (i.e. >50,000 cycles) for an increase in axial strain, are consistent with an increase in the material degradation.

Table 1. Summary of material properties.

Type	ρ_{bulk} (kg/m^3)	w.c. (%)
Soil	2020	12.8
Aggregate	2050	7.9
Foam Conc.	830	1.9

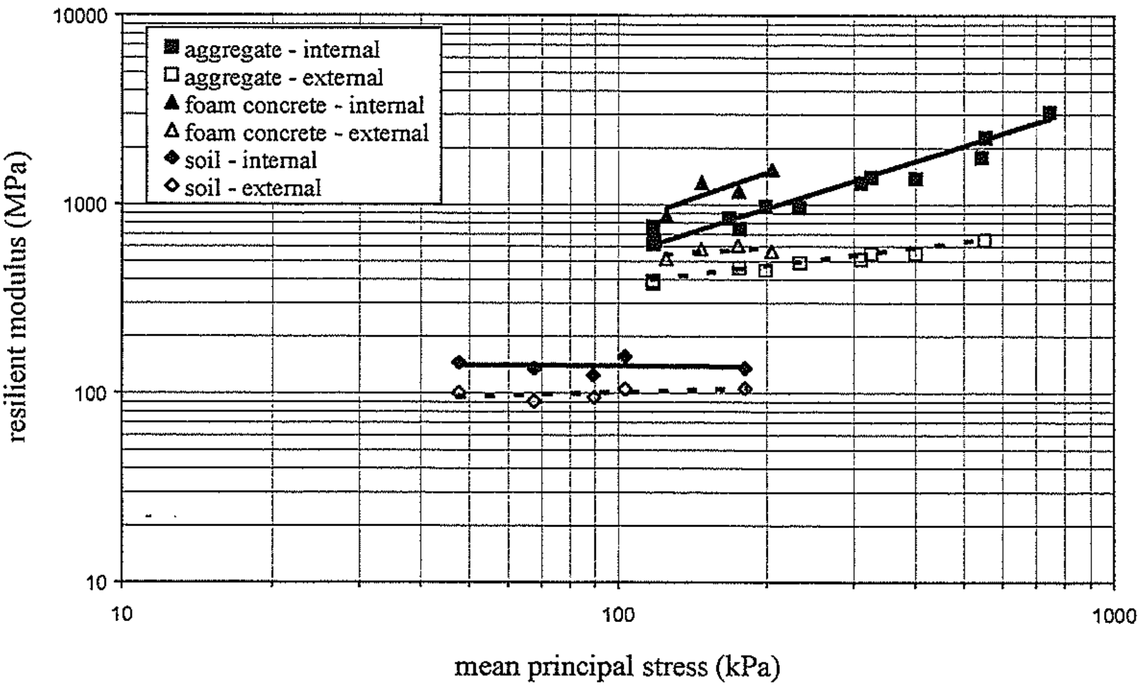

Figure 4. Typical results of resilient modulus at different stress levels for various materials.

6. HAVESINE LOADING

Figure 6 shows the distribution of stress in a road structure under a wheel load measured at the Road Research Laboratory - UK (Whiffin 1955) as reported by Grainger and Lister. The application of a havesine loading pulse to RLTT sample is considered to closely represent the wheel loading.

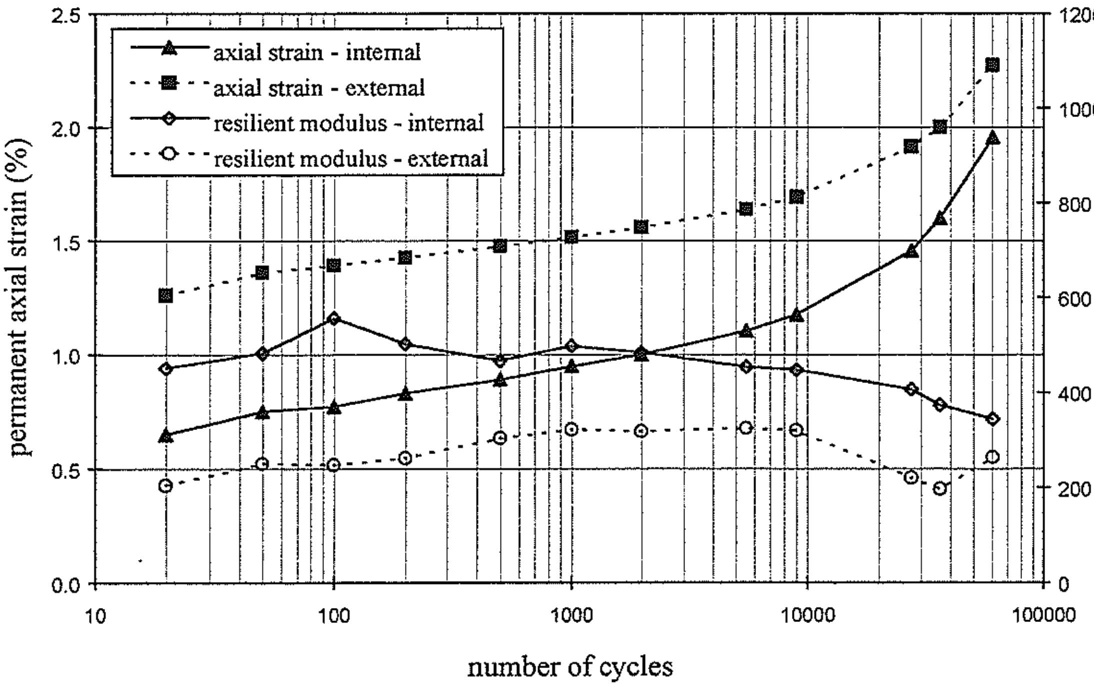

Figure 5. Typical results of the permanent strain and resilient modulus for repeated loading on an aggregate.

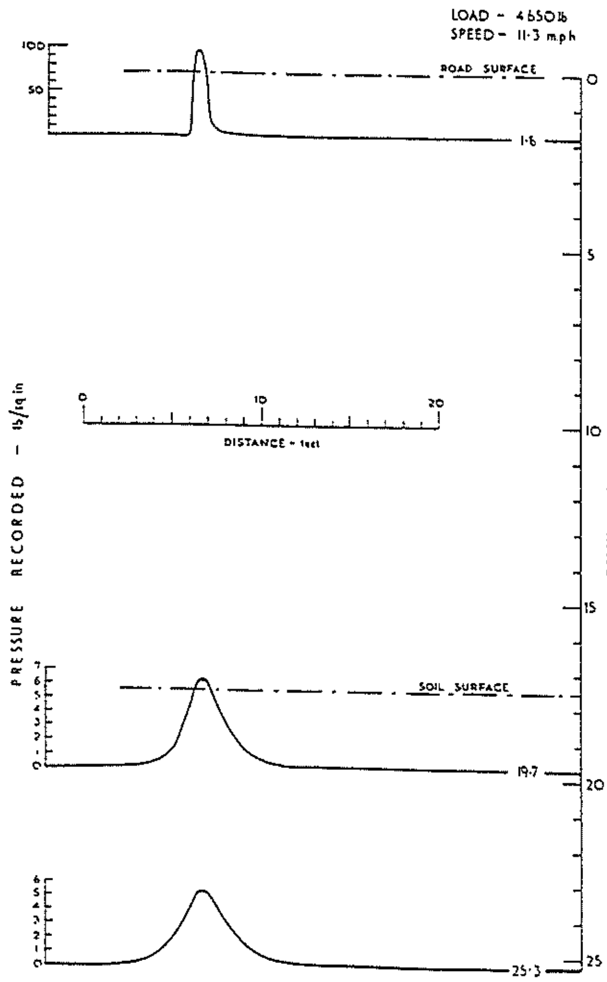

Figure 6. Typical instantaneous distribution of stress in road structure under a moving wheel load (Whiffin 1955).

Figure 7 represents the condition that the computer software considers in calculating the turning point of the hysteretic loading that occurs in the RLTT i.e. it finds the maximum (or minimum) and the change in direction and applies a linear extrapolation of the adjacent points, to determine the turning point on the stress/strain loop. With the application of a square wave (as per AS1280.6.8.1), the load (and unload) is permitted to be applied for up to 1 second.

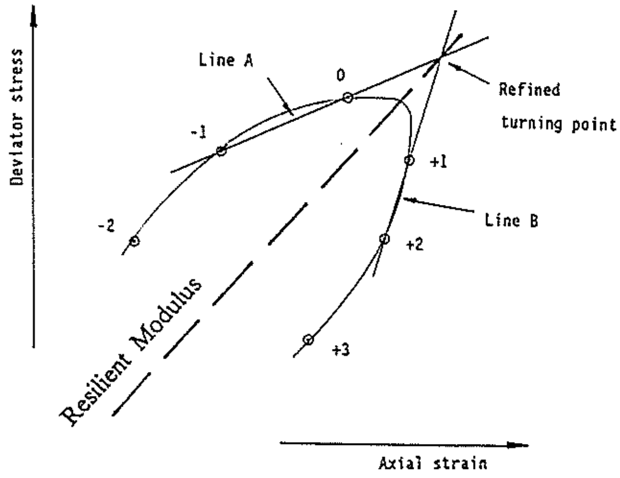

Figure 7. Turning Point refinement for a stress-strain loop.

Where a recording system has typically 30 - 50 points per cycle to define the load/displacement hysteresis loop, the square wave application will have approximately 30% of the points recording the maximum and minimum loadings. With feedback over-shoot or servo settling being possibly recorded during this interval, it is not uncommon for the software to attempt to calculate erroneous turning points of the load plateau when analysing the recorded data. This effect is minimised when the havesine pulse is applied to the sample as the load is continually changing and does not maintain a continuous peak stress state.

A comparison study of the square and the havesine loading was undertaken with a typical result shown in Figure 8. No appreciable effect has been detected on the resilient modulus. A simplification of the analysis process has however been determined.

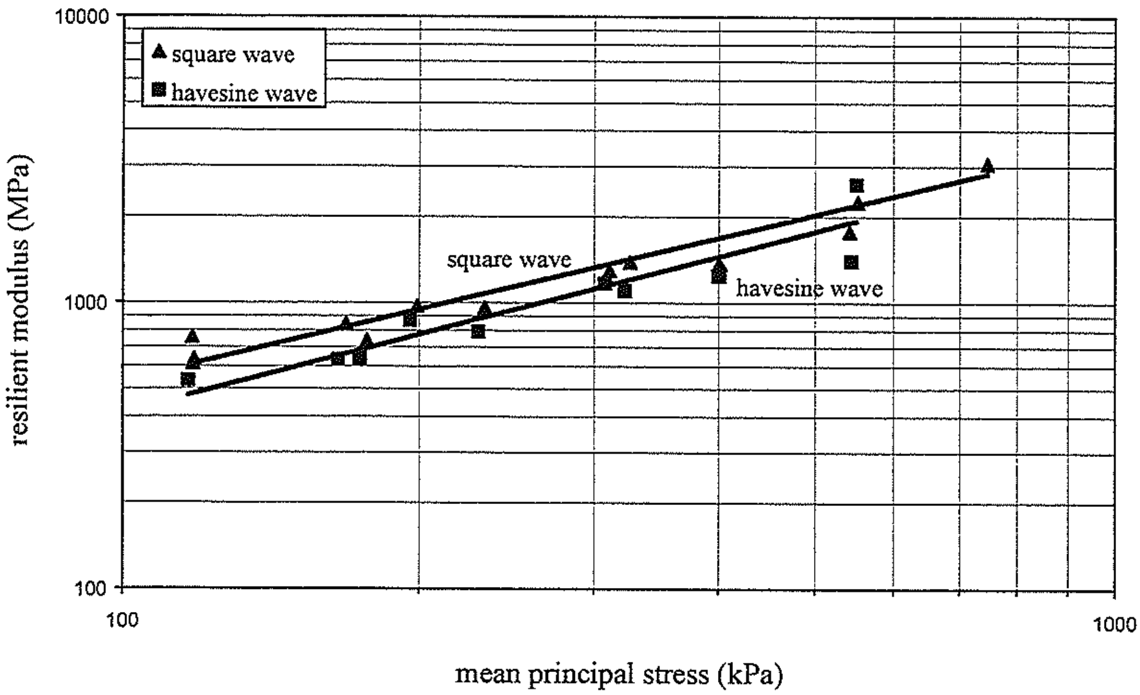

Figure 8. Resilient modulus comparison results for the square wave and havesine wave load application on an aggregate

7. CONCLUSIONS

1.Repeated load triaxial testing is a new tool for the road designer to apply a mechanistic design procedure, or an analytical design method to the structural analysis of a multi-layer pavement subject to wheel loadings, taking into account the elastic stress and strain behaviour of the material to be used, and to providing an assessment on such a structure to its possible life.

2. Typical test results for a soil, aggregate and foam concrete have been presented to show that the internal strain measurements are very important in determining the elastic modulus values for a roading material subjected to repeated loading.

3. Permanent strain values measured
internally for repeated loading on an aggregate, are considerably less than that determined for external measurements.

4. No appreciable effect on the measured resilient modulus is apparent from measurements made of two different loading waveforms. The analysis of the results gathered with the havesine waveform is simpler than that for the square wave, as the definition of the turning point is much clearer.

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USE OF REPEATED LOAD TRIAXIAL TEST FOR ASSESSING PROPERTIES OF NEW ZEALAND BASECOURSE MATERIALS

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SUMMARY

A repeated load triaxial test has been developed by Opus International Consultants to investigate the behaviour of typical New Zealand unbound pavement materials. Initial tests were carried out to assess the effects of test conditions, such as specimen diameter, particle size and drainage conditions. Best results on basecourse aggregates were obtained in 150 mm diameter samples prepared by removing particles retained on the 26.5 mm sieve and replacing them with an equivalent mass of gravel from 9.5 mm to 26.5 mm.

In fully drained tests on basecourse materials, the permanent strains grew linearly with the logarithm of load cycles. Elastic modulus values were higher than the presumptive value of 400 MPa for base quality gravel in the Austroads Pavement Design Guide. Saturated materials had much lower elastic modulus values and higher permanent strains when tested in undrained conditions.

1. INTRODUCTION

Transit New Zealand has now adopted the mechanistic pavement design procedures outlined in the Austroads Pavement Design Guide[1]. The design procedure is based on structural analysis of a multi-layered pavement subject to traffic loading. Estimates of the elastic properties each of the pavement layers are required information for the analysis. According the Austroads Guide, the preferred method for elastic characterisation of unbound granular material is the Repeated Load Triaxial Test.

The test can also be used to investigate the permanent strain behaviour of an unbound granular material. In the Austroads mechanistic design procedures for unbound granular pavements, performance criteria are related to vertical strain at the top of the subgrade. Deformation in unbound granular pavement materials is not considered by the model. However, excessive permanent deformation in the basecourse or subbase could occasionally lead to premature failure of the pavement.

2. TEST DESCRIPTION

In the repeated load triaxial test, a cylindrical sample of compacted aggregate is encased in a rubber membrane and placed within a sealed cell. The cell is pressurized to provide a confining force on the sample and a load pulse is applied repeatedly to the ends of the sample to simulate the passage of vehicle loads over a pavement.

The response of a granular material to repeated loading may be divided into two components: the resilient (also called recoverable or elastic) strain and the permanent (or plastic) strain. Two different types of repeated load triaxial tests are available, depending on which component is of greater interest.

The resilient modulus is the repeated axial pressure divided by recoverable strain. High resilient modulus indicates a stiff material that is likely to spread applied pavement loads over a large area and reduce the stresses in lower pavement layers and the subgrade. In the resilient modulus test, several different load levels are applied for a few load cycles at each stress level.

In the permanent strain type of test, one stress level is applied repeatedly for many loading cycles (typically 100,000 cycles). At low stress levels, the permanent strain grows at a decreasing rate, while the elastic component remains nearly constant. After many load applications the accumulation of permanent strain per load cycle is very small and the material is essentially elastic in its response to loading. At high levels of stress, some materials suffer a rapid increase in permanent strain that can be regarded as a failure condition for pavement design purposes.

3. MAXIMUM PARTICLE SIZE

According to Australian and American test procedures[2,3], the diameter of the test specimen should be greater than five times the maximum particle size. Oversize particles up to 5 percent of the material can be discarded without significant effect on the test results. For a common specimen diameter of 100 mm, these criteria effectively limit the application of the test to aggregates with maximum particle size of 20 mm

We considered several options to deal with the larger particles in typical New Zealand basecourse materials including the following:

- Ignore influence of oversize particles and test whole samples.
- Increase the size of the test specimen. Triaxial equipment, using a simple pneumatic loading system, was developed to test specimens up to 150 mm diameter.
- Remove (Scalp) oversize particles.
- Replace oversize particles with an equivalent mass of pea metal from the same source. This a common technique for compaction testing of gravelly soils.

The most consistent results on AP40 material were obtained in 150 mm diameter samples when the oversize particles were removed and replaced with an equal mass of pea metal (between 19 mm and 30 mm) from the same source.

The upper limit of oversize particles that can be replaced without significant effect on the test results is uncertain. We recommend that aggregates with up to 30 percent particles greater than 30 mm may be tested in a 150 mm diameter sample, using the replacement method of sample preparation. Using this method, all AP40 basecourse materials, and many subbase aggregates may be tested in the repeated load triaxial tests. Some very coarse aggregates (eg AP75) may not meet these criteria and cannot be tested with the existing apparatus and testing methods.

4. SAMPLE PREPARATION

According to current Transit New Zealand specifications, the density of basecourse should be at least 95 percent of maximum dry density, as measured in the vibrating hammer compaction test. Other target densities may be specified to investigate pavement failures suspected to be related to aggregate density.

We recommend vibrating hammer type of compaction for repeated load triaxial test samples. This method provides good control on the sample density and it reproduces field conditions under vibrating rollers and it is similar to the vibrating hammer compaction test. Other sample compaction techniques, such as tamping hammer or static compaction, may be specified if needed.

At low levels of saturation, the test results are insensitive to the moisture content of the sample. Moisture content can be controlled by drying or wetting the sample before compaction. In recent tests by Opus, the specified moisture was set at optimum water content plus 0.5 percent to allow for some drying during sample preparation.

Controlling the water content at high levels of saturation is difficult because excess water drains from the aggregate during compaction. However, the sample can be saturated after compaction by adding water through the drainage ports in the triaxial cell.

5. TEST STRESSES

Cell pressure and deviator stresses in the triaxial cell can be calculated to reproduce estimated stresses under a wheel load. However, both cell pressure and axial load must be pulsed during each load cycle to reproduce the complete stress path in a pavement basecourse. Also, triaxial tests cannot model the rotation of principal stresses as a wheel approaches, passes over and then recedes from a point in the pavement.

The complexity and cost of the repeated load triaxial test is greatly reduced by holding the cell pressure constant and repeating only the axial load during each load cycle. Figure 1 shows the estimated stress path of an element of basecourse under wheel loading, compared to the stress path in a constant cell pressure triaxial test. Test conditions are set to reproduce the stresses under the peak wheel load, but the cell pressure between load applications is higher than the overburden stress in the pavement.

Because most aggregates have a nonlinear response to loading, the measured resilient modulus depends partly on the stress levels chosen. If the permanent strain behaviour of the sample is not under investigation, the resilient modulus can be measured in a few loading cycles at several different stress levels in a single test.

The Australian standard for repeated load triaxial testing includes stresses that exceed the likely static shear strength of many aggregates. Low modulus values may result from these test conditions. The recommended test sequence shown in Table 1 is based on the American SHRP Protocol 46, scaled to allow higher stresses in typical New Zealand unbound pavements with thin surface layers.The vertical stress at the top of the basecourse in an unbound granular pavement is the same as the tyre pressure of the design vehicle. For subbases or other types of pavements the computer programme CIRCLY may be used to estimate the vertical stress at the top of the layer of

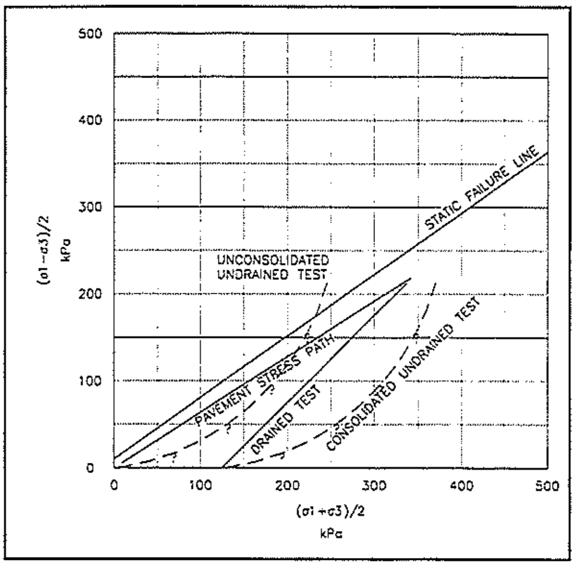

Figure 1 - Typical Stress Paths

Sequence	**Cell Pressure** σ_3	**Deviator Stress** $\sigma_1 - \sigma_3$
Precondition	0.375 p	0.375 p
2	0.075 p	0.075 p
3	0.075 p	0.150 p
4	0.075 p	0.225 p
5	0.125 p	0.125 p
6	0.125 p	0.250 p
7	0.125 p	0.375 p
8	0.250 p	0.250 p
9	0.250 p	0.500 p
10	**0.250 p**	**0.750 p**
11	0.375 p	0.250 p
12	0.375 p	0.375 p
13	0.375 p	0.750 p
14	0.500 p	0.375 p
15	0.500 p	0.500 p
p = estimated vertical stress at top of the layer		

Table 1- Recommended test stresses

interest.

A single combination of cell pressure and repeated axial stress must be selected for permanent strain tests. Previous test programmes on basecourse materials by Opus included permanent strain tests at cell pressure (σ_3) = 125 kPa and deviator stress (σ_1 - σ_3) = 425 kPa. For permanent strain tests on other pavement layers we recommend stresses similar to Sequence 10 in the table above.

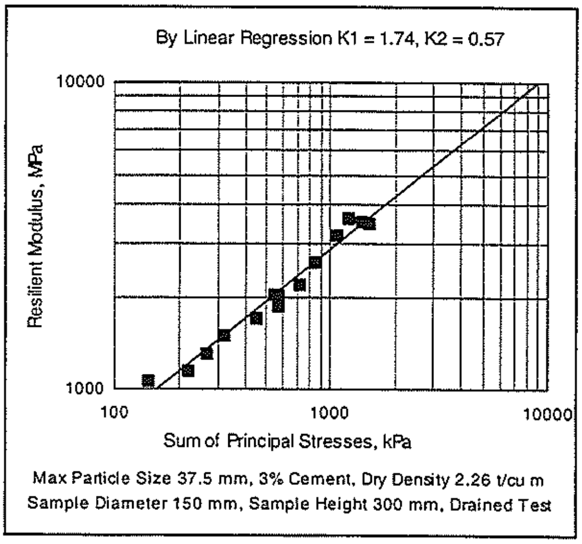

Figure 2 - Results of Resilient Modulus Test

6. DRAINAGE CONDITIONS

Drained triaxial tests allow excess porewater pressure within the sample to dissipate and the stresses acting on the soil particles (effective stress) are the same as the applied total stresses. In an undrained triaxial test the effective stress on the soil skeleton is modified by the excess pore water pressure within the sample. Since the components of the pore fluid (air and water) have different compressibilities, the pore pressure generated in an undrained test is sensitive to the saturation of the soil.

It is good design practice to include drainage features in pavements to ensure that basecourse layers are rarely fully saturated. For samples at low saturation levels, the air in the pore spaces compresses and most of the applied load is borne by the soil particles. Experience in Australia suggests that drainage conditions of the test do not affect results when the specimen is compacted at moisture contents drier than about 70% of optimum moisture content[4].

The Australian test standard specifies undrained test conditions during the initial consolidation and during the repeated load applications. Following this method for saturated soils pore pressures within the sample cannot be controlled and the effective stress on the soil particles is unknown. Initially, the confining pressure in the triaxial cell will be matched by a rise in the pore pressure within the sample and the sample will have very little, if any, effective confining stress on the soil particles. Low modulus values and large permanent strains are likely when testing saturated aggregates by this method and results may not be a true indication of aggregate performance.

An alternative test method is to allow pore water to drain into or out of the ends of the sample during consolidation and during repeated load triaxial testing. With this method some excess pore water pressure may be generated in the centre of the sample, depending on loading duration, permeability and compressibility of the sample. This situation models service conditions of a saturated aggregate that undergoes partial drainage under a moving wheel load. The load duration and drainage path (up to 150 mm) in the repeated load triaxial test are of similar magnitude to field conditions in a basecourse layer.

Another option is to carry out an undrained test with pore water measurements. Pore pressures within the sample cannot be controlled during the test, but they can be measured so that effective stresses during the test can be determined. Drainage should be allowed during consolidation so that the initial confining pressure on the soil particles can be set to the desired level.

7. RESULTS OF RESILIENT MODULUS TEST

Strains in granular soils generally are not a linear function of applied stress and the measured modulus varies with the size of the applied load. Several relationships between stress and modulus have been proposed. Many sources, including the Austroads Pavement Design Guide, recommend the following relationship between resilient modulus and stress:

$$E = K_1 \theta^{K_2}$$

where	$E =$	Resilient modulus
	$\theta =$	Sum of principal stresses, $\sigma_1 + \sigma_2 + \sigma_3$
	$K_1, K_2 =$	Constants depending on properties of the material

According to the AASHTO Guide for the Design of Pavement Structures[5], K_2 varies from 0.5 to 0.7 and K_1 is about 3,000 to 8,000 in psi units (equivalent to K_1 = 6 to 16 for θ in kPa and E in MPa).

Results of resilient modulus tests carried out at several stress levels can be plotted as a function of θ as shown in Figure 2. The test results could be used in a nonlinear mechanistic model (eg finite elements) to predict strains within a pavement. With a linear elastic model such as CIRCLY, each sublayer may be checked to ensure that the specified modulus is close to the measured modulus for the calculated stress level. Several trial runs may be required to match the calculated stresses with the resilient modulus from the repeated load triaxial test.

8. RESULTS OF PERMANENT STRAIN TEST

The Austroads mechanistic pavement design method assumes that the life of a pavement is limited by deformation in the subgrade or by the fatigue life of tensile layers such as asphaltic concrete or cemented materials. Repeated load triaxial testing is useful to confirm the design assumption that there will not be excessive permanent deformation within the unbound granular layers of the pavement.

Resilient modulus can be determined at only one stress level in the permanent strain test. The modulus is expected to remain steady throughout the test. For pavement design, this modulus can be used for the top of the layer and the modulus of lower sublayers can be estimated as described in the Austroads Pavement Design Guide.

Because the accumulation of permanent strains normally slows as the test progresses, it is convenient to plot permanent strain against the logarithm of loading cycles as shown in Figure 3.

Permanent strains in the first few loading cycles of a repeated load triaxial vary widely and the first few loading cycles are not good indicators of aggregate performance. After an initial settling period (up to a few hundred load cycles) the accumulation of permanent strain in each load cycle is much smaller than the resilient deformation and the material is essentially elastic.

Accumulation of permanent strain in the triaxial test may not be a reliable prediction of the growth in rut depth

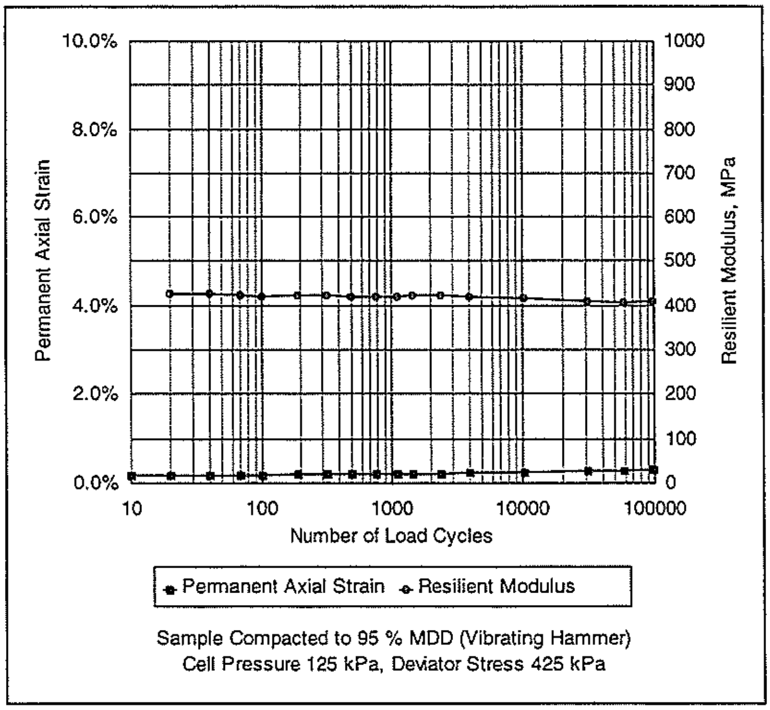

Figure 3 - Results of Drained Permanent Strain Test

in a pavement. The results should be used as an index of aggregate performance to confirm the design assumption that rutting in the unbound granular layers is not the limiting factor in the life of the pavement

Permanent strains in a high quality basecourse or sub-base should be small and they should remain linear with the logarithm of load cycles. In previous tests by Opus on good quality basecourse materials the permanent strain accumulated at rates of less than 0.05 percent per log cycle of loading for up to 100,000 loading cycles (see Figure 3).

Figure 4 shows permanent strain results of an undrained saturated test sample that may be expected to perfrom poorly as a basecourse layer under those conditions. The line showing permanent strain tends to curve upwards and significant levels of strain are recorded by the end of the test. In some tests, the permanent strain will be small for many loading cycles, and only show an upward break in the permanent strain - log cycles curve towards the end of the test. This may indicate that the material may have a limited fatigue life under the stress and drainage conditions of the test.

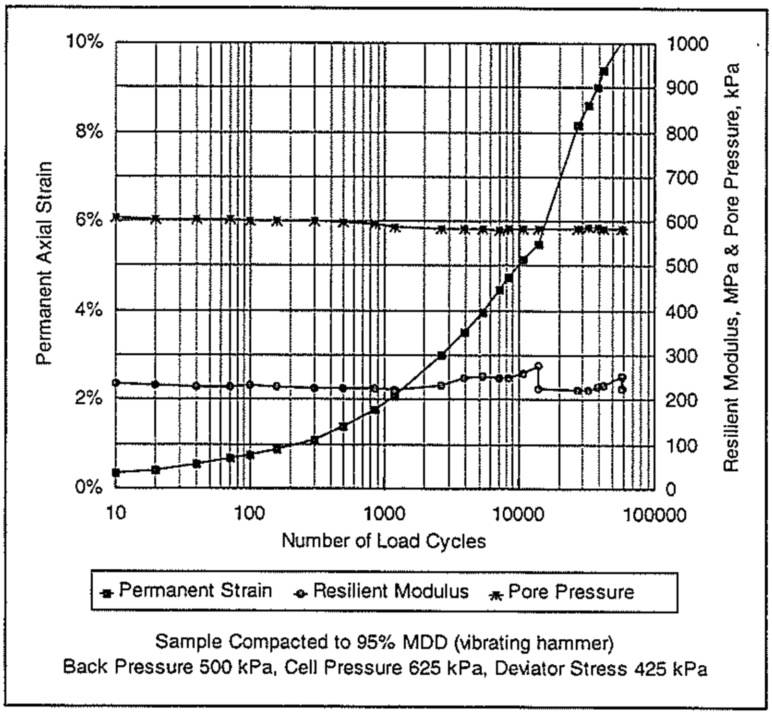

Figure 4- Results of Undrained Permanent Strain Test

9. CONCLUSION

The repeated load triaxial test has proven to be a useful tool to investigate the sensitivity of permanent deformations and of elastic modulus to various material parameters. Other properties that affect the performance of pavement aggregates, such as shear strength, permeability, and weathering potential, should be assessed by other means.

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MINERAL CHEMISTRY AS AN INDICATOR OF STRENGTH IN WEATHERED BASALT

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SUMMARY

The strength of earth materials used in roading technology plays a very important role in design and construction. It is also well known that determination of strength requires large quantities of material and expensive field and laboratory equipment. Mineral chemistry was known to affect the rock and soil strengths. This paper is based on a project attempting to determine precisely and in detail the role mineral chemistry plays in the strength properties of weathered basalt at Karamu, western North Island, New Zealand. Since mineral chemistry can be determined with a 0.03mm thin section and a few grams of powdered material it seems appropriate to use mineral chemistry as an indicator of strength.
The paper discusses relationships between parameters of mineral chemistry and strength of weathered basalt material. Relationships of compressive strength, shear strength, plasticity index, dry density, porosity and hardness with chemical factors are given emphasis and for most of them model equations are derived based on the data. These equations are given in the section headed as.conclusions and they should serve as clearly defined models in estimating strength properties with the available information on mineral chemistry.

1. INTRODUCTION

This project was an attempt to determine precisely and in detail the role mineral chemistry plays in the strength properties of weathered rock materials.
Road construction requires very large quantities of rock and soil materials in the form of aggregates. The strength of earth materials used in roading technology plays an important role in design and construction and it would be advantageous to develop a method to determine them at a low cost.
Since basalt is a common rock type for road aggregates, it was selected for this research. Due to availability of exposed weathering profiles, basalt at Karamu was chosen for the study. Karamu lies 17 km Southwest of Hamilton and the Project Area is enclosed within Longitudes: $175^0 5^1$ and $175^0 7^1$ East & Latitudes: $37^0 54^1$ and $37^0 55^1$ South.

2. GEOLOGY OF THE PROJECT AREA

Karamu is a volcanic centre belonging to the Okete Volcanics which is part of the Alexandra Volcanics Group (AVG). Alexandra is the southern-most Pliocene-Quaternary (2.7 to 1.6 Ma) basalt fields of northern North Island.The Okete volcanics form a basalt field of numerous monogenetic volcanoes that have a scattered distribution around Raglan - Pirongia. The Okete Volcanics have compositions which include basanites, alkali-olivine basalts and hawaiites (Briggs & Goles 1984).

The **Karamu** volcanic centre consists of basaltic lava flows and associated scoria material. The age of the lava flows are estimated at 2.03 Ma. The basaltic lavas are overlain by a thick sequence of weathered rhyolitic tephras ranging in age from 1.6 to 0.9 Ma. Tephras generally form layers which have a uniform thickness. When one such layer has formed ,and before the formation of the next tephra layer, a soil layer has been formed. These soil layers have turned into dark coloured paleosols and now we see a series of alternating tephra and soil layers. Due to deep weathering, the tephras have turned into soil with high contents of clay.

	Fresh Basalt	Slight. Weath.	Moder. Weath.	Highly. Weath.	Compl. Weath.
SiO_2 %	45.92	35.69	31.87	28.22	25.82
Al_2O_3 %	12.43	14.23	22.07	28.24	26.47
MnO %	0.18	0.19	0.32	0.36	0.23
MgO %	11.28	11.45	3.71	0.54	0.34
CaO %	10.18	6.11	0.64	0	0
Na_2O %	1.93	0.19	0	0	0
K_2O %	1.11	0.11	0.09	0	0
TiO_2 %	2.18	3.01	3.94	3.58	4.94
P_2O_5 %	0.38	0.33	0.57	0.83	0.51
Fe_2O_3 %	2.91	3.75	19.11	18.01	23.03
FeO %	9.76	12.54	1.01	0.95	1.21
L.O.I.	2.11	11.49	13.95	16.17	14.82
Zr ppm	202	228	299	355	337
pH	8.71	7.42	5.07	4.33	4.08
Miura W.I.	1	0.56	0.06	0.02	0.01
Parker W.I.	1	0.56	0.06	0.001	0.002
Mod. W.I.	2	1	0.1	0.02	0.01
Dry Density kg/m^2	2902	2492	1845	1787	1502
Porosity%	0.31	1.42	13.34	21.38	27.19
Compressive Strength MPa	298.8	84.94	13.52	4.12	4.38
Cohesion KPa	_	0	1	2	7
Angle of Int. Friction ^{0}	_	35	33	33	33
Undr.Shear Strength KPa	_	183	112.8	146.6	94.8

Table 1. Results of Chemical and Geotechnical Tests

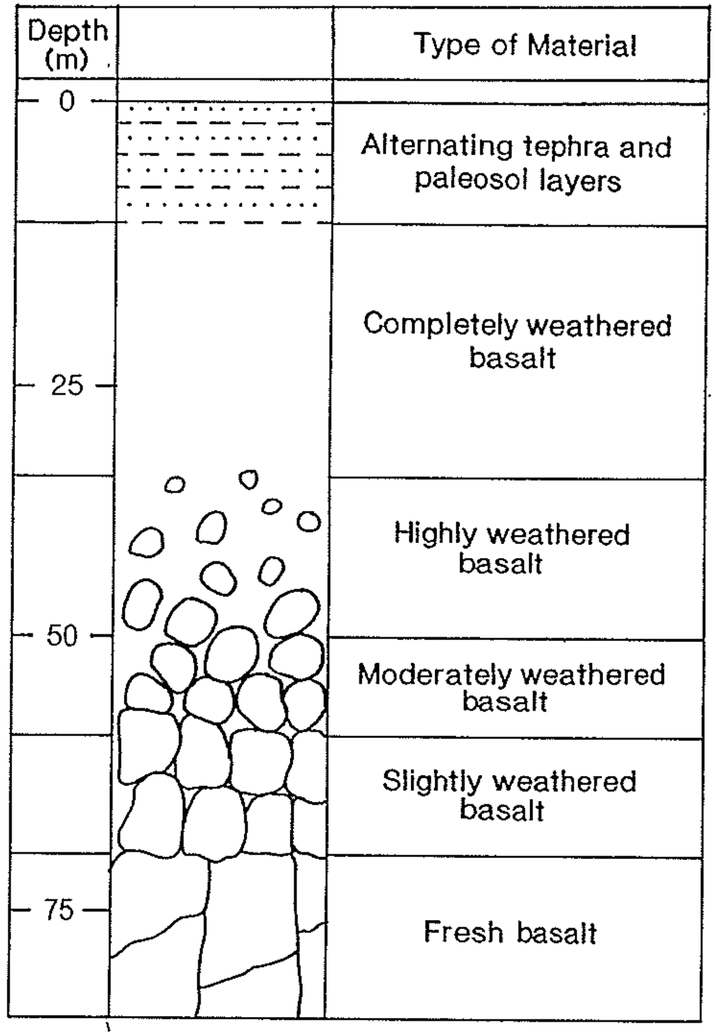


Figure 1: Weathering Profile at Kawerau

3. METHODOLOGY

At an abandoned open-cast basalt mine at Karamu, a complete weathering profile starting from fresh basalt , through slightly weathered, moderately weathered , highly weathered to completely weathered basalt was observed and mapped (see Figure 1).
In situ tests such as Schmidt Rebound Hammer Hardness Determination, Vane Shear Test and Standard Penetration Test were carried out in the field. Samples of each unit in the profile, in very large quantities , were then collected and prepared for further examination. In the laboratories, the following tests were carried out to determine geotechnical parameters: Uniaxial Compressive Strength Determination, Point Load Strength Index Determination, Shore Scleroscope Hardness, California Bearing Ratio, Water Content Determination, Dry Density, Particle Density, Particle Size Distribution, Direct Shear Test, Atterberg Limits and Permeability.
In order to evaluate the mineral chemistry of these materials the following studies were carried out: X-ray Fluorescence for Chemical Analyses, X-ray Diffraction for Mineral Identification, pH Values, Electrical Conductivity, Petrography on Thin and Polished Sections and Scanning Electron Microscopy.

4. OBSERVATIONS AND TEST RESULTS

a. Weathering Profile:
Figure 1 gives a description of the weathering profile . The bottom the profile. consists of black coloured, columnar jointed fresh basalt. Slightly weathered basalt with brown colour at the joints overlies that, followed by moderately weathered basalt with a visible brownish yellow clayey substance in thick layers at the joints which is overlain by highly weathered basalt which still has corestones in visible proportions in a matrix of clayey material. On top of it lies the brown coloured completely weathered basalt where the original structure could be seen but the minerals have altered beyond recognition giving rise to a new set of minerals. Several thin layers of alternating weathered tephras and paleosols are covering these layers in some places.

b. Mineralogy:
Microscopic Studies of the samples show that the fresh basalt consists mainly of the three minerals olivine, augite and labradorite plagioclase. These minerals are found embedded in a groundmass of glass and very fine crystals of plagioclase and magnetite. In the fresh basalts, the minerals are unaltered. In slightly weathered basalt, the mineral olivine starts to develop rims of iddingsite. In moderately weathered basalt , parts of the olivine crystals are removed due to weathering and augite shows evidence of alteration. Plagioclase is completely altered into clays. Clay minerals mainly smectites and in lesser quantities, illite could be detected by the X-Ray diffraction method. Hematite and phlogopite mica are also present as secondary minerals. Highly weathered basalt is a mixture of clay minerals (mainly smectite, illite and kaolinite) derived from plagioclase and iron oxides and hydroxides derived from olivine and augite with remnants of the old rock preserved at places. Mineralogy of the completely weathered basalt is complex with clays, iron oxides and hydroxides predominating and with only traces of olivine, augite or plagioclase left. Throughout the weathering profile, magnetite remains unaltered.

C. Test Results
For each layer with a different degree of weathering, some of the average values of the chemical analyses along with results of the tests Uniaxial compressive Strength, Direct Shear (cohesion and angle of internal friction), Vane Shear, Porosity and Density, are given in table 1. The values of Shore Scleroscope Hardness, thickness of olivine rims measured on thin sections, Electrical Conductivity, Atterberg limits, mineral percentages calculated with X-Ray Difraction Method and Schmidt Rebound Hardness were used in trying to establish relationships even though they are not included in the table.

The loss on ignition values given as percentages in the chemical analysis is the water retained within the mineral structures which is released on ignition at very high temperatures (over 1000 degrees centigrade) and they should not be mistaken for the usual water content which is an assessment of externally retained water in the pores of the rock and soil material. The samples were oven dried before they were analysed for chemical composition.

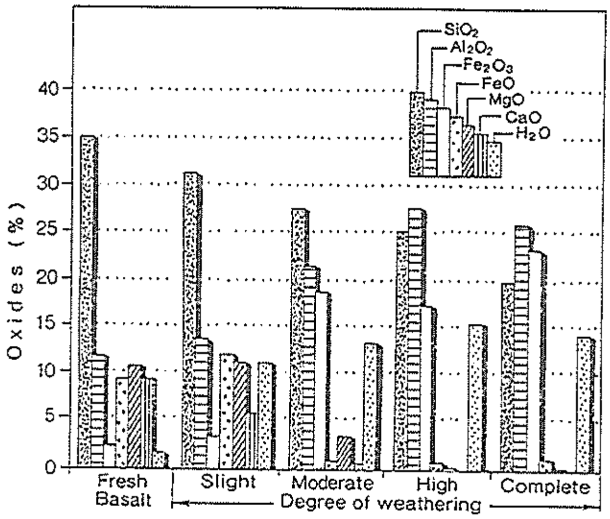


Figure 2: Changes in Chemistry

Figure 3. Plot of Norm. Miura Index vs Depth

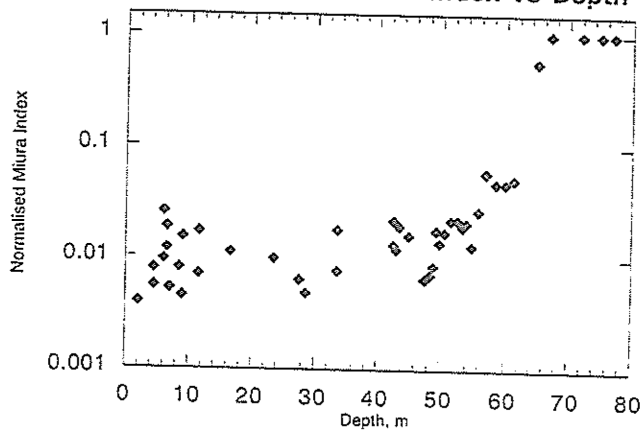


Figure 4. Plot of Compressive Strength vs Modified W.I.

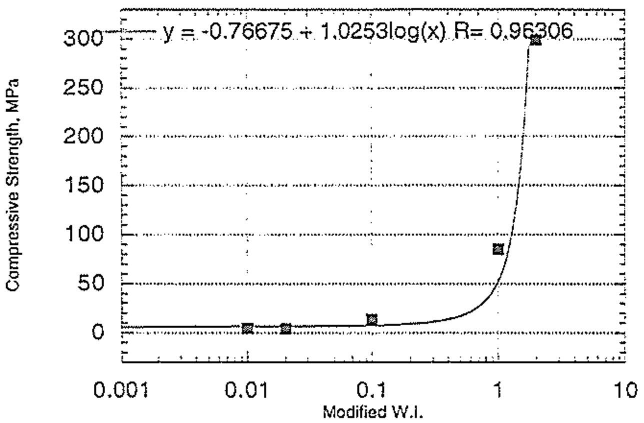


Figure 5. Plot of Cohesion vs Modified W.I.

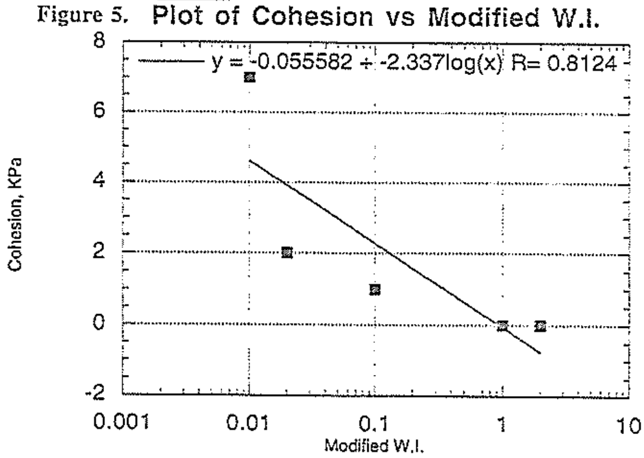


Figure 6. Plot of Dry Density vs Modified W.I.

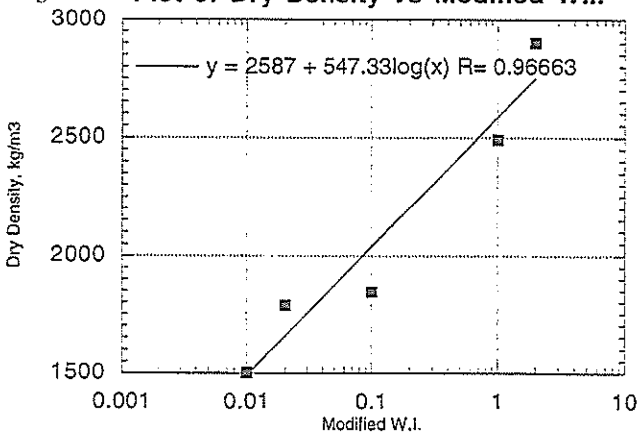
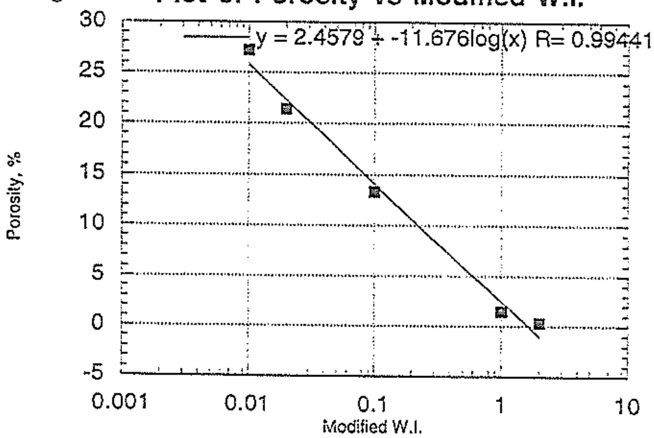


Figure 7. Plot of Porosity vs Modified W.I.



5. ANALYSIS OF RESULTS

Chemical composition of the samples are given as oxides and they show changes from fresh basalt to weathered. The calcium, bivalent iron and magnesium content decrease with the degree of weathering while the aluminium, trivalent iron and structural water content increase. Figure 2 shows this relationship.

Miura Weathering index (Wm) were calculated based on the chemical analysis. The values were normalised taking the values for fresh basalt as 1. The Miura weathering index, in logarithmic scale, show a gradual decrease upon weathering of basalt (see figure 3).

$$Wm = \frac{MnO+FeO+CaO+MgO+Na_2O+K_2O}{Fe_2O_3 + Al_2O_3 + H_2O}$$

Since manganese, sodium and potassium are available in negligible quantities the weathering index could be modified and rewritten as :

$$\text{Modified W.I.} = \frac{FeO+CaO+MgO}{Fe_2O_3 + Al_2O_3 + H_2O}$$

This modified weathering index clearly indicates strength changes. The figures 4-7 shows how Compressive Strength, Cohesion, Dry Density and Porosity change with the modified weathering index.

On basalt samples at diffierent stages of weathering, Schore Scleroscope Hardness were determined and on thin sections at these very spots, thickness of the iddinsite rims on olivine were measured. The figure 8 shows that with increasing thickness of the rim the hardness decreases.

Other comparisons were also made between chemical parameters and geotechnical parameters. Some interesting relationships could be observed between chemical and geotechnical parameters. The figures 9-13 show these relationships. Successful attempts were made to quantify them.

6. CONCLUSIONS

a) The weathering of basalt can be considered as a change in the mineral chemistry with the contents of aluminium, iron, magnesium, calcium and water playing major roles influencing strength.

b) Modified Weathering Index decreases with the intensity of weathering while Compressive Strength and Dry Density decrease and Cohesion & Porosity increase with the decrease in the weathering index.
Compressive Strength, MPa = -0.77 + 1.03 log(W.I.)
Dry Density, kg/m^3 = 2587 + 547.33 log(W.I.)
Porosity , % = 2.46 + (-11.68 log (W.I.))
Cohesion, Kpa = -0.06 + (-2.34 log(W.I.))

c) The degree of alteration of olivine, shown by thickening of the iddinsite rims, is an indicator of surface hardness measured by Shore Scleroscope.

d) The Undrained Shear Strength as well as the pH of the weathered material increase with depth.

e) With increased Zirconium content the Plasticity Index increases and with increasing Rubidium content the Schmidt Rebound Hardness and Unconfined Compressive Strength increase.
Plasticity Index = -18.59 + - 0.1 (Zr content, ppm)
Compressive Strength, Mpa = -25.4 + 7.05 (Rb content, ppm)
Schmidt Rebound Hardness = 18.32 + 0.85 (Rb content, ppm)

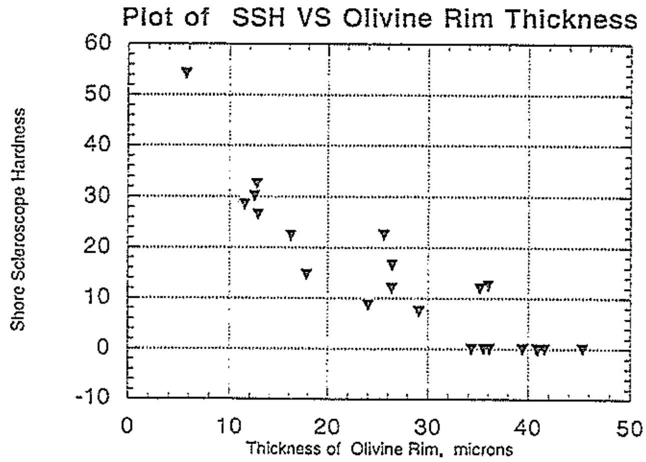


Figure 8

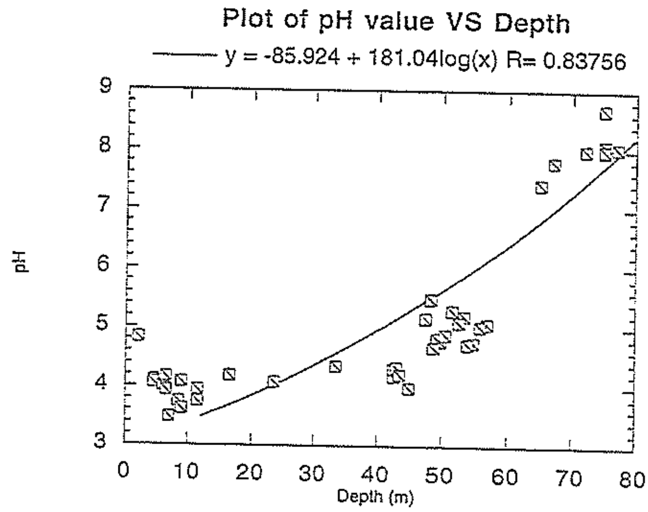


Figure 9

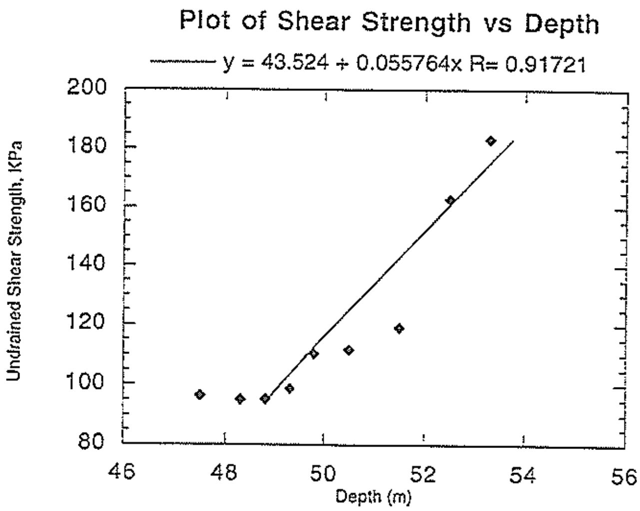


Figure 10

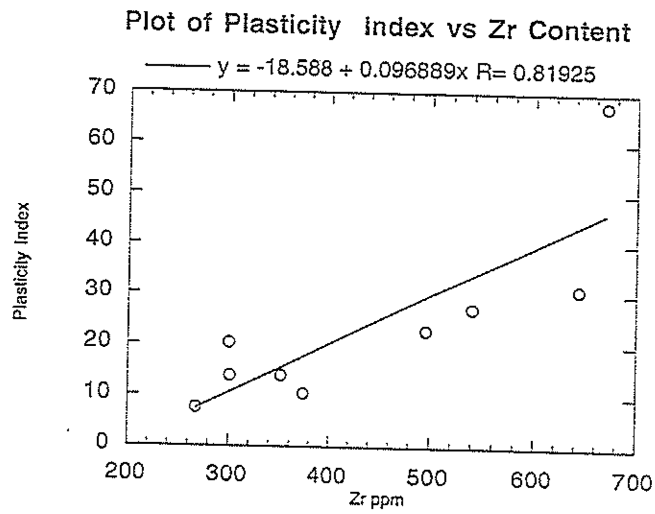


Figure 11

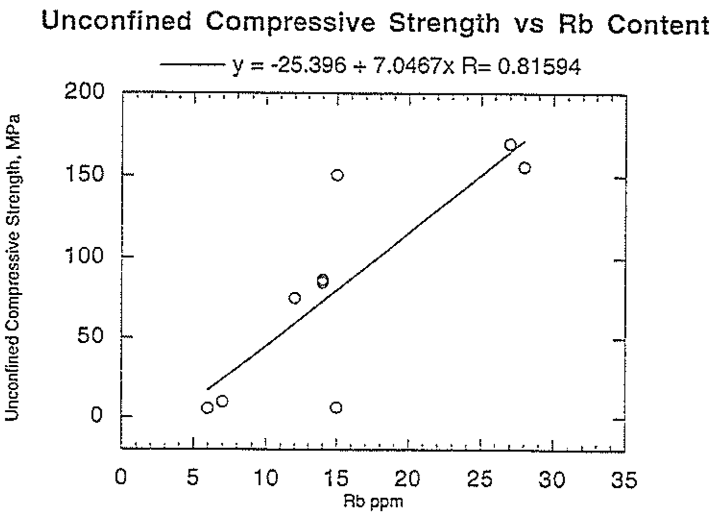


Figure 12

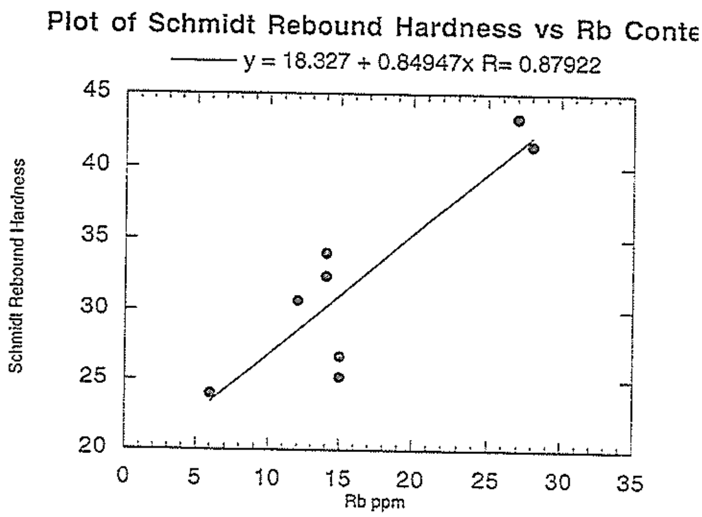


Figure 13

INVESTIGATION OF CRITICAL STRESS AND PERMANENT DEFORMATION OF SUBGRADE BY DYNAMIC TRIAXIAL TESTS AND EFFICIENCY OF GEOCOMPOSITES BY MODEL TESTS

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SUMMARY

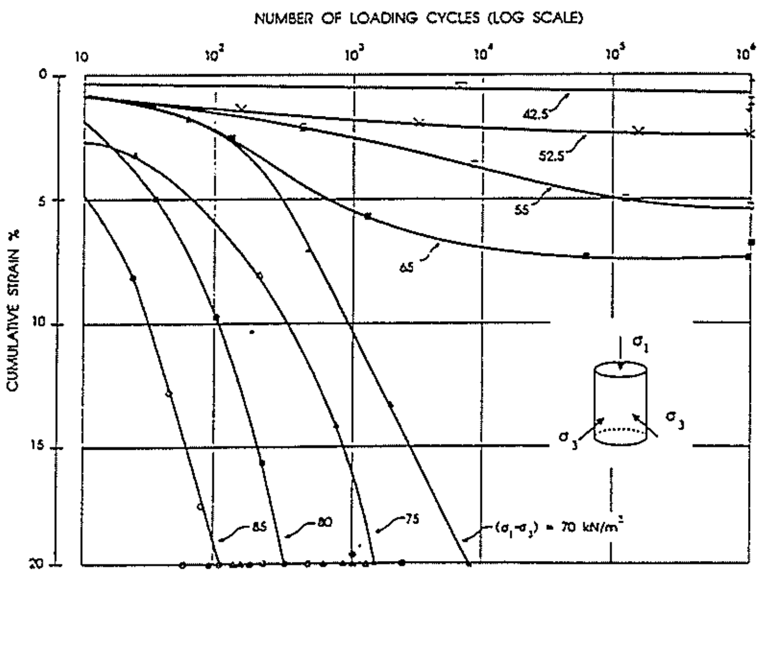
Excessive plastic and repeated elastic deflection in the subgrade often results in cracking of the pavement. Therefore, the critical stress and permanent deformation of soils attracts much attention. In this paper, the effect of loading frequency, confining pressure, loading amplitude, and loading cycles on the critical stress and the permanent deformation were investigated by dynamic triaxial tests. The efficiency of reducing the stress in the subgrade by geocomposites were investigated by model tests. Some conclusions are obtained. Finally, a method of determining construction depth is recommended according to critical stress and axle load.

1. INTRODUCTION

The pavement and subgrade constitute a multi-layer system. The subgrade is a very important part, because some load-associated cracking of pavement results from excessive plastic and repeated elastic deflection in the subgrade. Usually, In the design of pavement system, resilient modulus of the subgrade is an important index, hence it receives special attention. Many researchers have investigated the factors which affect resilient modulus (3). However, permanent deformation has a cumulative effect with loading cycles, therefore, much cracking is caused by permanent deformation. If the pavement and subgrade were a purely elastic system, then permanent deformation would not occur. In practice, permanent deformation is often found. How to reduce permanent deformation in design, namely how to determine minimum construction depth, is a key to assure road quality. Permanent deformation was often investigated using triaxial tests (1), and many important conclusions have been obtained. For example, critical stress , or the threshold stress, is an important concept. Figure 1(5) denotes the concept of the critical stress. In Figure 1, it can be seen that all results are separated into two distinct groups:

1. those in which the deformation is increaseing until complete failure of the specimem is reached.
2. those in which although the deformation is increasing, a stable condition is attained rapidly.

Figure 1. Relationship between Permanent Deformation and Number of Cyclic Loading



The stress difference (σ_1-σ_3) separating these two groups has been designated the threshold stress. If the stress in the subgrade caused by the vehicle load is less than the critical stress or the subgrade has sufficient strength, then the permanent deformation will be very small. After some duration, for example 10^5 loading cycles, the incremental permanent deformation is negligible.

The critical stress is affected by many factors, but the two most significant groups are:

1. soil properties: moisture content, saturation degree, plasticity index, specific gravity, and compacted density; and
2. confining pressure, loading frequency, number of loading cycles, loading amplitude, etc.

In the past, research has been done on the effect of these factors (1,2,6). However, the effect of frequency on the critical stress and some effects of cyclic loading on the permanent deformation still need to be investigated and evaluated. The first part of this paper investigates the effects of the confining pressure and the loading frequency on the critical stress and permanent deformation. Some relationships between critical stress and the factors are obtained. Finally, a design method of minimum construction depth using the critical stress is recommended.

For improving the subgrade state, two methods can taken. One is to increase the strength of subgrade. The other is to reduce the loading amplitude on the subgrade. The second part of this paper uses full scale model tests to investigate the efficiency of geocomposites in reducing loading peak value. The results of the tests also illustrate that permanent deformation is greatly reduced by using geocomposites.

2. DYNAMIC TRIAXIAL TESTING

For investigating the effects of the confining pressure and the loading frequency on critical stress and permanent deformation, dynamic triaxial tests were done.

2.1 Soil Properties, Specimen Preparation and Repeated Triaxial Testing

Its main index parameters of the subgrade soil are: soil classification is CL; natural density is 1840kg/m^3; liquid limit is 45.62%; plastic limit is 17.7%; After standard proctor tests, optimum moisture content is 20.3%; maximum dry density is 1642 kg/m^3. According to the test standard, all specimens were trimmed into 50 mm diameter by 100 mm high.

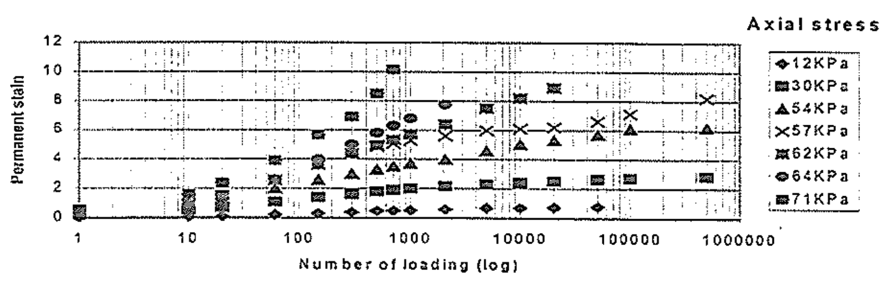
Repeated triaxial testing was conducted by a common triaxial testing method. For investigating the critical stress and permanent deformation, selected cell pressures were 0kPa, 25kPa, 50kPa, 75kPa, and 100kPa; and the selected loading frequencies were 2Hz, 5Hz, 10Hz, and 15Hz. The cyclic load was applied using an electropneumatic loading system, and an internal load cell and internally mounted linear variable differential transducers were used.

3. RESULTS AND EVALUATION ANALYSIS OF DYNAMIC TRIAXIAL TESTS

3.1 Characteristics of Critical Stress and Permanent Deformation under Repeated Loading

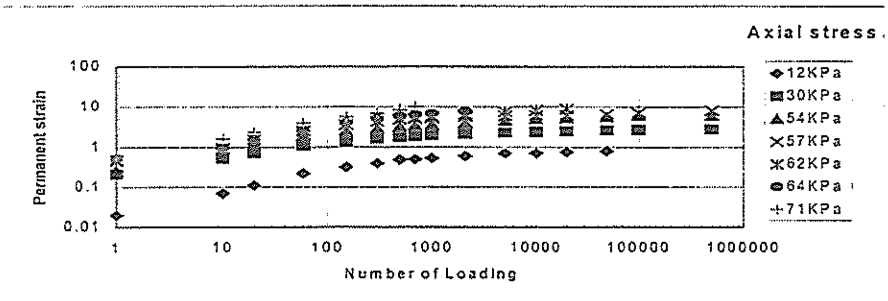
Under repeated loading, the subgrade had not only an elastic deformation, but also a permanent deformation. Compared with the elastic deformation, the permanent deformation is more important, because it may be the cause of cracking in the pavement surface. Therefore, attention should be given to investigating its characteristics. In terms of the experiment and based on experience, the confining pressure of the subgrade in the field is about 14-40 kPa. For a simple case, an unconfined test can be used. Test results are shown in Figure 2. It can be seen that critical stress is about 55kPa - 60kPa.

Figure 2: Relationship between Permanent Deformation and Number of Cycle of Stress



In semilogarithmic coordinates, researchers have shown that the relationship between the number of loading cycles and permanent strain is linear in sand (4). From the clay tests Fig. 3, the linear relationship is valid only for a low number of loading cycles. When the number of loading cycles exceeds 2×10^5, the permanent strain is nearly constant. The slight change is from soil fatigue. This illustrates that the clay has adequate strength to resist the load after the number of loading cycles is greater than 8×10^5.

Figure 3. Relationship between Permanent Strain and Number of Loading in Logarithmatic Coordinates



3.2 Effect of Loading Frequency on Critical Stress

The effect of loading frequency on the critical stress is very important, because road vehicle speed varies greatly. For investigating its effect on the critical stress, four loading frequencies (2Hz, 5Hz, 10Hz, and 15Hz) were selected for the tests. The relationship between critical stress and loading frequency is illustrated in Figure 4 which clearly shows the change of critical stress as loading frequency varies. When the frequency is low, the critical stress is high and as loading frequency increases, the critical stress reduces. When the loading frequency is greater than 15Hz, the critical stress changes only slightly.

Figure 4: Relationship Between Critical Stress and Loading Frequency

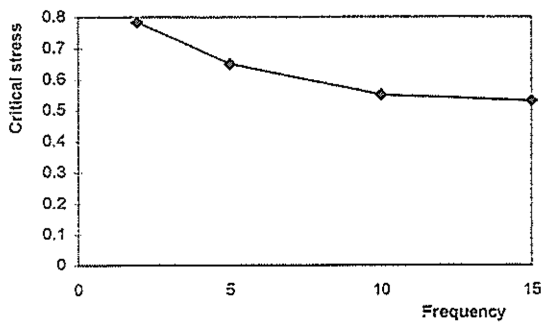
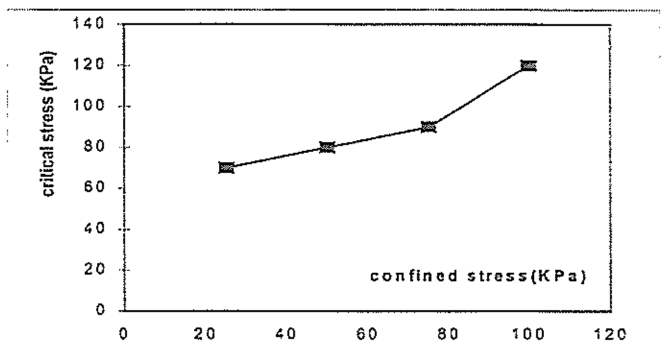


Figure 5: Relationship between Confining Pressure and Critical Stress



3.3 Effect of Confining Pressure on Critical Stress

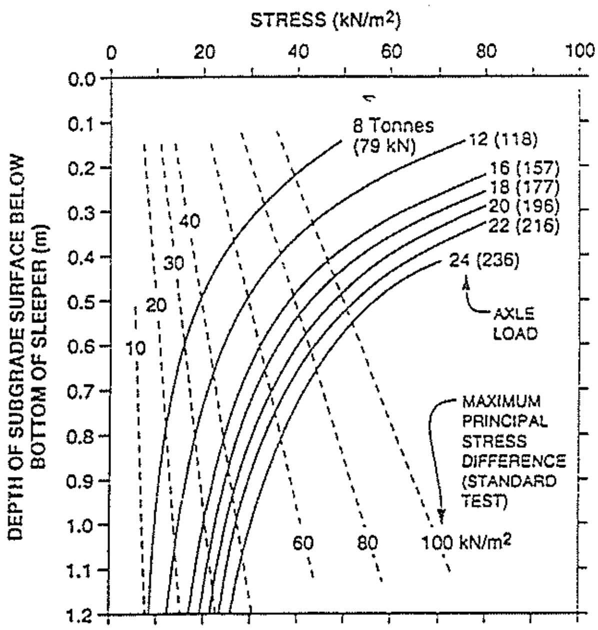
The effect of the confining pressure on the critical stress was also investigated. According to the four different confining pressure tests, four critical stresses were obtained. As the confining pressure increases, the critical stress increases. According to Figure 5, they have nearly a linear relationship which conforms to the relationship between dynamic strength and confining pressure.

The relationship is important for the subgrade, because as the depth of the subgrade increases, the critical stress increases. Therefore, the surface of the subgrade is the weakest part which is easily compressed, because confining pressure in the surface is the lowest. If the deformation state is to be improved, the density of surface of subgrade should be increased.

3.4 Minimum Construction Depth Determined by Critical Stress

After the critical stress for a specific soil type is obtained and design axle load is determined, minimum construction depth can be calculated. Selig and Waters(5) give a detailed procedure for railroad subgrade design. Figure 6 shows the basic method. For example, if the construction depth is expected to be obtained under an axle load of 16 tons, and a critical stress of 60kN/m^2, then the required construction depth is 600mm.

Figure 6. Variation of Subgrade Vertical Stress and Threshold Stress with Depth



4. FULL SCALE MODEL TESTS

Originally, the full scale model tests were designed to simulate the behaviour of a railway embankment under dynamic loads. The characteristics of the permanent deformation and stress distribution in the tests is similar to those of a road, the difference is only in the amplitudes of deformation and stress. The reason for using the tests is to explain the efficiencies of using geocomposites and how to change the components of embankment system to increase the critical stress and improve stability of the embankment.

In the tests, the four thicknesses of the subballast layer (similar to a subbasecourse) were selected as four kinds of model tests. They were 0mm, 100mm, 200mm and 300mm, respectively. Geocomposites (two layers of non-woven geotextile and a layer of geomembrane) were laid on the top of subballast. Its weight is 600g/m^2 and thickness is about 2mm. Subballast and geocomposites consisted of a complex layer.

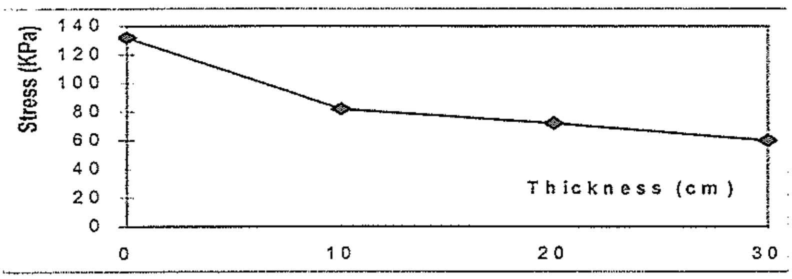
The model test was designed using a 1:1 scale. Because of the symmetry of the embankment, only a half section of the embankment was used. Displacement gauges, earth pressure cells and acceleration gauges were setup along the depth and the width. All tests were done by a servo-hydraulic system. Data were read by computer system.

5. RESULTS OF MODEL TESTS

5.1 Stress at Interface Between Complex Layer and Subgrade

The stress in the subgrade is very important in designing the subgrade. For comparing the efficiencies of a complex layer in reducing the stress in subgrade, four tests were done. Figure 7 shows the relationship between the maximum stresses on the surface of the subgrade. It illustrates that the complex layer reduces stress, but the efficiency will gradually reduce as the thickness of the complex layer increases. Therefore, the optimum thickness of the complex layer is very important in the embankment design.

Figure 7. Relation between maximum stresses on the surface of subgrade and thickness of complex layer



5.2 Distributed Stresses Along the Depth

The distribution of stresses along the depth of embankment will cause the settlement of the embankment. The complex layer reduces the stress at the interface between the complex layer and the subgrade (figure 7) and also affects the distribution of stresses. Figure 8 shows the results of model 1 and 2. On the top of the subgrade, the stresses in the model 1 are greater than the ones in the model 2. However, in the low part of the subgrade, the stresses in model 1 are smaller than the ones in model 2. This result denotes that although the complex layer reduces the stresses at the interface, the stress attenuation along the depth of the embankment in model 2 is slower than that in model 1. This should be given attention in soft ground, because it may be cause reconsolidation.
The effect of geocomposites on the embankment are very interesting. Figure 9 is the result in which stresses were measured above and below geocomposites. It efficiently reduces stresses below geocomposites and changes the value of the distribution of stresses along the depth of embankment. At the 100mm range above and below geocomposites, the stresses are reduced about 30%. These results show that a complex layer or geocomposites can improve the state of an embankment system.

Figure 8: Stress Distribution in model 1 and 2

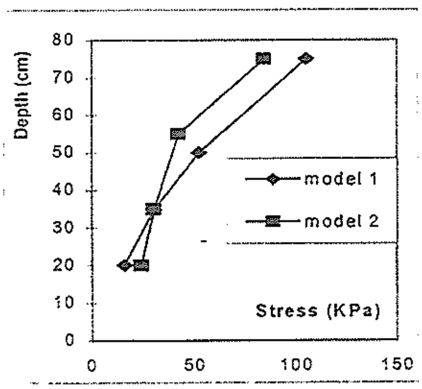
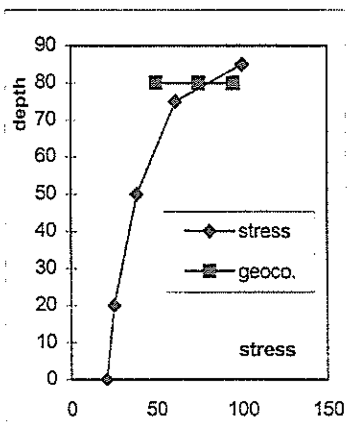


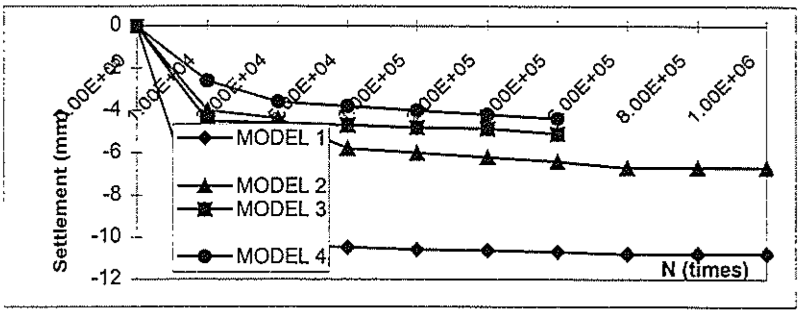
Figure 9: Effect of geocomposites on stress



3.4 Settlement of Subgrade

The settlement of the surface of the subgrade will affect the stability of the embankment and will increase the cost of maintenance in the future. Figure 10 shows the results of settlement in the four model tests. These results denote that initial settlement is the most part of total settlement, For example, initial settlement is 93% and 55% of the total settlement in the model 1 and 2, respectively. These results illustrate that before constructing pavement, completing initial settlement is a key to reduce settlement in the future. If the settlements in various models are compared with the settlement of model 1, the percent at model 2, 3, 4 is 37%, 52% and 59%. Because the complex layer is on the surface of the subgrade, it is seen that the settlement of subgrade is reduced greatly

Figure 10. Surface settlement of embankment



4. CONCLUSIONS

The critical stress and the permanent deformation were investigated using dynamic triaxial tests. The efficiencies of using geocomposites were investigated by full scale model tests. The following conclusions can be drawn from the study:

1. The curves of permanent deformation are separated into two groups. One is stable. The other is unstable. The critical stress is an parameter separating these cases.
2. The loading frequency has effect on the permanent deformation and critical stress. As the loading frequency increases, the critical stress decreases gradually. However, above a these hold loading frequency, its effect is negligible on the critical stress.
3. The confining pressure affects the permanent deformation and critical stress. As the confining pressure increases, the permanent deformation decreases, but the critical stress increases.
4. A method of determining minimum construction depth is recommended according to a railroad design method using the critical stress. The method may be useful in embankment design.
5.Geocomposites can effectively reduce the peak of stress on the surface of subgrade. In the tests, the reduction of stress as high as 30%.
6.The initial settlement provides most of the total deformation. It is best that before constructing the pavement, taking some measures to completes initial settlement.
7.The state of the embankment system is a key factor in roading design. For example, according to the test results, changing the thickness of complex layer will improve state of stresses in the subgrade. This may then affect pavement design.

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SCOPING GEOTECHNICAL INVESTIGATIONS FOR ROADING PROJECTS

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Kingston Morrison

Summary
What constitutes an adequate geotechnical investigation? How do we identify best investigation practise and scope the investigation works for real projects? The underlying theme of most documents and reviews on ground investigation methods suggests that unnecessary expense is incurred on capital projects due to inadequate investigations.

A study has been undertaken of geotechnical investigations for some typical roading projects in New Zealand, Australia, the UK and Fiji in order to compare and contrast the type and styles of investigation favoured. Some very interesting comparisons can be drawn on the type of fieldwork executed, the approach to laboratory testing and the relative costs.

Introduction
Over many years there have been numerous learned publications, standards and codes of practice which have advocated a structured approach to ground investigations for civil engineering projects. There is therefore no need to repeat, reiterate or reinvent what every geotechnical engineer knows to be the case; a "good" investigation comprises desk study, field reconnaissance, intrusive investigation, in situ testing, sampling, laboratory analysis and interpretation.

It is widely recognised that a phased approach to site investigations can be of substantial benefit. Studies in Scotland however suggest that phased ground investigations are rarely undertaken (Ref: 1).

The purpose of this paper is to consider the planning and scoping of geotechnical investigations for roading projects. A comparison has been made of actual investigations undertaken for a variety of roading projects in New Zealand, Australia, the UK and Fiji. Included are some observations on methodologies and relative costs from which conclusions are drawn about the "accepted" investigation philosophy in NZ. Recommendations are made on a means of improving current work practises.

Planning Investigations
In major roading projects, the project organisation can be commonly described by the flow chart on Figure 1 (Ref: 2). The areas of responsibility and the activities of each group are laid out on an approximate time line from left to right. Whilst special circumstances occasionally dictate some modification of this model the basic concept is of fundamental importance to the successful completion of any project.

Of the fourteen case studies included within this review only one relates to Phase 3 investigations and only the very large NZ projects appear to have included a Phase 1 investigation. The conclusion drawn is that most effort is still concentrated on undertaking a "comprehensive" investigation at Phase 2. There is an underlying trend or "mind-set" which dictates that the investigation specialist will only be allowed one hit at doing his job. The geotechnical engineer is expected to get all the information he needs in one go and he has to get it right first time. Numerous examples of this philosophy failing are found in arbitration hearings and claims courts around the world.

The Resource Management Act has probably had an impact on this mind-set, particularly on the larger roading projects in NZ, where the process of applying for a route designation, resource, building and earthworks consents will result in a number of phased reviews of the geology and geotechnical conditions. Unfortunately, each phase will be undertaken at minimal cost and may even be undertaken by different professionals so a consistent and rigorous approach is not guaranteed.

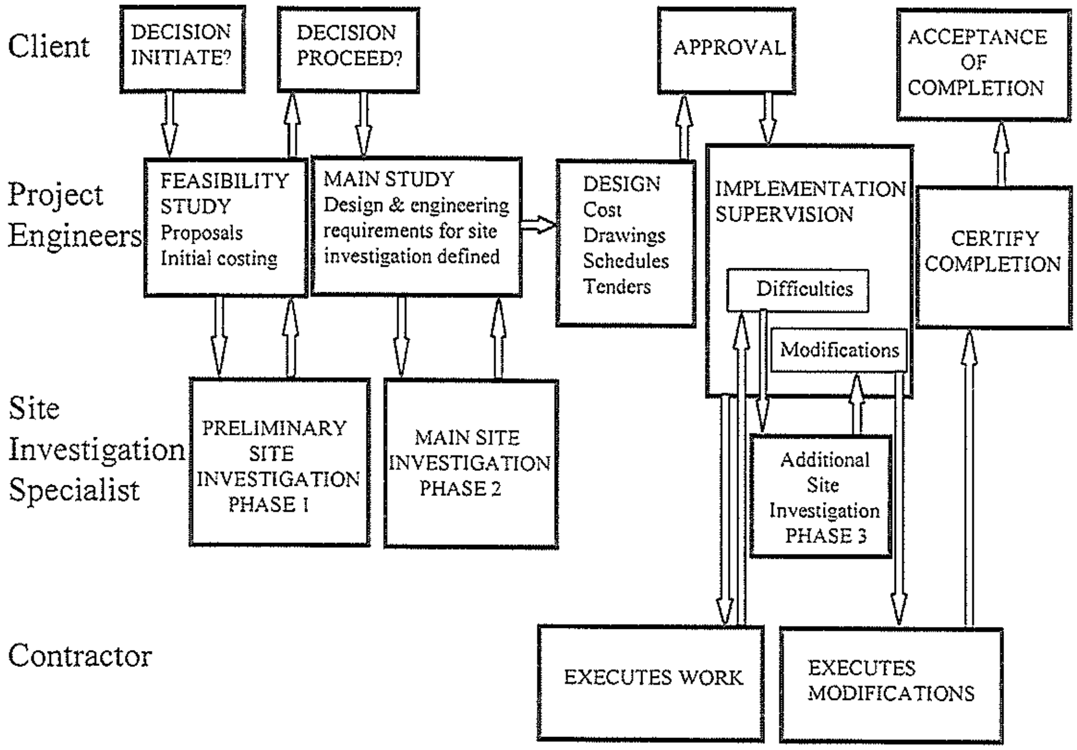

Figure 1

Investigation Investment

There have been a number of studies and reports on what constitutes an adequate investigation and what expenditure should be anticipated. In 1990 the Scottish Office reported that between 1971-75 on Scottish trunk road contracts less than 0.5% of the tender sum was spent on ground investigations (Ref: 3). Based on a survey of 86 contracts this figure effectively doubled between 1977 and 1988 to 0.9% of the tender sum. At that time the Scottish Office were anticipating a further increase of this ratio to 2.0% although they were not convinced that the extra expenditure would have any impact on final out-turn costs.

The interpretation and prediction by the Scottish Office is in direct conflict with the findings of IL Whyte in his paper on the financial benefits of a site investigation (Ref: 4). Evidence from tunnel and highway schemes in the USA and UK is reported which suggests that the level of uncertainty on out-turn costs decreases rapidly with increased investment in site investigation.

In 1993 the Site Investigation Steering Group (SISG) of the Institution of Civil Engineers issued a series of publications on site investigation in construction under the banner, or catch phrase, "You pay for a site investigation whether you do one or not." (Ref: 5). These documents reflect the widely accepted view that the largest element of technical or financial risk in a civil engineering project normally lies in the ground.

In identifying the scale of the problem the SISG reported the findings of the National Economic Development Office, the National Audit Office, the National House Building Council and the Public Accounts Committee. Their findings all demonstrate the financial implications of inadequate ground investigations.

In 1983 Tyrell reported (Ref: 6) that the average cost increase on ten major road contracts was 35% over the tendered sum and half of this was the result of poor planning, execution or interpretation of geotechnical investigations. These figures are similar to those quoted by the Scottish Development Department in their review of road projects (Ref: 3) where 35% of the additional costs were related to problems in the ground.

Investigation Satisfaction
Whyte (Ref: 4) also reports the results of another survey (Ref: 7) which suggests that 80% of geotechnical engineers in the UK were not satisfied with the general standards of ground investigation in construction. This high level of dissatisfaction with the quality of site investigations is blamed on clients for their reluctance to invest in geotechnical expertise up front or continually accepting the lowest tender for a ground investigation without checking the quality of service or product offered.

In New Zealand the same argument does not hold water. Unlike the UK, there are no specialist ground investigation contractors acting directly on a client's brief. The blame, if there is any, for inadequate investigations in New Zealand lies squarely at the door of the professional geotechnical engineers.

"There is hardly anything in this world that some man cannot sell a little cheaper and make a little worse. Those who consider price only are this man's lawful prey" John Ruskin, *Sesame and Lillies*, (quoted from Ref: 4).

Investigation Objectives
The British Standard Code of Practice BS5930 Site Investigations describes the following objectives for ground investigations.

- To assess the general suitability of the site for the proposed works.
- To enable an adequate and economic design to be prepared.
- To foresee and provide against difficulties that may arise during construction.
- To investigate the occurrence or cause of natural or created changes to ground conditions and the implication or impact on the proposed works.

These would appear to be fairly straightforward and simple objectives and it is therefore surprising how often failure to meet the second and third objectives results in cost or program overruns. It has often been stated that site investigation is a low cost activity and it would be prudent to simply increase expenditure.

Whyte (Ref: 4) suggested that an indicator of adequate investment on a ground investigation would be to identify the cost of a one week delay during the height of construction activity on the project. However, based on the review of investigations in this paper, the author is of the opinion that there is no means of judging the adequacy of a particular level of investment on any individual ground investigation. Throwing more money at ground investigations is not a solution, if the investigations themselves are poorly planned, directed, executed or interpreted.

Investigation Comparisons
A simple review of investigations has been undertaken for a variety of roading projects in New Zealand, Australia, the UK and Fiji. The purpose of the review was to identify any significant differences in the scope of works (Table 1) and level of investment (Table 2) for typical ground investigations in the different countries.

Fieldwork
Data was obtained on fourteen projects varying in capital value from $2.5M to $160M (all costs have been converted to New Zealand Dollars). Due to the variety of physical works proposed for each of the projects it is difficult to draw any significant conclusions on the differing approaches to the execution of fieldwork during the investigations. It is perhaps worth noting that it is only the investigations undertaken in NZ which appear to place a degree of dependence on qualitative investigation tools like the Scala Penetrometer.

A general rule of thumb which has been suggested by folklore rather than any learned publication or code of practice would have us believe that a borehole (or intrusive investigation location) every 100m of carriageway is generally adequate for a roading project. Remarkably, of the ten projects reported which involve a significant length of new or upgraded carriageway the average distribution of investigation locations is one every 110m.

Budgets

There is little to separate the investigations in terms of overall investment in ground investigation work relative to the construction costs (Range 0.1-1.7%, mean 0.9%). The review therefore confirms the approximate ratios reported in previous publications (Ref: 5).

Although the sample size from the UK is small the relatively low ratios reflect the fact that the SI costs reported are for the ground investigation contractors costs and do not include the professional fees, or Interpretative Reporting costs, associated with investigations in NZ. It should also be noted that Project 1 from the UK was a Phase 3 investigation as described in Figure 1 and therefore direct comparison of costs in this case are not relevant.

Laboratory Testing

Perhaps the most striking differences in the investigations reviewed are that the Australian examples include a significant amount of instrumentation and very little soil testing. NZ investigations show a similar trend in low investment on laboratory work. Further enquiries on this issue in Australia were met with the response that soil testing was of limited value and the results were seldom used in practice, "....we used to do a lot of testing but no one looked at the results".

There is a significantly different attitude towards laboratory testing in the UK where another common rule of thumb applied to ground investigation tender submissions on the part of cost estimators is to allow approximately 1/3 each for fieldwork, laboratory testing and reporting. This is clearly not applicable in New Zealand where laboratory testing costs rarely exceed 10% of the ground investigation total and even less in Australia.

There are three possible reasons for this difference in attitude which are described below in commercial, practical and technical arguments.

Laboratory Costs

Table 3 presents a cost comparison of routine laboratory soil tests which would normally be expected to be undertaken as part of a basic investigation for a roading project. No adjustment has been made for inflation in the prices of UK laboratory testing between 1987 and 1997 and the same exchange rate has been applied. In real terms the price of laboratory work in the UK has therefore reduced significantly over the last ten years. It could be concluded that NZ clients are paying extremely inflated prices.

Based on these cost ratios it may be understandable for a geotechnical engineer in NZ, coached in the inherent uncertainty of lab' test results and their value, to arrive at the conclusion that testing soil samples is simply too expensive. If, in order to get a representative range of classification test results for the soil types across a site it is going to push the cost of the investigation through the roof, why bother? This is the commercial argument for dropping laboratory testing.

Sampling

In order to undertake a rigorous laboratory testing programme demands the availability of a representative number of quality samples. With the drilling techniques that are commonly adopted in NZ it is often impractical to recover an undisturbed sample since the borehole diameter is too small to accept a sampling tube of adequate size to eliminate, or at least reduce, sample disturbance.

Whilst this is defined as the practical argument for dropping laboratory testing there is an element of commercialism inherent in this aspect. It is time consuming to interrupt the drilling process to recover an undisturbed sample. To the authors knowledge, NZ is one of the few remaining countries in the developed world where geotechnical drilling contractors are remunerated on a time related basis rather than productivity. Significant savings can be made if time on site is reduced by dropping any demands for precise sampling. If no quality samples are recovered there is no point in undertaking expensive soil testing.

Laboratory Testing Validity

This years Rankine Lecture by Professor David Hight may provide a clue to an alternative and more sophisticated reason why New Zealand and Australian geotechnical engineers spend less money on laboratory testing in comparative terms to our colleagues in the UK. In the lecture, Professor Hight touched on the subject of laboratory testing and its place in geotechnical design. The points made were (Ref: 8):-

- Where calculation methods rely on the results of laboratory tests, the right answer is obtained by a series of compensating factors. For example ignoring the effects of anisotropy and testing damaged samples in their strongest/stiffest direction (compression).

- With semi-empirical calculation methods, it makes no sense improving the quality of sampling without also changing the laboratory analysis technique to better reflect the stress state experienced by the soil in situ or you risk upsetting the balance that exists in the evolved design practise.

- The development of sophisticated numerical analysis has demanded parallel developments in sampling and laboratory testing so that all the key features of soil behaviour are modelled.

It could be argued that Australian and New Zealand Geotechnical Engineers appreciate that semi-empirical calculation methods derived from sampling, testing and construction/design experience in soils of fundamentally different geological formations is non-sensical. Therefore correlations and interpretations of characteristic engineering properties for NZ soils using laboratory testing methods adopted from elsewhere are not valid. This is the technical argument for dropping laboratory testing.

Observation Techniques/Empiricism

Unfortunately, it would appear that in NZ whilst we have accepted that the laboratory testing methods available are not appropriate to support the design tools at our disposal we still adopt the same constitutive soil models and empiricism in our designs. For example, the Building Industry Authority Revision of B1/VM4 (Ref: 9) makes extensive use of foundation/pile design criteria and earth retention design methodologies based on the traditional semi-empirical design experience reported in standard soil mechanics text books. The geo-mechanical fabric of the pumiceous deposits and residual soils of the Auckland Region rarely exhibit similar characteristics to the over-consolidated soils of London and the Thames Valley.

Thankfully, design and construction experience has shown that these methods "normally" work. Where construction failures occur it is usually as a result of an unforeseen condition rather than the selection of the wrong soil strength in a basic soil mechanics analysis. This leads to rhetorical questions beyond the scope of this paper. If there is some inherent conservatism in the design techniques, how much? And, are the failures the result of an over dependence on qualitative investigations rather than more rigorous quantitative tools?

Based on the review of ground investigation works included in this paper we could speculate that the technical argument described above maybe be the reason why our Australian colleagues stopped looking at laboratory test results and started to invest in field trials and instrumentation. By using accurate observational techniques they will validate their design assumptions and geotechnical design tools in their unique geological and climatic conditions.

Having verified, or amended as necessary, the constitutive soil models and empirical design tools to suit the regional circumstances it will then be possible to revisit the sampling and testing standards of the geotechnical investigation industry to identify the critical index properties, their means of measurement and how to apply them to the design process.

Conclusions

- There remains a tendency for effort and investment on geotechnical investigations to be concentrated within Phase 2 of the project model. Greater investment and a professional approach to Phase 1 investigations coupled with an acknowledgement that the need for a Phase 3 investigation is not always a professional failure is necessary.

- It is recognised that a poorly planned, managed or executed investigation will result in additional costs during construction. Unlike the UK, in NZ, the responsibility for such circumstances lies with the consulting engineers that manage the investigation process and not the client or his ground investigation contractor.

- Reliance on qualitative tools like the Scala Penetrometer is unique to NZ

- Rules of thumb regarding investigation intensity on roading projects appear to be valid as do the reported average ratios of ground investigations to construction costs.

- NZ prices for laboratory testing are artificially inflated and a lack of soil testing is being driven by commercial rather than practical or technical reasons.

- NZ geotechnical engineers are not compensating for a lack of understanding in the local soil properties by undertaking rigorous field trials or installing instrumentation. There is a demand for the publication of case studies and the back analysis of geotechnical engineering projects to validate the semi-empirical design tools adopted in the industry.

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Table 1

Project	Project Description	Geotechnical Issues	Ground Investigation Scope
1	7.3km of dual two lane motorway with associated slip roads and 16 structures	20m deep rock cuts, structural alterations to a bridge over an alluvial flood plain and 8m high approach embankments	43 Machine boreholes 31 Trial Pits
2	Upgrade of a trunk road including the construction of 3 new slip roads, underpinning an existing bridge and road widening.	Earthworks, piling and low subgrade capacity.	13 Machine boreholes 48 Trial Pits
3	2.2km of road realignment and widening	15m high fill construction and retaining walls	20 Trial Pits 14 Post holes & Scala's
4	10km of seal extension on an existing alignment.	Subgrade strength	45 Trial Pits 135 Scala's & Benkelman Beam Tests
5	11km of pavement rehabilitation	Subgrade strength	46 Trial Pits & Scala's Benkelman Beam Tests
6	Motorway interchange with two sets of on/off ramps and two overbridges.	Earthworks, retaining walls, abutment design and foundation strength	21 Machine boreholes 27 Trial pits
7	6.5km of dual carriageway with 2 bridge crossings and an interchange.	$1Mm^3$ of earthworks, slope stability & foundation design.	26 Machine boreholes 20 CPT's 28 Trial Pits & Scala's
8	15km of dual carriageway with 7 overbridges and 3 interchanges	$2Mm^3$ of earthworks, slope stability & foundation design.	69 Machine boreholes 94 CPT's 110Trial Pits 263 Scala's/Handaugers
9	Two lane bridge crossing a river estuary with safety shoulders and footpaths adjacent to an existing crossing.	Major earth retention structures to support cut slopes.	10 Machine boreholes & pressuremeter tests. Inclinometer installation 9 Scala's & Benkelman Beam Tests
10	Remedial stabilisation works at six sites over 8km of highway through mountainous terrain.	Earthworks, anchored sheet pile retaining walls and reinforced earth embankments	13 Machine boreholes 6 CPT's
11	14km of dual carriageway bypass including 2 major bridges and 5km of embankment on soft ground	Ground settlement, foundation design.	80 Machine boreholes 6 CPT's 30 Trial Pits Trial embankment and instrumentation.
12	12km of dual carriageway realignment including 1 major bridge and 2km of embankment on soft ground.	Ground settlement, foundation design.	45 Machine boreholes 65 Trial Pits 20 CPT's Trial embankment and instrumentation.
13	23km of dual carriageway in mountainous terrain.	Earthworks, cut and fill stability.	70 Machine boreholes 70 Trial Pits
14	7km of dual carriageway by-pass with two swamp crossings and minor bridges.	Earthworks, subgrade strength and ground settlements	30 Machine boreholes 28 Trial Pits Instrumentation

Table 2

Project	Location	Estimated Value	SI Cost	SI Cost as % of Est. Value	Laboratory Testing Cost	Lab. Costs as % of SI
1	UK	$114M	$129K	0.1%	$43K*	30%
2	UK	$14.3M	$99K	0.7%	$30K*	30%
3	NZ	$2.5M	$37K	1.5%	$1.0K	2.5%
4	NZ	$5.5M	$50K	0.9%	$1.0K	2.0%
5	NZ	$3.5M	$60K	1.7%	$1.0K	1.7%
6	NZ	$17M	$130K	0.8%	$15K	11.5%
7	NZ	$35M	$146K	0.4%	$6.2K	4.2%
8	NZ	$63M	$670K	1.1%	$127K	19%
9	NZ	$6.0M	$68.5K	1.1%	$1.0K	1.5%
10	FIJI	$2.5M	$37K	1.5%	$15K	39%
11	AUS	$135M	$835K	0.6%	$6.0K	0.7%
12	AUS	$105M	$775K	0.7%	$7.0K	0.9%
13	AUS	$160M	$1700K	1.1%	$2.8K	0.2%
14	AUS	$50M	$390K	0.8%	$3.9K	0.1%

* Estimated costs only, unable to be identified separately in data.

Table 3

Test Procedure	UK 1997	UK 1987	NZ 1997 (Range)	Ratio NZ/UK	UK 97/87
Natural Moisture Content	$5	$8	$10-30	2.0 - 6.0	0.6
Plasticity Indices	$46	$52	$130-230	2.8 - 5.0	0.9
Particle Size Analysis	$55	$46	$130-150	2.4 - 2.7	1.2
UU Triaxial	$61	$61	$150-250	2.5 - 4.1	1.0
CU Triaxial + PWP	$600	$725	$1100-1300	1.8 - 2.2	0.8
Consolidation	$121	$242	$425-500	3.5 - 4.1	0.5
CBR	$91	$70	$90-130	1.0 - 1.4	1.3
Standard Compaction	$152	$136	$360-380	2.4 - 2.5	1.1

MINE SUBSIDENCE HAZARD ASSESSMENT FOR FAIRFIELD BYPASS, SH1

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SUMMARY

A proposed motorway bypass of Fairfield township, crosses an area of abandoned underground coal mines with a record of crownhole subsidence and potential for future ground subsidence. An extensive literature search and drilling investigations defined subsidence mitigation options and the conditon of the workings. The subsidence hazard mitigation strategy proposed involves locating and treating mine shaft and adit entrances within the bypass corridor and incorporating a geogrid reinforcement system within the carriageway to minimise pavement deflections. Geogrid will be used where the depth to the mine workings is less than 10 times the height of the workings, corresponding to where most crown-hole subsidence has occurred in the past at Fairfield.

1. INTRODUCTION

A bypass of Fairfield township, 10 km south of Dunedin, has been planned since the 1960's. Preliminary investigations were carried out in the 1960's and early 1970's with Transit New Zealand initiating further design investigations in early 1995. The proposed bypass runs east - west joining the existing Green Island and Saddle Hill - East Taieri sections of the Southern motorway, a distance of 4 km (Figure 1). A 0.9 km long section of the proposed route is located over abandoned underground coal workings, including an overpass at Old Brighton Road. Much of this section will be in cuttings (up to 8 m deep), apart from the portion to the east of chainage 2970 which will be on a fill embankment (Figures 2 and 3).

The Institute of Geological & Nuclear Sciences Ltd (GNS) was engaged by Duffill Watts & King Ltd, the project design engineers appointed by Transit NZ, to carry out an assessment of subsidence hazards and mitigation strategies for the coal mine section of the proposed route. This paper is based largely on reports prepared by GNS (Stewart, 1995 and Stewart, 1996). Subsequent to these reports Transit NZ has further assessed the risk of mine subsidence and the value of conducting further investigations - aspects which are not discussed in this paper. The opinions and views expressed in this paper are those of GNS and not necessarily those of Transit NZ.

2. INVESTIGATIONS

(i) Desk study phase

An extensive literature review of mine subsidence hazard assessment, mitigation strategies and case histories of road construction practices in areas of underground mines was conducted during mid 1995. A review of geological/geotechnical work relating to the proposed route was carried out, the main studies being a report on the coalfield by Mutch (1982) and investigations conducted by the Ministry of Works for the bypass relating to an overpass at Old Brighton Road (MOW, 1973a) and to the coal mines (MOW, 1973b). All available mine plans, annual Mine Statements and historical aerial photographs were screened for relevant information. Long time local residents and an old miner who worked in the mine were also consulted. A preliminary hazard and risk assessment was then prepared,

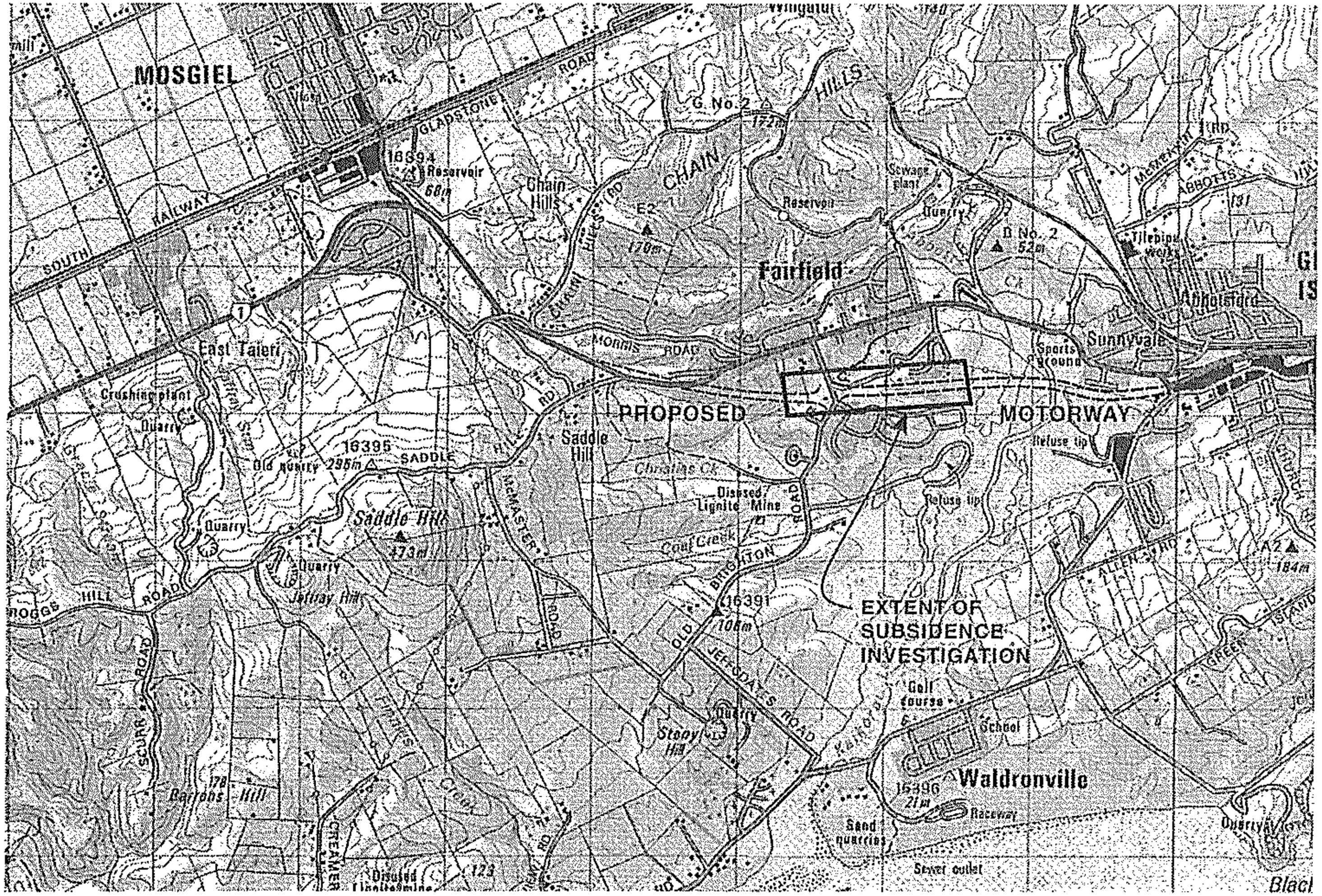

FIGURE1 LOCATION PLAN - PROPOSED FAIRFIELD BYPASS

along with recommendations for mitigating the subsidence hazard, including a site investigation program to better define subsurface conditions.

(ii) **Site investigation phase**

Subsurface investigations in November 1995 involved drilling and logging of four cored holes to just below mine level (up to 85 m) and trenching of a suspected collapsed area (Figure 2). During drilling, penetration rates were recorded and permeability tests conducted, in an attempt to identify subsidence disturbed ground. To allow for future geophysical surveying and groundwater monitoring, 100 mm PVC pipe was installed to mine level. The geological model and hazard assessment were revised based on findings.

(iii) A **review of further mine plans/case histories** was carried out in April 1996 (Stewart,1996b).

(iv) **Ground radar** - A ground probing radar survey was carried out in November 1997 over a shallow section of the mine in an attempt to identify voids.

3. SITE GEOLOGY

The geological sequence along the route (Figure 3) comprises soft rock sediments of lower Tertiary to Upper Cretaceous age overlain by a mantle of surficial materials (loess and localised solifluction deposits) up to 10 m thick. The loess is a cohesive clayey silt, with the underlying solifluction deposits characterised by the presence of basaltic boulders. The soft rock sequence, which dips gently to the southeast at 4-9 degrees, is comprised of siltstones, fine sandstones and greensands of the Abbotsford Formation overlying a layer of silty fine quartz sands (Fernhill Sand) between 5 and 10 m thick, in turn overlying coarse quartz sands of the upper Taratu Formation (10-13 m thick). These quartz sand layers, mined in nearby pits by Walton Park Sand Company, are generally poorly cemented to uncemented but include localised strong iron cemented bands up to 3 m thick. Below the quartz sands is a 15 m thick alternating sequence of thin (<2 m thick) lignite grade coal, mudstone and fine sand layers (Figure 3), which overly the main (worked) coal seam. The 'main' coal seam is 4 to 6 m thick.

Numerous holes were drilled as part of the MOW investigations (Figure 2). However, with one exception, only those drillholes within 150 m of Old Brighton Road penetrated more than 3 m below the surficial deposits, and only holes within 50 m of Old Brighton Road were drilled to the level of the main coal seam (Figure 3).

4. MINEWORKINGS

The main coal seam below the bypass corridor was mined between 1870 and 1955 (mainly by the Walton Park Colliery, 1870-1903) using "room and pillar" methods. Partial or full removal of pillars occurred locally but is believed not to have been common below the bypass corridor due to problems with water, and roof collapses, in the deeper workings and fires, CO^2 gas, floor heaving and collapses in the shallower workings.

Open cavities were encountered at the level of the main seam in all four drillholes in the current investigation and also in four drillholes sunk by the Ministry of Works in 1970 in the vicinity of the planned Old Brighton Road overpass (Figures 2 and 3). The overburden materials in these drillholes, including the mine roof, are generally 'unbroken', indicating that much of the mine is still uncollapsed. The mines are flooded except for the shallowest workings within 100 m of Old Brighton Road (Figure 3). The worked thickness indicated from drilling is up to 4.5 m, compared to the maximum of 2.1 m indicated by the Mine Statements. The latest mine plan was found not to be entirely accurate as a cavity

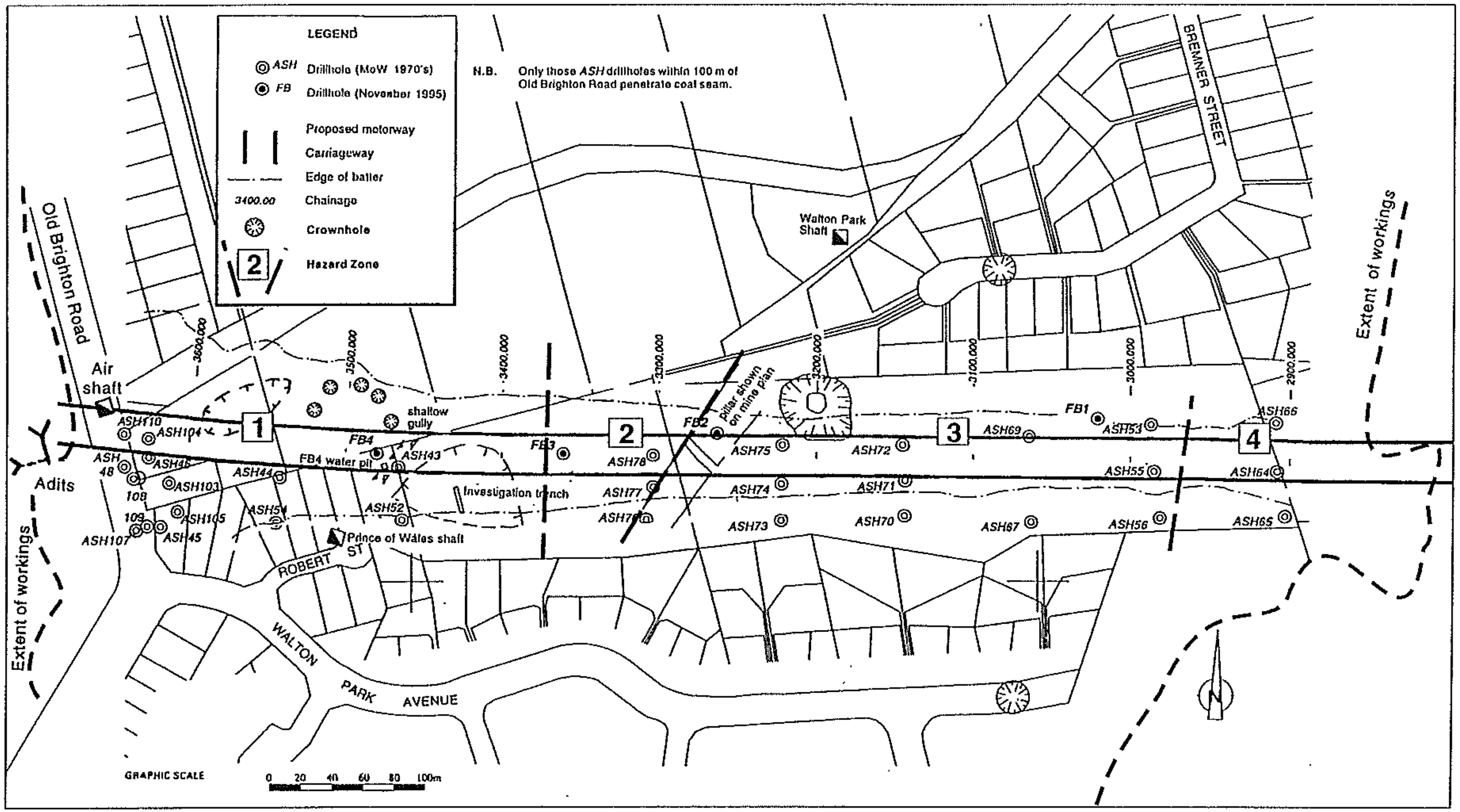

FIGURE 2 PLAN OF PROPOSED MOTORWAY, EXTENT OF COAL WORKINGS, INVESTIGATION DRILLHOLES AND HAZARD ZONES

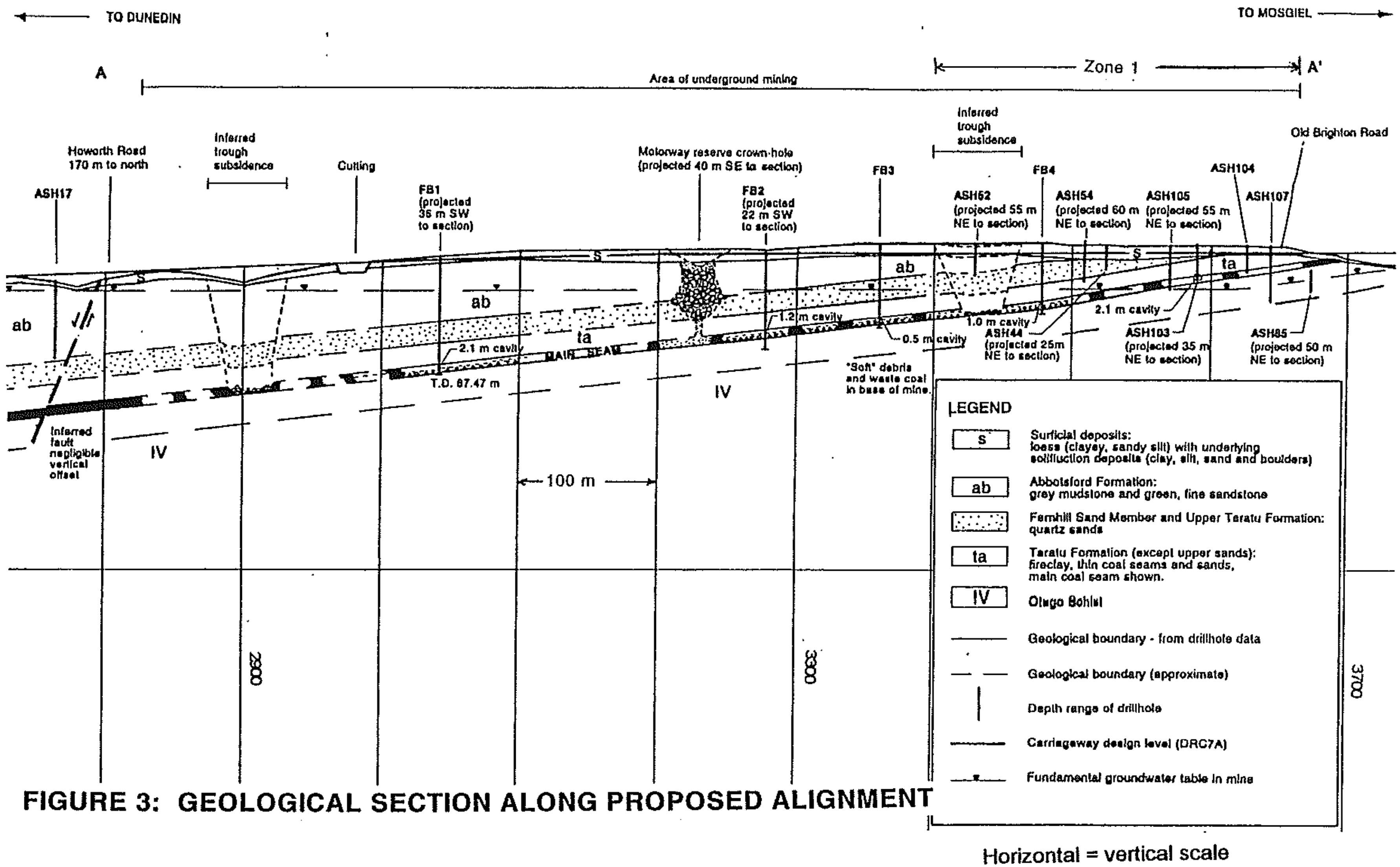

FIGURE 3: GEOLOGICAL SECTION ALONG PROPOSED ALIGNMENT

at the level of the main seam was encountered in drillhole FB2 where the plan showed a very large pillar (Figure 2).

Four mine entrances (two adits and two shafts) are indicated on mine plans within (or immediately adjacent to) the bypass corridor near Old Brighton Road (Figure 2).

5. SUBSIDENCE

(i) Occurrences

A number of circular depressions (crown-holes) up to about 12 m diameter are evident on historical air photos, or reported by Mutch (1982) and MOW (1973a), above or adjacent to the shallowest section of the corridor within 170 m of Old Brighton Road. These are believed to have resulted from roof collapses in the mine. The latest of these occurred in the early 1960's. In addition, three large (up to 45m diameter) crown-hole depressions are evident over deeper parts of the mine, one of which is partly within the motorway corridor (Figure 2).

Broader areas of general (trough) subsidence of up to about 1 m are likely to have occurred in places, particularly where partial or full pillar extraction took place. A small area of trough subsidence (c.60 m by 25 m) is indicated at chainage 3580 on 1942 air photo's, correponding to an area of pillar extraction noted on a 1924 mine plan (Stewart, 1996b). However, no conclusive surface evidence of such features has been noted on walkovers of the corridor or in the investigation trench excavated at chainage 3420.

(ii) Hazards/ hazard zonation

The main hazard to the proposed motorway project is crown-hole craters appearing at the road surface, either during or after construction, potentially resulting in serious accidents if vehicles are not warned of their presence. Trough subsidence of the carriageway poses a lesser degree of hazard, resulting in irregularity and probable fissuring of the road surface, damage to services / drainage systems, and consequent flow-on problems eg. water ponding.

Studies overseas generally conclude that, only around mine shafts and in the shallowest area of workings are the risk of subsidence hazards high enough to warrant extensive preventative treatment measures (eg. Statham and Treharne, 1991). Statistically the vast majority of crownholes occur where the depth to overburden is less than 6 times the height of the workings and virtually all within 10 times the worked thickness (Garrard and Taylor, 1988). Hazard zonations for many civil projects in mine areas have applied this "10 to 1" guideline to determine potential crownhole subsidence areas. Other projects noted have carried out treatment where workings are within a certain depth from foundation level; depths used range from 10 to 50 m.

At Fairfield three crown-holes are believed to have formed where the overburden ratio is greater than 10 to 1 and depths greater than 50 m (Figure 2). These crown-holes are thought to have developed following roof collapses where large volumes of saturated 'running' sands flowed through the workings (recorded in the 1890 Mine Statements). Due to the fluid nature of the sand, choking of the collapse void would not be able to occur, with formation of very large voids in the thick quartz sand layers at least 15 m above the workings. Subsequent collapse of the roof of the voids are believed to have resulted in these crown-holes forming at the surface, with the crater diameters enlarged by erosion of the loess sides of the holes. The three crownholes probably occurred during or soon after mining (pre WWI) and certainly had occurred by the time the 1942 air photos were taken. The possibility of further large voids still being present in the overburden above the workings has not been (totally) discounted, but future large crown-hole collapses are considered unlikely as given the weak nature of the overburden materials it is believed that collapse of any large void would have long since occurred and there is no indication of subsequent subsidence events in this zone in the past 55 plus years. Further investigations to locate any such voids would involve drilling and/or geophysics; however these may not necessarily provide conclusive results.

The hazard zonation developed for Fairfield (as shown on Figure 2) is given in the table below. In addition, hazard zones are proposed around the location of mine entrances.

Zone 1	covers area of past crown-hole subsidence, workings are at a depth of less than 10 times the worked thickness; potential for future crown-hole subsidence
Zone 2	no subsidence known in this area, workings are at a depth of greater than 10 times the worked thickness; trough subsidence possible
Zone 3	area containing 3 large crown-holes over deeper part of mine; further large crown-hole subsidence possible but considered unlikely
Zone 4	area above deepest part of mine with some past trough subsidence likely and potential for future trough subsidence

Table 1: Proposed subsidence hazard zonation for Fairfield Bypass

6. SUBSIDENCE HAZARD MITIGATION

Review of case histories in the literature indicated the following procedures have been used to mitigate the risk of subsidence to road construction projects in mine areas.

a. Subsurface investigations involving drilling and usually geophysics and where possible direct viewing/mine entry. These are typically used to better define the location and condition of voids. *At Fairfield, ground probing radar surveys were trialled in late 1997. No voids were located. However, the technique was found to be not particularly effective for this site due to the low penetration achieved in the clayey loess surficial materials.*

b Locating and filling or sealing mine entrances (shafts/adits)

c. Excavation of shallow workings (up to 5-7 m depth) and backfilling

d. Grouting or infilling of workings/voids

e. Concrete raft foundations above known or potential voids

f. Geosynthetic layers installed below road pavement/embankment
Geosynthetic reinforcement systems for roads are designed either to: 1) limit surface deformation if the cavity occurs and maintain serviceabiity, or 2) prevent catastrophic failure from taking place, but loose serviceability (Agaiby and Jones, 1996).

g. Warning signs, subsidence sensors/alarms and inspections

Proposed solution at Fairfield.

At Fairfield, Transit NZ and Duffill Watts & King have determined that the use of geogrid is the most cost effective subsidence mitigation solution. The proposed solution is to use geogrid reinforcement in a 300 - 400 m long section of the carriageway above the area of shallowest workings, where the risk of crown-hole collapse is considered to be greatest (Hazard Zone 1 on Figure 2). This zone covers the section of the route where the overburden to worked thickness ratio is less than 10:1, coinciding with where most crown-holes occurred in the past. In addition, two shaft and two adit entrances within (or immediately adjacent to) the corridor are planned to be located and capped or backfilled, and sealed. Over the deeper section of the mine (Hazard Zones 2-4) the future occurrence of crown-holes is considered to be unlikely and consequently no remedial measures are proposed. Further investigations within Zone 3 may include some non-cored drilling and geophysics. During motorway construction close surveillance is proposed for signs of subsidence.

Construction of the bypass is hoped to start during the 1998/99 summer, subject to approval of a revised motorway designation by Dunedin City Council and confirmation of funding by Transfund.

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RISK MANAGEMENT IN THE ECONOMIC DESIGN AND CONSTRUCTION OF ROAD CUTTINGS IN NEW ZEALAND

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SUMMARY

Road projects involve uncertainties as to the ground conditions and the expected performance of cut slopes. These uncertainties are pronounced due to the steep terrain and highly fractured, geologically young rocks predominant in many parts of New Zealand. Site investigations can be difficult and costly due to access restrictions. Taking a global approach to site investigations, design and construction of road cuttings will help optimise total costs and manage the risks. The limited funds available for road construction and the low volumes of traffic on rural roads, make effective management of risks important for road development in New Zealand.

A number of methods of risk management are illustrated using recent projects. These include the adoption of a coherent design and construction philosophy, use of tools such as decision analyses, and an observational method during construction. These are valuable in managing the risks from geotechnical conditions affecting road cuttings.

1. INTRODUCTION

The design and construction philosophy for road cuttings in New Zealand requires careful thought. Geologically young rocks in steep terrain formed by tectonic movements are predominant in New Zealand. Roads in such rugged terrain have historically been formed by cutting into the hillside and formation of sidling fills. Road improvements often require further cutting into the hillside.

The low traffic volumes, particularly in areas outside main centres, mean that the tangible benefits from road improvements in terms of benefit / cost ratios are often modest. Only projects with a benefit/cost ratio exceeding four are currently able to be financed given the limited funds available for road improvements or construction. The slope angle of cut slopes can have a significant impact on the cost of earthworks and land acquisition.

Under the circumstances, the selection of appropriate cut slopes can be critical to the viability of road projects. If conservative cut slopes are chosen to minimise the risk in the face of uncertainty, leading to increased project costs, then :

- the project may not meet the minimum benefit/cost ratio for funding
- the limited funds for road development may not be effectively utilised.

On the other hand, steeper cut slopes with a higher risk could lead to failures during and after construction with associated escalation of construction and ongoing maintenance costs. There is the added disadvantage that important lifeline roads are more likely to be blocked by failures, during natural hazards such as earthquakes or unusually wet weather, in slopes which are at best marginal in normal conditions.

Therefore it becomes important to consider and manage the risks in the design and construction of cut slopes. Managing the risk requires partnership between the road authority, designers and the contractors. The designers will need to clearly identify and present the risks to road authorities so that they can make informed decisions. The road authorities and contractors will need to co-operate to manage risk during construction. This paper presents the philosophy associated with the management of the risks, and methods which have been adopted by Opus International Consultants in recent road projects. Examples from recent projects are cited where appropriate.

2. THE CHANGING SCENE

New Zealand has undergone significant changes in the development and management of public assets during the past ten years. This has also meant changes in the way roads are developed and maintained. Historically, roads have been managed by government departments such as the National Roads Board and local authorities. Government agencies such as the Ministry of Works and Development and its predecessors have been responsible for the design, construction and maintenance of State Highways and major roads. Risk decisions were made implicitly, and appropriate designs were adopted.

Changes in the procurement of road design and construction and funding have been introduced since about 1990. Under this new environment, the risks have to be identified explicitly so that the road authority is aware of, and accepts the level of risk associated with the design. Consultants who design and manage the construction of road projects need to assess and present the risks and help the road authorities make decisions on the level of risk appropriate for each project. The commercial pressures and potential liability make the identification and acceptance of risk an important consideration.

Road reforms being proposed will make this risk decision making process even more important. The longer term total maintenance contracts being contemplated will also require such decisions to be made by both "maintenance contractors" and road authorities.

3. THE GEOLOGICAL ENVIRONMENT

New Zealand is in an area of high tectonic activity and a number of faults transect the terrain. There are large areas with rugged terrain with steep hillsides. The central part of New Zealand has particularly high tectonic activity. Tectonic activity has been associated with movement along faults in the brittle hard sedimentary rocks in these parts of New Zealand. Such movements have resulted in complex highly fractured and fault disturbed rocks, with fault, crush and shear zones. These defects in the rock often have a major adverse effect on the stability of highway cuttings.

Such conditions contribute to the uncertainty and risk associated with road cuttings. Hancox and Brabhaharan (1995) presented geological aspects of cut slope failures in the realignment of State Highway 58 in Wellington. This illustrates the importance of a coherent design and construction approach to deal with the uncertainties in design arising from geological factors.

4. RISK MANAGEMENT

The risk from cut slopes can be managed by a combination of :

- an appropriate scope of site investigations
- geotechnical engineering assessment and design
- risk assessment and decision analyses
- observational method during construction
- monitoring and maintenance.

The effective management of risk involves the use of an appropriate combination of the above measures to optimise the risks and costs for a particular project.

4.1 Site Investigations

It is well established that site investigations will enable the uncertainties to be reduced and hence the risk to be better defined. That will in turn enable more informed decisions on risk. Therefore an appropriate site investigation programme is essential for the design and construction of cut slopes. The site investigations can be optimised by considering the overall project approach. For example, an observational approach during construction may lead to better management of risk rather than a very detailed and expensive programme of site investigations.

4.2 Geotechnical Engineering Assessment and Design

This forms part of the risk management process. It is important to identify the mechanisms of failure, assess the likelihood of potential failures and design an appropriate cutting. However, the complexity of the analyses must be compatible with the information available and the benefits from such analyses.

4.3 Risk Assessment and Decision Analyses

Risk assessment is a valuable tool in the design of road cuttings, and comparison of options with lower and higher risks. Often, with rock cuttings, the risk assessment may be qualitative, and be presented as recommended by the Australian / New Zealand Standard on Risk Management AS/NZS 4360:1995 (Standards New Zealand, 1995). Consideration of the likelihood of failure and the consequences to derive the risk will provide information for road authorities to decide whether the risk is acceptable and to select an option with an acceptable level of risk.

Decision analyses such as using an event tree (Whitman, 1984) could be useful if there are a number of courses of action with associated risks and costs. This will help assess the likely cost and consequences of choosing various options. Probabilities have to be chosen for various outcomes, and the sensitivity of the outcomes to variations in probability can be verified. The analyses presents a useful way to manage the risk in an informed manner.

More sophisticated approaches using specialist software are now available. These models enable probability distributions to be placed on important input variables, and through a process of Monte Carlo type analysis, are able to produce a more comprehensive picture and better understanding of the resulting outcomes. This in turn enables the impact of various factors on risk and project cost to be better understood and helps with the selection of an acceptable level of risk and cost.

4.4 Observational Method During Construction

Where there is uncertainty as to the ground conditions at design stage, the risk could be managed by carrying out a design based on the available information, and then putting in place observation during construction to verify the actual conditions exposed. Typically for rock cuttings, the observation will involve engineering geological mapping of the defects exposed and observing the behaviour of the cut slopes. This would enable the cut slopes to be changed or stabilisation measures to be incorporated. Brabhaharan and Fleming (1996) described the application of the observational method for the design and construction of rock cuttings for roads in highly fractured rocks.

4.5 Monitoring and Maintenance

The risk could be further managed by monitoring road cuttings after construction and carrying out stabilisation measures as part of the maintenance programme after construction. This may be an appropriate form of risk management where the consequences of failure are relatively low and the cost of risk reduction is high.

5. CASE STUDIES

5.1 Cuttings for State Highway 6 Whangamoa North Realignment, Nelson

The project involved the realignment of about 3 km section of State Highway 6 through the rugged terrain associated with the Whangamoa Hill, between Nelson and Blenheim. A number of major cuttings through two topographic saddles, as well as many large sidling cuttings along the face of steep hillsides were constructed. At early design stages of the project, it was clear that the ground conditions were poor, given the presence of highly fault disturbed fractured rocks associated with the Whangamoa Fault, and the variable nature of the rocks along the route.

During the design of the project, Transit New Zealand expressed a willingness to consider a higher risk, lower capital cost scheme, which was considered appropriate given the rural location of the highway and the relatively low traffic volumes. Risks associated with a lower capital cost scheme were identified as:

(a) risk of escalation of construction costs due to design changes required during construction
(b) risk of long term failure
(c) risk of increased maintenance costs.

The higher risks associated with a lower capital cost scheme with steep slopes were discussed with Transit New Zealand, and an *observational method* was chosen to manage the risk. This enabled less conservative designs to be adopted, in spite of the uncertainty in ground conditions, as it allowed the ground conditions to be verified during construction by inspection and engineering geological logging.

Cut slopes ranging from 45° to 63° to the horizontal were adopted in the bedrock, with slopes of 33° in colluvium, till, and terrace alluvium deposits. The cuttings were regularly inspected by an engineering geologist, and the observational approach was effective in identifying poor unfavourable ground conditions, and making design changes for individual cuttings. This enabled the capital cost of the project to be optimised and an acceptable level of risk to be achieved. The observational method adopted was reported in detail by Brabhaharan and Fleming (1996).

5.2 Cutting of Bluff at State Highway 1 - Newlands Interchange, Wellington

Cutting into a 40 m high bluff was required for the Newlands Interchange project on State Highway 1 in Wellington, New Zealand. The bluff in the Ngauranga Gorge was in highly to completely weathered highly fractured Wellington Greywacke sandstone and argillite with a number of shear zones associated with faults. The project is also within 0.5 km of the active Wellington Fault. The bluff stood at a slope of about 65°, with the geometry indicating a number of past failures, suspected to have occurred along shear surfaces. The State Highway at the foot of the bluff is a busy section of motorway with over 60,000 vehicles per day. There are a number of houses at the top of the bluff.

Site investigations comprised engineering geological mapping, with trial pits and boreholes near the foot of the bluff. The investigations showed the presence of numerous shear zones dipping at between 45° and 88°, some in an unfavourable direction towards the highway. The rock was also closely jointed, but these joints were generally not persistent through the rock mass.

An engineering geological assessment identified a risk of slope failure due to the presence of adverse defects. Such a failure would present a significant hazard to road users as well as the properties at the top of the bluff. In addition there would be a high risk of earthquake induced slope failure affecting this important road. Given the height of the cutting and the presence of the properties at the top, the cost of forming the slope at a preferred 45° cut slope was high. The client was faced with two options - adopt a 45° slope with a lower risk, or proceed with a steeper 63° slope with a higher risk, with further investigations and an observational method to manage the risk. Should unfavourable conditions be encountered this may require the slope to be reformed at a flatter slope, say 45°.

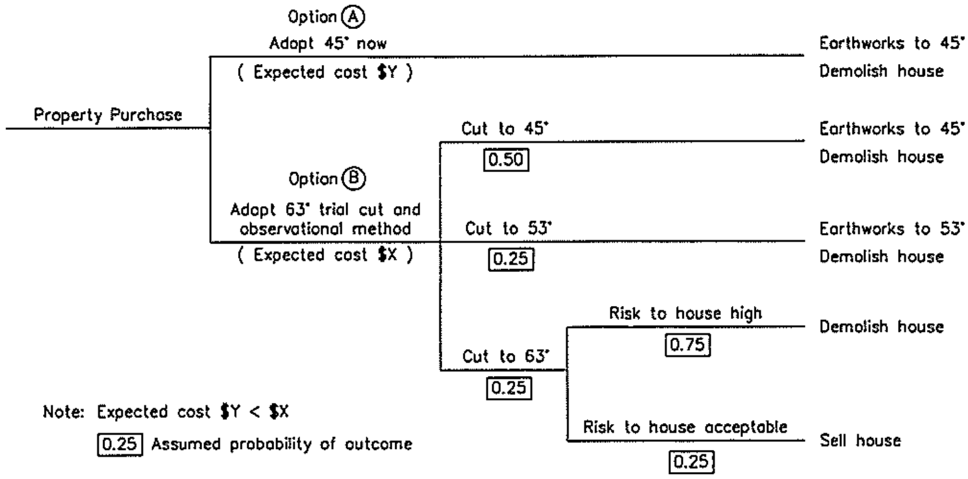

Figure 1 Decision Analysis using Event Tree for Cut Slope at SH 1 Newlands Interchange, Wellington

A decision analysis was carried out to assess the expected cost of both options, and this showed that the expected cost of the 45° slope option was cheaper. Sensitivity analyses indicated that even if the probabilities were assumed more unfavourably, the expected cost of both options may still be the same. These decision analyses enabled the client to decide to adopt a 45° slope, rather than run the risk of proceeding with a steeper 63° slope, incurring additional costs for further investigations and observational method, and possibly having to recut the slope with an overall higher cost.

The cutting was successfully constructed in 1997 and is performing as expected.

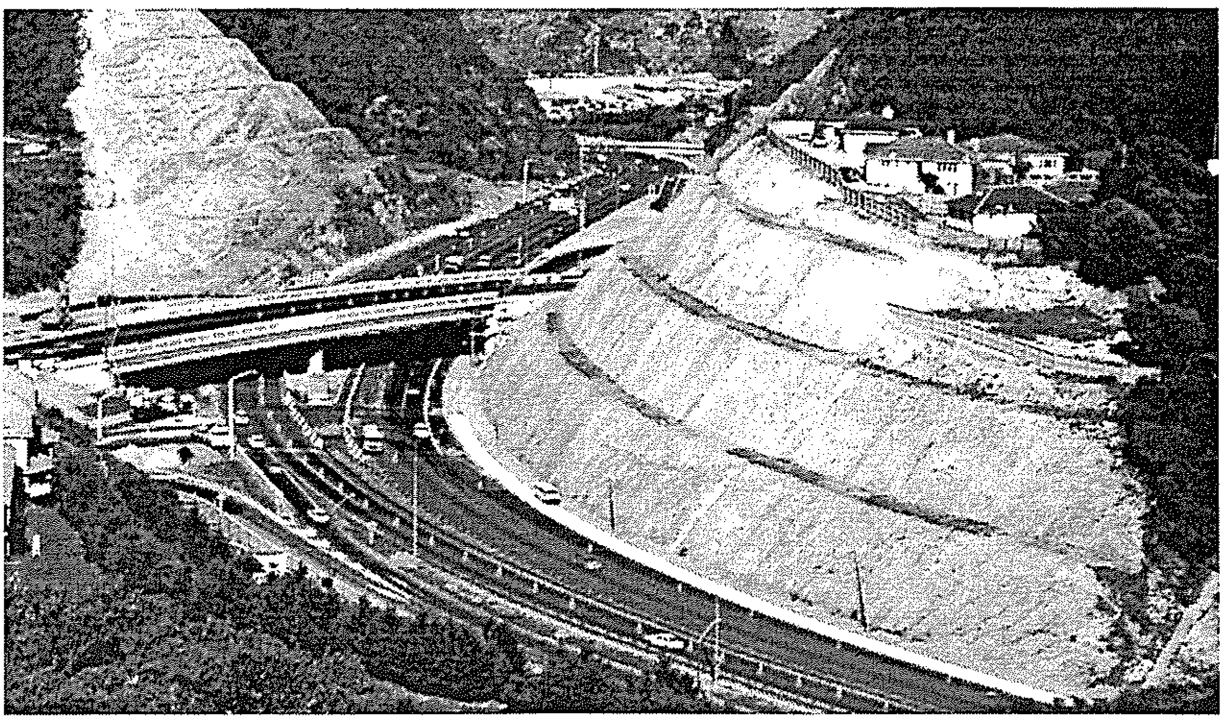

Figure 2 Newlands Interchange, Wellington : 40 m High Cutting

6. CONCLUSIONS

Design and construction of road cuttings involve significant uncertainties, particularly where the ground conditions are variable and uncertain. The project cost is sensitive to the cut slopes chosen for the cuttings as these involve significant quantities of earthworks where high cuttings are required in steep terrain. The economic design and construction of these road cuttings requires the management of risk, to achieve an acceptable level of risk and cost.

Risk management techniques enable road development authorities to achieve economy on the total project costs, and to make decisions as to an acceptable level of risk and cost. Some techniques which have been effectively used by Opus International Consultants on recent projects are :

- decision analyses on slope options and investigations / design / construction processes
- observational approach to design and construction

The form of risk management for a particular road cutting will depend on the individual circumstances. To achieve the most appropriate form of risk management and achieve economic road construction, it is important to consider the total project from site investigations to maintenance. By considering this at an early stage, an appropriate programme of site investigations, design, risk assessment, construction and maintenance can be chosen. The success of the risk management programme will depend on the choice of an optimum combination of measures.

It is emphasised that a global approach to consider the total project including investigation, design, construction and maintenance is important for the effective management of the risks, and to achieve an economic design for roads. Road authorities should give consideration to allowing for flexibility in the procurement and management of road design and construction services to facilitate such a global approach.

7. ACKNOWLEDGEMENTS

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RECENT CASE STUDIES OF SLOPE FAILURES IN ROADING

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SUMMARY

Methods employed for site investigations for slope failures in roading are typically dictated by the importance of the road, the risks associated with the level of remediation, and the costs/benefits of the investigation and repairs.

On high profile, major roadways such as motorways or arterial links, the importance of the works and the consequence of failures, commonly provide a larger budget that allow the use of heavy mechanised drilling rigs and/or Dutch Cone Penetrometers. Alternatively, for lesser volume, rural roads, sealed or not, it can be difficult to justify the costs of such investigation techniques, with the investigating engineer resorting to the more economical hand auger, shear vane and/or scala penetrometer techniques, with their associated limitations.

This paper presents selected case studies demonstrating the usefulness of the Pennine Dynamic Probe in cost effectively developing more reliable stratigraphic profiles, to aid in the design of appropriate remedial works.

The Tool

The Pennine Dynamic Probe is manufactured in the United Kingdom, generally in accordance with the European standard DIN 4094, Part 1, and can use a range of hammer weights (commonly 30 or 50 kg) and drop heights (commonly 0.5 metres), to drive a 90 degree cone with an end area of 15cm^2 into the ground, while the operator records the blow count per 100mm of penetration, to get an indication of the soil strength profile with depth. The probing rods are in 1 metre increments, and torque readings are taken prior to adding each new rod, to give an indication of the parasitic effects of skin friction. With its available driving energy, the probe is generally able to achieve greater depths of penetration into dense or stiff materials than traditional hand operated tools.

The first Pennine Dynamic Probe arrived in New Zealand in October 1995, and since that time, has proved its worth on a variety of slope failure investigations, often in conjunction with the traditional hand techniques. It is for example, small and light enough to be manhandled and/or winched down very steep slopes, yet powerful enough to punch through road pavement, and it can probe to considerably greater depths than conventional hand augering.

Case Study 1: Farm Accessway, Whitford

On a scale based on available funds for investigation and remedial works, slips such as occurred on this rural site must languish near the bottom. Thus, one useful, low cost tool used very effectively here, was a ground operated helium blimp, to take aerial photographs of the site, from which we could read, and onto which, mark up, important surface features.

During a period of high intensity rainfall in September 1997, a large area of land slipped below, and extended back across an existing farm access-way to the residence. In many rural scenarios like this, the land owner would often just dress the land up with a bulldozer, in the hope of no repeat events, at the smallest possible cost.

On this site however, the owner was making plans to construct new utility buildings just beyond the head of the slip scarp, and there was concern that topping up the slumped ground would simply trigger repeat movements, and possibly extend the failure zone, unless an understanding of the slip mechanisms and stratigraphy were obtained first, to evaluate the likely need for, and nature of suitable stabilisation works.

On this job, the owner was able to install a series of standpipes, and monitor groundwater levels himself, so we could map the likely influences on groundwater on the stability. To establish depths to stable ground, it was necessary to carry out some form of subsoil investigation, but the presence of rock fill and the possibility of missing thin planes of weakness, indicated that hand augers might not be suitable.

Instead, a series of six dynamic probes were put down along the periphery of that part of the land estimated as most likely able to be cost effectively retained, which enabled us to develop the following long section.

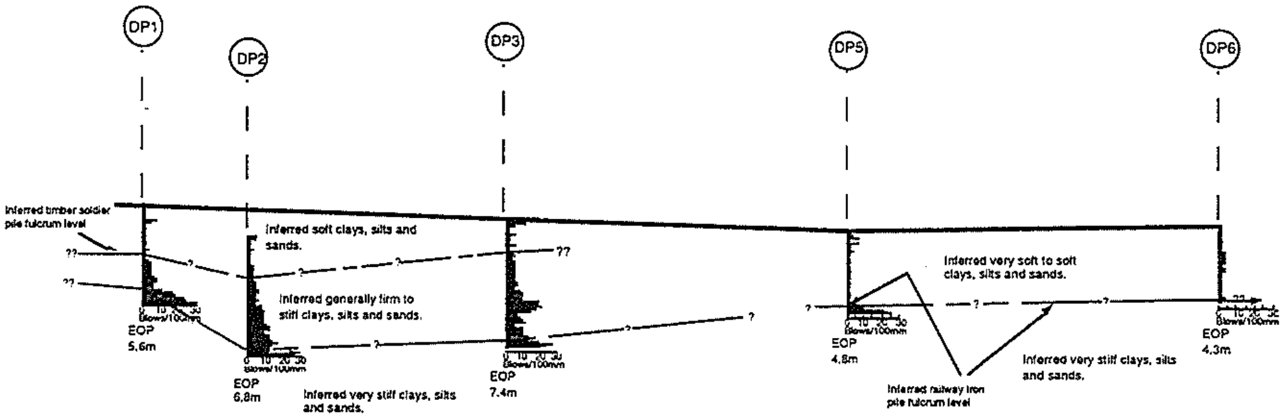


Figure 1: Long-section strength profile

From this profile of depths to various strength strata, we could quickly see that relatively low cost timber pole soldier piles could be installed in the vicinity of probes 1 to 3, but that near probes 5 to 6, the depths to likely pile fulcrum points were not only becoming too great for commonly available timber poles, but also, the ground at that depth was becoming extremely hard, with blow counts per 100mm of better than 25 (≈SPTN=50), indicating the presence of sandstone bedrock which might be difficult to auger. Seemingly, a different approach would be required here, probably utilising steel piles driven in holes pre-drilled by a percussion rig.

Case Study 2: Run Road, Wharehine

This section of road is constructed on an embankment formation, and seems to slump with nearly every heavy rain. Historically, each slump has been patched up as part of the Territorial Authority's maintenance programme, because its rural location and relatively light use has previously made it difficult to justify a large capital expenditure to prevent repeat slumping.

However, the costs of frequent repairs has now reached a level where some preventative measures are considered necessary. An investigation was carried out using the dynamic probe, complemented with a single borehole to confirm material types, and again, a long-section of the soil strength profile was able to be determined, together with a typical cross-section.

From that inferred long-section profile of strengths, it was apparent that soldier piles would have to cantilever for heights of up to approximately 4 metres, which, with additional surcharges, was beyond the practical limits of timber soldier piles.

As a result, given the low risk/low priority nature of this rural site, the solution adopted was to design for the use of driven used railway irons, at 1.8 metre centres on a trial and error basis, with provision for infill piling if the road formation continued to move.

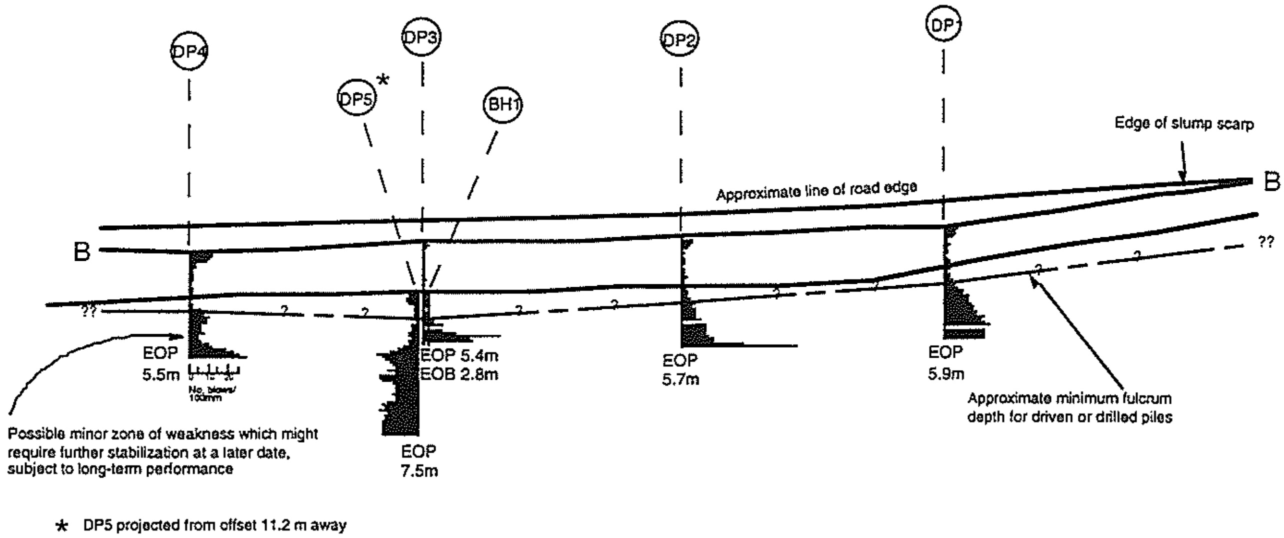


Figure 2: Run Road long-section strength profile

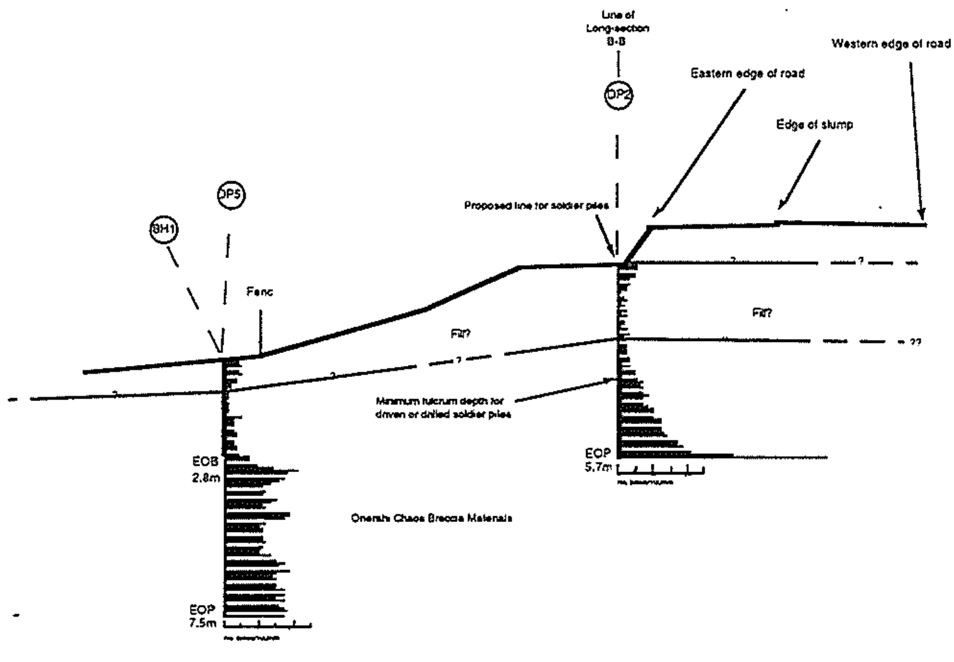


Figure 3: Run Road typical cross-section

Case Study 3: Wharehine Slip, Kaipara Coast Highway

Here, when the road had been upgraded previously, a large quantity of spoil had been placed in the head of a steep-sided gully below the road, possibly with the idea of providing additional support to the road alignment itself. However, based on our inspections of the subsequent subsidence, it appears likely that there may have been insufficient measures installed to control the existing groundwater regime, and also, the standard of pre-filling preparation is unknown. In any event, it appears that the new fill became saturated and possibly contributed to the road subsidence.

This site is on State Highway 16, not far from, and is similar in nature to, that subsidence at Run Road, and figure 4 shows the inferred long-section strength profile.

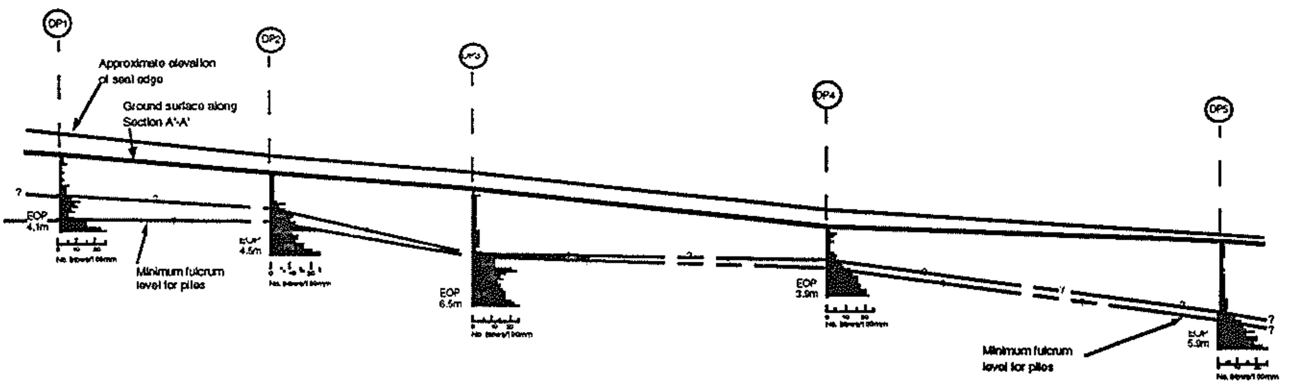


Figure 4: Wharehine (SH16) long-section strength profile

A similar system of driven railway irons was proposed here, but due to budget constraints, interim stabilisation measures to date have been limited to passive works, by way of a cut-off drain installed uphill of the road, where the depth to rock is only about 1.5 metres.

Case Study 4: Cleasby's Hill, Kaipara Coast Highway

This site is also on State Highway 16, but about 10 kilometres further north, where the road more or less follows an elevated and very narrow ridge, as it approaches a scenic look-out on the high point of the highway. Here, the topography and folding visible in roadside cuttings, suggest that the slumped area overlies an ancient infilled syncline carrying a defined aquifer, implying the likely presence of a greasy-back. Exposures of somewhat weathered parent bedrock in those cuttings also suggests that the greasy-back should be relatively shallow.

However, the local geology is that of Onerahi Chaos, which consists of chaotic bedding of mudstone, siltstones, limestones and shales, produced by large scale movements of sedimentary beds, that have resulted in an inversion of older rocks overlying younger rocks.

Furthermore, the vertical magnitude of the slumping was such that the failure plane appeared to be relatively deep. Observations of the topography below the site, also suggested that the movement was deep seated, and extensive.

A total of six probes were put down across the slump zone, to map the sub-surface profiles and a key feature noted in the soundings was the apparent presence of interbedded stiff and soft zones, considered to be typical of the Chaos formation.

The inferred depths to the likely failure planes were up to 6 metres, and given the restrictive nature of available road reserve, coupled with the depth and severity of the slip, Council here felt they had little option, but to install a line of reinforced concrete soldier piles.

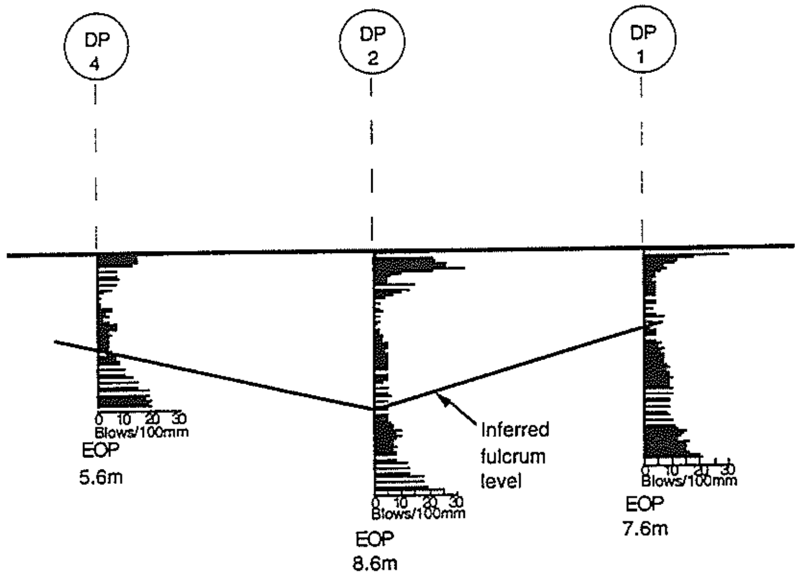


Figure 5: Cleasby Hill (SH16) long-section strength profile

Case Study 5: South Head Road Slip

This is another low volume road, in the extreme, in that the site is located approximately 50 metres from the terminal end of South Head Road, below the Kaipara Harbour. Unlike the last three case studies, this was a massive slip, rather than a slump, which encroached into the unsealed road pavement by approximately 2 metres, over a road length of about 15 metres, and for which the scarp and slump debris extended downslope for about 25 metres. Typical grades of the surrounding intact slopes were measured at nearly 1V:1H.

The site was investigated by putting down three dynamic probes to depths of up to 11.7 metres, and two, 50mm diameter hand augered boreholes to depths of up to 4.0 metres. Also, logging of the exposed slip scarp showed the site to be underlain by an old gully, that had been filled predominantly with local silty sands, but which also contained a single layer of boulders, with observed diameters ranging from 0.4 to 1.0 metres.

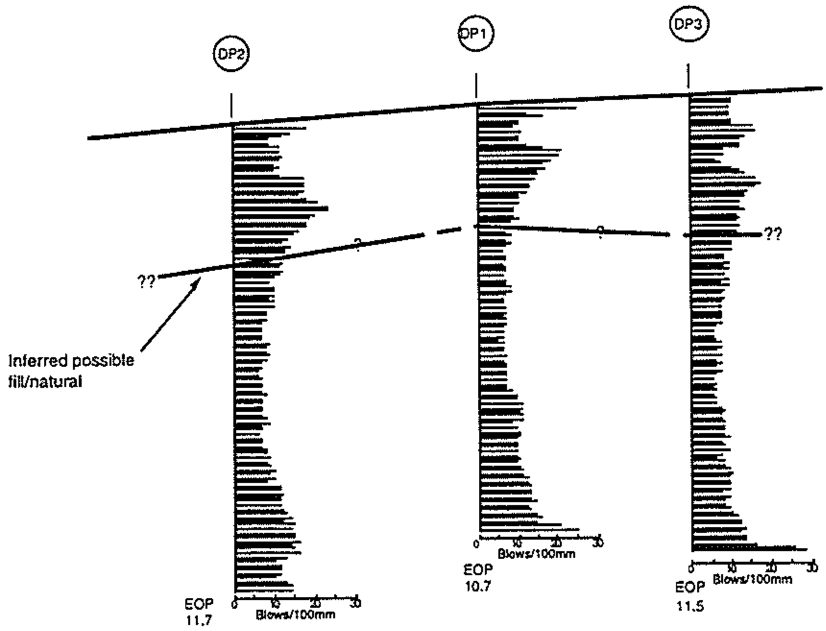


Figure 6: South Head Road Slip long-section strength profile

An interesting feature of the dynamic probe results, when compared to those of other sites with cohesive soils, is the comparatively uniform blow count profile with depth, and the relatively consistent, and moderately low torque readings measured in the sandy soils. Typically in cohesive materials, (layers of weakness aside,) the blow counts increase with depth, as do the rod string torque readings, with the latter often showing a decrease when the water table is encountered. Reference to the literature indicated that the local geology is that of late Pleistocene fixed dune sands, and the steep local topography suggests that it must have a reasonably high level of inherent stability, at least in the absence of excessive water. The marked difference in the nature of the soundings at this site therefore confirmed to us that we were dealing with substantially non-cohesive materials.

The slip has effectively reduced the road to one lane,and left a potentially dangerous escarpment for local traffic to negotiate. However, road patronage is particularly light, with its main usage being limited to stock trucks and fishermen wishing to access the Kaipara Harbour South Heads. The challenge then, was to identify an effective means of returning the road to two lanes, at an affordable cost/benefit ratio. To date, that, combined with the severe topography, has ruled out the use of conventional reinforced concrete soldier piles.

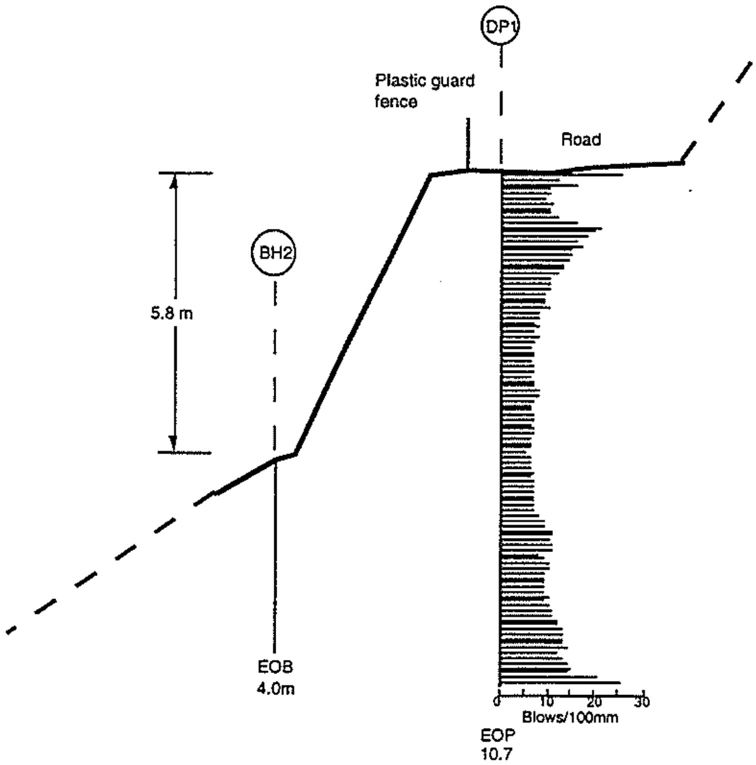


Figure 7: South Head Road cross-section

The current proposal under consideration is to divert the road by cutting back into the hillside, to create a slope angle of around 65 degrees, which can be stabilised by soil nailing. The success of soil nailing in any situation is heavily dependent on the actual nail pullout resistance exceeding the theoretical design figure, and so it is important to be confident of achieving success before committing any significant monies to insitu pullout resistance testing. However, soil nailing is a system that is especially well suited to non-cohesive materials, and the writer is extremely confident of its potential here.

SLOPE REPAIR DESIGN & CONSTRUCTION ON LOW VOLUME ROADS HOW DO WE COMMUNICATE THE RISKS?

William Gray, Principal Geotechnical Engineer, Opus International Consultants
Tim Jowett, Engineering Geologist, Opus International Consultants

All our engineering and geological training teaches us that in order to design and construct safe slope repairs alongside roads, or indeed anywhere where life and property are at risk from failure, that we must ensure that adequate geotechnical investigation and design is carried out. In this day of competitive design and cost conscious clients/owners, the investigation and design of repairs or control of slope failures is often completed on a shoe string budget. Who takes the risk in this situation?

In Hawkes Bay and Gisborne, slope repairs are often carried out on roads carrying low traffic volumes. The cost of repair can be high. Transit New Zealand and other road controlling authorities are often placed in the difficult situation of having to spend considerable sums of money repairing roadside failures on roads that conventional Benefit/Costs analysis would suggest are "uneconomic". Obviously to the farmer at the end of the road, maintaining safe access is essential.

Over the last few years, our small geotechnical group based in Napier has developed a close relationship with client road controlling authorities. We aim to ensure that the client is fully informed of the risks involved with a project. The client can then decide on the level of risk that they are prepared to take. Rather than seek the safest, and possibly most expensive solution to slips affecting the roading network, Transit New Zealand in particular has a policy to reinstate the road to the original standard using professional engineering advice and sound construction techniques, together with an agreed Factor of Safety against future failure. The objective is to achieve the least repair cost option that is consistent with good engineering practice and acceptable risks. It is essential that good communication between client and consultant is maintained if this approach is to work effectively, and fairly.

1. The Geology of the East Coast

Along the East Coast, in Hawkes Bay and Gisborne, we are often building and maintaining roads on steep hill country whose geology is predominately sedimentary rock and soil. The slopes are often steep with little soil or vegetation cover. Adverse groundwater conditions, low strength and sensitive materials and extreme rainfall events combine to create often large and complex slope failures.

Following the tropical cyclones in 1987 (Bola) and 1995/96 and 1997, a large number of dropouts and slope failures affected State Highways 2,5,35 and 38, as well as many Local Authority Roads. Those of us who can remember Cyclone Bola, can recall many kilometres of roadway closed by slips. Whilst many of these can be simply cleared, dropouts and failures below the road often require repair or road realignment.

2. Methods of Repair

Traditionally, repairs of dropouts and down-slope failures have included road retreat, stormwater control, subsoil drainage, and gabion/concrete/timber gravity walls. In recent years significant use has been made of reinforced earth technology.

All these repair methods require the designer to have access to, and to have considered some or all of the following information:

- Access to roadside land, and site survey information
- Geological mapping of the site
- Subsurface investigation data
- Material soils properties
- Road geometry and traffic requirements
- Traffic safety issues both during and after construction
- Resource Consent and Building Consent Requirements
- Long-term security of the site
- Economic considerations

The costs and time involved in obtaining this information, even for small dropouts or slope repairs, can be significant. In order to design "safe" repairs at a reasonable cost, we have found that involving the client in the decision process, at all the stages of the investigation, design and construction of slope repairs, can be an effective way of achieving good solutions. We will discuss some of the ways we have used this approach in the following sections.

3. Preliminary Site Assessment

As soon as a dropout or slope failure occurs, our geotechnical group will be asked to look at the site to determine whether the site is unsafe, and what measures should be taken immediately to ensure the safety of the travelling public. Options can involve leaving the road as is, restricting the width of the road, or in extreme events, road closure. The client is actively involved in this decision process, and any decision will be made in consultation with the client, the network maintenance consultant and the geotechnical engineer.

At the same time we will carry out a preliminary geological appraisal, and possibly limited subsurface investigations (Scala Penetrometer) to determine what possible remedial works options are available, and what investigation and design options are considered necessary. These aspects will again be discussed with the client, and a decision made on how best to proceed, both in terms of costs and timeframe. We will often produce a preliminary Options Report that will identify repair options and rough order costs. This report will enable the TNZ or the LA to seek funding from Transfund while the detailed investigation and design works are carried out. This speeds up the repair process.

The preliminary Options Report also identifies the investigation and design fee estimate for each project. This enables the client to control overall project costs on a site by site basis.

4. Survey and Site Investigations

Having good quality survey and site information helps the design of slope repairs. Total station based survey is used by our team to provide contour and cross section information for subsequent slope stability and design work. Often several dropout repairs can be surveyed in one day.

In most cases, subsurface investigations are carried out using a combination of geological walkover, Scala Penetrometer and Hand Auger investigations, and truck mounted mechanical Dutch Cone Penetrometer investigations. Particular attention will be given to groundwater conditions. We will often discuss the investigation results with the client as the work proceeds. If changes in the investigation process are required, these can often be agreed to quickly because the client is aware of the work, and the need for the change.

In recent years more use has been made of specialist laboratory testing for material properties. With an increasing use now being made of reinforced earth technology, tests such as direct shear tests on backfill materials are being carried out with the support of clients. This enables us to consider using a greater variety of more economical backfill materials, rather than the "safe" granular backfill option, which are very expensive on the East Coast.

5. Design Process

The design process we follow involves using the survey and subsurface investigation data to build slope stability models of the site in SLOPEW. This enables us to back calculate estimates of material properties and groundwater conditions that were present at the time of failure. Our repair solutions are then built up out of these models.

We have found that an important part of this design process is again to discuss the results with the client. Options and solutions can be determined in consultation with the client. In the case of smaller slip or repair sites, this consultation can mean that a formal final report is not required, therefore saving time and expense. For larger projects, a formal Design Report is usually produced. In all cases it is essential that a good record is kept of the investigations and design process, and of any decisions which are made, particularly the "risk" decisions.

In some of the more complex slope repair sites we have worked on in the last three years, the close working relationship with TNZ in particular, has meant that the "risks" associated with the investigation and design for particular sites and solutions have been understood by all parties.

These risks can include:

- The long-term and short term "safety" of the site. Site safety during construction can often be an issue, particularly during construction. With modern design methods "neat" solutions can often be dreamed up and drawn. But can these be built?
- Understanding material properties used in design. Low risk, expensive solutions can be achieved using conservative material properties, or granular backfill.
- Our level of understanding of complex site geology. Although much historic information is available, without detailed geological and subsurface investigation often we need to made assumptions about geology at complex sites.

6. The Construction Process

As consultants, we are increasingly being asked to provide construction "observation or surveillance" rather than supervision. Often important geotechnical information about a site can be obtained during construction. This information may affect the "solution" and require changes to the design. Alternatively this information may confirm the assumptions made in design. Whatever the outcome, geotechnical information obtained during design will help our overall understanding of the site and the area. The amount of information obtained during construction will need to be balanced between the "clients" willingness to pay and acceptance of associated risks, our own "thirst" for information, and our assessment of the risks involved.

The same can be said for material and site testing during construction, and quality management. End result specifications require contractors to take much more responsibility for the quality of the job. In the case of slope repairs on complex sites, this means that the contractor needs to understand the "design" and be aware of the risks associated with such things as material selection and use, site management (e.g. how long cut slopes will stand up), and groundwater and surface water management. In our experience on the East Coast, contractors do not have the required level of understanding.

Who then takes the risk if during construction, vital geotechnical signs may be missed because the consultant is only able to provide "observation" of the site.

Over the last few years we have tried, and are still trying to write contract documents which explain the "design process" and the "assumptions made" in design. All subsurface and geological information obtained from the site is included in the contract documents.

Is this process similar to how others around New Zealand treat construction "observation"?

7. Case Study : Mohaka Hill, SH 2

The Mohaka Hill site on SH 2 between Napier and Wairoa is a Case Study which we hope illustrates some of the issues we have raised above.

The Mohaka Hill dropout occurred on 3 July, 1997, after approximately 5 days of heavy rainfall. The dropout closed the road for about two days. This required traffic to make a 20-minute detour along narrow unsealed roads. The road was only reopened after our geotechnical engineer had visited the site the day after it happened and recommended that a temporary road could be cut out of the steep hill batter above the road. This bypass then remained unsealed and one way.

The dropout site is on a section of SH 2 that traverses the side of a hill which is known to be unstable. At the foot of the hill, approximately 18m below the road, a culvert fed stream is actively cutting into the toe of the slope.

The backscarp of the landslide extended to the inside edge of the road, over a length of about 45m. The landslide was approximately 60m long, , and about 18m downslope.

The site was known to have high groundwater levels and low strength weathered subsoils. The exposed slip material were very weak and saturated. The dropout could not be left as it was. The options for retreating the road away from the slip were limited by steep unstable slopes above the road, and road geometry and safety. The site is situated on a steep, straight downhill road section in which a sudden "corner" would have been very unsafe. Therefore any retreat would have involved considerable changes to road geometry on both the approaches.

The investigation and design process we used on this site was as follows:

- Following an initial site investigation just after the dropout occured, our preliminary appraisal suggested that the re-establishment of the fill slope below the road would involve excavation and replacement of some 9000 m^3 at a cost of between $350,000 and $750,000. The range of costs reflected out uncertainty about the level of the foundation upon which we could base the repair. The repair would also need to include drainage control works. The client TNZ was made aware of the scale of the project at an early stage. Because of difficulties with obtaining sufficient fill, and potential problems with toe erosion and foundation conditions, we rejected this option at an early stage.

- Our preferred solution included re-establishing the previous road geometry using geogrid reinforced fill and downslope drainage controls. However, because at this stage we had no subsurface information, we were not able to confirm the geometry of the reinforced fill.

- This preferred "option" was discussed at a very early stage with the client, and with the consent authorities such as the Regional Council, local iwi and landowners so that they could have input into the design. This early consultation greatly helped the process.

- Site investigations included a geological walkover and mapping of the site, and truck mounted CPT tests. From this information a slope model was built in SLOPEW to assess material properties at failure. An early assessment of groundwater conditions had to be made as these significantly affected the solution.

- The CPT results indicated that the depth to competent foundation material (assumed to be weathered siltstone) increased markedly with distance from the road. Below the road, the competent material was 9 metres below the road. At about 14 metres out from the road, the competent material was 21 metres below road level.

- Preliminary design options were then discussed with the client, TNZ. Based on the CPT results it was decided that a "high level" geogrid wall constructed as close as possible to the existing alignment was the most cost effective option.

- The final design solution was chosen which balanced traffic safety, construction safety, cost and aesthetics. The factors that had the greatest impact of the design were the depth to a suitable foundation in the highly weathered subsoil materials and groundwater conditions.

- The final solution involved an 11m reinforced earth slope, on top of which was placed a reinforced wall with concrete facing. This enable us to put the road back where it used to be, and include active safety measures such as roadside barriers. The solution also included piping and controls on stormwater.

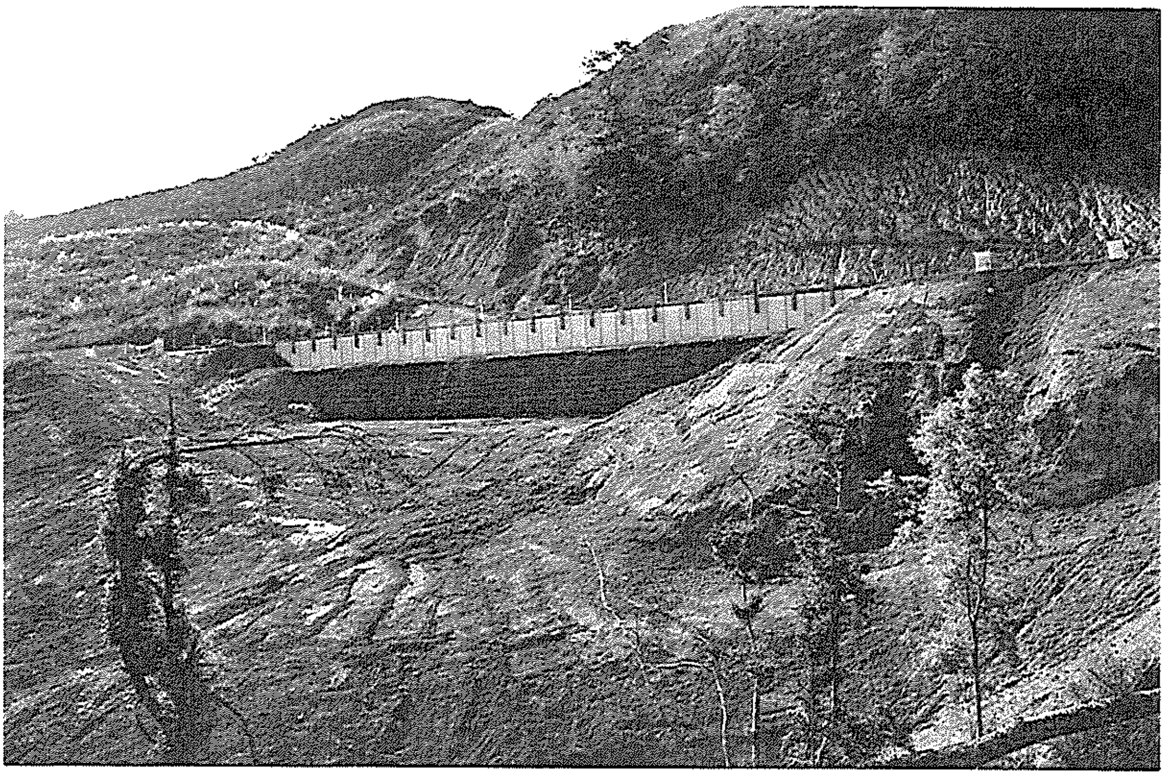
- The construction process at this site has included a number of "hold" points, in which in particular the condition of the foundation and construction backslope have been assessed. The construction required a temporary batter of 0.5H: 1V over 11m be cut in the back of the site. One way traffic continued at the top of this batter. We considered that this was "stable" but allowed a contingency plan in the contract for the detouring of traffic if the situation changed. A suitable road detour had been found and discussed with the affected Local Authority before construction started. This meant that the detour could be operated at any time, and at short notice.

- Even though site testing was required in the contract (foundation and material testing, particularly compaction control) we have been disappointed with the contractors understanding of and completion of testing.

- The repair was completed in May 1998, at a cost of approximately $400,000.

The following photographs give you some insight into this project.

Photograph 1: Mohaka Hill, lower excavation and beginning reinforced slope



Photograph 2: Mohaka Hill, nearly finished repair

8. Conclusions

The investigation and design of slope repairs and dropout repairs on low traffic volume roads can be affected by the available budget, particularly if the client is seeking the "perfect" solution without being prepared to fund adequate investigation works. As a small geotechnical group involved with small to medium sized repair projects (30K to 500K) we have found that by maintaining a close open relationship with an informed client we can produce "cost" effective solutions without being required to carry too much "risk". Even though we strive to provide sound, safe solutions, by working with the client on the issues associated with projects during the design process we believe that the majority of projects completed in recent years have been "cost effective " and appropriate to the low traffic volume environment in which they have been built.

References

SLOPE\W version 3.03. GEO-SLOPE International, Calgary, Alberta, Canada.

HIGHWAY SLIP REMEDIAL MEASURES WITHIN THE AUCKLAND/NORTHLAND ROADING NETWORK

By: Glyn R W East
OPUS International Consultants Ltd

SUMMARY

A review has been made of the range of slip remediation methods carried out on the Auckland/Northland roading networks in recent years.

Slips on highway networks have been accepted in the past as part of an ongoing maintenance and shape correction regime but in recent times the avoidance of disruption to traffic flow, urbanisation pressures, and the requirements of the RMA have lead to more positive permanent repairs. In addition, slips that could be repaired with a relatively simple earthwork and drainage approache have been repaired, and the remnant require more technical solutions.

The slips encountered in the Auckland/Northland highways are medium in size. Dimensions in the order of 30 to 50 metres length and 10 to 15 metres height, affecting some 50 metres of highway would be typical of the larger underslip dropouts on the highways. As such, they have practical and economic solutions for repair with a construction price within the range of $100,000 to $300,000. Examples discussed will include shear key "digouts", the "deep pit-bored horizontal drain" system, gabion and steel mesh or geogrid reinforced earth, and close centred cantilever and tied back pile walls. It will be shown that each method is unique to the geology and topographic conditions. The review also includes an outline of the practicality and design philosophy relative to the topography and geology. Construction pitfalls and typical costs of each system will be outlined.

INTRODUCTION

The repair of highway slips has become more sophisticated in recent years. More permanent, lower risk, and less disruptive solutions are required. The reasons for this include:-

- Highways have become more efficient with denser traffic and any unnecessary disruption has become unacceptable and costly.
- The historic slips that could be repaired by relatively low cost methods using drainage, buttressing, and retreats have been repaired, leaving the more complex slips that require more costly methods.
- Designers and highway owners are more conscious that drainage methods of repair should be able to be monitored and maintained which may lead to some forms of drainage being impractical.
- Land take for retreat/realignments has become more difficult for both environmental and cost reasons.
- Urbanisation pressures and seal extensions have made slips unacceptable on roads with rapid increases in traffic volumes.
- Historic slips that have high maintenance costs are being repaired.
- Slips where low cost or inappropriate repairs were utilised have subsequently failed.
- First time event slips that are the result of cyclones need to be repaired expediently and permanently to avoid traffic disruption.

As in all cases of remediation of slips the cause and source of the instability needs to be assessed with some degree of certainty. A knowledge of the geology and hydrogeology is essential. Site investigation is almost essential and the cost of such can be a high percentage of the ultimate repair cost. Investigation costs in excess of 10% of the repair cost would not be uncommon for the medium sized slips that occur in the Auckland/Northland roading network. Without site investigation no confidence can be give to any remediation repair.

The following outlines the range of typical methods of slip repairs undertaken on underslips or dropouts on the Auckland/Northland highway networks with the order of all up cost that would be expected. Some examples are attached. In all cases some form of effective drainage is added to the solution to control the ground water level and further enhance the repair.

SHEAR KEY "DIGOUTS" (UNDERSLIPS)

Setting: Situations where the instability in a moderately steep slope has been initiated at the toe and some remnant material remains on the slope. In general the highway is partially within a slump graben and the slip is progressively regressing. Highway sideling fill over unstable slopes are a typical example.

Construction Methodology: A key in the form of a designed excavation is progressively excavated and backfilled into the lower portion of the slip mass and through the slide zone, preferably into "bedrock". A well graded durable rock up to 100 mm size is used as backfill. "Brown" rock has been found to be acceptable as a backfill. Shear keys with depths of up to 6 metres have been constructed and appropriate safety precautions are required as part of the design and construction.

Basic Geotechnical Philosophy: To restore the stability by increasing the shear resistance of the shear zone using a high strength material.

Geology: The method is appropriate in situations where the instability is sufficiently shallow near the toe that the slip zone can be intercepted by an excavation below the toe. The topography must be such that there is sufficient near horizontal ground beyond the toe to reconstitute the batter with a stable slope angle appropriate for the resulting composite shear strength.

Requirements: That the shear strength of the key material (generally granular) and quantity of this material develops sufficient shear resistance such that, combined with the residual strength of the remnant slip plane, the overall slip surface(s) have sufficient composite shear resistance to maintain stability of the reconstituted road and slope.

Design: Traditional stability analysis appears adequate for design. Back analyses of the remnant slip is used to assess the residual strength of the existing slip zone. Care is required to model realistic conservative piezometric conditions.

Practical considerations:

- Compaction of the key is limited due to the time the excavation can be left open at any one time without inducing further instability, and the practicality of compacting in a deep excavation. Compaction will be largely due to end dumping (say 70% to 80% maximin density) and friction angles of no more than 30 to 35 degree should be used for design. Sensitivity analyses should be carried out over a range of values.

- Drainage of the shear key is not always practical and effective stress conditions will result in a reduction of the shear resistance of the key

Costs: These are highly dependant on the aerial extent of the instability and the volume of the material required to rebuild the road and slope. From past repairs this can be based on \$60/m^3 to \$80/m^3 of replacement material.

DEEP PIT DRAINAGE (SPIDER DRAINS)

Setting: Situations where the instability is intermittent and deep seated with most of the remnant material remaining on the slope. In general the highway is partially within a slump graben. Creep type instability in Northland Allochthon (Onerahi Chaos) soil/rock would be a typical example, where the intermittent movements can be put down to fluctuating piezometric levels.

Construction Methodology: A shaft of nominal 3 metres diameter is taken down to beyond the slip plan and sectionally lined to form a chamber. A horizontal boring machine is lowered into the chamber and a water egress drain is bored downslope before a fan of drains are bored into the slip mass. Commercially available 40 to 50 mm diameter slotted PVC (Class C) has been used for the drains. Appropriate air circulation is required. Depths of up to 10 metres have been used. A proprietary "kitset" chamber is made in Whangarei.

Basic Geotechnical Philosophy: To rectify the instability by increasing the effective shear resistance of the shear zone by reducing the piezometric head within the slip plane zone.

Geology: The method is appropriate where the instability is caused by a high piezometric head within a weak soil mass. It has most promise in confined aquifer situations such as within the Northland Allochthan (Onerahi Chaos) sheared rock geology.

Requirements:

- That the slip plane and/or position of aquifers are identified such that the pit depth and bored drain length and angle can be targeted. In the very fractured siltstone of the Northland Allochthon inclinometer monitoring may be required.

- In general the bored horizontal drains are fanned such that drains are at some 3 to 5 metre spacing where the arc on drains intercepts the head scarp. Vertical angles on the drains are intermittently varied from the target angle by 2 to 5 degrees and/or aquifers are progressively targeted with a "hands on" approach. To alleviate regression of the slip the drains are taken some 5 to 10 m beyond the head scarp.

- For practical drain spacing the permeability of the target areas should be greater than 10^{-7} m/sec to successfully control the aquifer.

Design:

- Traditional stability analyses appear adequate for design. Back analyses of the remnant slip is used to assess the residual strength of the existing slip zone.

- Care is required to model realistic conservative piezometric conditions that will develop from the drainage. Bored drains decrease the piezometric head around the drain with a cylindrical distribution but may not significantly alter the top line. A 50% overall piezometric drawdown on the slip plane seems an appropriate approximation in typical situations.

Practical considerations: To date this method has been surprisingly free of construction complications. The limitation to the method is the aerial coverage from one "deep pit". The range of the light boring machines is some 50 to 70 metres length and with a typical 120 degree drainage arc, the maximum length of head scarp that can be drained is some 150 metres. In a number of cases more than pit has been used.

Costs: These are dependant on the area of the unstable mass. The all up cost of a systems having one deep pit incorporating a fan of drains covering a triangular area of some 2500 m2 (50mx50mx100m) is some $100,000 to $150,000 including the road reinstatement.

MECHANICALLY STABILISED EARTH WALLS FOR UNDERSLIPS

Setting: In general MSE walls are only practical in situations where the slip mass within and adjacent to the highway has been completely removed by the instability and the reinforced "block" is used to reconstitute the highway. The fully saturated remoulded slurry type slips are the likely candidates.

Construction Methodology: Two types of MSE reinforced earth walls have been utilised predominantly on rural roads.

- The gabion facing with the steel mesh ("Terramesh") has been the usual method. The steel mesh requires a granular backfill within the reinforced "block". Recent developments and research has indicated that the mesh can be used with a low plasticity fine grained soil but such soils are not readily available in the Auckland or Northland

- More recently the "wrap around" thermoplastic geogrid reinforced walls with a nominal 70 degree face slope have been used. This type of reinforcement allows the used of fine grained cohesive soils conditioned to the appropriate water content for a compacted backfill within the reinforcement "block". Once completed the face can be grassed. Such walls require positive drainage at the back of the "block" which is the major difference between the two types. A 13 metre high "wrap around" wall has recently been constructed near Mangawhai.

- One of the choices required to be made between the two types of reinforcement is the relative cost between importing granular fill, and the conditioning of the local cohesive borrow soil.

- A further recent development is the use of steel ladder s with a keystone or mesh panel face. The economics of this system for rural type highway slip has yet to be established.

Basic Geotechnical Philosophy: To stabilise the highway by completely reconstructing the slope using an unnaturally steep batter made of material reinforced by strips of thermoplastic geogrid or corrosion resistant steel strips ladders or mesh. The overall stability and the batter slope stability is maintained.

Construction Methodology: The method is appropriate where a firm stable base of sufficient width for the lower reinforcement length is available at or near the base of the slope. Some undercutting is generally required to penetrate the remnant slump material or to bench in the base.

Requirements: A firm stable base of sufficient width for the lower reinforcement length

Design: Traditional MSE analyses (Jewell) appears adequate for design although suppliers of proprietary systems are using variations of this type of wedge analyses and supply software for their individual products. .

Practical considerations:

- Little problems have been experienced with the steel mesh gabion system apart from sourcing an economic rock backfill. Contractors are familiar with the gabion type systems.

- Apart from the requirement to condition the soil the wrap around geogrid reinforcement system has some difficulties problems for an inexperienced contractor. The wrap around nose is difficult to construct in a controlled manner without shutters. While this nose does not play a significant part in the design it must be well constructed to allow the next layer to be compacted. Construction sequence problems can also occur with the drainage layer behind the wall. This drain, of nominal width 300 to 600 mm, is progressively laid behind each tier of the wall and contamination must avoided and continuity maintained. The use of a separation geotextile is considered impractical.

Costs: These are largely dependant on the length and depth of the unstable mass (area of retention) and the type of reinforcement backfill. A steel grid/mesh system that require the importation of granular material is generally more costly than a plastic geogrid system that can use local soils for backfill but this can be out weighed by the requirement to condition the local soils and the somewhat more difficult construction procedures of the "wrap around". As the volume of soils within these systems are generally proportional to the height of the walls costs can be related to the area of the exposed face. An all up cost of $600/m^2 to $700/m^2 of wall face would be typical inclusive of road reinstatement. To distinguish between the two reinforcement systems is site specific.

BURIED OR PARTIALLY BURIED PILE WALLS (PILE PINNING UNDERSLIPS)

Setting: In general these are only practical in situations where a toe driven regressive slip mass within and adjacent to the highway is largely still in place. The cyclic movement and creep graben type slips are typical candidates.

Construction Methodology: To install piles at close spacing to allow arching (< 2.5D to 3D) embedded at a designed depth below the slip plane. The piles can be cantilever or tied back and constructed of reinforced concrete or timber (or steel) depending on the design conditions. Tie back piles have used a deadman footing at a designed depth and distance behind the pile heads. The ties are mild steel bars connected to a whaler and drilled under the road. Slips with dropped grabens have the piles extended to highway level with ground support planks. The system retains the road isolating it from the slip but in most cases bored horizontal drains are installed below the wall to enhance the downslope stability. .

Basic Geotechnical Philosophy: To stabilise the highway by supporting that portion of the remnant slip that contains the road leaving the lower portion of the slip unstabilised.

Geology: The method is appropriate where a harder strata underlies the unstable zone in the upper portion of the slip. Maximum depths to hard of some 12 metres have allowed an economic design

Requirements: That the slip plan is identified such that the retention length and pile depth can be designed and constructed.

Design: Traditional solder pile retention analyses appears adequate for design with the proviso that the active thrust is obtained from a stability analysis of the retained portion of the slip. Depending on the size and geometry of the retained section of the slip the forces can far exceed traditional Active Force (Pa) conditions. Although drainage can be undertaken in front of the wall to alleviate downslope instability the wall is design with no support in front of the wall (Pp =0) above the slip plan, on the assumption that this material will continue to remain unstable and move away from the front face.

Practical considerations:

- Little problems have been experienced with the piling. Contractually the piling contractor must have a knowledge of the material to be drilled through. The investigation should include a rig bore for this reason in addition of obtaining strength parameter for embedment design. Some care is required regarding the line and vertical angle of the pile group as it is very difficult to line up the planks if adjacent pile are not parallel.

- The tie back deadman can be buried some 3 to 5 metres in a relatively restricted road reserve. The excavation can endanger the highway and adjacent property unless the design and contract calls for shoring and deadman reinforcement cage is segmental such that the excavation can be undertaken progressively.
- The tie rod are typically placed through holes drilled under the pavement to avoid traffic disruption. Problems have occurred when large size aggregate used to develop the road have been encountered in the drilling. The alternative method of trenching can be substituted and the traffic disruption and pavement reinstatement allowed for in the contract.

Costs: These are largely dependant on the length and depth of the unstable mass (area of retention). A decision to use a deadman tieback system in place of a cantilever system requires an economic assessment of the particular site as well as the practical aspects of installation. For a given condition the use of a tie back system reduces the size and embedment length of the pile and the costs can be similar. As the depth of retention reflect the costs the costs are proportional to area of retention. An all up cost of $800/m^2 to $1000/m^2 of retention height would be typical for a reinforced bored concrete pile system including reinstatement of the road.

EXAMPLES

Shear key "digouts" (underslips)

Fergus Slip - SH1 Brynderwyn Hill - See attached plan.

Construction: A partial slurry type slip in mixed Greywacke rock and embankment fill that occurred during a cyclone Fergus when a large diameter culvert blocked. The failure left a scarp face of some 15 metres high above a stream between two greywacke spurs taking the full highway width overnight. A temporary road was constructed around the slip. The remedial system consisted of a granular shear key and a reconstruction of the slope including a chimney drain behind the new fill. Included in the construction was a new culvert interconnecting the upslope gully with the stream. Bored drains were also installed in the adjacent natural spurs.. The volume of reinstatement was some 5500m^3.

Cost: $320, 000 all up including the road reconstruction, ie unit cost of $60/m^3 of volume replaced

Deep pit drainage (spider drains)

Prides Slip SH16 - see attached plan.

Construction: A slip in Northland Allocthon (Onerahi Chaoes) soil/rock where the depth of the slip plane was identified by inclinometer monitoring. The remedial system consisted of one deep pit chamber installed at 7 metre depth with 24 x 50 metre long bored drains over and a 150 degree arc and inclined at some 3 to 5 degrees ie an area measurement of some 3300 m^2.
Combined flows of some 2000 litres per day resulted from the system.

Cost: $120, 000 all up including the road reconstruction, ie unit cost of $4/$m^2$ of area drained.

Mechanically stabilised earth walls for underslips

<u>Gabion - Steel Mesh Reinforcement</u>

Hinua Road Muriwai - see attached plan.

Construction: A slurry type slip in sideling fill that occurred during a cyclone that left a gapping scarp taking most of the road. The slurry had deposited over the lower private property some 6 metres below. The remedial system consisted of a 6 tier steel mesh gabion system over 18 metres length. Face area of retention is 108 m^2

Cost: $70,000 all up including road reinstatement, ie unit cost of $650/$m^2$

<u>Wrap Around - Geogrid Reinforcement</u>

Mangawhai Road - see attached plan.

Construction: A slurry type slip involving natural weakly cemented Waitemata Group soils and sideling fill that occurred during a cyclone Fergus and left a scarp face of some 13 metres high above a gully, taking half of the road. The remedial system consisted of 34 400mm to 600mm tiers of geogrid system over a 60 metres road length. A further 3 tiers were keyed down to rock. Triangular face area of retention is 400 m^2.

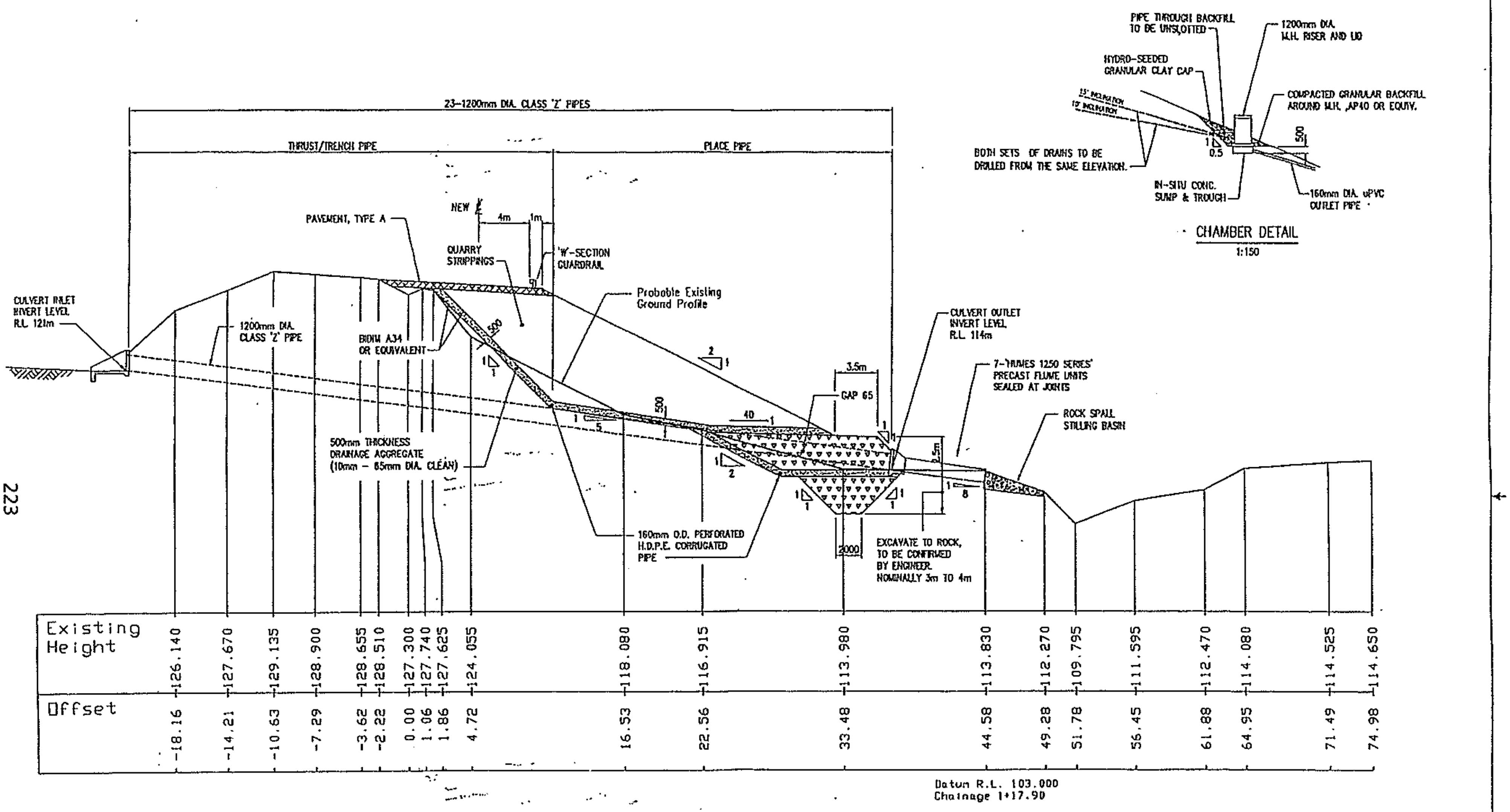
Cost: $270,000 all up including road reinstatement, ie unit cost of $670/$m^2$

Buried or partially buried pile walls (pile pinning underslips)

Barratt - Boyes - Waiwera SH1 - See attached plan.

Construction: A tied back bored reinforced concrete pile wall of 13 piles over 24 metres (800mm diameter) supporting 11 metres of slip mass (old fill) over Waitemata Group "bedrock". The graben required the piles with planks to extend 2 metres above ground to reconstruct the highway. Face area of retention is some 260 m^2. Although not critical to the designed bored hotrizontal drains were also installed below the wall to alleviate further movement of the slip mass below the wall.

Cost: $215, 000, ie unit cost of $830/$m^2$ of face area.



CROSS-SECTION A-A

1:150

	BY	CHECKED	DATE
SURVEY			
DESIGN	S.HOOKS	A.BROWNE	4-97
DRAWN	S.CORNTHWAITE	K.MEHAN	4-97
RECOM'D	G.EAST		4-97
APPROVED:			

R1 OUTLET STRUCTURE AMENDED 4-97

STATUS CONSTRUCTION

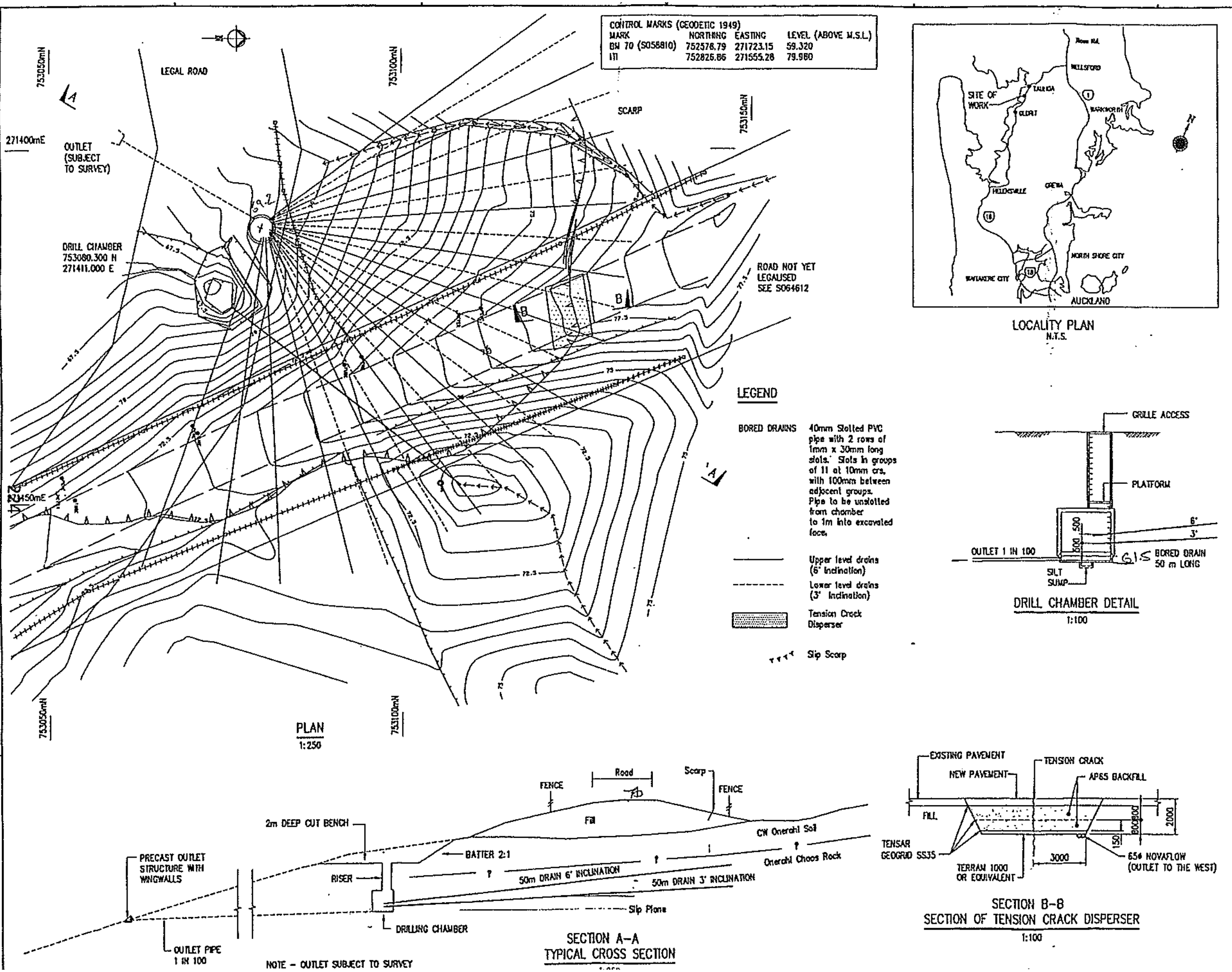
TRANSIT NEW ZEALAND

OPUS Auckland
PO Box 5848 Auckland, New Zealand
Tel: (09) 355 9500 Fax: (09) 355 9590

TITLE: S.H.1 REGION 1 R.S 203
SLIP REPAIRS AT R.P.203/13.70
SOUTH SIDE OF BRYNDERWYNS FERGUS SLIP

REMEDIAL MEASURES – SECTION A-A

SCALE	PLOT DATE	JOB	CODE	SHEET
AS SHOWN	21/4/97	9/ 17 /35	7104	2
		FILE 11196	CAD FILE 11196c06	REVISION R1



Rodney DISTRICT COUNCIL

WORKS Consultancy Services
Auckland

REVISIONS	BY	CHECKED	DATE
SURVEY			
DESIGN			
GEOMETRICS			
DRAWN	LH & P.D.W		3/25
RECORD			
APPROVED			

STATUS: PRELIMINARY

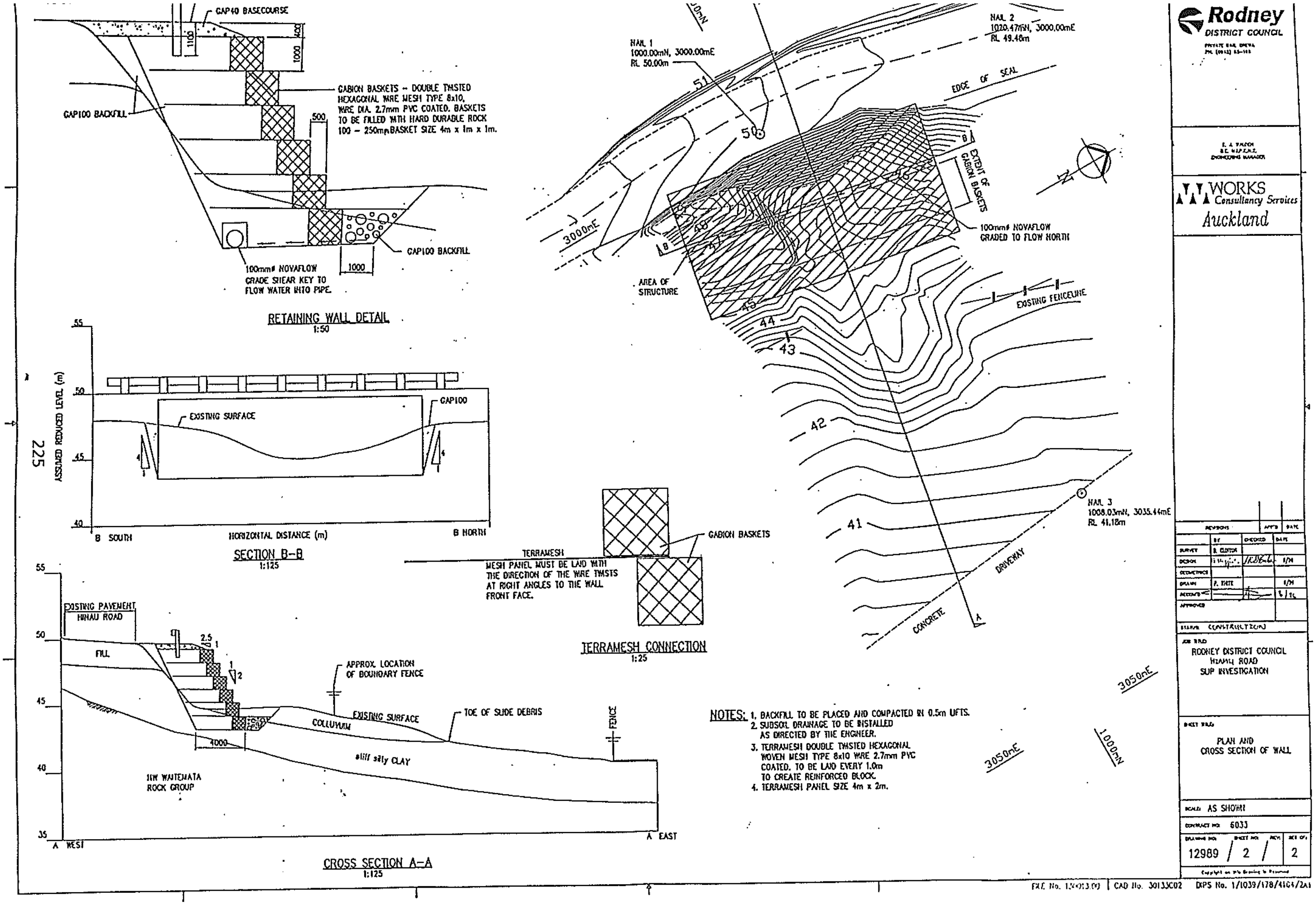
JOB TITLE: RODNEY DISTRICT COUNCIL
KAIPARA COAST HIGHWAY
PRIDES SLIP

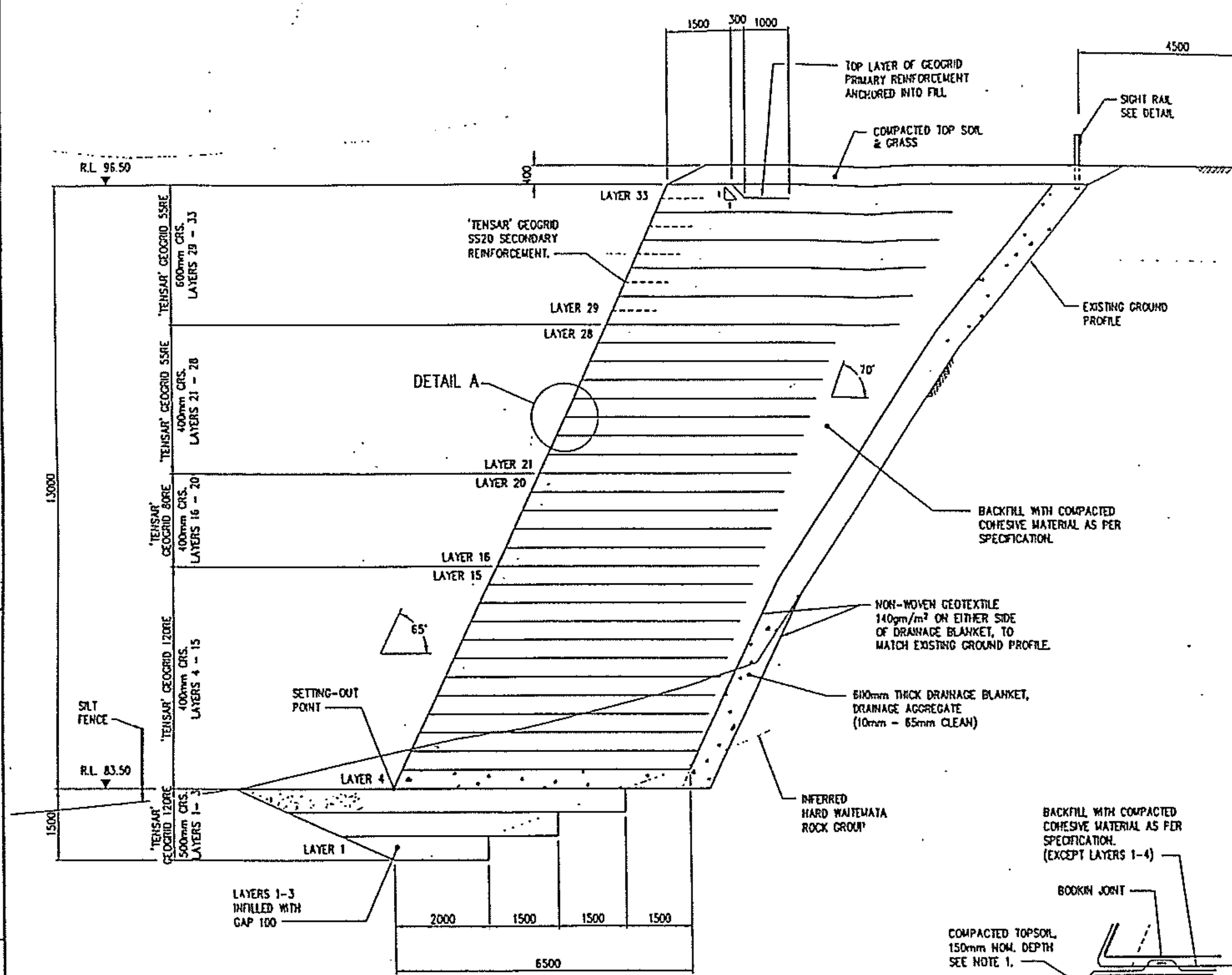
SHEET TITLE: PROPOSED REMEDIAL MEASURES

SCALE: AS SHOWN

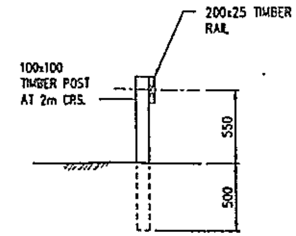
CONTRACT NO: 4046

A1


CAP40 BASECOURSE
GABION BASKETS – DOUBLE TWISTED HEXAGONAL WIRE MESH TYPE 8x10, WIRE DIA. 2.7mm PVC COATED. BASKETS TO BE FILLED WITH HARD DURABLE ROCK 100 – 250mm BASKET SIZE 4m x 1m x 1m.
CAP100 BACKFILL
100mm⌀ NOVAFLOW GRADE SHEAR KEY TO FLOW WATER INTO PIPE.
RETAINING WALL DETAIL
1:50
ASSUMED REDUCED LEVEL (m)
EXISTING SURFACE
CAP100
HORIZONTAL DISTANCE (m)
B SOUTH
B NORTH
SECTION B–B
1:125
EXISTING PAVEMENT
HINAU ROAD
FILL
APPROX. LOCATION OF BOUNDARY FENCE
EXISTING SURFACE
COLLUVIUM
TOE OF SLIDE DEBRIS
FENCE
stiff silty CLAY
HW WAITEMATA ROCK GROUP
A WEST
A EAST
CROSS SECTION A–A
1:125
NAIL 1
1000.00mN, 3000.00mE
RL 50.00m
NAIL 2
1020.47mN, 3000.00mE
RL 49.48m
NAIL 3
1008.03mN, 3035.44mE
RL 41.18m
EDGE OF SEAL
EXTENT OF GABION BASKETS
100mm⌀ NOVAFLOW GRADED TO FLOW NORTH
AREA OF STRUCTURE
EXISTING FENCELINE
DRIVEWAY
CONCRETE
3000mE
3050mE
1000mN
TERRAMESH
MESH PANEL MUST BE LAID WITH THE DIRECTION OF THE WIRE TWISTS AT RIGHT ANGLES TO THE WALL FRONT FACE.
GABION BASKETS
TERRAMESH CONNECTION
1:25
NOTES: 1. BACKFILL TO BE PLACED AND COMPACTED IN 0.5m LIFTS.
2. SUBSOIL DRAINAGE TO BE INSTALLED AS DIRECTED BY THE ENGINEER.
3. TERRAMESH DOUBLE TWISTED HEXAGONAL WOVEN MESH TYPE 8x10 WIRE 2.7mm PVC COATED. TO BE LAID EVERY 1.0m TO CREATE REINFORCED BLOCK.
4. TERRAMESH PANEL SIZE 4m x 2m.
Rodney DISTRICT COUNCIL
WORKS Consultancy Services
Auckland
STATUS CONSTRUCTION
RODNEY DISTRICT COUNCIL
HINAU ROAD
SLIP INVESTIGATION
PLAN AND CROSS SECTION OF WALL
SCALE AS SHOWN
CONTRACT NO. 6033
12989 / 2 / 2
CAD No. 30133C02
DPS No. 1/1039/178/4104/2A1

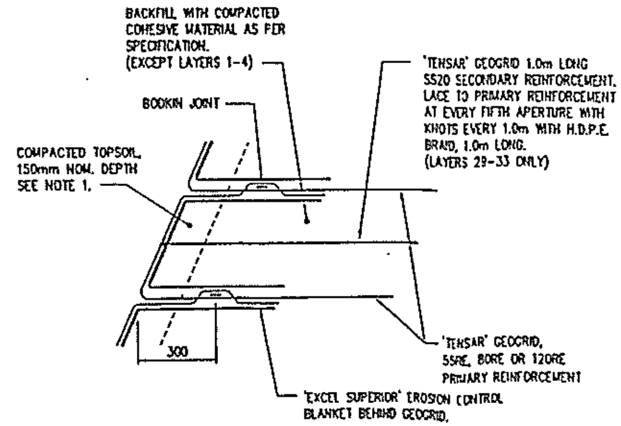


A – A
1:50



REFER TO SPECIFICATION

SIGHT RAIL DETAIL
1:20



DETAIL A
1:10

NOTES:

1. 50mm THICK WELL ESTABLISHED TURF MAY BE USED AS AN ALTERNATIVE TO THE EROSION CONTROL BLANKET AND TOPSOIL.
2. DESIGN LEVELS SHOWN ON SECTIONS ARE INDICATIVE ONLY. FINAL LEVELS TO BE DETERMINED FOLLOWING EXCAVATION.
3. WALL TO BE FOUNDED ON HARD WAITEMATA ROCK LAYER.
4. FOUNDATION TO BE CLEAR OF WATER AND LOOSE MATERIAL PRIOR TO PLACING OF GRANULAR FILL.
5. EXCAVATION MUST BE CARRIED OUT BY A METHODOLOGY THAT AVOIDS FURTHER

Rodney DISTRICT COUNCIL

Auckland

OPUS

STATUS CONSTRUCTION

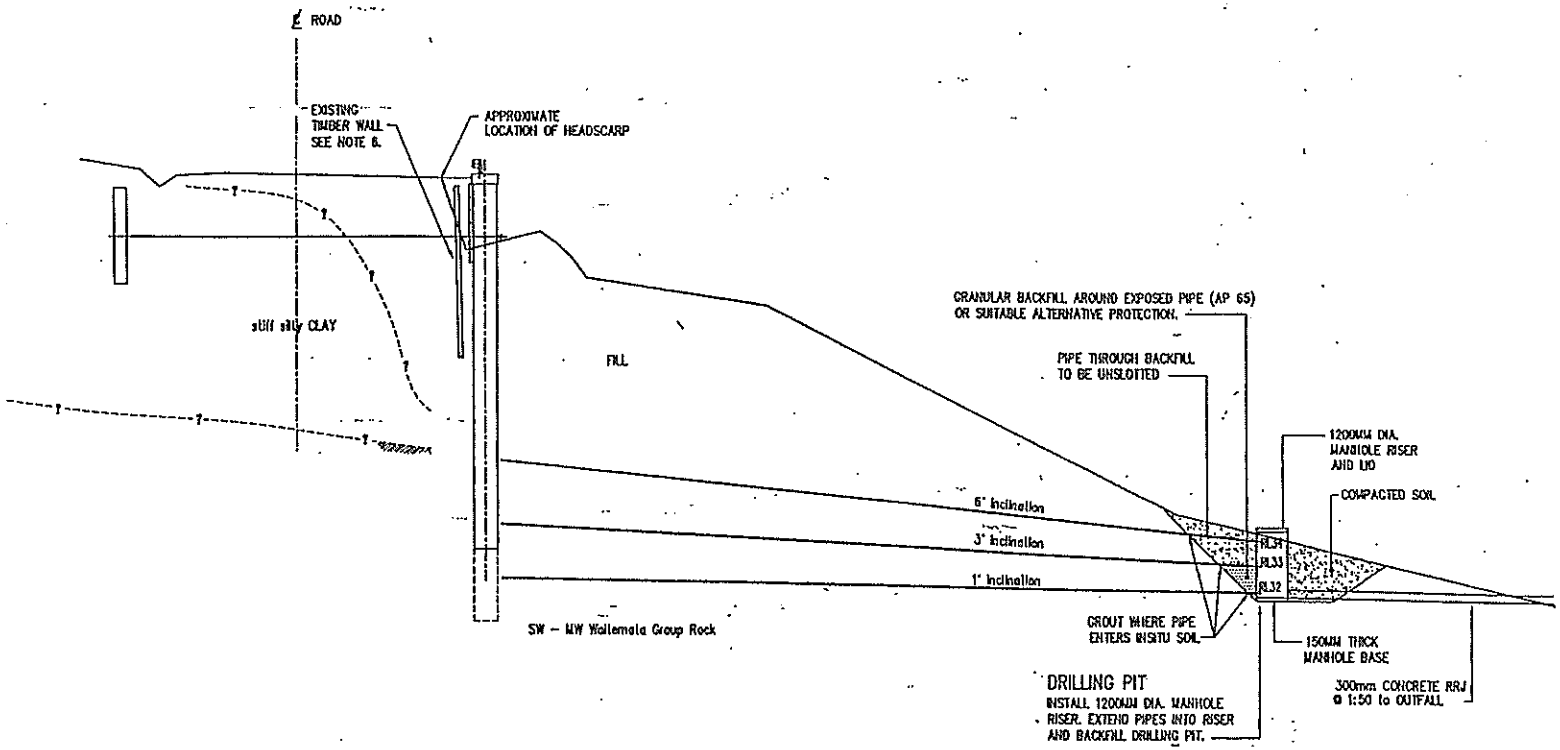
MANGAWHAI/CAVES ROAD SLIP REPAIR

TYPICAL SECTION

SCALE AS SHOWN

CONTRACT NO 6061

A1



A – A
1:100

FOR LOCATION SEE SHEET 1.

4. COORDINATES ARE IN TERMS OF MT. EDEN GEODETIC DATUM 1949 700000mN, 300000mE.
5. ALL LEVELS ARE IN TERMS OF MEAN SEA LEVEL AUCKLAND 1946.
6. EXISTING WALL TO BE REMOVED IF IT AFFECTS NEW CONSTRUCTION.

	BY	CHECKED	DATE
SURVEY			
DESIGN			9/96
DRAWN	A.S.C. / P.D.W		9/96
RECOMD			

TRANSIT NEW ZEALAND

WORKS Consultancy Services
Auckland

TITLE: S.H.1. REGION 2 R.S.268
SUP REPAIR AT R.P.268/1.45
BARRATT-BOYES SLIP

LONGITUDINAL SECTION A – A

JOB | CODE | SHEET

SH73 OTIRA VIADUCT PROJECT SLOPE STABILITY CONSIDERATIONS

Graham Ramsay

Beca Carter Hollings and Ferner Ltd

SUMMARY

Transit NZ is currently constructing a 454 m long viaduct on State Highway 73 down the upper section of Otira Gorge. The viaduct will bypass a section of road, referred to as the Zig Zag, which crosses actively retreating slopes in the toe area of a massive rock avalanche which dammed the valley some 2000 years ago and was subsequently breached by and, is being actively eroded by the Otira River. The evaluation of alternative options for a new road alignment at acceptable grades and also the design of the northern approach to the adopted viaduct solution has included consideration of the assessment of the stability of slopes which have been formed by natural processes which have left them in a state of uncertain and indeterminate stability. This paper describes investigations undertaken and the philosophy with respect to the slope stability of the cuts and reinforced earth fills on the viaduct approaches.

1. INTRODUCTION

State Highway 73 (SH73) is one of three routes crossing the mountain ranges that extend the length of New Zealand's South Island. The route is currently being upgraded by Transit NZ to provide a secure and safe route capable of taking conventional legal truck and trailer units. The upgrading consists of works to improve alignment, control and/or provide protection from rockfall, and to secure the route against natural river erosion and slope processes that currently threaten the long term security of the route.

This paper addresses one section of the upgrade, referred to as the Zig Zag, where the road crosses an activity retreating slope above the Otira River and a project is underway to move the road alignment onto a 454 m long bridge running essentially parallel to the river. At this point the Otira River was dammed some 2000 years ago by a massive rock avalanche which the river subsequently breached and is progressively cutting down and eroding. The southern approach to the new bridge involves a sidling cutting through the end of the former rock avalanche dam, immediately adjacent to the steep (20% gradient) channel cut down through the dam by the Otira River. Beyond the northern end of the bridge, the new road will traverse slopes in the rock avalanche materials, and is supported on a combination of reinforced earth walls and a short bridge.

This paper discusses the investigation and analysis of the slope stability of the cut slopes at the southern approach, the reinforced earth wall supported sections at the northern approach; and the slope stability considerations of an alternative alignment which involved a major slope cutting on the opposite bank.

2. GEOLOGICAL BACKGROUND

The geological history of the site is discussed in technical details in papers by Paterson [to come]. This section provides a general description of the conditions that exist in the project area with reference to the slope stability considerations.

Figure 1 shows an oblique aerial view of the project area with localities indicated. Some 2000 years ago a series of major rock avalanches occurred from the Hills Peak Ridge some 600 metres above the Otira River. These avalanches dammed the valley at Deaths Corner and created a reservoir which was infilled by the river to form the Peg Leg flat area. The rock avalanche material as observed and encountered in investigation boreholes comprises very strong (100-250 MPa UCS), very abrasive greywacke sandstone with sizes ranging from sand sizes to large boulders, metres in dimension, and including large widely

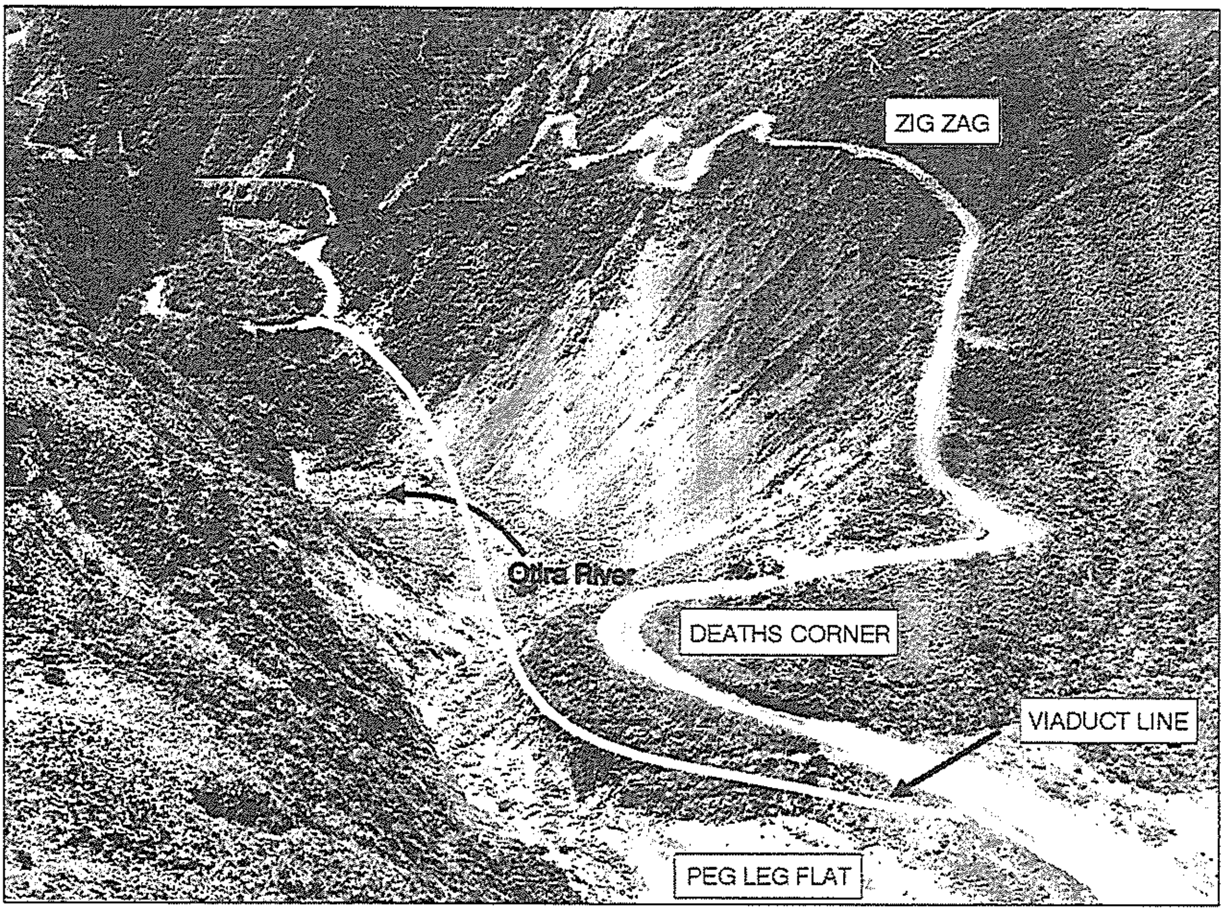


Figure 1 - Oblique View of Site

jointed and dilated slabs tens of metres in dimension. Slope angles in the eroded faces of the rock avalanche range from typical 35-38° in the actively eroding amphitheatre below the Zig Zag to 45-56° in the vegetated slopes below the Death Corner lookout.

Investigation drilling on slopes (as opposed to the valley floor) suggested the rock avalanche debris is generally free draining, as there was little if any return of drilling fluid and no indications of a hydrostatic water table. The drilling at Deaths Corner through the former landslide dam encountered substantial voids where the drill rods dropped and it is considered that in this area the finer sizes may have been piped out during the period when there was a full dam at this point. Currently a large proportion of the Otira River flow between Peg Leg flat and the gorge below the site occurs as underground flow. Investigation drilling below Deaths Corner and near Pier 1 encountered strong artesian ground water flows and there are also large springs discharging from the banks into the river in this area.

Since breaching the rock avalanche formed dam, the Otira River has cut itself a new channel on the western side at Deaths Corner and has actively eroded the toe of the rock avalanche deposits, which has resulted in local toe oversteepening, and slope regression. As a result of this regression it has been necessary to progressively relocate the SH73 roadway which currently crosses the top of these slopes. Over the period 1929-1992 the roadway has been moved horizontally some 64 metres eastward into the slope.

3. PROJECT BACKGROUND

Transit NZ and its predecessors have been aware that the active retreat of the slopes below SH73 at the Zig Zag would continue and the stage would be reached where moving the road further upslope and eastwards would become impractical in terms of the road gradients and alignments. In the mid 1980's a number of options were considered for an alternative route at the Zig Zag section. The options included an embankment against the toe of the slope (which would have required major river protection and upslope provisions to control rock falls and slumps), tunnels through the slopes on the western side of the valley (which were substantially more expensive solutions) and a number of bridge and tunnel/bridge options. The options are shown in Figure 2.

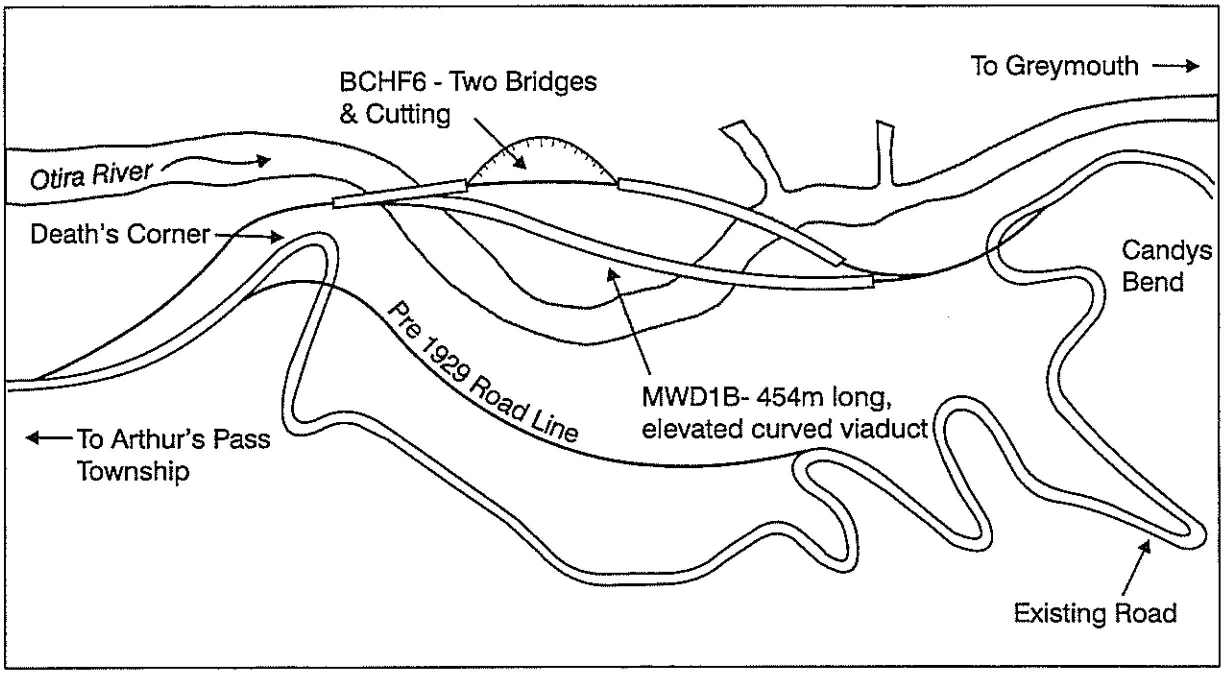


Figure 2 - Options Considered

In 1993 Transit retained BCHF to proceed with investigation design and construction of the MWD 1B option involving a bridge approximately 450 m long down the centre of the valley. Investigations and hydraulic evaluations disclosed complex and potentially difficult foundation conditions and a need for extensive allowances for scour provisions and erosion protection at pier locations and against the northern approach fills. -

Subsequently, Professor Christian Menn, a Swiss bridge design expert retained by BCHF as a peer reviewer, recommended that, if possible, a solution should be adopted which avoided the risks associated with the construction of and scour prediction provisions for piers near the highly active Otira River. As a result, west bank options involving a major cutting and shorter bridges spanning the river channel were re-examined. These options, BCHF5 and BCHF6 are discussed below.

4. GENERAL INVESTIGATION AND DESIGN APPROACH

4.1 GENERAL

Slope stability assessments were required to address three significantly different situations.

(a) a cutting within the rock avalanche mass at Death Corner

(b) a cutting for the west bank BCHF5 option which was expected to be in the bedrock on the side of the valley

(c) stability of slope modifications on the northern approach involving the construction of a reinforced earth block on a bench cut in the slope.

The form of investigation and analysis was different in each case and reflected the ability to characterise the materials present in a form suitable for analysis.

4.2 SOUTHERN APPROACH SLOPE STABILITY

At Death Corner observations indicated the presence of large boulders in the rock avalanche material. One borehole was drilled as close as possible to the cutting location and also indicated large size material with apparent voids. Survey of the slopes on the northern flank of Deaths Corner indicated natural slopes of around 45°. The form of the topography meant that the volume of earthworks would not increase dramatically with flatter slopes. A slope profile of 1½ horizontal to 1 vertical (33°) was adopted on the basis of the observed slopes with the major concern being the manner in which large boulders encountered in the cut slopes would be treated. Conditions exposed during construction involved a greater than anticipated volume of large boulders and blocks (used as rip rap) but with less than anticipated voids and a greater degree of matrix. No particular problems were encountered in constructing the batter slopes.

4.3 WEST BANK OPTIONS

The BCHF5 option was originally proposed on the assumption that a steep cutting could be formed into the western side of the valley in bedrock. The proposal assumed removal of the veneer of rock avalanche debris together with any surficial relaxed zones of the bedrock. Contours of the bedrock/rock avalanche interface were predicted by extrapolating between the observable surface exposures of bedrock. Examination of the condition of bedrock on the western side of the Otira River at Deaths Corner, and in the sides of Curtiss Creek immediately north of the proposed cutting suggested that bedrock would be competent and likely to stand at slopes around ½-¼ horizontal to 1 vertical.

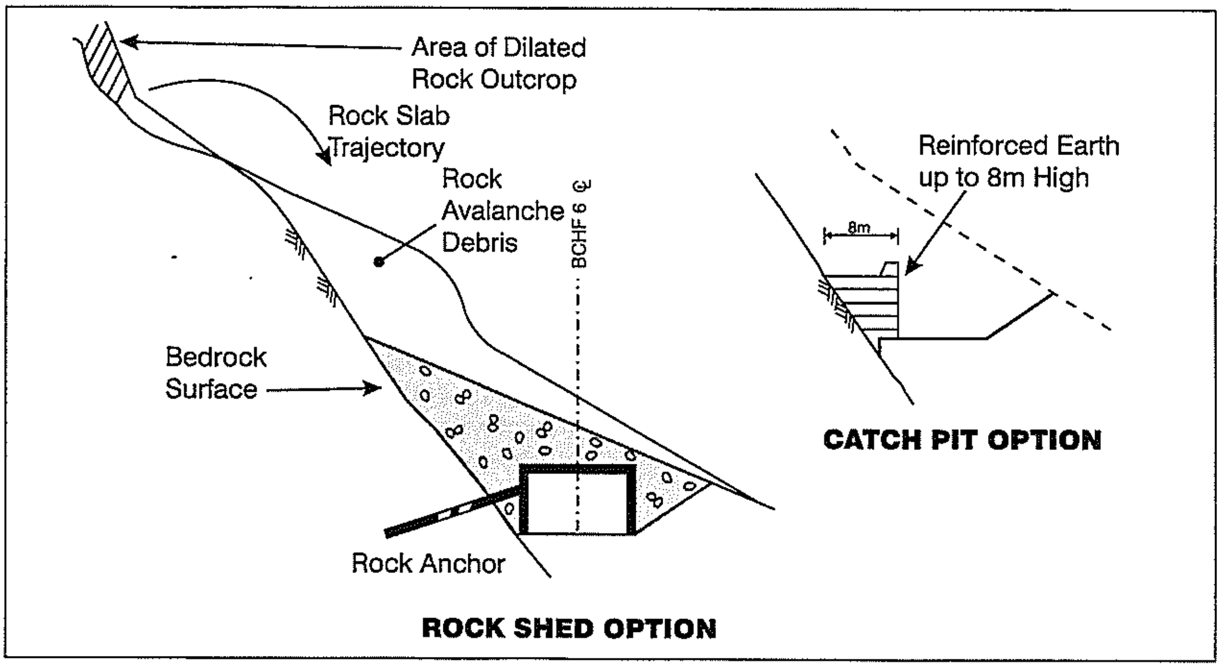


Figure 3 - West Bank Alternative-Rockfall Protection Options

Investigations, to confirm the thickness of the rock avalanche veneer and the anticipated bedrock competence, were carried out in the form of inclined boreholes drilled from benches established on the slope. Core recovery of the bedrock with core orientation, was programmed. In the event, the investigations demonstrated that the rock avalanche veneer was thicker than anticipated and the bedrock was found to be relaxed and heavily weathered near the surface of the natural slope beneath the avalanche material. As a consequence it was concluded that the BCHF5 alignment was not practical and an alternative alignment, BCHF6 was developed which allowed the road to be positioned at the toe of the original natural slope profile which would be uncovered and scaled as part of the works. A concern with this option was the risk of continuing rockfalls from the exposed natural slope. In comparing the BCHF6 option with the original bridge solution, allowance was made in the estimates for either a rock shed or catch pit elevated on a reinforced wall as shown in Figure 3. The costs of the BCHF6 option and the long bridge option were similar and on review it was concluded that the bridge option provided lower risk.

4.4 NORTHERN APPROACH

The northern approach to the viaduct sidles along slopes of the rock avalanche which are vegetated and exhibit no indications of current or recent major instability. In this area the rock avalanche toe is on a previous river terrace with the Otira River being entrenched into a gorge in the bedrock. The accommodation of the road along the slope has been achieved in some sections by a reinforced earth wall up to 7 m high where this can be benched into the slope, and by a short bridge over one section.

The rock avalanche materials are widely ranged in size and it is difficult to assess typical strength parameters for analysis. The existing slopes have overall slopes of 38° minimum, and locally where there are large blocks, or the slope has been temporarily oversteepend by slope retreat, much steeper slopes occur. The local slope has survived a number of major earthquakes, and therefore the existing slope angle is very conservative estimate of the strength parameters. For the analysis of the Reinforced Earth walls, three aspects were considered:

(i) Local stability near the reinforced earth block (Figure 4a).

(ii) Overall slope stability (Figure 4b).

(iii) Internal stability, base shear and overturning of the reinforced earth block.

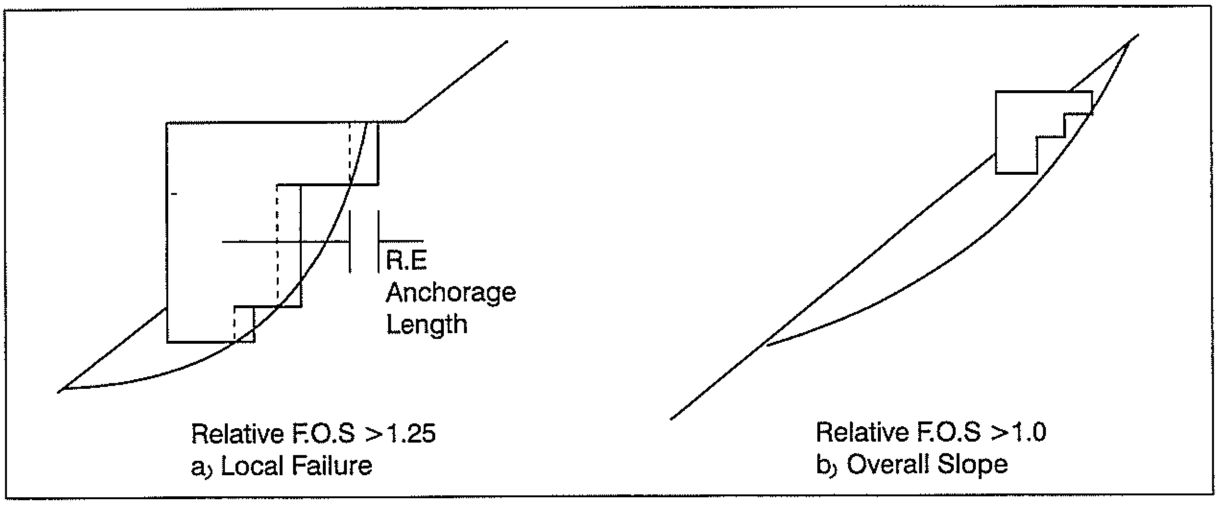


Figure 4 - Northern Approach-Failure Mechanisms Analysis

A minimum size of reinforced earth block was determined assuming C'=O,Ø=38° and a Factor of Safety of 1.25 against failure along the surface shown in Figure 4a. Local shallower failure surfaces which would normally have a lower factor of safety are not critical as the surfaces cross and are restrained by the reinforcing. The reinforcing strips extend beyond the minimum block size specified.

For the overall slope stability of the type shown in Figure 4b, the analyses indicated a factor of safety of greater than 1.0 (relative to the existing slope) for failure surfaces extending 40 m down slope. For longer failure slopes the factor of safety is less than 1.0 but for slope failure over that length, factors such as three dimensional effects will minimise the effect of the localised RE block. The design of the reinforced block was undertaken by the contractor.

No specific slope analyses were undertaken for seismic effects although the Reinforced Earth wall blocks were designed and detailed for internal seismic earth pressures. Analyses indicated that horizontal movement of the RE block of up to 100 mm might be anticipated in the design earthquake.

The rationale for acceptability of seismic security of the overall slopes was that there was no evidence to suggest recent movement of the slopes, apart from localised shallow slumping. It was not considered possible to establish realistically parameters for seismic slope stability analyses and the scale of the slopes meant that conventional 'pseudostatic' analyses approaches were not likely to be meaningful. Finally the works undertaken (localised reinforced earth walls) are insignificant in the overall scale of the slopes.

5. ROCKFALL ANALYSIS

In addition to slope stability assessments the project required the assessment of rockfall risks and where necessary the provision of protection for the road and bridge piers. The current road is subject to rockfall particularly during earthquakes. These rockfalls originate from the bluffs on the Hills Ridge. An analysis of rockfall trajectories was undertaken by Golder Associates using the CRSP computer program. The analyses indicated that rockfalls could reach the Pier 2 location on the opposite side of the river. However, because of the shape of the slopes, it was assessed that rockfalls from the Hills Ridge slopes would be 'focussed' towards the middle of the active failure amphitheatre and would not reach the other piers or the approach roads. A bund structure was provided to protect Pier 2 from impact by rockfalls.

The Pier 1 location is at the toe of an actively eroding slope that has several large blocks partially exposed. It is believed that ongoing natural erosion of the matrix material is likely to result in these blocks being released during the life of the bridge. It is anticipated that the blocks will roll/slide rather than bounce and to protect Pier 1 from impact, a deflector structure comprising a triangular pattern of piles with a capping beam was provided.

6. CONCLUSIONS

The Otira project is similar to many roading projects in major river gorges in that the sides of the valley on which the road is to be constructed have been formed by slope failure. The geological conditions (in this case the rock avalanche origin of the material and its large range of sizes) make it impractical to establish representative soil and groundwater parameters and thus carry out the deterministic analyses of slope stability factors of safety required or implied by the Building Act and other design criteria.

For the Otira project analysis of relative factors of safety of the northern approach slopes indicated that near surface stability can be enhanced by the presence of reinforced earth wall blocks which effectively interrupt shallow failure surfaces.

7. ACKNOWLEDGMENTS

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EFFECTS AND MITIGATION OF ROCK-FALL HAZARD SH73 ARTHUR'S PASS HIGHWAY, SOUTH ISLAND, NEW ZEALAND

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SUMMARY

Rock fall presents a serious hazard to travellers using Arthur's Pass Highway (SH73), particularly along the Otira Gorge-Zig Zag section where dilated rock and unconsolidated slope deposits on steep slopes above the highway have been destabilised by recent local earthquakes. Earthquakes in 1994 and 1995 initiated rock falls in numerous locations and have led to ongoing highway repairs and slope stabilisation measures. A programme of monitoring dilation of tension fractures above the Zig Zag, the source of the Otira rock avalanche, was initiated after large blocks of rock detached from the main scarp and impacted on the highway 600 m below. Although completion of the Otira Viaduct should reduce the hazard of similar events, additional rock-fall protection measures will be required in Otira Gorge where the highway is to be upgraded.

1. INTRODUCTION

State Highway (SH) 73 is one of New Zealand's main transalpine highways which crosses the Southern Alps and links the main centres Christchurch and Greymouth, located respectively on the east and west coasts of South Island, New Zealand (Fig. 1). Since the route was first established for horse-drawn coaches in 1866, it has been subject to numerous closures from natural events such as landslides, earthquakes, snowfalls and rainstorms, particularly the section of highway between Arthur's Pass and Otira villages, within Arthur's Pass National Park, where the terrain is highly mountainous.

The highway serves as a strategic route of national importance and is a popular scenic route for tourists visiting the National Park, or crossing the South Island. Increased use of the highway for passenger traffic and road freight has attracted significant funding for major highway improvements, including the Otira Viaduct, currently under construction, as well as several bridge replacements. The hazardous, partially single lane section through the upper Otira Gorge, immediately north of the Otira Viaduct site, will be upgraded in the near future, resulting in removal of the existing 13 m length restriction.

Local earthquakes in 1994 and 1995 caused significant damage to the highway mainly from debris falling onto the highway. Although the highway was closed while repairs were carried out, the most significant effect of the earthquakes was the longer term destabilisation of slope debris and dilated bedrock above the highway which necessitated ongoing highway repairs and slope stabilisation measures.

2. REGIONAL SETTING

Between Arthur's Pass and Otira villages, where highway problems are most severe, the highway is flanked on either side by rugged mountains that rise to 1950 m, 1000 m above the highway. Above valley floors, glaciated slopes are generally steep and dissected. Basement rock of Torlesse Terrane, which extends along the highway throughout Arthur's Pass National Park, consists of predominantly highly indurated, pale grey, greywacke sandstone and minor black, fissile mudstone (argillite) of Late Triassic age (c. 200 Ma), collectively referred to as "greywacke". In general, bedding adjacent to the highway strikes north-northeast, slightly oblique to Otira and Bealey Valleys and dips steeply either towards east or west (Fig. 1). Bedding orientation plays a major role in controlling the drainage pattern and behaviour of the rock mass along the highway.

Along the highway corridor, as in most areas of Torlesse rocks, closely spaced joints are highly developed in the rock mass. In general, the rock mass is open jointed near the ground surface due to stress relief resulting from deep glacial and fluvial erosion. Where steeply dipping bedding occurs in interbedded sandstone and argillite exposed on steep valley slopes, flexural toppling has occurred on lithological contacts due to combined stress release and gravitational creep. This occurs on steep slopes immediately above the highway in Otira Gorge, and is also evident, on a larger scale, near ridge crests where flexural toppling may have formed ridge rents, or uphill facing surface scarps (Fig. 1).

A geomorphic feature of major significance to the highway is the Otira rock-avalanche deposit, which partially blocks Otira Valley a short distance north of Arthur's Pass (Fig. 2). To cross this obstruction from the southern side, the highway climbs 65 m to avoid a steep, ravelling slope undercut at the toe by the Otira River, and descends 200 m in a series of hairpin bends into the upper Otira Gorge. The 440 m long, Otira Viaduct is being constructed along the valley floor to replace the Zig Zag, which is endangered by continuing erosion of rock avalanche debris and from rock fall from the main scarp 600 m above the highway.

Major active strike-slip faults, the Alpine, Hope, and Kakapo Faults, occur within 20 km of Arthur's Pass (Fig. 1). The Alpine Fault, a section of the Australian-Pacific plate boundary, is located 20 km northwest. The Hope Fault passes along Taramakau Valley at the northern boundary of the park, and the Kakapo Fault crosses SH 73 6 km north of the pass. On the southeastern side of the Alpine Fault, uplift of the Southern Alps is taking place at 6-10 mm per year. This uplift is matched by a high rate of erosion which, combined with poor rock condition that characterise greywacke in the Southern Alps, contribute to aggradation problems along the highway. In addition to major tectonic faults, minor shear and crush zones are common throughout the rock mass - these zones tend to be oriented parallel to bedding and are often contained within incompetent argillite beds.

Arthur's Pass lies within one of the most active seismic regions of New Zealand and this contributes greatly to landslide hazard potential in the area. Large earthquakes have occurred nearby in historical times, including the M 7-7.3 Glynn Wye earthquake of 1888, and the M 7.1 Arthur's Pass earthquake of 1929 (Fig. 1). On 18 June 1994, a magnitude M6.5 earthquake centred c. 20 km southwest of Arthur's Pass, caused considerable damage to sections of SH73 in Arthur's Pass National Park (Paterson & Bourne-Webb 1994). Further damage to the highway was caused by the 29 May 1995, magnitude M5.5 earthquake, which was centred 14 km west of Arthur's Pass (Paterson & Berrill 1995). Recent investigation of paleoseismicity on the central section of the Alpine Fault indicates occurrences of magnitude $M \approx 8$ earthquakes in 1425±15yr AD, 1620±10yr AD and 1717 AD (Yetton 1998). Similar future earthquakes from this source are likely to produce high seismic shaking intensities at Arthur's Pass.

3. ROCK FALL HAZARD ALONG THE HIGHWAY

Detailed assessment of landslide hazards along Arthur's Pass highway was undertaken in the 1980s during investigations for Arthur's Pass Roading Project carried out by Works Consultancy Services Limited on behalf of Transit New Zealand. Investigations included geomorphic hazard assessment of the entire section of highway through Arthur's Pass National Park (Whitehouse & McSaveney 1992), and detailed, site specific, engineering geological investigations (Paterson 1987). The occurrence and general effects of debris flows, rock falls and rock avalanches on SH73 through Arthur's Pass National Park, are described by Paterson (1996).

Rock fall is common along the highway throughout the park. Small rock falls onto the highway occur on a daily basis, although the number and magnitude of events are significantly greater during rainstorms. Small rock falls of detrital material from excavated batters, or adjacent slopes occur frequently during periods of freeze and thaw. Large rock falls (> 100 m^3) can occur at any time, but generally are associated with strong earthquakes or heavy rain. Sections of the highway subject to greatest rock-fall hazard are the Zig Zag and Otira Gorge where fatalities have occurred.

3.1 Rock-fall hazard at the Zig Zag

Frequency of minor rock fall is difficult to assess because they can easily escape detection. Small rock falls from dilated rock on the main scarp above the Zig Zag are probably a daily occurrence, if not more frequent, but these rarely affect the highway. Extensive scree deposits above the highway are testimony to continuous rock fall and debris flows that have occurred during the last 2000 years since the last known phase of rock avalanching at the Zig Zag (Fig. 3).

The record of historical rock fall is limited to major events such as the 1929 Arthur's Pass earthquake. During this event, a section of road at the Zig Zag collapsed, and other sections of the road were buried with rock debris, including blocks in excess of 100 tonnes. According to newspaper articles, the earthquake mobilised many thousands of tonnes of scree deposits above the highway. Numerous impact depressions on the road indicated that large blocks landed on the road and continued downslope into the Otira River. Based on the total area affected by landsliding, the earthquake magnitude was estimated to be approximately M7.0, and the earthquake was attributed to movement on a section of the Kakapo Fault (Fig. 1). From historical records of the 1929 Arthur's Pass earthquake, Reyners (in Paterson et al. 1992) estimated that shaking intensity reached MMVII or greater in Arthur's Pass area. Analysis of historical and paleoseismicity data indicates return periods of 100-300 years for earthquakes of shaking intensity MMIX or greater, in Arthur's Pass area (Yetton 1998). Rock-fall activity initiated by these events is likely to be greater than that experienced during the 1929 Arthur's Pass earthquake.

Association of landslides and earthquakes has been studied locally and extensively outside New Zealand. In a review paper documenting 40 historical earthquakes from around the world, Keefer (1984) suggested that the smallest earthquakes likely to cause rock fall and rock avalanches in epicentral areas are $M_L \approx 4.0$ and $M_L \approx 6.0$, respectively, and minimum shaking intensity for generating rock fall and rock avalanches are MMVI and MMVII, respectively. This threshold of shaking intensity for initiation of rock fall appears to be at least one level lower than experienced during historical earthquakes at Arthur's Pass, even after taking into account local effects such as orientation of valleys in relation to direction of earthquake wave propagation, and topographic amplification. Differences in threshold could be due to climatic variations, as well as other factors.

Rock fall at the Zig Zag from the 18 June 1994, M6.5 Arthur's Pass earthquake was minor compared with extensive landsliding that occurred in Otira Gorge immediately to the north. During the M5.5 earthquake on 29 May 1995, numerous blocks of rock fell onto the highway at the Zig Zag, mainly from adjacent scree slopes, but several blocks 1.5 m diameter, caused impact depressions on the highway indicating an origin from considerable height above the highway. Sources of these blocks, and new tongues of rock debris on upper scree slopes, were traced to the main scarp at the ridge crest (Fig. 3). This increased rock-fall activity could be due to the closer proximity of the 1995 earthquake epicentre; different directions of seismic wave propagation relative to the valley slopes; or predisposition of slope materials to failure (Paterson & Berrill 1995).

In December 1995 and on 18 March 1996, 6-10 months after the 1995 earthquake, several individual blocks of rock from the main scarp above the Zig Zag impacted on the highway. In May 1996, several additional large blocks (>10 tonnes) from the same source caused further impact damage (Fig. 3), necessitating a five-day closure of the highway while stability of the source area, and hazard mitigation measures were investigated. These blocks of rock consisted of hard, massive, greywacke sandstone which survived rebounds from *in situ* rock and impacts from stationary blocks. Numerous blocks, including some that may have overtopped the highway without a trace, came to rest on the valley floor.

Since May 1996, rock fall at the Zig Zag has been limited to a few isolated blocks. However, it is recognised that loose scree deposits and dilated rock across the main scarp could give rise to further major rock falls or rock avalanches during severe rainstorms, or earthquakes of shaking intensity MMVII or greater. Effects of such events on the existing highway could range from impact damage to collapse of the most vulnerable sections of highway. Completion of the viaduct should protect highway users from rock fall of the magnitude experienced since the road was formed. However, a strong (M>7) earthquake could initiate another rock avalanche which could cause extensive damage to the highway including the viaduct.

3.2 Rock-fall hazard in Otira Gorge

In Otira Gorge immediately north of the Zig Zag, the narrow highway bench crosses a steep rock face which in recent years has been subject to several large rock falls and numerous minor rock falls. Natural debris chutes also occur at several locations along this section of highway (Figs. 2, 4). During storms, rock debris from slopes c. 800 m above the highway travels rapidly down narrow chutes and falls onto the highway from a height of 30 m or so, causing panel and windscreen damage to vehicles. At Reids Falls during rainstorms, debris stored temporarily in a small drainage basin 30 m above the highway, is flushed through a narrow chute onto the highway at the end of a single lane section of highway where vehicles pause to give way to uphill traffic (Fig. 4). Further down Otira Gorge for 2.5 km, the highway follows the riverbank not far above flood level and is founded mainly on fill or alluvium/colluvium, except around rock spurs where the highway is cut into bedrock. Between spurs the highway is exposed to rock fall, debris slide/flow and stream aggradation, whereas rock fall from oversteepened bedrock slopes at the spurs has been a recurring problem since the 1994 and 1995 earthquakes.

4. ROCK-FALL PROTECTION MEASURES

4.1 Zig Zag

Rock-fall protection measures at the Zig Zag are constrained by the irregularity and unpredictability of life threatening rock fall; inaccessibility of the extensive rock-fall source; and the long, steep, rock-fall path above an unprotected section of highway. Any protection measures implemented would be required only until late 1999 when the viaduct is due to be completed and the Zig Zag highway closed. Protection measures such as stabilisation of the rock-fall source, or installation of rock-fall deflection/arresting structures, are probably not feasible at this site, although to date, conditions have not deteriorated to a point that such protection measures were necessary for the safety of highway users.

During rock-fall activity in 1996, rock-fall mitigation options considered included temporary closure of the highway, particularly during darkness; construction of a bund or gabion wall along the uphill side of the most vulnerable section of highway; and a combination of rock-fall surveillance and supervised travel over the Zig Zag. Rock-fall trials, using a helicopter and painted blocks of rock, were carried out to determine travel times between release of blocks from the ridge crest and their arrival at the highway. Recorded rock-fall travel times of c. 30 seconds (with average velocities ≥ 15 m/s) were considered to be insufficient for vehicles to cross the Zig Zag safely between rock-fall events. As the exposed section of highway is partially cut into the toe of scree deposits, difficulty in constructing an effective rockfall barrier above the road, and associated high costs of such a structure, ruled out this option as a short term solution. The highway was closed for five days while stability of the source area and hazard mitigation measures were investigated, and for several weeks after reopening the highway, on-site surveillance was maintained to monitor rock-fall activity and if necessary, to close the highway at short notice. A "no stopping" zone established across the most vulnerable section of highway will probably remain in force until the viaduct is completed.

During investigation of the rock-fall source area in 1996, open tension fractures in bedrock along the ridge crest were examined and monitoring pins were installed on a fracture closest to the edge of the scarp near a source of previous rock fall, to detect any dilation that may occur as a precursor to further rock fall (Fig. 3). Monitoring of this fracture at six monthly intervals indicates that minor changes recorded (±0-3 mm) are probably due to seasonal variation or measurement error. Two additional sets of vertical tension fractures oriented subparallel to, and located within 50 m behind the rock avalanche headscarp, have been incorporated in a survey monitoring network, to determine whether dilation occurs during any future major earthquake (Fig. 3). The proximity and subparallel orientation of these features to the main scarp indicate that they may have formed during the last phase of rock avalanching c. 2000 years ago, or during subsequent strong earthquakes. It is difficult to establish the age of these features because of the resistance of greywacke sandstone to weathering, and the lack of development of soil and vegetation on the ice-sculptured ridge crest.

4.2 Otira Gorge

Rock fall along the Otira Gorge highway is confined to numerous individual sites some of which are reactivated during frequent heavy rainfalls and others that are initiated by infrequent extreme events such as earthquakes and high intensity rainstorms. Sites affected by extreme events often continue to give rise to frequent rock fall which diminishes as the source of unstable material contracts and vegetation reestablishes. In general, sources of repeated rock fall consist of mainly slope deposits and dilated bedrock. Rock fall also occurs from upper margins of steep highway batters. At several sites, debris travels down narrow chutes or channels from eroding upper ridges, and rebounds off lower rock slopes onto the highway (Fig. 4). Rock-fall protection at these sites is more difficult to achieve than at incised channels where the flow of water and rock debris onto the highway is confined. Designing appropriate protection systems for each site necessitates assessment of the source and path of rock fall to determine the type and volume of potentially unstable material and the hazard it creates at highway level, and to assess whether it constitutes a short term or long term problem.

In general, rock-fall protection systems fall into three categories - stabilisation of source material; protection shelters or barriers above or adjacent to the highway; or relocation of the highway outside the danger zone. In Otira Gorge, relocation of the highway is not feasible because of the restricted width of the valley and the position of the highway at the base of, or cut into, slopes oversteepened by the river. To date a combination of source stabilisation and protection barriers has been used, depending on characteristics of the site, the nature and extent of source material, and the level of hazard. Because of proximity of the highway to the base of the slopes and the difficulty in erecting effective rock-fall barriers on slopes above the highway, the emphasis has been on stabilisation of source material, although it is now recognised that protection shelters or specialised fences may be required at sites where rock fall has been identified as a long term problem.

When rock fall occurs, if a ground inspection is inadequate or not feasible, a helicopter is used to trace and examine the source of unstable debris so that the immediate hazard to highway users and remedial options can be assessed. Photographs of the site are taken from the helicopter for future reference. After the inspection, remedial measures are implemented immediately if urgent action is required to protect the safety of highway users.

Road closures cannot be avoided during major events such as earthquakes and rainstorms when large volumes of material cover the highway. Inspections may be required to determine whether the highway can be reopened safely after the highway is cleared. This is often necessary when material is continuing to fall from sources not visible or accessible from the highway. If unstable loose debris is a serious threat to traffic, a helicopter, with monsoon bucket, may be used to sluice the face clean. This can be successful on steep rock slopes immediately above the highway, or in narrow debris chutes where loose debris can be sluiced progressively down slope. This method is not appropriate where large volumes of material or large blocks need to be moved, or where falling debris is not a direct threat to traffic. However, in some situations it is a very efficient method of mitigating a serious rock-fall hazard.

If sluicing is not feasible and debris is not falling directly onto the highway, i.e. rock fragments roll out onto the road, a bund of rock debris or gravel placed along the road verge and located slightly out from the base of the slope may be used to trap debris. The height of the bund and its location can be varied to increase effectiveness, although it may encroach onto the highway pavement, restricting road width. This may not be a serious problem if approach visibility is unrestricted, or if the road width can be increased - reduced width may be acceptable if remedial measures are short term and the slope is expected to stabilise.

At sites where blocks fall directly onto the highway, stabilisation can be achieved by removing unstable rock or by erecting a "blanket"of steel mesh to prevent loose blocks from bouncing onto the highway - the latter has been used at only one site. Several methods of scaling unstable rock have been used, including abseiling and hand barring; hand barring on a safety line and harness from a helicopter; trim blasting using abseiling techniques or cherry picker for access; and pulling loose blocks and overhanging trees with a wire rope and machine. In most cases, the sources of unstable debris are above the reach of hydraulic excavators.

The 1994 and 1995 earthquakes loosened the rock mass at numerous sites through Otira Gorge requiring extended services of professional abseilers and necessitating frequent road closures. At one site where rock fall was a continuing problem, attempts to install rock-fall protection fences were abandoned in favour of debris shelters which may be erected for long term protection during a future upgrading contract.

5. CONCLUSIONS

Rock fall has been a serious problem along Arthur's Pass highway since the occurrence of recent earthquakes, and is likely to continue, possibly at an abated rate, in the foreseeable future. Prior to 1994, slope instability problems were less widespread and tended to be focussed on a few specific sites, except during major floods and during the 1929 Arthur's Pass earthquake when damage was serious and widespread.

Although improvements such as the Otira Viaduct and the upgraded section in upper Otira Gorge will reduce rock-fall hazard on this route by eliminating or reducing the problem at high risk sites, maintenance costs will continue to be high because of the mountainous terrain, severe climate, highly fractured rock mass and high level of seismicity.

Sites in Otira Gorge north of the upgraded section may give rise to further rock fall which may need to be rectified by applying more permanent solutions on a site by site basis, because of expected increased traffic when present length restrictions are removed. Rock-fall protection measures such as mesh, rock bolts, and rock-fall protection fences may need to be used more extensively than at present to improve safety along this route. These methods should conform to standards acceptable within Arthur's Pass National Park.

6. ACKNOWLEDGEMENTS

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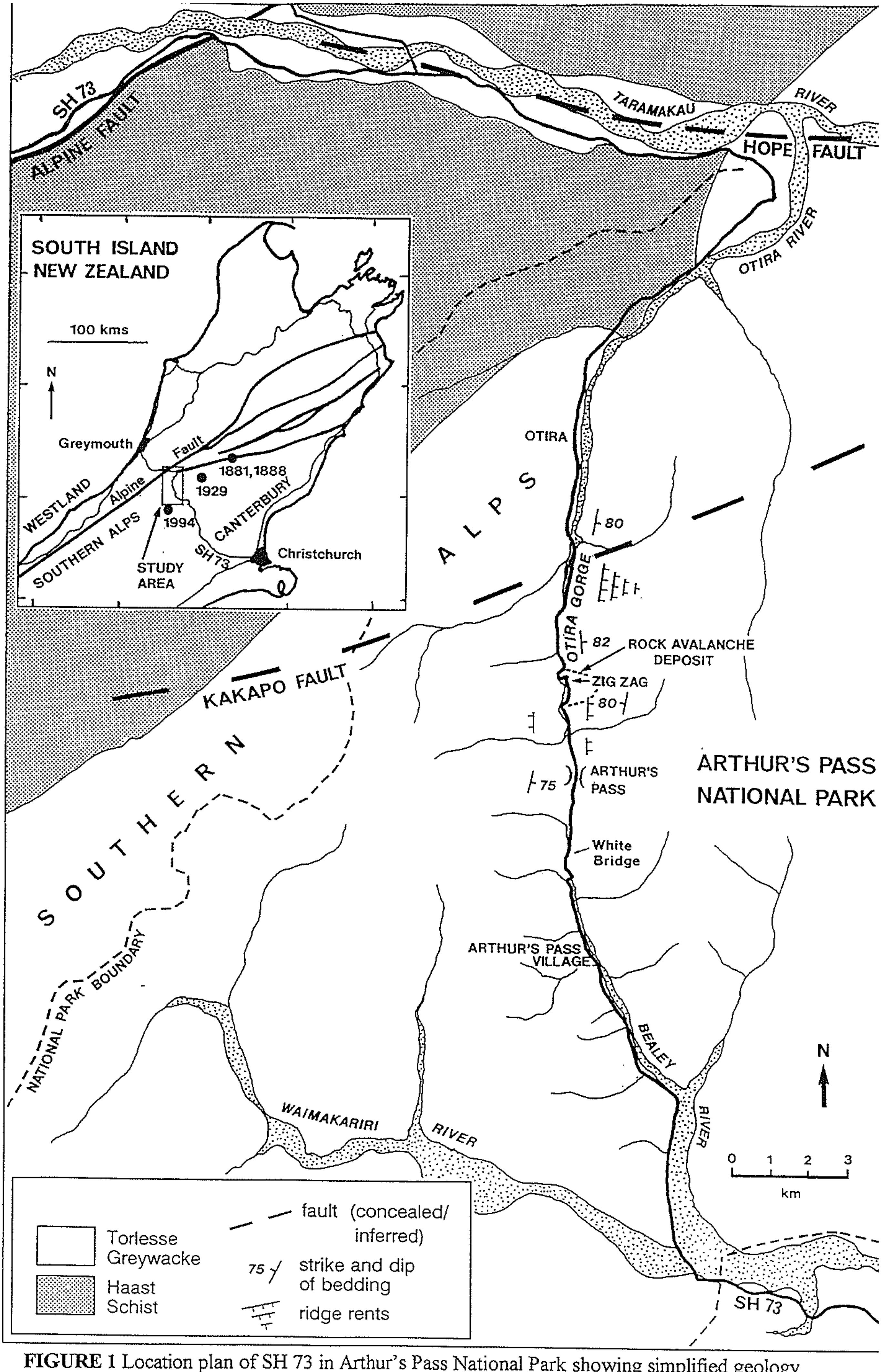

FIGURE 1 Location plan of SH 73 in Arthur's Pass National Park showing simplified geology. Inset shows location of site, major active faults and epicentres of major historical earthquakes.

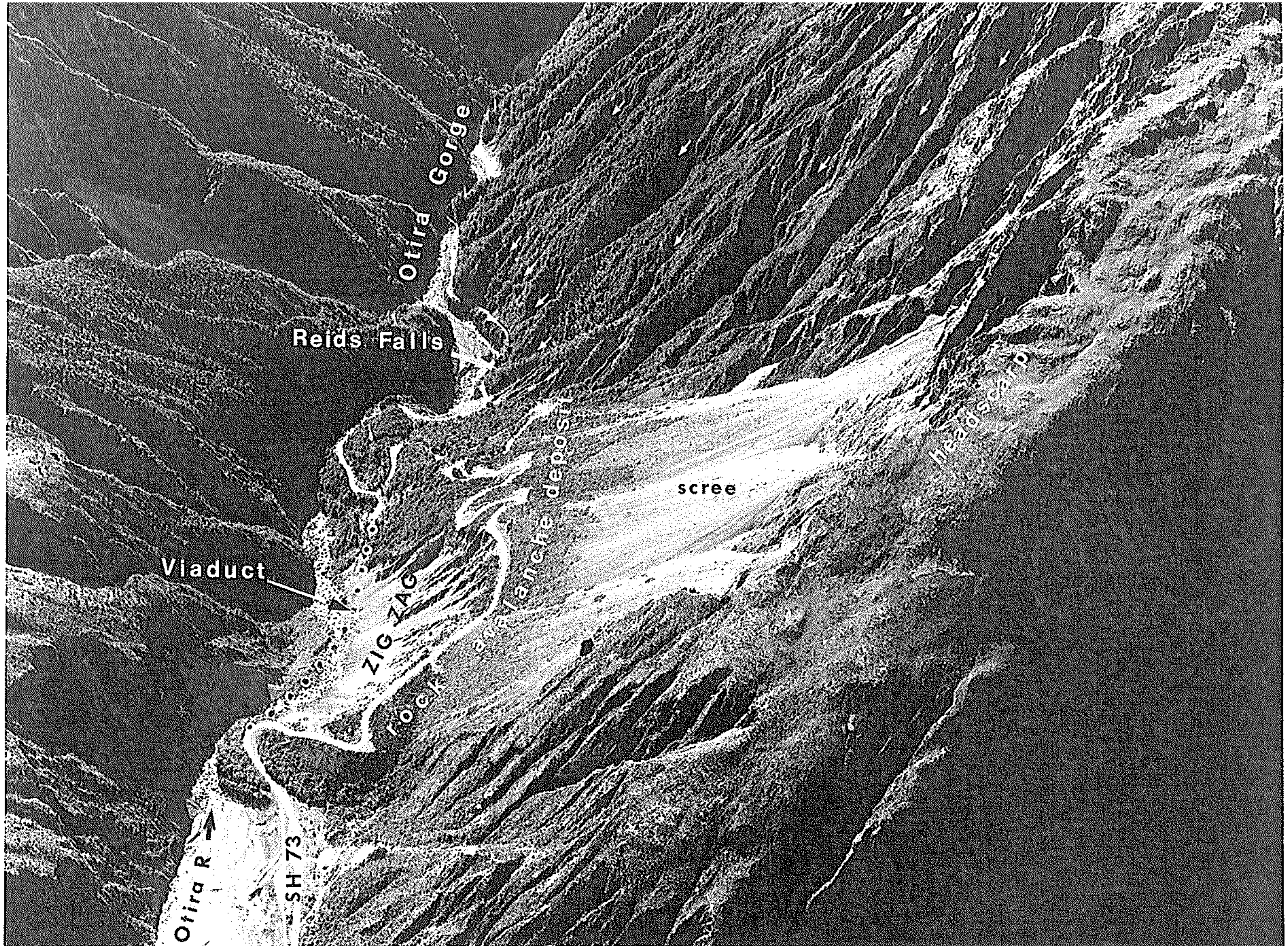

FIGURE 2 View looking North down Otira Valley, showing the viaduct alignment below the Zig Zag and numerous debris paths on steep slopes above upper Otira Gorge highway. Note the extent of rock avalanche and scree deposits. *(Photo: D. L. Homer)*

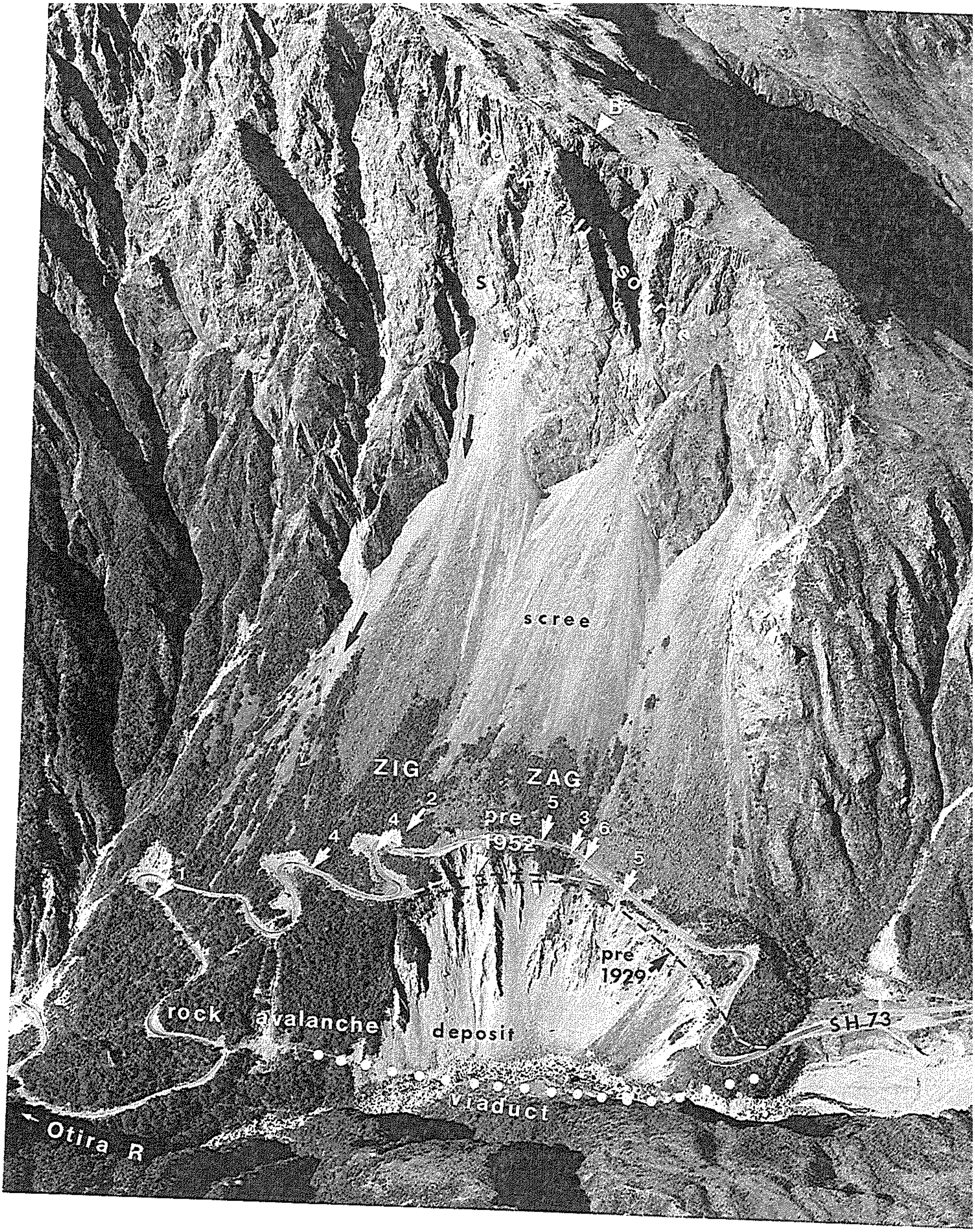

FIGURE 3 Existing and previous alignments of SH 73 at the Zig Zag, and the alignment of the viaduct, under construction. Numbered white arrows locate impacts of large blocks from 1995,96 rock fall, with sources on the headscarp above scree deposits. Other features are: source (s) and path of 1995 earthquake-generated rockfall, and locations of monitored open fractures (A-C).
(Photo: D. L. Homer)

FIGURE 4 Partially single lane, Upper Otira Gorge section of SH 73 which is subject to rock fall (rf) from highway batters and debris fall from drainage/debris chutes (dc) with sources high on the ridge (see Fig. 2). This section of highway is to be regraded, widened and protected from rockfall

ROCK BURST IN THE HOMER TUNNEL

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SUMMARY

Following recent widening of portions of the Homer Tunnel on SH 94 near Milford Sound in Fiordland, a series of rock burst events have been experienced. While no injuries were caused, the potential safety implication for motorists was very apparent and Opus was engaged by Transit NZ to develop a remedial programme to reduce the risk of further failures. The mechanism of failure has been found to be similar to that experienced during the original construction of the tunnel. Effective confinement of the very high residual stress in the massive crystalline rock has been found to be the key parameter in managing the problem, and the remedial treatment based on pattern bolting with 2.4 m and 3.6 m stressed rock anchors is described. Differences between the usual kinematic stability considerations of jointed rock masses, and the rapid fracturing of the highly stressed strong massive rock present in the zone of failure is briefly discussed.

1. INTRODUCTION

The Homer Tunnel is situated on SH 94 between TeAnau and Milford Sound in Fiordland. The tunnel was constructed over the periods 1934 to 1941 and 1951 to 1953 between the Hollyford and Cleddau Valleys to give road access to Milford Sound. The Homer Portal (Hollyford Valley) is at an elevation of 914 m, and the 1275 m long tunnel descends in a westerly direction at a 10% grade to the Cleddau Portal. An essentially circular unlined profile of 7.3 m diameter was adopted, allowing two way traffic for light vehicles, but this width has been found to be restrictive for the large number of tour coaches now using the route. Two passing bays within the tunnel have been recently completed by local widening of the cross section.

The geological setting has been described by Blattner & Coates (1989). The dominant rock type within the tunnel is very strong unweathered biotite norite, with numerous vertical pegmatite and hornblende veins or dykes. The igneous complex was formed between 230 and 100 million years ago, and in the last 6 million years the area has been uplifted some 18 000 m, with associated erosion. The current landform has been shaped by glaciers up to 800 m deep, with melting of the ice occurring some 15 000 years ago. Located within a wedge formed by the major tectonic boundary on the Alpine Fault some 30 km to the NW, and the Hollyford Fault 10 km to the east, the site is still subject to tectonic action. Residual stresses associated with the uplifting (F R Gordon, 1992), possibly together with regional stresses caused by the current tectonic processes, are thought to be the primary cause of the rock burst phenomenon that has been experienced.

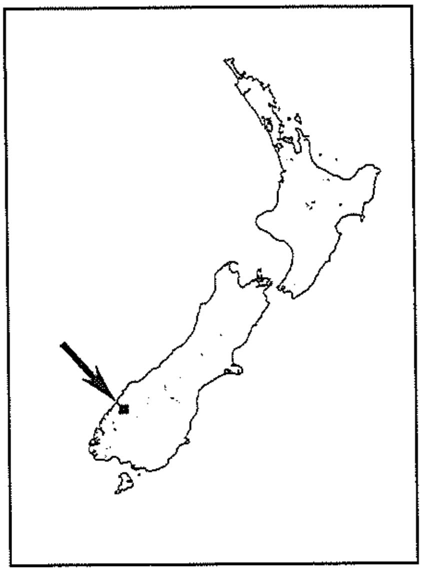

Figure 1 : Site Location

2. ORIGINAL CONSTRUCTION EXPERIENCE

Construction experience has been documented by various authors, with a concise summary presented by F R Gordon (1992). The rock jointing was found to be quite variable, with areas of both massive intact rock, and zones of closely spaced jointing requiring support in the form of bolting together with mesh and gunite lining. Immediately following excavation in massive rock zones, the apparently sound surface has been reported as becoming "drummy", and explosive rock burst failure would subsequently occur. The incidence of this violent spalling decreased progressively until after a period of four years a new state of equilibrium appeared to be established.

Both regional and tunnel joints have been mapped (Macfarlane 1982). Six joint sets are recognised:

1	340°/40°	strike sub parallel to the Alpine Fault
2	110°/50°	
3	285°/75°	strike sub parallel to the Hollyford Fault
4	145°/45°	strike sub parallel to the Alpine Fault
5	045°/10°	
6	030°/42°	Platy (stress relief) joint set

The orientation of the tunnel alignment is approximately 276°/5.7°.

Joints are generally moderately widely spaced, although spacing does vary markedly in some areas. Kinematic stability of blocks and control of relaxation processes have been addressed in the past by strategic rock bolting and gunite lining.

3. WIDENING PROJECT

The tunnel was recently widened at two locations to improve passing opportunities. The locations selected were:

- 140-240 m and
- 1040-1140 m from the Hollyford Portal.

The cross section profile adopted for the widening is shown on figure 2 below. A very planar crown was adopted in areas of very widely spaced jointing, rather than the near circular arch section which is typical of the "mature" original profile. Significant rock falls were experienced over the months following excavation, with the zone of particular concern is extending from 1090 m to 1130 m from the Hollyford portal. These rock falls were not associated with conventional kinematic instability of a jointed rock mass, as the failure areas were essentially free of jointing and consisted of very strong rock. Fracturing parallel to the rock surface was experienced to a depth of 300 mm, with rapid release of energy on final failure. This rock burst phenomenon appeared to be triggered by a rapid temperature drop in the tunnel, and the sound of cracking could be clearly heard.

This zone of massive rock appears to be carrying very high stresses compared to the zones of generally more closely jointed rock. There appears to be a strong longitudinal component to the massive rock orientation and stress vector, with the principal compressive stress aligned at a bearing of approximately 258° and dipping at 15° through section 1085 m (RH crown) to 1125 m (LH invert). A further zone of highly stressed strong rock was observed in the RH wall near station 212 m.

4. REMEDIAL DESIGN CONCEPT

The rock burst mechanism experienced in this tunnel during and after construction, has been well documented, but no allowance appears to have been made for this phenomenon when the widening work was undertaken. The rock burst mechanism is unlikely to occur once a concave (arch) surface is established, as the energy cannot be released due to the natural arch resistance that is then available. The 7.3 m diameter tunnel (pre widening) appeared to have reached this stable condition within a few years of construction, but the recent widening had created zones which were not naturally arched or adequately restrained by other support. The effective arch in the widened areas is equivalent to a 9 m diameter excavation, so remedial measures could make use of the arch action available provided the surface rock was adequately restrained.

Air temperature drop is expected to be a trigger factor in initiating rock burst events. The highly stressed rock is close to failure in the absence of restraint, and the rock contraction on cooling will increase the secondary tensile stresses which promote fracturing, leading to surface rock burst. The very low air temperatures experienced in this tunnel would be sufficient to create a temperature gradient in the rock surface zone.

As the rock is assessed to have an unconfined compressive strength in the range 100 to 200 MPa, the residual stresses present must be substantial. It did not appear to be practical to directly resist the forces using a structural lining, or to disrupt traffic further by excavating back to the larger circular arch. A pattern bolting solution was therefore adopted to secure the rock potentially subject to further rock burst events. A design based upon the use of readily available 2.4 m rock anchors was discarded in favour of a design using a combination of 2.4 m and 3.6 m anchors positioned to reach beyond the compressive arch zone as shown in figure 2 below.

Pretensioned fully encapsulated resin anchors were adopted to achieve a small degree of prestress into the surface rock and thereby suppress fracture initiation. The 800 mm anchor zone utilised a flash setting resin to facilitate stressing, and the 24 mm diameter anchors had a capacity of 250 kN. A 1.2 m by 1.5 m pattern was used to achieve a reasonably uniform confining effect over the treated area. Proprietary strapping bands suited to 1.5 m support centres, plus two layers of triple twist high tensile mesh (terramesh) was installed to contain any minor spalling that may occur between the anchors.

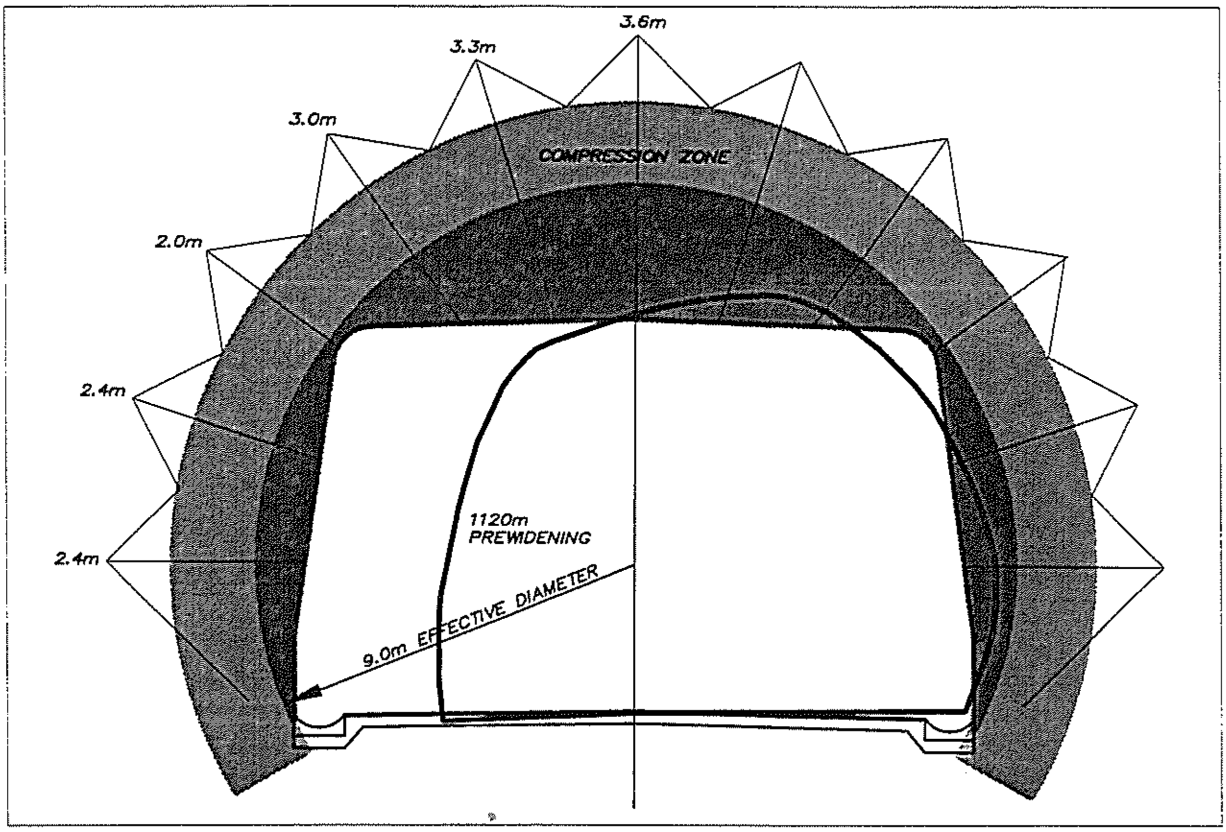

Figure 2 : Pattern Bolting Cross Section

The remedial works were undertaken at night when there is very little traffic on this section of SH 94. Drilling was undertaken using an air leg supported on an elevated cage. Protection from rock fall was able to be provided over this cage which was mounted on a wheeled loader. Due to the use of flash setting resin, it was found that the 3.6 m anchors were near the limit of practical installation using light equipment. A modified track mounted drill boom was used to reliably seat these anchors.

5. CONCLUSIONS

The strong massive rock present around the 1100 m section of the Homer Tunnel appears to have lead to a false sense of security during the design and construction of the passing bay widening project. Original construction experience with rock burst events associated with the high residual stress present in the rock mass has been well documented. However, the widening profile adopted removed the arch action needed to confine the rock carrying these high stresses. Planar surfaces replaced stable arch profiles, with the result that rock burst events last seen shortly after original construction were reactivated.

The rock burst phenomenon is quite different from the usual kinematic stability applied to shallow tunnels in jointed rock. High residual or tectonic stresses, combined with strong massive rock masses, are required to be present in conjunction with adverse geometry; ie, non restrained or planar excavated surfaces. The rock burst occurs as fresh fracturing develops parallel to the exposed surface under the induced secondary stresses. Explosive failure can occur.

The conditions associated with rock burst are present in the Homer tunnel most obviously around station 1120 m, but there is evidence of less severe but similar conditions existing at other positions, including around 210 m. Most of the remainder of the tunnel is located in more closely jointed rock which requires conventional treatment to prevent kinematic rockfall hazards but which is not subject to the high stresses associated with rock burst.

Temperature drop does appear to be a trigger factor in initiating rock burst events.

The remedial work undertaken on the Homer Tunnel following the passing bay widening project was focussed upon rapidly removing the potential danger to motorists, rather than seeking to fully understand the subtleties of the rock stress regime through an investigation programme. In this context, a conservative rock bolting approach was taken with no specific insitu rock stress measurements. These remedial measures to secure the tunnel profile have so far proved to be successful in controlling rock burst events in the treated area.

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CLIFF FACE STABILISATION WORKS ROCKS ROAD NELSON

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Paul Denton
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SUMMARY

State Highway 6, Rocks Road, Nelson traverses the base of a 40 m high sea cliff. Sections of this cliff face have been subject to ongoing local failures that have caused temporary closure of the state highway. The stability issues have been three fold:

(a) Creep movement of fill at the top of the cliff.
(b) Potential failure of a rock block on unfavourably oriented discontinuities.
(c) Ravelling of the rock surface.

To address these issues a reinforced concrete pile palisade wall, dowels, inclined drains and rockfall protection netting have been provided. This paper discusses the design and construction of the stabilising works.

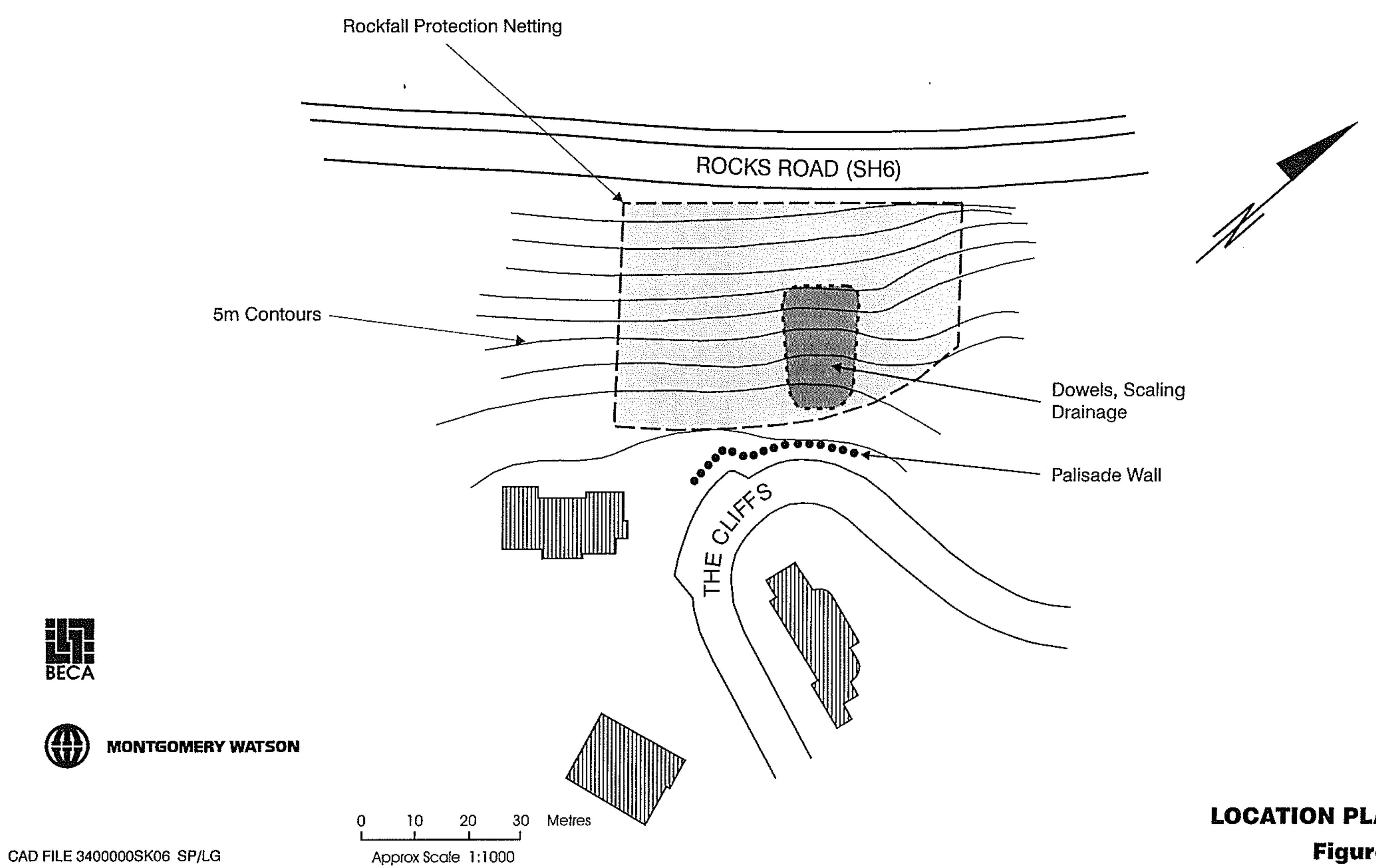
1. INTRODUCTION

State Highway 6, Rocks Road, follows the foreshore southwest of Port Nelson. A cliff face rises from the road edge at an angle of typically 50º to the horizontal to a height of 40 m. A section of this cliff face has been subject to ongoing local failures that have caused temporary closures of Rocks Road. A slope failure in August 1990 comprising 500 m^3 and closing Rocks Road for several days was one such event.

Traffic disruption and the risk of injury or damage to vehicles led Transit New Zealand (TNZ) to engage Beca Carter Hollings & Ferner Ltd (BCHF) to undertake detailed investigation and design and construction monitoring of cliff face stabilisation works. Royds Garden Ltd (now Montgomery Watson New Zealand Limited (MW)) were engaged as a subconsultant to assist with engineering geological investigations and construction monitoring. This paper discusses the design and construction of the stabilisation works.

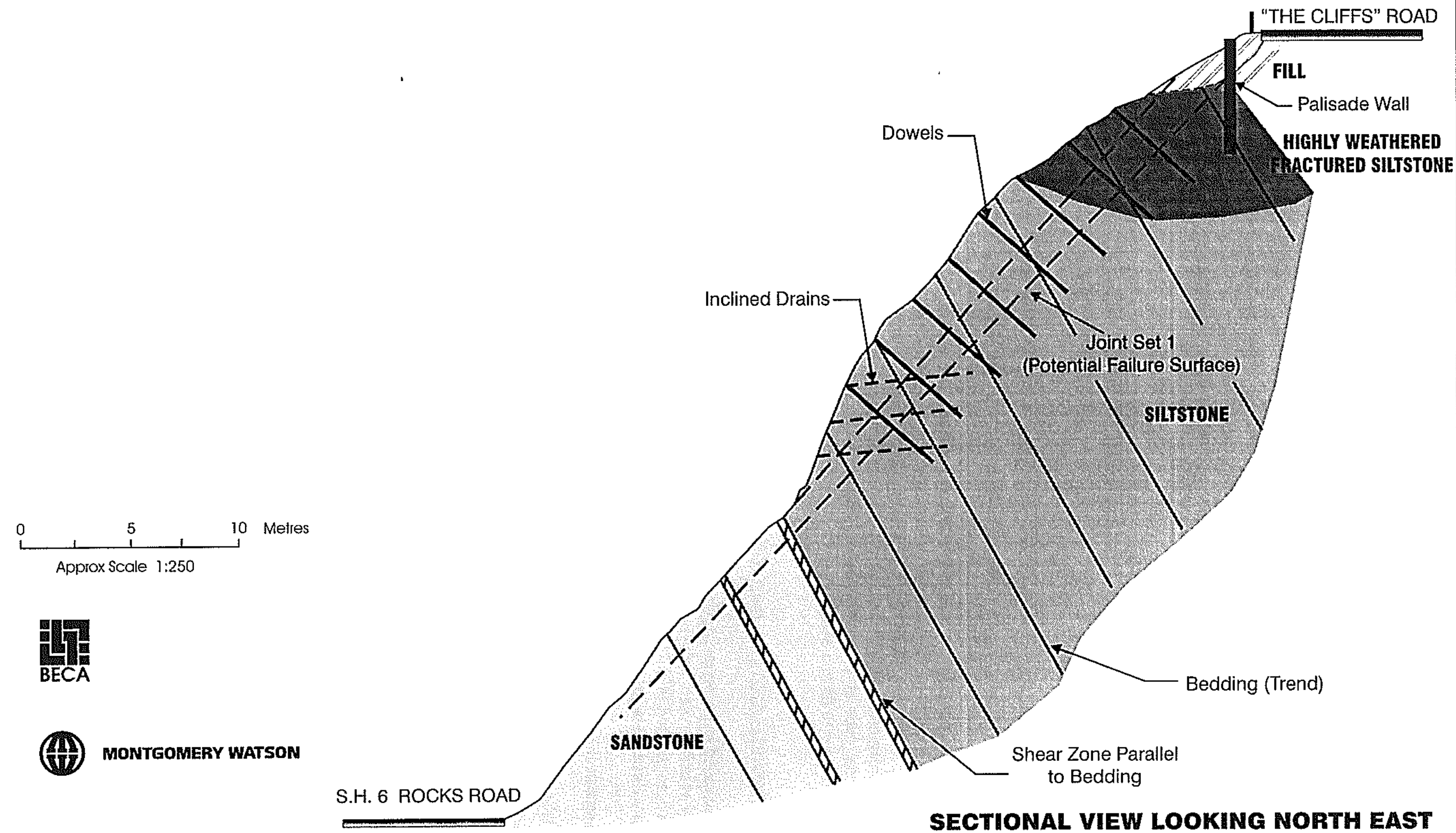
2. GEOLOGY

The geologic structure of the cliff face is summarised on Figure 2. Sandstone, siltstone and mudstone of the Magazine Point Formation dips into the cliff at 60º with a strike subparallel to the cliff face. In the Nelson area the total thickness of this unit probably exceeds 1500 m and was deposited by turbidity currents and submarine debris flows in an early Tertiary sea. It is predominantly a moderately well indurated competent sandstone, well exposed on the foreshore in front of the cliff at low tide. However, at the project site, higher up the stratigraphic column the unit is more deeply weathered, less competent and dominated by siltstone which has given rise to rockfalls and slumping. Bedding ranges from 50 mm to 1 m thick. Joint defects include a dominant set sub-parallel to the cliff face and daylighting from the cliff face. This joint set is identified as joint set 1 on Figure 2. The cliff is 40 metres high having been formed by erosion and undercutting of the sea during the post glacial period over the past 6000 years. Erosion and subsequent oversteepening of this old sea bluff continued until construction of State Highway 6 in 1893.



LOCATION PLAN

Figure 1



SECTIONAL VIEW LOOKING NORTH EAST

Figure 2

CAD FILE 3400000SK07 SP/LG

3. MECHANISMS OF INSTABILITY

Three mechanisms of past and/or potential instability had been identified. These were; (a) creep movement of fill at the crest of the cliff, (b) block failures of rock on the joint set sub-paralleling the cliff face, and (c) ravelling of the rock face. Respective techniques employed to remedy these mechanisms of instability were; (a) a palisade of reinforced concrete piles, (b) dowels and inclined drains, and (c) scaling and rockfall protection netting. Figures 1 and 2 show the areas of potential instability in plan and section.

4. PALISADE OF BORED PILES

Some 4 m thickness of fill and "The Cliffs" road had been constructed at the crest of the cliff during residential and subdivisional development in the 1970's. The fill comprises soft silty clay with its outer surface standing at 40º to the horizontal. Creep type movement of this fill is evidenced by cracking in The Cliffs road surface.

Options considered to stabilise this fill included: excavation and replacement with geogrid reinforced fill, excavation and construction of a crib retaining wall, and construction of a palisade of reinforced concrete piles. The palisade of piles was selected because it was assessed to be the most cost effective option and had the added benefit of causing minimum disturbance during construction. The stabilising works could be constructed without disturbing the road and underground services. 500 mm diameter reinforced concrete piles at 1.5 m centres embedding 3 m into the colluvium and rock underlying the fill were constructed to form a 30 m length of pile palisade.

5. DOWELS AND INCLINED DRAINS

Increased shearing resistance along the potential failure planes of joint set 1 (refer Figure 2) was provided by installing steel dowels and inclined drains. The steel dowels comprised HD40 reinforcing bars of 6 m length grouted into 75 mm diameter holes drilled perpendicular to the joint set. These steel dowels were provided on a grid of 3 m spacing downslope and 3 m spacing across slope. The dowels are to act as "shear pins" and were sized to increase the factor of safety against shear failure for both the static and the earthquake design case.

It was noted that past instability of this section of rock face has coincided with heavy rain fall events. It is probable that build-up of water pressure in the rock joints during and following rain contributes to cause the instability. Inclined drains extending beyond the daylighted joint sets were installed on a grid of 3 m spacing downslope and 3 m spacing across slope. The drains comprise 7 m long 50 mm diameter slotted PVC pipe installed in 75 mm diameter drilled holes.

The section of rock face identified as requiring dowel and inclined drain stabilising works measured 15 m across slope and 30 m downslope.

6. ROCKFALL PROTECTION NETTING

A 60 m long section of the 40 m high cliff face was identified as presenting a high potential hazard of rockfall and thus was treated with rockfall protection netting.

Galvanised and PVC coated double twisted mesh was used. The mesh was hung from an anchorage system, at the crest of the cliff, comprising a steel rope catenary spanning between anchors at 1 m spacings. The anchorage system was designed to have a greater holding capacity than the mesh it supports. At the base of the cliff a similar anchorage system comprising steel rope, strops and anchors, allows the netting to lift 300 mm to expel debris. It can also be disconnected for periodic removal of any collected debris.

With this rockfall protection netting in place any rock dislodged from the face is able to fall in a controlled manner beneath the netting to collect at the base. Prior to permanently placing the netting loose rock was scaled from the cliff face.

7. CONSTRUCTION

The site works were started in mid April 1994 by Hampton Construction of Paraparaumu using a small, and slow but effective, 2 man crew. Once vegetation was cleared an engineering geologic inspection was undertaken to confirm joint set orientation and set out the extent and location of construction works. For reasons of safety, control and cost effectiveness the cliff face was inspected by abseiling. Introductory abseil training was provided through the local caving group and rope anchors were installed, tested and approved for the "white knuckle" geotechnical inspection. Owing to the slope steepness (50° to near vertical), the height above the highway (40m) and the heavy vehicle traffic below special techniques were employed whenever personnel were moving around on the cliff face. These included:

- double line rope access (a working rope and a safety tie off)
- positioning 'spotters' in radio contact on the highway below (to control traffic when material became dislodged)
- securing all loose equipment with rope/rubber cord (ie rock hammer, Brunton compass, cans of spray paint etc)
- continuous, regular radio contact with backup personnel at the top when beyond line-of-sight.

The design work was undertaken by BCHF in Wellington with the Nelson office of Montgomery Watson handling implementation of the geotechnical investigation and the day to day construction monitoring. As a consequence a well co-ordinated, transparent flow of communication between the two partners was critical to the success of the project.

Modifications to the original construction work set out and extent were confirmed early on and the completed stabilisation works consisted of:

- 23 bored piles to 6 m (30 m of pile palisade)
- 48 grouted rock dowels
- 19 inclined drains
- 59 top net anchors (to 3 m)
- 19 detachable bottom net anchors (to 1.5 m)
- 2700 m^2 rockfall protection netting (60 m length)
- 280 m^3 scaling
- 45 metres of 1.8m high security fence at the top
- 60 m^3 additional scaling outside of the project site

A total of 5 variations were approved and implemented during the extended construction period including:

- an extension of the depth (and number) of top anchors due to unfavourable ground conditions. A reinforced concrete strip was added at the head of the anchors
- removal of loose materials (deep soil cracks) elsewhere along the cliff top identified during a 400m inspection traverse (Royds Consulting, 1994)
- an extension of the netting 18m to the south.

On completion of the initially proposed rockfall protection netting a slump of approximately 120 m^3 occurred at the southern limits of the works. That portion of the slump beneath the netting was contained resulting in a controlled release of rock and soil some 40 m below. As a consequence of the nettings excellent performance of the netting and the potential for further rockfall to the south it was decided to extend the netting 18 m south.

8. CONCLUSION

This project employed traditional rock slope stabilising and protection techniques which are not commonly used in New Zealand. The stabilising and protection works proved successful with there being no rock block failures or debris falling onto this section of the state highway since the construction of the works in 1994. While issues of instability remain to be addressed elsewhere along portions of SH-6 to the north, this work has been very successful in effectively addressing the most immediate stability issue. A total of 60 m length of cliff face was treated at a cost of approximately $250,000 (1994).

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DESIGN OF THE McARTHURS BEND REALIGNMENT

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SUMMARY

The combination of wet cut to fill material and a poor drying environment at the McArthurs Bend Realignment on SH 1 north of Dunedin has required the development of unusual design solutions for this project involving earthworks totalling some 600 000 m^3. Details of the 28 m high embankment fill and the cut batter slopes are presented, together with foundation and subgrade characteristics. Spatial variability in the faulted terrain containing both volcanic and tertiary sediment materials influenced the design concept. The degree of weathering of the volcanic rock together with the presence of ash deposits and highly reactive clay minerals in some of the tertiary sediments were significant considerations. The utilisation of very wet cut to waste material to provide support to the low strength embankment fill has allowed maximum use to be made of the local materials.

1. INTRODUCTION

The McArthurs Bend Realignment Project is situated north of Dunedin city on SH 1 as shown on figure 1. The topography is hilly with the site of work situated at the top of long steep grades in each direction. The original highway alignment over a length of some 2.4 km included two 70 km curves which required modification to improve traffic safety standards. The adjacent section of highway to the north had been realigned several years earlier, and construction difficulty was experienced with the wet volcanic soil encountered. Similar materials were expected to be encountered on this project, and relatively poor drying conditions were known to exist for much of the year.

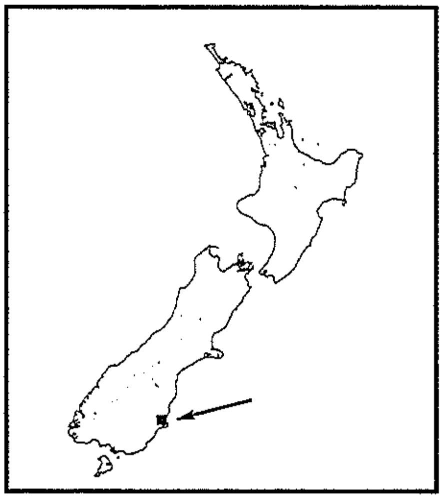

Figure 1 : Location

The realignment project undertaken for Transit New Zealand involved the excavation of a major cutting through the southern section of the site, and the construction of a 28 m high embankment. The cut batters were designed to be excavated at moderate slopes to limit the degree of shading during winter, thereby generating adequate fill quantities for the embankment. The plan layout is shown on figure 2.

This brief summary of the project only discusses geotechnical design aspects of the solutions arrived at to address the problems encountered.

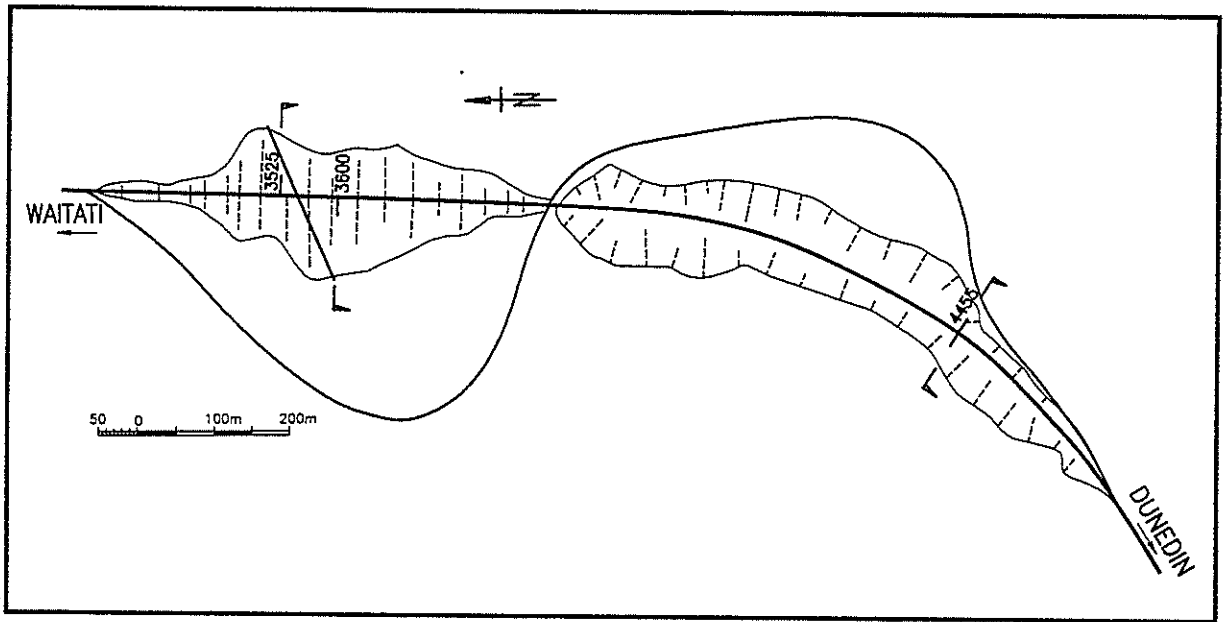

Figure 2 : Site Layout

2. CONSTRUCTION MATERIALS

Fault traces were mapped passing through the site, and tertiary sediments were expected to be present to the south, with weathered lava flows to the north. Five fully cored boreholes were drilled in the cut section and three wash bores were drilled in the embankment foundations. A series of test pits were also excavated along the alignment to obtain samples for laboratory testing. A schematic geological

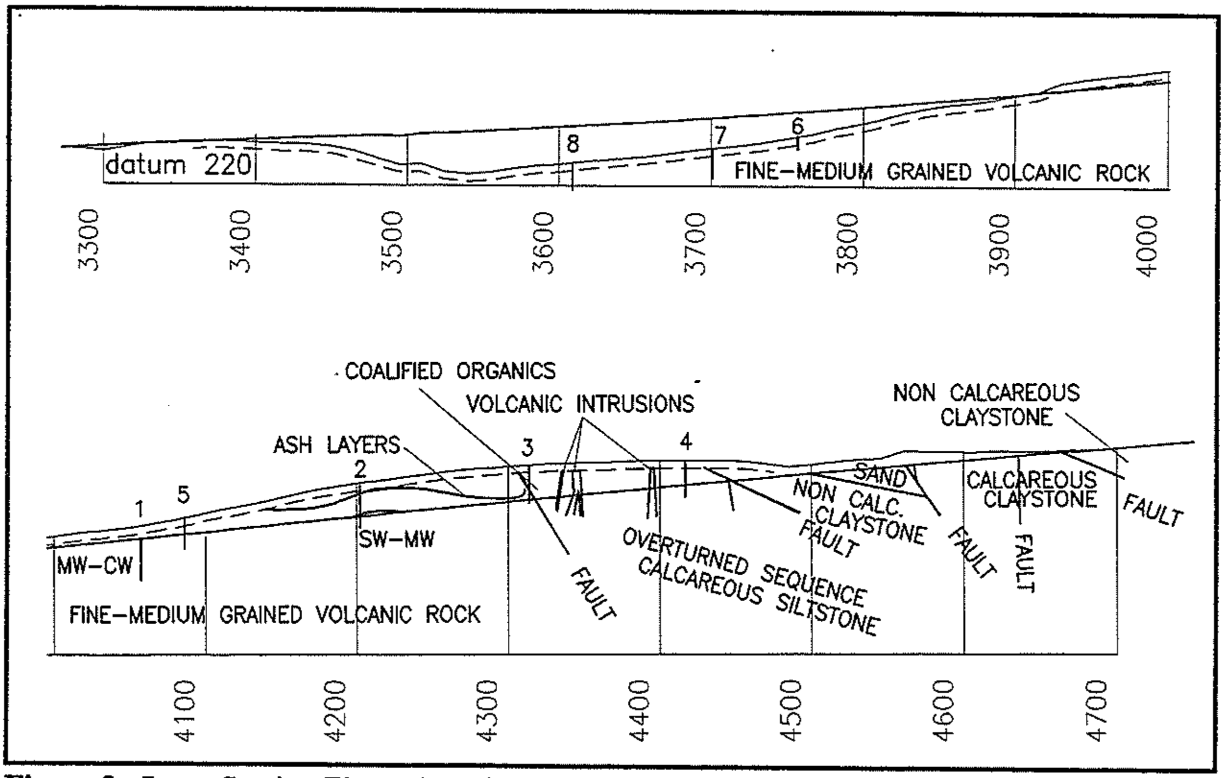

Figure 3 : Long Section Through Major Earthworks Section

model through the main earthworks areas is shown in figure 3. Faulting had created a complex distribution of materials throughout the southern portion of the cutting, with some portions of the sedimentary sequences overturned and modified by volcanic intrusions. The lava flows exhibited a highly variable degree of weathering in the vesicular rock, and weak saturated ash pockets were present. Colluvium and loess cover were found to mantle the site to a typical depth of 3 to 5 m, increasing to at least 7 m in the area of reworked alluvial deposits around station 3550 m.

Laboratory testing included the determination of compaction characteristics which revealed that the field moisture content of some cut to fill material was up to 15% above the optimum moisture content. Loess and calcareous siltstone on the other hand was found to be generally close to optimum conditions.

Results from ring shear testing of weathered volcanic soil indicated very little effective cohesion (0.3 to 0.6 kPa) and an effective friction angle from 32 to 33 degrees. Consolidated undrained triaxial tests on yellow clayey silt produced effective strength results of 2 kPa / 34 deg and 19 kPa / 30 deg. These clayey silts comprised from 22% to 27% clay, and Plasticity Index values ranged from 20 to 26. CBR testing on remoulded soaked samples gave swell responses ranging from 0% to 1%, and CBR values of 6 or better. A strong response to the addition of lime was obtained, with soaked CBR values exceeding 30. Consolidation and strength testing was also undertaken on the embankment foundation materials.

An off site source of high quality aggregate was available from the Grit Bin corner quarry situated adjacent to SH 1 some 2 km away.

3. DESIGN

Key design considerations for the embankment included:

- Shear strength of toe foundation under elevated construction pore pressure and ground water seepage
- Pore pressure rise and effective strength reduction in fill materials placed wet of optimum
- Post construction settlement of the embankment in response to the gradual dissipation of excess pore pressure
- Handling, compaction and plant movement on wet materials

These issues were addressed by the design cross section shown in figure 4 overleaf. Subsoils drains were placed on the stripped foundation at all potential seepage locations and at positions chosen to reduce effective drainage paths for excess pore pressure. Inspection points were provided on these drains for long term maintenance. Vibrating wire piezometers were installed in the compressible foundation materials and in the base lifts of the embankment fill to monitor pore pressure rise during fill placement. The weaker foundation materials under the toe zone were removed, and a free draining

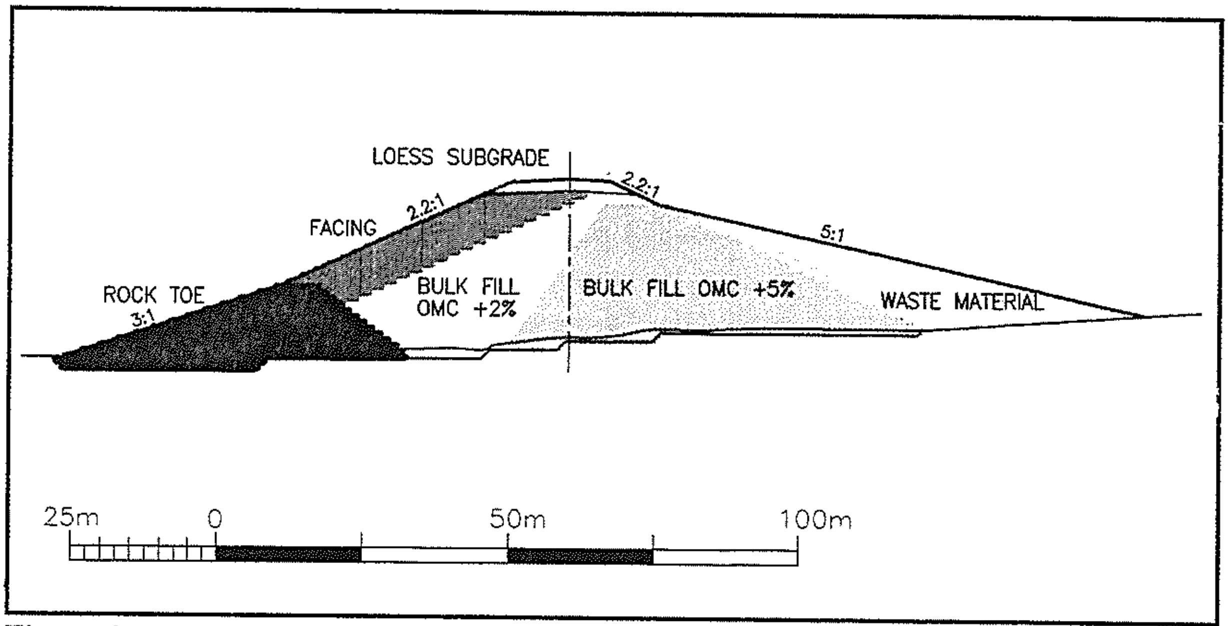

Figure 4 : Embankment Cross Section

rock fill toe was adopted to distribute the load over an enlarged footprint and to provide a base for the following shoulder placement. The geometry was established on the basis of stability being maintained under an allowable construction pore pressure rise of 50% of the embankment fill height. Rather than place the wet waste material from the cutting in a separate dump site, it was decided to utilise this material to increase stability of the upstream batter. The 1200 mm diameter aluminium culvert needed to be extended by some 50 m to accommodate this addition to the cross section, but this approach permitted less drying to be used on material placed within the upstream shoulder.

4. CONSTRUCTION PERFORMANCE

Shear vane testing of the prepared embankment foundation revealed that lower peak strength was available than that expected from interpretation of the investigations. Rather than extend the excavations to a greater depth, it was decided to rely upon the strength improvement obtained as the soil was loaded by the embankment fill. Only one of the installed piezometers (#2 @3540 m) showed a substantial pore pressure response to fill placement, and at this location it was not possible to keep the rise within the 50% limit because of the very slow rate of dissipation. The initial response was around 100% (ie 1 m pore pressure head increase per 1 m increase in fill height), and now some 24 months after full height was reached the pressure is at 26% which may be approaching equilibrium.

Post construction settlement of the embankment was monitored by survey levelling methods over the winter periods following progressive fill placement. Crest settlement was very much less than that predicted from the laboratory consolidation test results, being only in the order of several cm.

Even with the relaxation of the usual drying requirement for some of the embankment fill zones, substantial construction delays were incurred when weather conditions limited even the reduced rate of moisture reduction.

The cut batters generally performed well as they were designed at a moderate slope for reasons other than geotechnical considerations. Stability was expected to be adversely affected in the areas adjacent to faulting, but only one feature (shown in figure 5) required significant remedial attention. Blackish brown non calcarious claystone present around station 4450 m contains highly reactive clay minerals which give clay index test results in the range 10 to 20. When exposed to moisture variation or when disturbed by remoulding this material provides very little strength. The presence of this material in combination with a low angle fault resulted in a translational slide developing in the RHS cut batter. The repair strategy adopted was based on undercutting the failure surfaces and replacing the materials with non cohesive engineered fill which was predominately sand from the excavation. The batter needed to be unloaded somewhat to avoid further failure during the remedial work, and drainage features were incorporated to control seepage.

The shallow batter drains located under the topsoil layer were found to be necessary as anticipated. The location of concentrated seepage flows exiting from the cut batters following heavy rainfall were not easily located before the event, and several repairs were necessary. Uncemented sand exposed on a large cut batter was particularly vulnerable to erosion before drains were installed and vegetation cover was established.

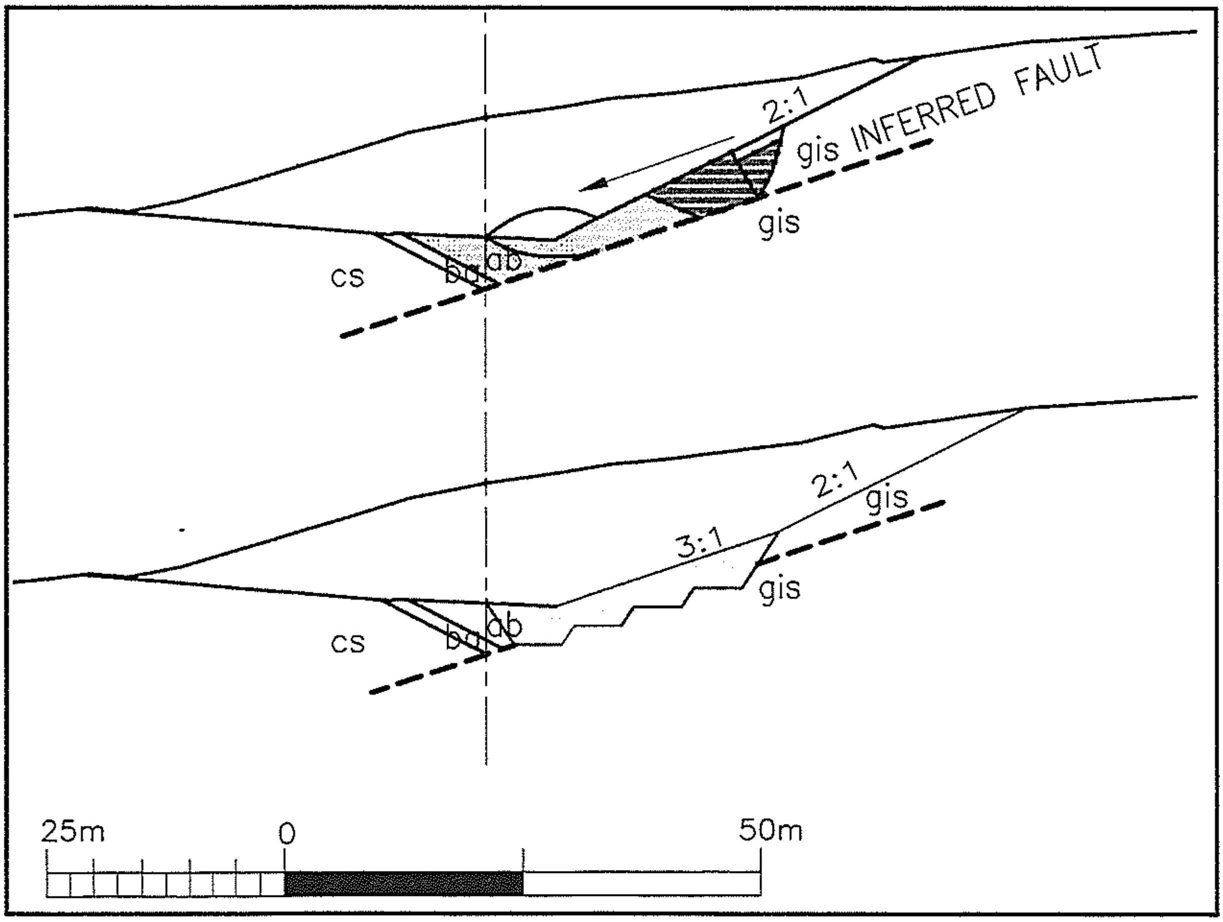

Figure 5 : Cut Batter Stabilisation

The embankment subgrade constructed from yellowish brown clayey silt (loess) did not perform to expectations derived from laboratory soaked CBR testing. Very large deflections were measured during proof testing of the prepared subgrade, and these effective low stiffness characteristics could not be accounted for by inadequate compaction or moisture control. Response to lime stabilisation was also poor, and well below expectations established from tests conducted on the borrow. Modified Benkelman beam testing utilising a lightly loaded truck (40% of the usual axle load) and low tyre pressure was used to examine the subgrade stiffness under a stress level similar to in service conditions, and thereby minimise the influence of non linear effects. A stiffness modulus of as low as 2 to 5 MPA (Ev) on the poorest layer between 400 mm and 1000 mm depth was obtained by back analysis, with an inferred CBR of around 10 on the stabilised surface and 6 below 100 mm depth.

The weak layer was compacted close to zero air voids, and it was subject to heavy rainfall following compaction. Remoulding is thought to have subsequently occurred in this saturated condition from the heavy construction traffic loading. Ironically the poor performance of the lime stabilised subgrade was thought to be due to inadequate moisture being available for full hydration.

As the end of the construction season was approaching, a risk assessment was commissioned by Transit New Zealand. The risk of major delay arising from reconstruction of the fill subgrade was considered to be too high, and an alternative pavement design incorporating 30 kN/m grid reinforcement was adopted. The additional aggregate depth required to be supported on the limited width embankment formation was retained by wrapping the grid around the shoulders.

5. CONCLUSIONS

Notwithstanding the construction progress difficulties that were experienced on this project due to the highly variable nature of the materials and the poor drying conditions, the embankment has performed better than expected and the use of zones of higher than usual moisture content has proven to be successful. Laboratory testing undertaken before this approach was adopted showed the expected substantial drop off in engineering performance as the placing moisture content was increased. However, provided that construction plant can operate satisfactorily, and the design takes account of the lower performance material, the end result can provide adequate levels of stability and stiffness.

The potential adverse effects of over compaction on sensitive subgrade was highlighted on this project. The operation of heavy construction plant can place unacceptable loading upon soil that has very low air voids, and laboratory derived design parameters may not be achieved. The use of a modified Benkelman beam test has been found to be useful in monitoring construction performance through the matching of in service stress levels on the layer under test. By minimising the influence of non linear effects associated with higher loadings, more confidence can be placed in back analysis results obtained from linear elastic programs such as CIRCLY. For subgrade testing, an unloaded truck with an axle load of around 40% of the standard test load is suggested, and a 70% axle load applies to testing subbase layers. Deflection readings around 0.3 m to 1.0 m positions are required in addition to the standard test positions, and tyre pressure should be lowered to the safe minimum. Allowance also needs to be made for the duration of test loading relative to the pulse of a moving highway vehicle.

EMBANKMENTS ON SOFT SOILS SETTLEMENTS AND SURCHARGE

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SUMMARY

The soil mechanics methodology for embankments on soft soils is well known. Design includes consideration of stability and settlement, with the latter extending into the means of accelerating settlement by the use of preloading, surcharging and vertical wick drains.
This paper sets out a useful method to simplify the analysis procedures, enabling embankment design heights to allow for settlements and enabling the benefits of surcharge loading to be easily evaluated.
The paper also includes a simple method for design of the drainage blankets which receive the water from vertical drains. Recent studies have shown that inadequate drainage blankets severely restrict the effectiveness of vertical drains.

1. INTRODUCTION

The purpose of this paper is to provide some simple practical methods to aid certain aspects of the design of embankments on soft soils, particularly in relation to settlements. The first concern of a designer may well be the stability of the embankment, but that is not the primary focus of this paper. After stability, performance requirements are usually expressed in terms of settlements following construction, i.e. the "in-service" settlements. A designer must not only allow sufficient embankment height to compensate for the settlement but also provide means of eliminating much of the settlement during the earthworks period, in order to meet the in-service settlement constraints. This paper sets out a method of developing a simple design chart for this purpose which enables the benefits of surcharge loading to be easily evaluated.
With soft soils, however, it is rarely possible to achieve the necessary degree of consolidation during a reasonable construction period without some means of accelerating settlement, such as the use of vertical wick drains. There has been much debate on the effectiveness of wick drains, with some case histories showing no detectable increases in rates of settlement. The second objective of this paper is to illustrate the importance of the drainage blanket design in this regard.
It is not the intention here to explain how settlements or rates of settlement are calculated or how stability is assured. It is assumed the reader is familiar with such methods through general knowledge or other sources.

2. EMBANKMENT HEIGHT

The required embankment height in generally controlled by alignment constraints and/or drainage and flood prevention. However, the geotechnical engineer must provide for sufficient fill to allow for settlement, and possibly add a bit more (the "surcharge") to help eliminate settlement during the set construction period.
Clearly this can be done by "trial-and-error" or with the aid of computers but the following method may be useful:

- Step 1: For a given subsurface design profile, calculate the load-settlement relationship and plot settlement(s) against height of fill (H) on a log scale.

- Step 2: Plot another curve of height-of-fill-minus-settlement (H_o = H-s) against height H. The H_o scale is where the designer starts with the required height of formation level above foundation level, and the corresponding value of H is the required height of fill to allow for settlement.

- Step 3: On the first of the graphs of Step 1, plot the associated 95%, 90% (etc) consolidation curves. (There is usually little point in showing anything less than 80% consolidation). Using these curves, draw a line of constant settlement from the design height (H) on the 100% line to (say) the 90% line. The difference on the H-scale is the surcharge which will permit a lower degree of consolidation (e.g. 90%) for the same final settlement.

- Step 4: Compute the secondary settlement for an appropriate period (e.g. 10 years) and plot the combined primary and secondary relationships alongside the curves of steps 1 and 2. These will then allow fill heights and surcharge heights to be determined providing for specified secondary settlements as well as primary.

The above steps are illustrated in Figure 1 and an example of a full design chart is shown in Figure 2. In using the design charts, the following factors become evident:

i) Because of the log scale of H, the surcharge requirement tends to be large compared with the original height requirement (H).

ii) In order to limit surcharge heights to something practical, it will be necessary either to aim for degrees of consolidation greater than 80% during construction, or to accept significant in-service settlement.

To achieve the required degree of consolidation, the use of vertical wick drains would likely be necessary. It is not intended to dwell on this aspect here but the design charts produced by the makers of the proprietary drain types are generally suitable for the purpose. An example of such a chart (produced by Mebra-Drain) is reproduced in Figure 3.

3. DRAINAGE BLANKET

A drainage blanket between the fill and the foundation soils is generally necessary to accept the expelled pore water from the drains. Often, attempted savings in this component destroy the effectiveness of the drains.

It can be shown (see the Appendix to this paper) that the pressure head in a drainage blanket with confined flow is given by:

$$h_L = \frac{1}{2} \cdot L^2 q/kt \quad \text{...} (1)$$

where:

h_L = pressure head in the blanket at distance L from the exit point (m)
q = flow into the drainage blanket per unit area (m^3/s per m^2)
k = permeability of drainage blanket material (m/s)
t = thickness of drainage blanket.

The unit flow rate (q) is simply the rate of settlement which gradually reduces with time. The average rate of settlement for the first 25% consolidation is approximately five times the average rate over the 90% consolidation range. It is necessary to design the drainage blanket for the initial rate or there will be severe impedance to drainage.

Also, it is necessary to maintain the pressure head in the blanket as low as possible to ensure the vertical wick drains work efficiently. Anything more than one or two metres (or say 10% of the applied load) would significantly slow the rate of settlement.

Consider an example:

- A road embankment for a dual carriageway, together with stabilising berms and flat side slopes, could well have a drainage path (L) of about 40 metres.
- Assuming a design requirement to achieve an average settlement rate of one metre in a year, the design flow rate would need to be at least five times this, which gives $q = 1.6 \times 10^{-7}$ m/s.
- Assume a fine sand is available for the drainage blanket, with a permeability $k = 10^{-4}$ m/s.
- Using Equation (1), the thickness of drainage blanket required to limit the maximum pressure head in the drainage blanket to (say) 1.5 m would be 850 mm.

So, any temptation to reduce the blanket to less than this wold severely inhibit the rate of consolidation. However,

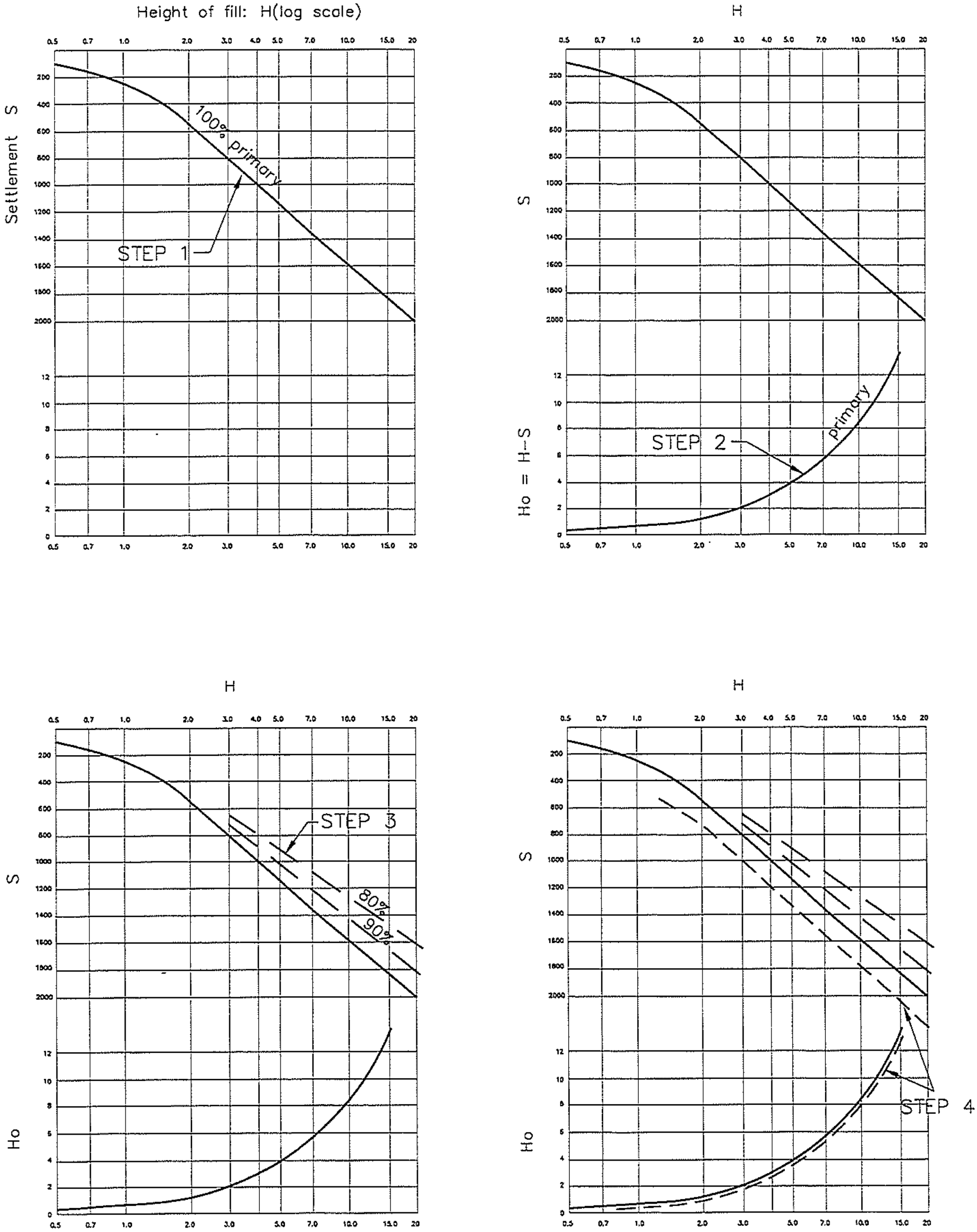


FIGURE 1: PROCEDURE FOR SURCHARGE DESIGN CHART

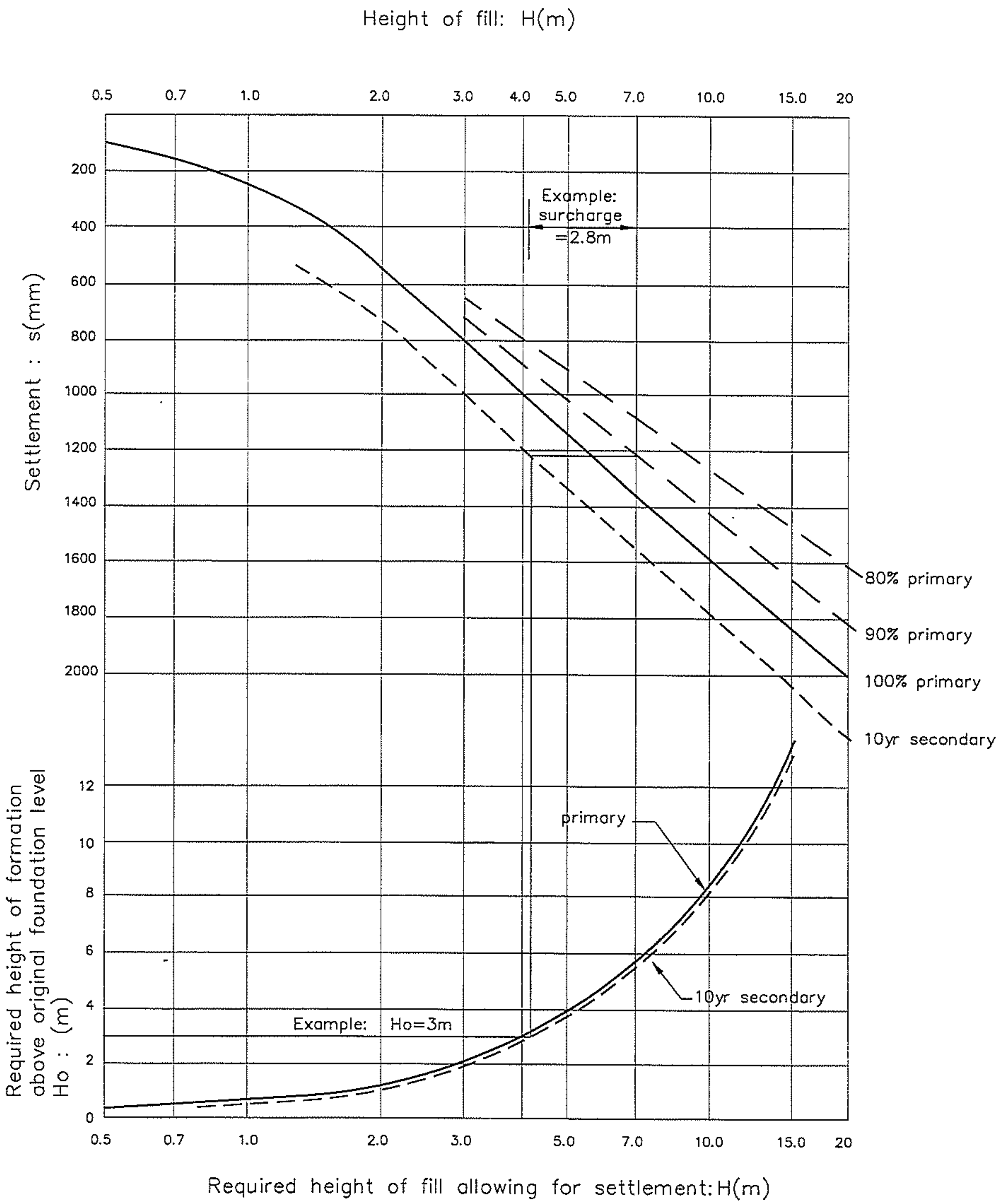


FIGURE 2: EXAMPLE OF SURCHARGE DESIGN CHART

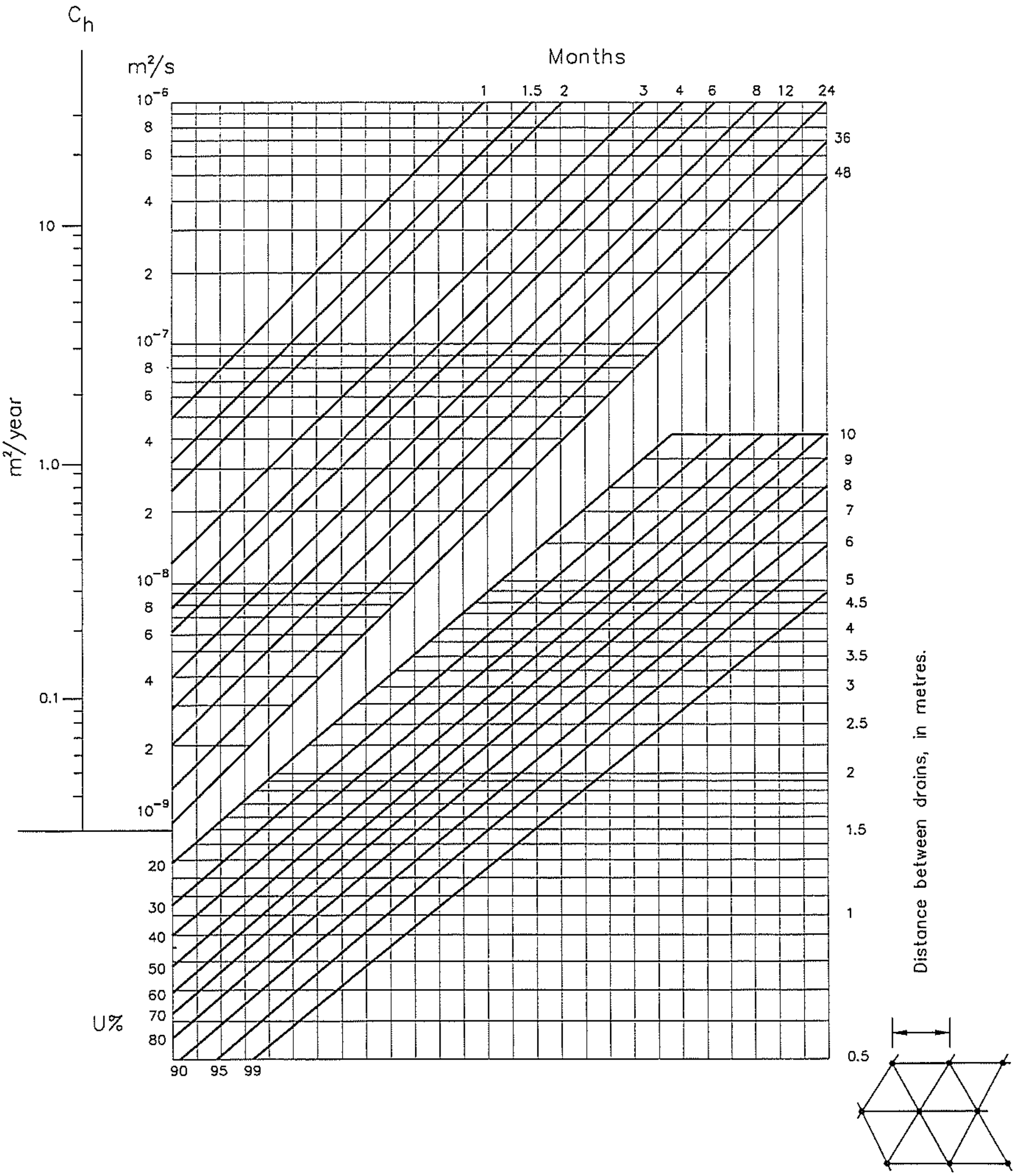


FIGURE 3: DESIGN CHART FOR 100mm BAND (WICK) DRAINS
(Derived from Mebra–Drain brochure)

it is common practice to use drainage blankets of 400 to 500 mm thickness, a cost saving which may completely negate the benefit of the drains. A small investment in a thicker or more permeable blanket may be the difference between satisfactory performance and failure.

Equation (1) is for confined flow throughout the blanket. If the blanket thickness in sufficiently large, the flow will be partially unconfined and there would be a more complex relationship. Assuming zero head at the exit point, wholly unconfined flow would occur if:

$$t \geq L\sqrt{q/k} \qquad (2)$$

For the example above, this would apply with a blanket of 1600 mm thickness.

The two equations give a useful and simple method of assessing the efficiency of a drainage blanket.

4. CONCLUSIONS

The paper considers two aspects which might be considered for the design of embankment on soft soils.

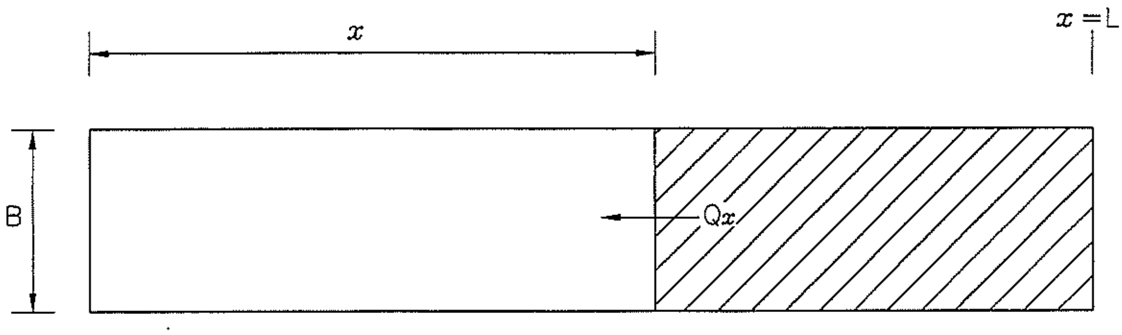
i) A simple method is described enabling fill heights to be determined to allow for primary and secondary settlements, and to enable the effectiveness of a surcharge to be evaluated. It is evident from this that a surcharge needs to be a significant proportion of the overall embankment height for it to have a marked effect.

ii) Two simple formulae are given which enable the effectiveness of a drainage layer to be assessed. Inadequate provision for drainage for vertical wick drains will seriously impair their effectiveness.

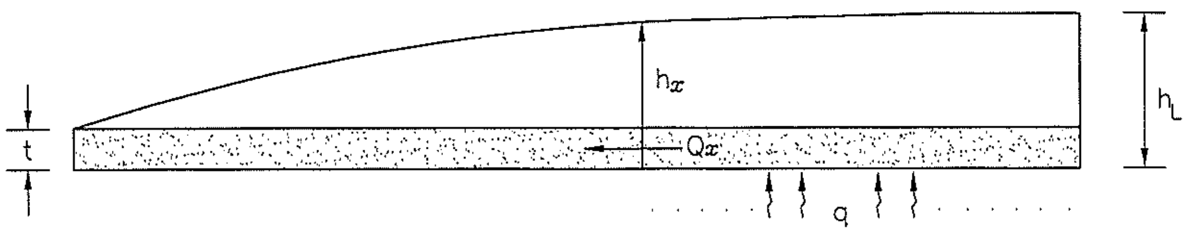
5. APPENDIX: Capacity of Drainage Blanket

Consider a strip of blanket of length L, width B and thickness t, as shown in Figure A1:

- Assume flow from wick drains $= q$ $m^3/s/m^2$ (m/s)
- At distance x, flow Q_x = flow into shaded areas = $(L-x).B.q.$
- Also, by Darcy's Law, $Q_x = k.B.t.\, dh/dx$ (for confined flow)
- so $dh/dx = q(L-x)/kt$
- and integrating: $h = \frac{1}{2}.q.(2L-x)x/k.t.$
- and at x=L, $h_L = \frac{1}{2}.L^2.q/k.t.$
- Alternatively: $Q_x = k.B.h.\, dh/dx$ (for unconfined flow)
- so $h.dh/dx = q(L-x)/k$
- and $h^2 = q(2L-x)x/k$
- and at x=L $h_L^2 = q.L^2/k$



PLAN



SECTION

FIGURE A1: DRAINAGE BLANKET

GEOGRID REINFORCED LIGHT WEIGHT EMBANKMENT ON STONE COLUMNS

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SUMMARY

As part of the South Eastern Arterial Road project in Auckland, a 2m to 4m high geogrid reinforced embankment with steep side slopes was built as an alternative to a section of the bridge structure to span the soft ground for the Carbine Road section of the route. The embankment is underlain by soft ground and is in close proximity to three adjacent structures. Stone columns were installed along the edges of the embankment to strengthen its foundations and reduce the influence of ground movement on the adjacent buildings. Light weight sand fill was used in the embankment to reduce the ground settlement. Extensive stone column loading tests and ground monitoring were undertaken. Ground level measurements confirmed that the design successfully eliminated the influence of the embankment settlement on the adjacent buildings.

1 INTRODUCTION

Auckland City, the Principal, called for design-build tenders for the South Eastern Arterial Road project. Connell Wagner was engaged by Downer Construction Ltd as its design consultant for the two major bridge structures; the 475m long Sylvia Park Viaduct and the Southern Motorway Underpass. In design-build projects, designers are required to develop cost effective design solutions to assist the contractor to win the contract. At the eastern end of the Sylvia Park Viaduct, Connell Wagner proposed to strengthen the soft ground with stone columns and construct a geogrid reinforced embankment as an alternative to using the bridge to span the soft ground. The proposal shortened the viaduct structure by two spans and resulted in substantially saving in the construction cost of the route. Three existing structures are situated between 2m and 10m from the edges of the embankment.

The embankment makes use of the following techniques (i) four rows of stone columns around the perimeter of the embankment to strengthen the ground, (ii) light weight sand fill to reduce the imposed load and hence ground settlement, (iii) reinforcement of the 1:4(horizontal:vertical) slopes with geogrids (iv) geotextile basal reinforcement to increase the short term stability and the seismic resistance of the embankment and (v) wick drains within the central unreinforced zone of the embankment to increase the consolidation rate. A high frequency vertical vibration compaction technique was developed for compacting the stone placed in the pre-bored stone column holes to minimise the installation effects on nearby structures. This paper presents the design philosophy, methodology, requirements and monitoring results for the embankment.

2 GROUND CONDITIONS

The site is underlain by a 22m thick layer of alluvium which slightly increases in strength with depth, and is interbedded with a 2m to 3m thick layer of basaltic lava at a depth of 6m to 7m below the ground surface level. The ground surface layer is a 1.5m thick firm to stiff volcanic ash. Below the ash layer and above the basalt material, a 4m to 5m thick layer of soft to firm alluvial clayey silt/silty clay is present. The upper alluvial layers are generally under-consolidated and occasionally thinly interbedded with peat and sand layers. Typical vane shear strengths of 20kPa to 35kPa were measured in the clayey silt/silty clay layer. Cone penetration tests (mechanical cone) indicated the cone tip resistance is generally below 0.6MPa in this material. The groundwater level is close to the ground surface during the winter.

Below the basalt layer there is 10m to 12m of firm to stiff alluvial silty clay with typical standard penetration test N values in the order of 5 but values of 10 to 15 are occasional encountered. Consolidation test results generally indicated that this lower silty clay layer is over-consolidated with an over-consolidation ratio of 3-4. The alluvium is underlain by the so-called Waitemata formation comprising interbedded layers of mudstone, siltstone and sandstone. The weathering of the Waitemata formation decreases rapidly with depth.

3 DESIGN PHILOSOPHY

It was estimated that the ultimate bearing capacity of the upper alluvium under the 23m wide embankment is in the order of 100kPa to 150kPa. However, analyses showed that there was an insufficient margin of safety against rotational failure for the proposed 1:4(Horizontal:Vertical) side slopes for an embankment higher than 2.5m. Because the lower silty clay layer is over-consolidated the induced settlement of this layer is likely to be small provided the embankment load transferred to it is less than its pre-consolidation pressure. Conservative analyses showed that the settlement of this layer would be less than 20mm for an embankment height of less than 4m. In consequence strengthening the upper layer would be sufficient to ensure the stability of the embankment and eliminate the influence of the embankment settlement on the adjacent structures. A typical cross section of the embankment is shown in Figure 1. Based on these considerations, the combination of the following techniques were employed to improve the performance of the embankment.

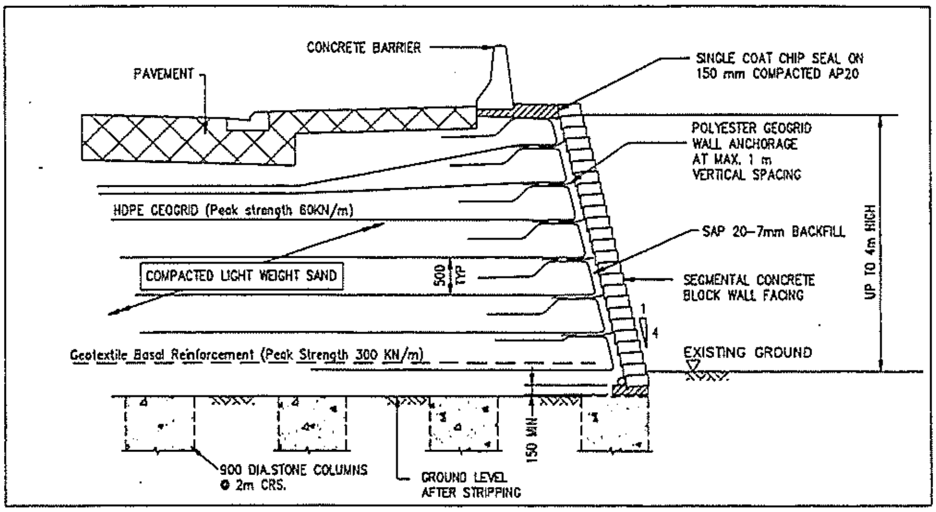

FIGURE 1 Typical Cross Section of Embankment

Embankment side slope geogrid reinforcement - The width available for embankment was tightly constrained by the site boundaries necessitating the use of 1:4 batter slopes to the embankment. The, internal stability of the embankment could be ensured by using HDPE geogrid reinforcement with a peak tensile strength of over 55kN/m at a vertical spacing of 500mm. Segmental concrete block wall (Keystone) facing was tied by high modulus polyester geogrid layers extending from the embankment for the protection of the HDPE geogrids. Because of the expected consolidation of the embankment during the construction stage, the segmental block wall facing was installed after completion of the consolidation settlement of the embankment.

Stone Columns and Basal Geotextile Reinforcement_ - The design requires that the stone columns on their own will ensure the side slope stability of the embankment after full consolidation. A layer of woven geotextile with a 300kN/m peak tensile strength was placed at the base of the embankment as basal reinforcement to ensure an acceptable margin of short term stability during the two stage construction with a minimum holding period between stages. The basal reinforcement layer will become redundant after consolidation under normal loading. Under extreme loading conditions such as during a seismic event, the basal geotextile reinforcement will come into effect again to maintain the stability of the embankment. Failure of stone columns under seismic loads is unlikely. It has been found that stone columns exhibit excellent capacity to resist horizontal shear in simulated seismic ground acceleration experiments, Engelhardt and Golding (1975).

Stone columns of 900mm diameter were arranged in four rows at 2m centres around the perimeter of the 2 to 4m high embankment No ground improvement was carried out where embankment height was less than 2m. Stone columns were also designed as vertical drains to shorten the time required for porewater pressure dissipation of the surrounding soil.

Temporary sheet pile walls - The stone columns and basal geotextile reinforcement were designed to reduce the ground deformation. Steel sheet pile walls were employed as a precautionary measure to isolate the three adjacent structures from the effects of the construction of the stone columns and the settlement of the embankment.

Wick Drains - In the central part of the embankment where stone columns are not installed porewater pressure may have built up under the embankment loading therefore wick drains were installed to reduce the time for the porewater pressure dissipation.

4 DESIGN REQUIREMENTS

Because of the limited construction time available for the entire project, it was necessary to design the embankment to have a satisfactory factor of safety during embankment construction which had the minimum holding periods between loading stages. In consequence the stone columns were designed to maintain the side slope stability with a minimum FOS>1.3 for the short term stability and a FOS>1.5 for the long term. The spacing and the diameters of the stone columns were designed to reduce the predicted ground settlement to negligible amounts at the adjacent structures. Basal geotextile reinforcement was found to be required if the FOS for the short term stability and the seismic resistance of the embankment were less than 1.3. The spacing of the stone columns and the wick drains was also designed to ensure that the 90% of the primary consolidation was complete within one year of imposing the fill load.

5 DESIGN METHODS

Design of Stone Column Group in Clay

For stone column groups in clay under an embankment, the stress conditions within the clay-stone columns are very different from that of a single column. Shear failure of individual columns is unlikely to occur and field observations and centrifuge testing by Osborne (1994) and Stewart and Fahey (1994) indicate that stone column groups performed satisfactorily under a 20m high iron ore stockpile and failure of individual columns was unlikely. However, when columns are placed under the side slopes of an embankment, gross bearing capacity failure is likely to be the critical condition. Slope stability analyses may be carried out using a conventional method considering the properties of the composite material of the foundation.

The "equivalent soil-stone column block" and "equivalent trench" approaches can be used for the modelling of soil-column foundations. The "equivalent soil-column block" method proposed by Almedia and Parry (1985) assumes that the reinforced zone can be represented by an equivalent soil block with strength properties based on the contribution of the foundation soil and the columns. The properties of the block are influenced by the relative stiffness of the in-situ soil and the column materials. The ratio of the vertical stress in the stone column σ_s to that in the clay σ_c is defined by the stress concentration ratio n as defined below:

$$n = \sigma_s / \sigma_c \qquad (1)$$

Mitchell and Katti (1981) suggest that n is in the range of 2 to 6 and usually has a value of 3 to 4 for stone columns in clay. The replacement ratio a_s representing the area of the clay foundation replaced by a stone column is given by:

$$a_s = A_s / (A_s + A_c) \qquad (2)$$

where A_s denotes the cross sectional area of the stone column and A_c the plan area of the clay foundation per column. Under an average applied pressure σ from the embankment, the stress taken by the clay foundation and stone column is given by:

$$\sigma_s = \beta_s \sigma \quad \text{and} \quad \sigma_c = \beta_c \sigma \qquad (3)$$

where $\beta_c = [1 + (n-1) a_s]^{-1}$ (4a)

and $\beta_s = n \beta_c$ (4b)

in which σ_s and σ_c represent the stresses taken up by the stone column and the clay foundation respectively. Based on the parameters obtained from equations (1) to (4), the equivalent block properties as given by Almedia and Parry (1985) are:

$$\tan \phi'_e = m \tan \phi'_s + (1 - m) \tan \phi'_c \tag{5a}$$
$$c'_e = (1 - m) c'_c \tag{5b}$$

where

$$m = a_s \beta_s \tag{6}$$

ϕ' and c' denote the effective friction angle and cohesion, respectively. Subscripts, e, s and c denote the parameters for the equivalent clay-column block, stone column and clay foundation respectively.

In the equivalent trench method, Stewart (1995) proposed that the individual columns are defined in the slope stability analysis with their dimensions modified to convert the three dimensional layout to an equivalent two dimensional geometry. For example, the centre to centre spacing of the columns should remain the same, but the column width should be reduced to equivalent trenches in the analysis. The equivalent trench width is given by:

$$W = A_s / S \tag{7}$$

where S is the column spacing as above. To account for the stress concentration in the columns under the embankment load in the analysis, the friction angle of the stone columns is factored up by the β_s value in Eq. (4b) as given below:

$$\tan \phi'_w = \beta_s \tan \phi'_s \tag{8}$$

Settlement Prediction

The elastoplastic finite element method for the estimation of the settlement of stone column groups proposed by Balaam N P and Booker J R (1985) was used in the design. In the method, the elastic settlement of the stone columns was first calculated from a set of analytical equations. The estimated elastic settlement was then corrected using design charts to account for the plastic deformation of the column material for the calculation of the total settlement. The stiffness of the stone columns is dependent on that of the surrounding soil. Typically, published values of the Young's modulus for stone columns is in the range of 20 to 40 times that of the surrounding clay. The stone column load versus settlement relationship can be calculated from the method. A two dimensional finite different analysis using the computer program FLAC was also carried out to evaluate the ground deformation outside the embankment.

Stone Column Materials

Uniformly graded materials with a particle size in the range of 20mm to 60mm are typically used for stone columns however, the ineffectiveness of stone columns in providing drainage and reducing settlement because of clay infiltration from the surrounding soil into the stone column has previously been reported, Garga and Medeiros (1995). In consequence, in this project, gap graded minus 65mm greywacke crushed rock with less than 20% of material passing 4.75mm sieve was used. By adding some finer materials to reduce the open voids of the stone column material, the amount of clay infiltration into the stone columns was reduced. The grading was designed to ensure good drainage and mobilisation of the column strength after a relatively small settlement.

Consolidation Time

The stone columns installed under the edges of the embankment served as vertical drains to accelerate the consolidation process. The coefficient of consolidation of the soil has been assumed to be in the range of 1 to 3 m^2/year. Based on the analytic solution proposed by Barron (1948) for radial drainage, the time required for the soil around the stone columns to reach 90% of consolidation would have been in the range of 6 months to 1 years after applying the fill load.

Embankment Stability and Basal Geotextile Reinforcement

Analyses of both the internal and external stability of the embankment supported on the stone column reinforced foundation was carried out using the slope stability analysis program SLOPE. Geogrid reinforcement for maintaining the stability of the side slopes (internal stability) and basal geotextile reinforcement were modelled. Both equivalent soil-column composite block and equivalent trench methods were used for comparison in the design.

6 DESIGN PARAMETERS

In the analyses, the following material parameters were used:

Embankment

Density of light weight sand fill	= 14kN/m³
ϕ' of sand fill	= 30°
Surcharge on top of embankment	= 10kPa

Stone Column

Column Diameter	= 0.9m
Column Spacing	= 2.0m
ϕ' of column material	= 35°
Stress concentration ratio, n	= 3

Clay Foundation

Undrained shear strength of clay	= 20kPa
Young's Modulus	= 1.5MPa
ϕ' of clay	= 18°

HDPE Geogrid Side Slope Reinforcement

Peak tensile strength	=60kN/m
Long term design strength	=18kN/m

Woven Basal Geotextile Reinforcement

Peak tensile strength	= 300kN/m

7 CONSTRUCTION

The stone columns were installed in 900mm diameter pre-bored holes taken down to the basalt rock at about 6m depth. After boring the hole a steel casing was lowered into it to maintain the stability of the hole walls. The stone column material was poured into the hole in 1m lifts. The casing was then withdrawn to the top of the loose material. The material was compacted for four minutes under a 800mm diameter hammer head connected to a vibrating hammer above the ground via a steel I beam. The vibrating hammer had an eccentric moment of 0.05kN-m at 50Hz. It was found that the loose material was compacted to the range of 40-70% of its loose lift height. During the stone column construction, very little ground vibration was felt beyond about 10m from the column under construction.

The embankment was constructed in two stages. The first stage brought the fill height up to 2m high at a rate of 1m per week. Embankment settlement and porewater pressure responses were monitored on a weekly basis. After a 1 month holding period, the embankment was constructed to its full height at the rate of 1m per week. One year after the embankment construction was completed measurement showed that the embankment settlement was complete. The segmental concrete block wall facing was then constructed.

8 RESULTS

Plate bearing tests with up to 250kN of axial loads were carried out on 20 stone columns. Measured stone column settlements under the test load were generally less than 15mm. No indication of failure of stone column was observed. Back analysed Young's modulus values of the stone columns were in the order of 160MPa. This is about 100 times of that of the surrounding clay and is significantly higher than the published values of 20 to 40 times.

Measurements of settlement gauges installed in the unreinforced zone of the embankment indicated ground settlements of 100mm under 2m of the light weight fill. This was in good agreement with the predicted settlement. The embankment settlement rate became very small 6 months after construction. Within the stone column reinforced zone, ground settlements of 40mm to 70mm were measured under the 4m high embankment load. At a distance of about 2m outside the embankment area, no detectible ground deformation was recorded in the deformation surveys.

9 CONCLUSIONS

This paper presents the design philosophy, methodology, requirements and monitored results for a geogrid reinforced light weight sand embankment with steep side slopes on a stone column strengthened foundation. Uniformly graded 20 to 60mm gravels are commonly used for stone column construction however, because of clay infiltration into stone columns, ineffective drainage and settlement reductions were previously reported. To overcome these shortcomings, finer materials were used to reduce the open voids in the stone column material. Load testing of stone columns indicated that the stone column settlement was generally less than 15mm under a 250kN test load. Back calculated Young's modulus values for the stone columns were in the order of 160MPa and were about 100 times that of the surrounding clay.

The monitored results indicated that the stone columns successfully strengthened the soft clay site enabling the construction of an embankment of 2m to 4m high. It was founded that the embankment settlement had a negligible influence on the adjacent structures although the structures were within 2m to 10m from the edge of the embankment. Furthermore the adoption of a vibrating hammer to compact the stone columns virtually eliminated nuisance from vibration during the stone column construction.

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GEOTECHNICAL DESIGN OF THE SOUTHERN MOTORWAY UNDERPASS

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SUMMARY

The Southern Motorway Underpass takes the new South Eastern Highway beneath the Southern Motorway (State Highway 1), one of the busiest roads in Auckland. One of the major requirements for the construction of the Southern Motorway Underpass was to minimise disruption to the motorway traffic. This was done by building the northern half of the bridge and moving traffic onto it before building the southern half of the bridge and also by adopting the top down method of construction. The top down method was achieved by initially constructing the soldier piles for the abutment walls, tying back to deadmen with pre-stressed anchors and initially excavating just sufficient material between the bridge abutments to construct the piers and place the bridge beams. To reduce the depth of temporary sheet piling required between the north and south halves of the bridge the remaining bulk excavations beneath the bridge deck were only carried out once both halves of the deck were opened to traffic. The skewness of the bridge meant that the distance between the bridge abutments and deadman had to be unusually short to keep the deadmen within the motorway embankment. Because of the soft ground conditions, there was potential for the 6.5m deep excavations to induce lateral abutment wall movements which would not have been acceptable to the bridge or the pavement of the motorway. This paper address the geotechnical design and construction aspects of the project.

1 INTRODUCTION

The Southern Motorway bisects the industrial area of Penrose in the west and the light commercial and residential area of Mt Wellington in the east. Prior to the completion of the South Eastern Highway , the nearby Mt Wellington Highway was one of the most heavily trafficked urban roads in Auckland and it was usually heavily congested during peak hours. The South Eastern Highway provides a convenient traffic link between the western and eastern areas via the Southern Motorway Underpass beneath the Southern Motorway.

The Southern Motorway (State Highway 1) is one of the busiest roads in Auckland and any significant disruption of the traffic due to the construction of the underpass was unacceptable. The first step in the construction of the underpass was to divert the northbound traffic on the motorway away from the bridge onto what was later to become a part of the South Eastern Highway and moving the southbound traffic onto the northbound carriageway . After the diversion was completed the northern half of the underpass which would carry the future southbound motorway lanes was constructed using the top down construction technique. Soldier piles and the dead men were constructed for the northern half of the bridge. The prestressed anchors between the abutments and the deadmen were placed in directionally drilled holes and stressed. Only enough soil was excavated between the bridge abutments to construct the bridge piers and place the deck beams. After the completion of the northern half of the underpass southbound traffic was moved onto it and construction was then commenced on the southern half of the bridge. Once both bridge decks were completed and opened to motorway traffic the bulk excavations beneath the new bridge were commenced. As excavation progressed the gaps between the soldier piles were closed with fibre reinforced shotcrete arches. Finally precast concrete panels were placed in front of the soldier piles to enhance the appearance of the bridge.

The underpass is a highly skew two span bridge designed to carry for a total of 7 lanes of traffic. Its construction involved a 6.5m deep excavation in soft ground, Figures 1 and 2. The bridge decks carrying the north and southbound carriageways are separated along the centerline of the motorway. Because of the bridge's skewed geometry it was not practical to use the decks as props for the abutments. Instead, the soldier pile abutment walls were tied back to deadman anchors by prestressed anchors. The combined resistance of the piers and abutments was needed to provide sufficient seismic restraint for the decks. This was achieved by connecting the decks to the piers with fixed bearings and gaining further resistance by utilising the shear stiffness of rubber bearings at the abutments. As a result only a limited amount of abutment movement could be accommodated. As the motorway is on an embankment, the very high skew of the bridge meant that the deadmen had to be placed closer to the bridge than normal practice would require. In order to address these two aspects one dimensional and two dimensional elastoplastic soil modelling were employed in the design to obtain an understanding of the interactive behaviour between the bridge abutment, the deadman anchor and the ground movement caused by the excavation.

2 GROUND CONDITIONS

The ground underlying the underpass site varies from one end to the other. Based on information from four boreholes, the ground profile may be generalised by a four layer system. A typical ground profile is shown in Figure 3. The near surface material is a volcanic ash layer varying from 2m to 4m thick. Beneath the volcanic ash is an alluvial silty clay layer varying in thickness from 7m to 14m with standard penetration test SPT N values in the range of 4 to 10 however N values as low as 2 were also recorded in one of the boreholes. The alluvium is underlain by a layer of residually to moderately weathered Waitemata mudstone, siltstone and sandstone. Its thickness varies from 5m to 10m with N values typically between 7 to 30. The underlying slightly weathered Waitemata formation generally has N values of over 50.

3 SOIL PARAMETERS

Drained and undrained analyses were carried out in the design. The following effective stress parameters which are based on a limited number of triaxial test results around the project site, were used in the drained analysis:

Volcanic ash	$c' = 3$kPa	$\phi' = 30°$
Alluvial silty clay	$c' = 15$kPa	$\phi' = 18°$
Residual to moderately Weathered Waitemata formation	$c' = 10$kPa	$\phi' = 24°$
Slightly Weathered Waitemata formation	$c' = 15$kPa	$\phi' = 35°$

For the undrained analyses, undrained shear strength Su values for the soils were estimated from the SPT N blow counts measured in the boreholes based on Su = 10N kPa. Undrained Young's modulus values were estimated to be related to 200Su. The total densities of the soils are assumed to be $1.85 t/m^3$.

4 DESIGN PHILOSOPHY

The underpass passes obliquely through the embankment beneath the existing motorway. Because of its skewed geometry, the earth pressure from the two abutment walls will generate a torque and cause the underpass to rotate in plan if the decks are used to support the abutment walls. One option to eliminate the torque is to tie back the soldier pile abutment walls with either rock anchors anchored in the slightly weathered mudstone and sandstone materials at about 20m depth or anchors tied to deadmen located in the upper soil layer.

It was estimated that the cost for the rock anchors was likely to be in the order of 500% higher than the deadman anchors. The deadman anchors were therefore employed on the project. The adoption of the deadman anchors required careful consideration for the following reasons:

- The excavation for the underpass is within a softer soil zone and it could induce both lateral ground movements affecting the completed bridge and settlement of the existing motorway pavement.
- As the deadmen are located just above the softer material the excavation could induce the deadman and the wall to move together.
- As a result of the skewness of the bridge, the deadmen have to be brought so close to the abutments as to render them ineffective.

5 STRUCTURAL LAYOUT

The sizes of the soldier piles and the deadmen and the capacity of the tie-back anchors adopted for the underpass are given below:

- 0.9m diameter reinforced concrete piles at 2.6m centre-to-centre spacing within the abutment wall area founded at ±20m depth within the slightly weathered Waitemata mudstone, siltstone and sandstone formation.
- 2m high deadman located about 12.5 m behind the abutments and buried 1m below ground surface
- Tie back anchors of 1750kN minimum breaking capacity were installed at 5m spacing.

6 DEADMAN DESIGN

The ultimate bearing capacity of the deadman in the horizontal direction was derived from Prantl's solution or the so-called "punching shear" problem of classical plasticity theory. In the design, the undrained shear strength of the surrounding soil was assumed to be 50kPa and the contribution of the soil confinement behind the back of the deadman to the bearing capacity was ignored. The method assumes that no soil movement occurs outside the shear zones enclosed by the shear surfaces in front of the deadman. Soil excavation in front of the soldier pile wall may however induce small ground movements around the deadman. To mobilise the bearing resistance of the deadman, the actual deadman movement must be larger than the excavation induced ground movement. If the relative movement between the deadman and the ground is small, the ultimate bearing capacity of the deadman cannot be developed. Therefore, the ultimate bearing capacity of the deadman may not be a governing factor.

7 SOLDIER PILE WALL DESIGN METHODS

The geometry and the layout of the underpass's soldier pile abutment walls and the deadmen are shown in Figures 1&2. In the design, sensitivity analyses were undertaken at various cross sections along the abutment walls using a range of selected soil parameters and the methods listed below.

(i) One dimensional limit equilibrium method for soldier pile walls without tie back anchors.
(ii) One dimensional elastoplastic finite element method for an embedded cantilever wall with and without tie back anchors using the computer program Wallap
(iii) Two dimensional elastic finite difference method using the computer program FLAC for the soldier pile walls subjected to stage excavation.
(iv) Two dimensional elastoplastic finite difference method using the computer program FLAC for the soldier pile walls subjected to stage excavation

One Dimensional Limit Equilibrium Method

The method proposed by Burland et al (1981) which is commonly used for embedded cantilever walls was used. To ensure that the abutment walls have sufficient stability at all times, the soldier pile walls were considered as an "unpropped" wall. A minimum factor of safety of 1.5 against overturning failure was selected. Both drained and undrained analyses were carried out to determine the governing condition. When the active earth pressure in undrained cohesive soil is small or negative, a minimum earth pressure calculated from a minimum equivalent fluid density of $5kN/m^3$ was used in accordance with codes of practice CP2.

One Dimensional Elastoplastic Finite Element Analyses

The analyses were undertaken to evaluate the bending moment envelope for the soldier pile wall and the load at the deadman anchors caused by the excavation. In the method, the wall was considered as an elastic wall while the soil was assumed to remain elastic until its stress state reached the given yield envelope. Analyses for the both free cantilever and deadman tied back conditions were also carried out. Deadman movement of 1-5mm were also modelled to check for the influence of the deadman movement on the bending moment of the wall. The analyses showed that the undrained soil condition generated the most unfavourable bending condition in the soldier pile wall.

Two Dimensional Elastic Finite Difference Analyses
To evaluate the interactive behaviour between the bridge abutment, the deadman anchor and the overall ground movement caused by the excavation, two dimensional finite difference analyses were performed. Both the pile and the soil were modelled as elastic materials. The results of the one dimensional finite element model were used to calibrate the validity and accuracy of the 2D model. The following construction stages were analysed:

(i) Excavate 2m of the upper soil for the installation of the tied back anchors
(ii) Pre-stress the tie back anchors to the design load estimated from the one-dimensional Wallap analysis.
(iii) Excavate the soil to the final excavation depth of 6.5m.

Two Dimensional Elastoplastic Finite Difference Analyses
A Mohr Coulomb elastoplastic soil model was also used in the analyses. In the elastoplastic model, the material response was assumed to be elastic for any stress state of the soil inside the defined yield envelope. Plastic flow would occur when the stress state reached the yield envelope. The results of the 2D elastic model were first used to calibrate the validity and accuracy of the elastoplastic model. Parametric studies using a range of soil parameters and various anchor lengths were then undertaken to evaluate the interactive behaviour between the bridge abutment, the deadman anchor and the overall ground movement caused by the excavation.

8 RESULTS OF ANALYSES

A series of undrained analyses were undertaken based on soil parameters calculated from the SPT results from the boreholes at the four corners of the underpass. The two dimensional finite difference model is shown in Figure 4 and the calculated ground movements after the completion of the excavation at the location of borehole BH11, where softest ground condition is located, are shown in Figure 5. Generally, it was found that the differences between the 2D elastic and elastoplastic models were not significant indicating that the stress system within the ground after the excavation remained elastic. The range of results for the excavation induced movement on the deadman tied back soldier pile wall from both one and two dimensional models are summarised below:

Table 1 Comparison of Results

	Calculated Movement	
	1-D Wallap Analyses	2-D FLAC Analyses
Top of wall movement after 2m excavation	3-10mm	5-15mm
Top of wall movement after 6.5m excavation	10-40mm	12-45mm
Deadman movement after 6.5m excavation	1-5mm (Assumed input parameter)	3-10mm

Generally, the 2-D finite difference analyses indicated that minor deadman movement due to the excavation is unavoidable because of the presence of the softer layer below the deadman level. The analyses showed that, after the installation of the anchor, subsequent soil excavation would induce about 5% additional anchor load. It also indicated that moving the deadman further from the wall would result in very little advantage over the adopted scheme because a longer anchor tie would have a greater elongation under the excavation induced load.

The results generally indicated that it was not possible to eliminate the soldier pile wall movements induced by excavation. It was therefore desirable to carry out as much excavation as possible prior to the placing bridge deck beams. Furthermore the results showed that the excavation induced ground movements would increase the anchor tie load above the initial pre-tension load. It was therefore important to allow for the load increase in determining the pre-tension load to avoid overstressing the anchors ties.

9 CONSTRUCTION AND MONITORING RESULTS

From the results of the analyses, it was believed that soldier pile abutment wall movements in the order of 10mm-30mm could to be expected due to the excavation. It was therefore decided that after the installation of the soldier piles and the construction of top capping beam as much excavation as possible should be undertaken in the central part of the underpass before placing the deck beams so that subsequent wall movement could be reduced to a minimum. It was determined that if the monitored abutment wall movement was in excess of 5mm the bridge beams would be lifted to allow the rubber bearings at the abutment to release their accumulated shear deflection. De-stressing of the anchor would also be required if the abutment movement caused the stress in the anchor ties to exceed 60% of their ultimate tensile capacity.

The excavation was first carried out to 2m depth to enable the installation of the tie-back anchors. The anchor ties were stressed to the design value of 30% of their ultimate tensile capacity giving a short term allowance of 30% (20% long term) for any excavation induced increase in the tension load. Excavation at the centre part of the underpass was then carried out to leave a 3m wide ledge at the anchor level with a 45° cut slope to protect the retained soil from squeezing through the gaps between the piles. After placing the deck beams and casting the deck slab the bridge was opened to traffic and excavation between the abutments continued. The soil was cut to a semi-circle between each pair of the soldier piles and sprayed with fibre reinforced shotcrete.

Regular deformation measurements were carried out as the excavation progressed. It was found that the excavation before placing the bridge beams induced less than 3mm of wall movement. After the completion of the bridge deck, the removal of the remainder of the excavation induced only 2mm to 3mm of movement and it was not necessary to re-seat the rubber bearings. A total abutment movement of less than 6mm was measured which was below the predicted lower bound movement of 10mm. The under estimation was probably due to (i) the Young's modulus profiles being slightly underestimated and (ii) the 3-dimensional layout the abutment and the wing walls was stiffer than 1 and 2 dimensional models.

10 CONCLUSIONS

The geotechnical design and construction aspects of the Southern Motorway Underpass have been discussed in this paper. One-dimensional finite element and two dimensional finite difference modelling were employed to evaluate the interactive behaviour between the soldier pile bridge abutment wall, the deadman anchors and the overall ground movement caused by the 6.5m excavation in soft ground. The results of the analyses indicated that the excavation induced abutment wall and deadman movement could not be avoided. It was therefore decided that to carry out as much excavation as possible before the placing of the bridge deck beams so that subsequent wall movement due to the removal of the soil temporary supporting the soldier pile abutment wall could be reduced to a minimum. Construction monitoring showed that 50% of the abutment wall movement occurred before the placing the bridge beams. The wall movement induced by the removal of the remaining soil was within acceptable limits for the rubber bridge bearings and it was not necessary to reseat them or de-stress the anchor ties. Tying back the top of the abutment walls to the deadmen was still effective even though the deadmen were much closer to the abutment than would normally be the case. Numerical modelling was found to be an important tool in developing a suitable design for the tied back soldier pile abutments.

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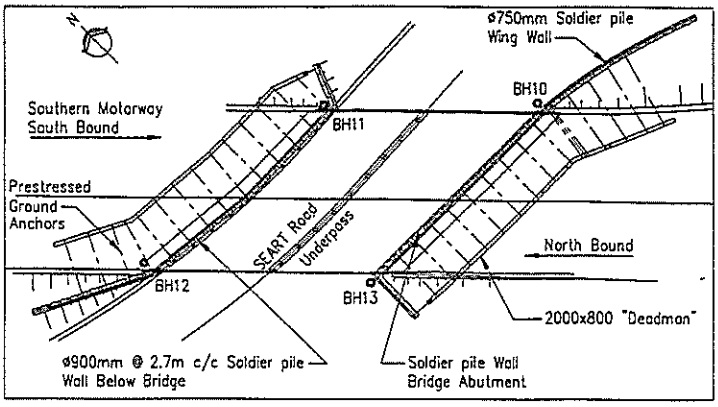

Figure 1 Underpass Layout and Borehole Locations

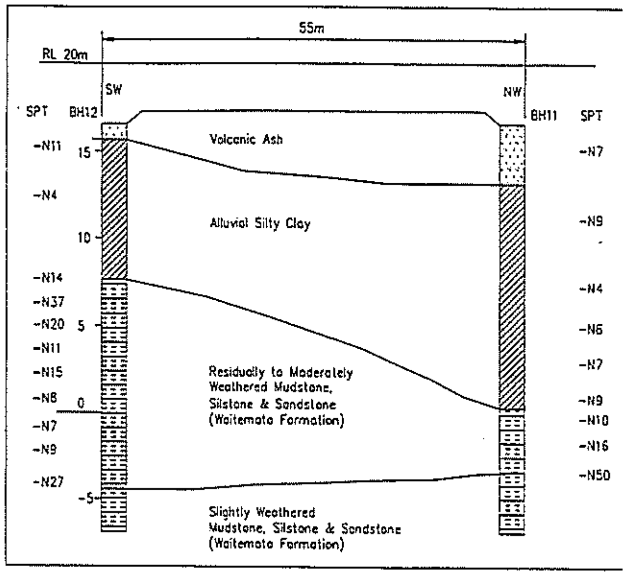

Figure 3 Typical Ground Profile

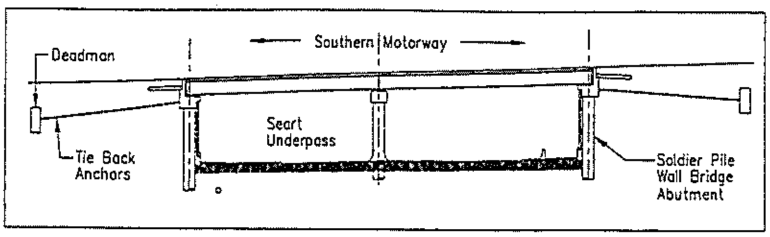

Figure 2 Cross Section of the Underpass

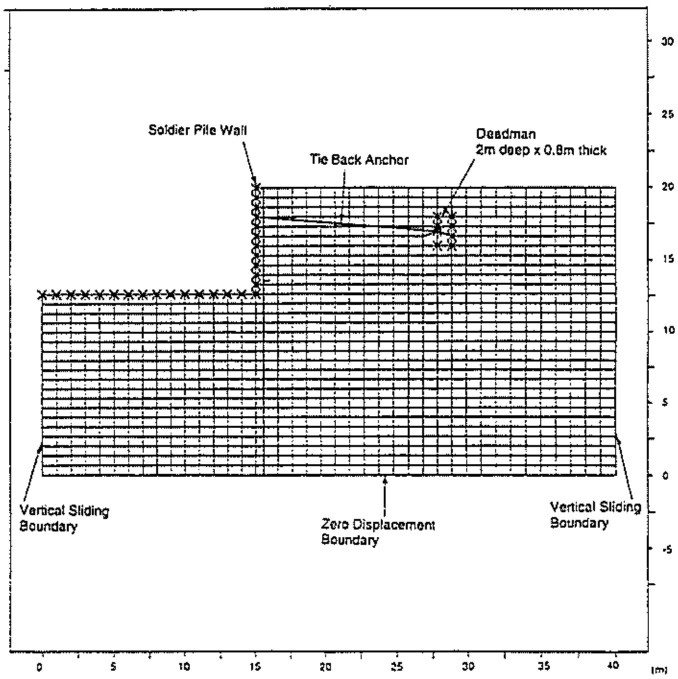

Figure 4 Two Dimensional Finite Difference Model

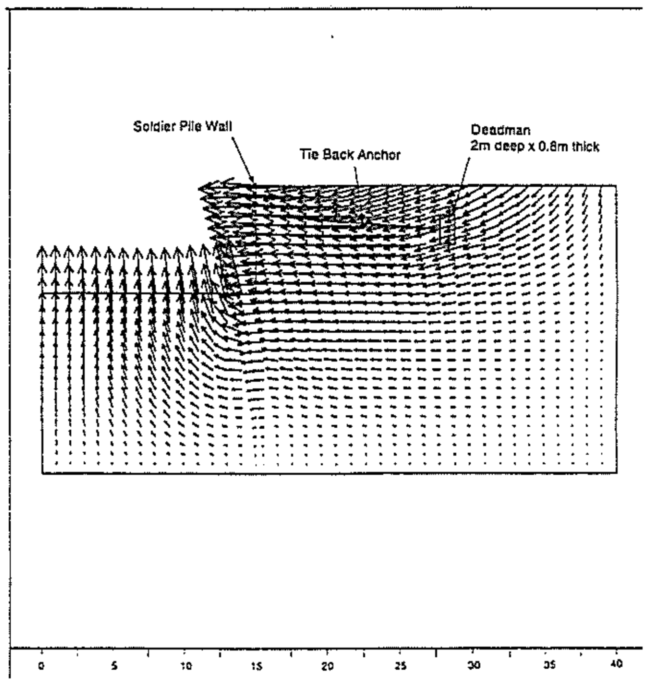

Figure 5 Ground Movement Pattern After Excavat

GEOGRID REINFORCEMENT OF ROADS IN NEW ZEALAND CONDITIONS

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SUMMARY

A description of the history and concepts behind the design of geogrid reinforced roads is given together with comments on construction of roads over soft subgrades. The situations where it may be advantageous to use geogrids when compared to other options of improving subgrade conditions are discussed.

We have selected seven NZ projects to describe briefly which are typical of many geogrid roading designs in NZ. The first describes a recent pavement construction for an industrial subdivision. The next three describe projects on the State Highway system where soft and difficult subgrade conditions were encountered. The fourth project involves a heavily loaded container yard, and the fifth project is an unsealed heavily loaded road in a forestry application. The final project describes reinforcement of asphalt overlay on motorway ramps.

1. INTRODUCTION

Extruded polymeric nets and meshes were originally produced in the 1950's but incorporated potentially weak overlapping joints, and were low strength by comparison with today's geogrids. In the 1970's Netlon Ltd carried out development on a new generation of polymer grids which are formed by stretching a punched polymer sheet. The resulting 'Tensar' geogrids have strong monolithic joints and higher strength and stiffness.

This type of geogrid has been used to reinforce road pavements worldwide since 1980. They were initially used in NZ in the early 1990s in remedial works on County roads over peat which were badly distressed, and in city streets where poor subgrade conditions were encountered. By 1995 as acceptance of their performance grew, geogrids were being used on the State Highway system. The main usage to date has been on highways crossing soft peat subgrades, and particularly where services have been installed at shallow depths beneath existing pavements, and in industries where heavily loaded vehicles must traverse soft ground.

2.0 GEOGRID REINFORCEMENT

Geogrid reinforcement performs two functions within the pavement. It provides both a stiff tensile layer, and a lateral constraint. When granular material is compacted over the grid the stones project through the apertures and create an interlocking action between the aggregate and the grid. This interlock enables the grid to resist horizontal shear from the aggregate and mobilise the maximum bearing capacity of a soft subgrade.

Overseas studies have shown that in order to achieve the aggregate / grid interlock, it is essential to have strong monolithic joints which achieve a high in plane torsional resistance. The ribs should be thick (in the order of 0.8mm), and rectangular in shape. The shape and thickness of the ribs appears to have a significant effect on the ability of the geogrid to achieve good grid / aggregate interlock and hence on the ability of the geogrid to reinforce the pavement. Comparative testing of different types of geogrids available have shown variable effectiveness. Ref 2

During the 1980s extensive research was carried out including work done by Dr Ralph Haas at the University of Waterloo, Ref 1. More recently in 1992 the US Corps of Engineers carried out full-scale pavement trials using a variety of geogrids available in the US, Ref 2. The trials assessed the structural contribution made by different grid types placed within a granular sub-base / aggregate basecourse layer. The results of these and other full scale trials have led to the typical practice of reducing the granular layers in a reinforced pavement by one third to give an equivalent design life to the unreinforced thicker pavement.

Geogrid reinforcement is generally laid at the subgrade level. If the reinforced pavement depth exceeds 400mm it is necessary to install an additional layer of geogrid to maintain the reinforcing effect. This layer is generally installed at mid level of the pavement.

In addition to being used to reduce the thickness of conventional pavements, geogrids have been used to facilitate the construction of roads over very soft subgrades where it is difficult to traffic the earthmoving equipment. The geogrid together with the roading aggregate forms a raft enabling compaction to be carried out on subsequent layers.

Another area that geogrids are being used on the NZ roads is in reinforcing asphalt overlays. Both geogrids, and geogrids bonded to geotextiles are laid immediately beneath the overlay to resist reflective cracking.

3.0 OPTIONS TO IMPROVE SUBGRADE CONDITIONS

If a marginal subgrade for a pavement exists, the options are generally to

- increase the pavement thickness
- stabilise with lime/cement
- provide additional drainage
- reinforce with a geosynthetic grid and/or filter cloth

The main factors influencing the decision are:

- cost
- past performance for the soils in the area of a particular method
- the purpose of the pavement and the specifications to be met
- construction difficulties especially trafficability, weather conditions and limited available depth for pavement construction

It would appear that lime and cement stabilisation is still the most commonly used alternative. Where trafficability, unsuitable soils, weather or limited construction depth are significant issues, reinforcement by geogrid becomes an attractive option.

4.0 NZ PROJECTS

4.1 Construction Methods

Most of the geogrid reinforced roads in NZ have been constructed over subgrades with a CBR of less than 5%. Historically such subgrades have been undercut or stabilised. There are a number of techniques that have been found to be important when constructing directly over these subgrades. It is preferable to install roading drainage as early in the construction as possible to obtain the optimum construction conditions. Although geogrids can be installed directly onto the subgrade, it can be difficult to achieve good geogrid / basecourse interlock when constructing over particularly wet soft subgrades. It is good practice in these circumstances to install a separation geotextile directly on the subgrade. A thin, typically 50mm to 100mm, layer of granular material should be laid between the geotextile and the geogrid to maximise the friction on both sides of the geogrid.

In order to achieve good interlock the aggregate should suit the geogrid aperture. For a typical grid aperture of 40mm, a well graded aggregate with a significant proportion of particles between 5 and 50mm is appropriate.

Often soft subgrades are also sensitive and can soften further with working. It is usual that static rollers only be used on the initial 300mm depth of aggregate on subgrades with CBR's of less than 3%. Above this level, standard vibrating compaction equipment can normally be used. Associated with this sensitivity is the need to sometimes allow time for the road to "set up". Deflections may be high immediately following construction, but tend to decrease as time is allowed for drainage to take place. Care should be taken to install good drainage systems for these roads.

Although the design for low volume roads can require thin pavements, where CBRs are less than 5% it is considered sound design practice to have a minimum aggregate thickness of 300mm. Benkelman Beam requirements on the thinner pavements may also need to be addressed. Refer section 4.9.

4.2 Woodcocks Road Project, Warkworth (March 1997)

This pavement was for an industrial subdivision in Warkworth. The local Council required Benkelman beam testing on the pavement. The soil conditions at the site consisted of Pleistocene and recent Holocene deposits (clays to sands), with extremely variable strengths. In the worst areas, springs existed with ground water levels at or close to the ground surface. Corresponding shear vane values in the silty clays were as low as 20 kPa, with small branch inclusions common.

The options considered for the pavement included:

- Undercutting of soft material
- Lime stabilisation
- Geogrid reinforcement

As trafficability in the worst area was a significant issue, the geogrid option was preferred over conventional stabilisation. The effectiveness of conventional stabilisation with organic inclusions was also doubtful. The geogrid option also had cost savings over the option of additional undercutting.

A summary of the design details of the pavement shown in the table 4.9, and a typical detail on figure1. The design utilised a filter cloth in the softest areas to minimise the risk of "pumping".

The measured Benkelman beam deflections ranged from typically 1.1 to 1.8 mm. There appeared to be no significant correlation with subgrade strength, implying that the increased pavement thickness/geogrid combination accounted for this factor.

4.3 State Highway 1 King St., Timaru (March 1995.)
This section of the State Highway had suffered extensive pavement failure with increasing traffic, and the shape of the road needed upgrading. The subgrade was a sandy silt material with an inferred CBR of 3%. The design loading for the reconstruction was 2.8 x 10^6 EDA per lane. This would have necessitated a pavement depth of 620mm. Sewer, stormwater, gas and water services lie close to the surface beneath the existing pavements. Tensar SS30 geogrid was used to reinforce the pavement at subgrade level reducing the required depth of metal to 420mm.

Geogrid reinforcement was used in preference to other options to reduce the uncertainty of damaging and replacing services within the additional depth that would be required with an unreinforced pavement option. Benkelman beam deflections of 0.5 to 1mm were recorded on the completed work at the southern end,

4.4 State Highway 1 Kawiu Rd, North of Levin (May 1995)
The widening of SH1 at Kawiu Rd intersection just north of Levin necessitated construction over a soft swamp. Site investigations showed the proposed subgrade to comprise black topsoil with large rotting stumps and logs, underlain by extensive peat deposits. The groundwater table was up to the surface. Strip drainage was laid along the edge of the road widening to a depth of 1m below the subgrade. The road was designed for a CBR of 1% and a design loading of 5.0 x 10^6 EDA. The total pavement thickness was 550mm reinforced with two layers of geogrid see Fig 2. Reduction of the pavement depth by using geogrid reinforcement provided a more economical solution than other options of subgrade improvement.

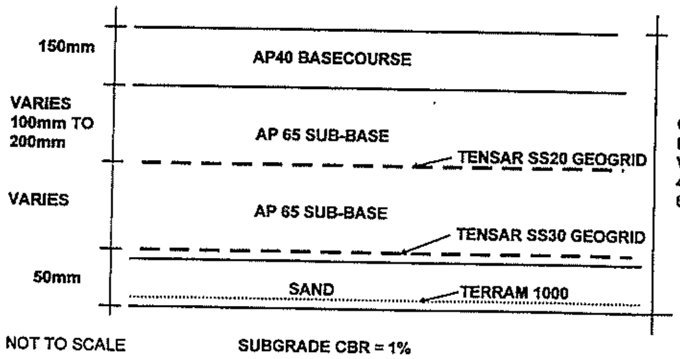

Fig 1 ; WOODCOCKS RD WARKWORTH

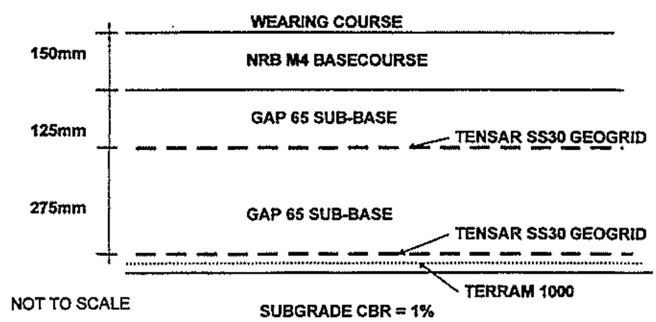

Fig 2 ; STATE HIGHWAY 1 KAWIU RD

4.5 State Highway 1, Milton (January 1996)

State Highway 1 passes directly through the business district of Milton. The reconstruction works was made more difficult by the presence of services including watermains at 600mm below the surface. The subgrade comprised variable clays and included a black organic river silt. The difficult and variable subsoil conditions were illustrated on an occasion where the road pavement dropped over 100mm overnight in a soft area. This was topped up the following day and no further movement was observed.

CBR's were measured to be a low as 1%. The design loading for a 25 year life is 2.5 x 10^6 EDA per lane. The design for an unreinforced road would have necessitated a pavement depth of 810mm which would have resulted in disruptions and costs associated with shifting services. However, when reinforced with two layers of geogrids, the pavement depth was reduced to a total of 540mm. See Fig 3. At this depth the excavation just skirted the tops of the watermains necessitating only some removal of valves. Another problem that would have been encountered with the conventional deep pavement would have been disposal of ground water, as this would have been below existing stormwater outlets.

Benkelman beam deflections on this road in the winter after construction were measured to be a maximum of 1.2mm.

4.6 Goods Yard, Rotorua (March 1994)
A marshaling yard located in an area of variable volcanic soils was redeveloped to accommodate Tranzrail's Container Freight operations in Rotorua. This area is subjected to the heavy loads of a forkhoist lifting containers off the train tracks and transferring them on to the road transport. The subgrade CBR was assessed to be 4%. Geogrid reinforcement enabled the depth of this heavily loaded pavement to be reduced to a total of 530mm See Fig 4.

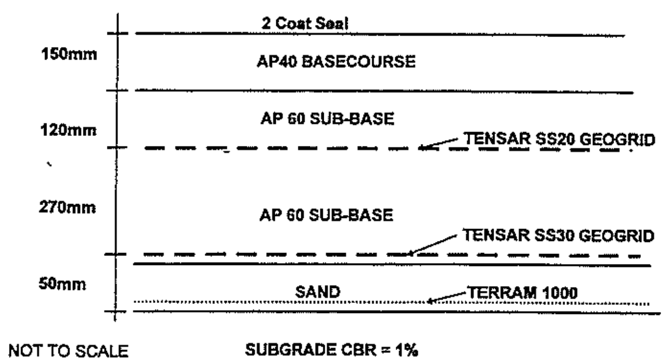

Fig 3 ; STATE HIGHWAY 1 MILTON

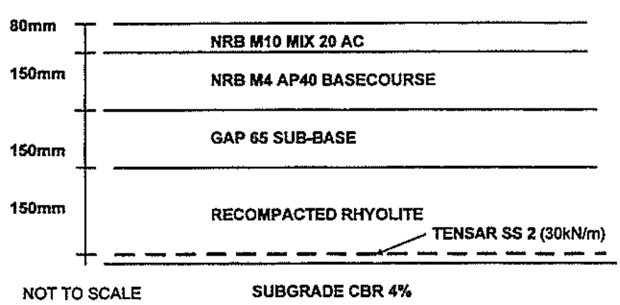

Fig 4 ; GOODS YARD ROTORUA

4.7 Whangapoua Forest Roading (April 1997)
Forestry Harvesting Managers have one of the most difficult tasks that involves creating serviceable roads that can sustain very high loadings. Sometimes the roads are in service for a short time only whilst a block of forest is being harvested, and so the Manager also needs to keep the design as economical as possible.

Ernslaw One Ltd's forest at Whangapoua in the Coromandel peninsula is located in deeply weathered, strongly rolling to moderately steep Andesitic hills of very low fertility and poor soil structure. Access is currently required to approximately 50,000 tonnes of timber which is to be harvested over a two to three year period. During initial investigation it became apparent that a section of the road had a very wet, weak subgrade. Generally the CBR of the subgrade was no more than 1 or 2% within the first 300mm, increasing to about 4% at 600mm and 4 to 6.5% at 700mm depth.

Two sections of the road have an adverse gradient of 1 : 6 which is tested to the limit when traversed by fully loaded logging trucks. These sections were constructed by laying Geotex 2002 geotextile covered with 50mm to 100mm of GAP100. A layer of Tensar SS30 geogrid was laid and covered with a further 200mm to 300mm of GAP 100 metal. The reinforcing action of the geogrid enables the minimum amount of metal to be used.

During the summer of 1997, there was an intense period of use for two months when 8 to 9 fully laden (gross weights of 44 to 46 tonnes) logging trucks per day used the road, and for the remainder of the summer 4 to 6 trucks per day. The Skyline hauler, with a gross weight of over 60 tonnes, for the logging operation also has to negotiate these roads. The roads constructed with the geogrid reinforcing have performed extremely well to date with virtually no movement or wheel ruts.

4.8 State Highway 1 Auckland Southern Motorway Off Ramp (October 1996)
The original pavement at the Papakura off ramp comprised a cement stabilised sub-base approximately 350mm thick overlain by approximately 50mm asphaltic concrete. Transverse and longitudinal block cracking had appeared at 3 to 4m intervals. Previous applications of thin overlays showed reflective cracking within 12 months. Tensar ARG geogrid/geotextile was used to provide both a waterproofing layer of geotextile, and a structural ability to span cracks beneath the new 50mm thick overlay. Following this project several other on or off ramps have been similarly treated.

4.9 Benkelman Beam Deflections
Benkelman beam test results are still widely used by local authorities as an acceptance criteria for pavements. This can cause difficulties for designers because typical design approaches use alternative criteria, which relate required pavement thickness to subgrade CBR and traffic loading. We have shown on the attached table recorded beam deflections as a function of subgrade CBR and pavement thicknesses for various sites. As would be expected there is very wide scatter and variability in the results. The most significant of these are:

- It is likely the soil type/geology has an influence on the relationship.

- There can be significant differences in how CBR is assessed i.e. correlations between shear vane/penetrometer, and CBR may give different results. Also the degree of conservatism in assessing CBR may be significant i.e. a single CBR figure is a crude approximation of subgrade strength.

- Beam deflections are usually time and moisture dependent.

- The strength properties of the base courses also influences the results.

Table 4.9
Results of Benkelman Beam Tests on Geogrid Reinforced Pavements

Location	Pavement Depth (mm)	Subgrade CBR (%)	Beam Deflection (mm)
Great North Road, Auckland	650	0.8 to 1	2.0
Woodcocks Road, Warkworth	700 425	1 4	1.2 to 1.6 1 to 1.5
SH1, Milton	540	1	1.2
SH73, Rangiora	430	2	2.0
SH1, Timaru	420	3	0.5 to 1
Great South Road, Auckland	625	3	<1.0
Pulham Road, Warkworth	550	2	0.6 to 1.2

5.0 CONCLUSIONS

Geogrid reinforcement of pavements is becoming more widely used in NZ particularly in areas with soft subgrades where:

- Trafficability of construction plant is difficult and/or weather conditions are adverse
- Economies can be gained by allowance of a reduction in design depth of pavement aggregate due to the reinforcing action of the geogrids.
- The presence of services or other constraints lead to a thinner pavement being advantageous.

There is a wide variation in Benkelman beam deflections due to the many different factors which may influence this measurement. Perhaps other test methods should also be considered to predict the likely future pavement performance of geogrid reinforced roads.

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2; Webster SL *Geogrid Reinforced Base Courses for Flexible Pavements for Light Aircraft: Test Section Construction, Behaviour Under Traffic, Laboratory Tests and Design Criteria.* Geotechnical Laboratory, Department of the Army, Waterways Experiment Station, Corps of Engineers, Mississippi 1992

Road Construction in an Active Geothermal Field: Rotorua, New Zealand

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SUMMARY

The Whakarewarewa thermal area in Rotorua, New Zealand, is an active geothermal area and major tourist attraction. A recent upgrade of the facility included a 1.8 km paved and suspended timber deck track involving three shallow cut and cover tunnels, two new bridges and an upgrade of an existing glulam bridge, as well as several new buildings and an expanded carpark area west of the existing complex.

The majority of ground conditions along the track alignment presented quite difficult challenges for pavement design including low bearing pressures and very low subgrade CBR values, sensitive collapsing soils in some of the track cuts and ubiquitous geothermal alteration and activity making construction very difficult in some areas. Development proceeded using an "observational methodology" approach which allowed for design variations during construction.

1. INTRODUCTION

The Whakarewarewa thermal area, located at the southern end of Rotorua, is an internationally recognised geothermal area and major tourist attraction. A 1.8 km paved and suspended timber deck track was built to carry tourists in electric powered vehicles around the thermal reserve both to open up new areas of the field for tourist access as well as to better manage the people flow through the facility. The track development involved three shallow 'cut and cover' tunnels at inverts of up to eight metres depth, two new bridges spanning up to 78 metres and an upgrade of the existing Pohutu Geyser bridge as well as specialist construction measures to deal with geothermal ground conditions.

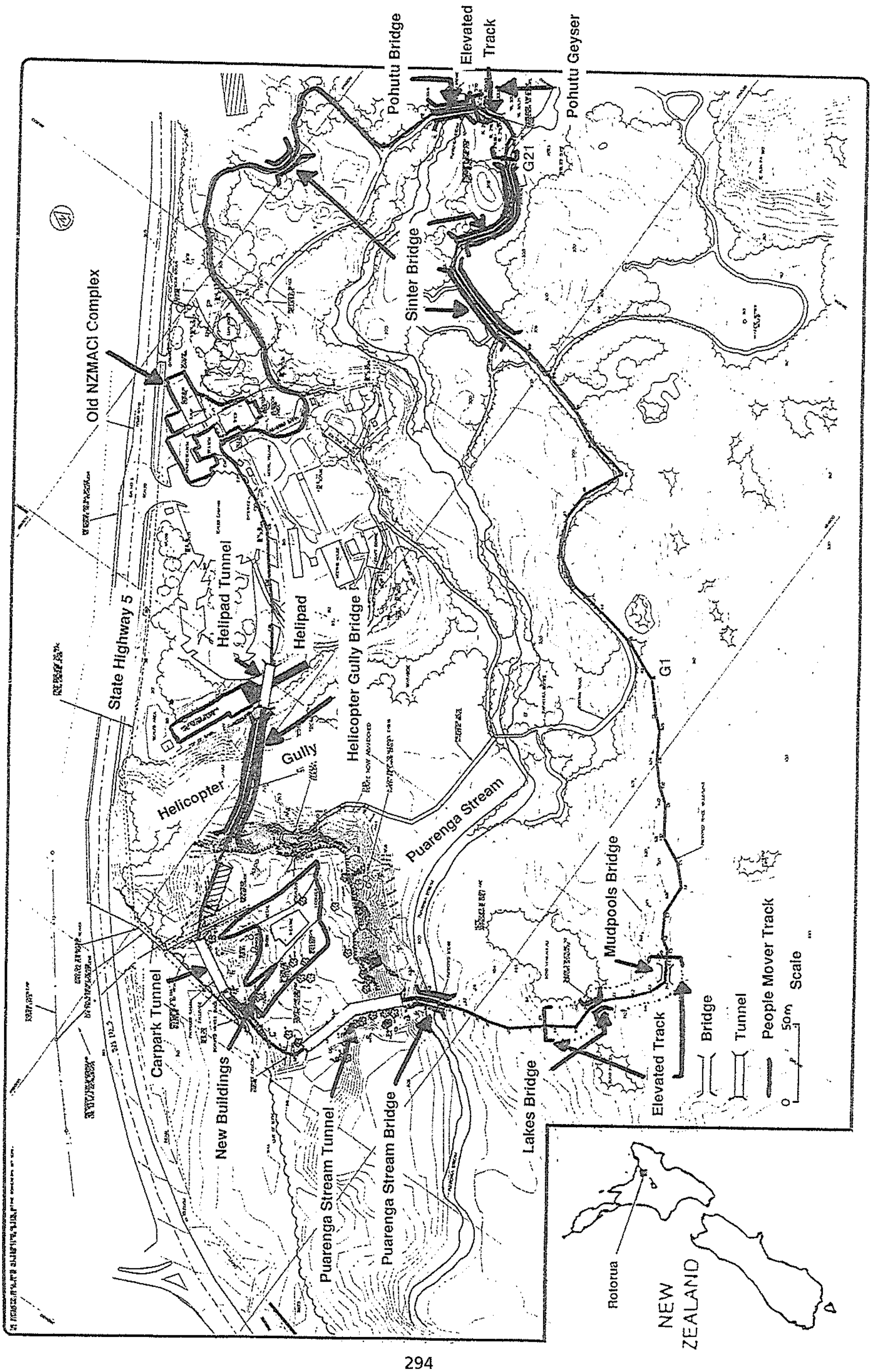
Detailed geotechnical and geological investigations were undertaken over a period of four years prior to and during track design to establish suitable design parameters, refine construction methodologies and obtain the best track location relative to exposed ground conditions.

An "observational method" approach was considered essential during the geotechnical reporting and design phase of the development to account for variations in ground conditions and unforeseen construction difficulties.

2. GEOLOGY

The Whakarewarewa thermal area (Figure 1) is located inside the southern wall of a large circular rhyolitic caldera, about 15 km in diameter, with Lake Rotorua offset towards the northern end. The caldera formed following collapse during and after the eruption of the Mamaku Ignimbrite some 220 ka ago and has subsequently been heavily modified by eruptions from adjacent volcanic centres, varying lake levels and deposits of primary and reworked volcanic sediment. A wide range of volcanic soil and rock types are present along the track alignment including at least two surficial volcanic airfall ashes comprising loose, uncompacted silty sands/sandy silts underlain by a 2 to 3 metre thick coarse gravel pumice ash.

Underlying the above is a sequence of stiff, moderately dense, alluvial (lake terrace) fine grained silica silts and pumice lapilli, forming much of the higher ground over the site. This unit is highly impermeable, with some constituent beds exhibiting thixotropic properties when disturbed. Overall it acts as a cap to the underlying geothermal activity contained within the basal Huka Group sediments, which essentially consist of hydrothermally altered and silicified alluvial sands, silts and coarse pumice/rhyolite gravels. As an example of the diversity of soil types present shear strengths range from 20 to 30 kPa within some of the surficial ashes to unconfined compressive strengths of 2.2 to 30 MPa within the basal Huka group materials.


Pohutu Bridge
Elevated Track
Pohutu Geyser
G21
Sinter Bridge
Old NZMACI Complex
State Highway 5
Helipad Tunnel
Helipad
Helicopter Gully Bridge
Helicopter Gully
Puarenga Stream
G1
Mudpools Bridge
Elevated Track
Lakes Bridge
Puarenga Stream Bridge
Puarenga Stream Tunnel
New Buildings
Carpark Tunnel
Bridge
Tunnel
People Mover Track
Scale
0
50m
Rotorua
NEW ZEALAND

The area has a high vertical conductive heatflow with thermal activity concentrated along the many faults which parallel the Puarenga Stream and criss-cross the area (1). Geothermal activity includes alkali-chloride springs, including geysers (typically high fluid discharges at or near boiling point), and semi-stagnant acid sulphate mudpools, with little or no discharge (2). Alkali-chloride springs precipitate silica to form sinter deposits while the mud pools are indicative of ongoing ground decay due to steam heating and sulpuric acid attack. Field evidence of the latter include ongoing decomposition of rock and soil minerals, multi-coloured clays stained with iron minerals, gradual void or cavity formation, subsidence and boiling mudpools.

From an engineering viewpoint, widespread and ongoing hydrothermal alteration has created areas of fragile ground conditions with low bearing capacity, potential for extensive total and differential settlement and potential for rapid ground collapse.

3. FIELD & LABORATORY INVESTIGATION

Fieldwork for the Whakarewarewa project began in late 1993 as part of a geotechnical feasibility study for the overall development. More intensive investigations were carried out in November 1996 with additional investigations in early and mid 1997. Investigations were structured to obtain information for bridge foundations, tunnel cuts, suspended track and on-grade pavement areas. Investigation methods included machine and hand auger boreholes, standard penetrometer testing (SPT) and cone penetrometer testing (CPT), plate load and insitu CBR tests, scala penetrometer tests and over 300 metres of ground penetrating radar to look for cavities and buried acid mud pools through a known area of thin sinter crust. Due to access difficulties much of the investigation was either carried out by hand or supported by helicopter with all of the plate load test sites and at least two machine boreholes being flown.

CPT results were variable, partly due to ground conditions but also due to temperature affecting the cone electronics. Scala penetrometer and SPT results were also highly variable, and in places were at odds with actual exposed ground conditions during construction.

Laboratory testing included Atterberg limits, standard compaction and solid density, effective stress and unconfined compression testing as well as particle size analyses, soil pH and nuclear density testing on compacted fill and hard track basecourse materials.

Plate load tests at 20mm deflection for geothermally altered soils ranged from 55 to 159 kPa, while loads at 15mm deflection for airfall ashes ranged from 65 to 110 kPa. Six insitu California Bearing Ratio (CBR) tests carried out adjacent to the plate load tests returned CBR values ranging from 1.0 to 1.5. Equivalent CBR values from scala penetrometer testing ranged from considerably less than 1.0 (typically 1 to 3 blows for 1500 to 2000mm penetration) to greater than 10 on areas of silicified sediments and/or sinter.

The ground radar survey indicated there were at least seven areas along the existing track from G1 and G21 of thin sinter crust overlying boiling mud or alkaline springs which would not be able to support the people mover design loads without significant additional support. These were confirmed by additional hand auger boreholes and scala penetrometers and led to the use of fibreglass reinforced concrete in these areas.

Table 1 summarises in some detail the soil types and their engineering properties encountered across the site.

4. PAVEMENT DESIGN

The people mover fleet consists of five battery powered electric vehicles, each seating up to 36 people. The articulated trams are 14 metres in length and are expected to operate continuously for a ten hour day throughout the year, charging at the main station during passenger pick up/drop off periods, and overnight. It is estimated the people mover will transport around one million visitors annually, the technology being a world first in terms of its innovative engineering design.

Typical axle loads are in the order of 2.0 tonnes (20 kN), approximately 25% of a standard design axle load. Considerable effort was expended in pavement design taking into account lower vehicle induced pavement stresses due to lighter axle loadings and low to very low equivalent design CBR's. Several design options were considered to prevent subgrade failures and limit pavement distress including increased basecourse hardfill thickness, use and selection of appropriate geotextile reinforcing, subgrade cement stabilisation and the use of specialised structures to bridge over the worst geothermally altered and active areas.

Table 1: Summary of ground conditions encountered at Whakarewarewa

Unit No.	Geological Description	"Engineering" Description	Distribution	Engineering Properties	Comment
Type 1	Whakatane and Mamuku ashes	Airfall volcanic ashes. Soft, loose, friable, non plastic, orange/brown/yellow fine sandy silts and silty fine sands.	Surface to between 1.6 & 2.2m depth. Average thickness approx 1.8m. Mantles remnant terrace areas across Whakarewarewa thermal area.	Loose, compacts under vehicle tyres. Shear vanes (SV's) range from 10-60 kPa, average 30 kPa. Sensitivities to disturbance 1.8-7.0, typical values 3.5-4.5. Equivalent CBRs almost all <1 for top 0.8-1.2 m. SPT results all N=0.	Seriously unfavourable load bearing characteristics. Hand auger boreholes 3 & 5 have 4+ m of this unit at northern end of building platform - material was hot and moderately altered. Airfall deposit mantles pre-existing topography, expect variable thicknesses across site.
Type 2	Rotoma and Rotorua ashes. Note Rotoma ash locally absent in some areas	Airfall volcanic ashes. Loose, non-plastic, orange/cream/brown, fine to coarse pumice sands & fine-medium pumice gravels with inclusions of fine gravel size rhyolitic lithic fragments. All particles angular to subangular.	Typically between 1.8 & 3.6m depth but ranging from 1.6 & 2.2m to 3.1 & 4.1m depth. Average thickness 1.8m.	Loose pumice sands and gravels. Highly permeable. Equivalent CBRs range <1 to 5, typical values CBR=3. Note values quite variable across building platform. SPT results all N=0 or at best N=1.	Highly permeable unit inferred to rapidly drain rainfall infiltration and hence enhance bank stability. Loose gravelly nature likely to substantially compact where exposed. Airfall deposit - mantles pre-existing topography, expect variable thicknesses across site.
Type 3	Oruanui breccia. Note erosional break between overlying ashes (Type 1 and 2) and this unit/soil type	Lacustrine terrace deposit, range of soil types: Soft to firm, non to moderately plastic, orange/red/ pink/ brown/cream silts and clayey silts. Beds and bands of 5-15mmØ lapilli pumice gravels. Bands of abundant amorphous silica, sensitive, thixotrophic - collapses on disturbance. Areas of hydrothermally altered montmorillite clays, swelling potential when unloaded. Increasing strength with depth, becoming stiff to very stiff and weakly cemented below 10-12 m depth	Top of unit typically 3.6m depth but ranging from 3.1-4.1 m. Base of unit variable range approx 11.1-13.6 m in machine boreholes. Mean thickness 8.5 m ranging from 7.2-10.4 m.	Firm to stiff, becoming weakly silica cemented at depth. SV's range from 10 to >140 kPa, mean approx 50 kPa. SV values higher than in two overlying units. Sensitivities to disturbance range 1.8 to >20 (extremely sensitive - quick). Equivalent CBRs range 3-10+, typical values CBR = 5. SPT results variable, range N=0-N=21, average N=6	Lacustrine terrace deposit , deposited approx 22,000 years BP when lake level was approx 40 metres higher. Some sedimentary structures seen in stream bank exposures. Sorting and grading of particles by size and density seen in borehole core. Highly impermeable unit effectively caps and seals off underlying geothermal aquifer and consequent activity.
Type 4	Huka Group sediments	Pre 65,000 year BP lacustrine sediments. Very weak to weak, weakly to moderately silica cemented fine to coarse grained silica sandstones, cemented gravels and weak, weakly cemented siltstones. Variable cementation.	Exposed along Puarenga Stream banks at bridge and in base of some machine boreholes. Inferred top of unit 11.1-13.6m depth but difficult to define precisely due to altered and weakly cemented nature of overlying Oruanui breccia unit	Very weak to weak, (non) to weakly to moderately cemented, soft rock. Bands of uncemented sands, interbeds of silicified sediments and sinter. SPTs range from N=3 to N>50. UCS range 2.2-29.9 MPa.	Good founding conditions for Puarenga Stream bridge but variable depth to top of competent founding surface. Undercutting and backfilling with site concrete or aggregate likely to be required.
Type 5	Acid geothermal muds and silts, hot barren ground, geothermally altered terrace deposits and airfall ashes	Soft to very soft, non plastic, cream/grey silts, often moist and hot. In places muds, actively bubbling. Thin, 0.2-1.0m thick crust of cool to warm, non plastic (white) silts, typically firm, in places friable, loose, prone to collapse where undermined by active acidic activity. Often mantled with 0.4-0.8m of moderately altered orange/brown/yellow ashes (type 1 soils)	Areas of site typically below RL 310m.	Very difficult founding conditions, poor ground bearing and high settlement characteristics coupled with elevated temperatures and on-going ground decay and settlement. SV's range 10-30 kPa, mean approx 15-20 kPa. Equivalent CBRs <1 (in places considerably <1!). Actual CBRs (done in conjunction with plate loads) 1-1.5. At best CBR = 1 available for design. Some ground modification will be required in some areas.	Ground conditions at absolute limit of conventional geomechanics applications.
Type 6	Boiling alkaline springs, active geyser areas, alkaline sinters	Loose, non-plastic, cream/ grey coarse silty sands and sandy fine gravels. Weak, weakly cemented cream/grey silica sinters interbedded with type 1, 2 and 3 soils above. Actively depositing amorphous silica sinter from boiling geothermal fluid.	Predominantly existing developed area of Whakarewarewa, southern abutment of Pohutu Bridge. Some older sinters adjacent to Puarenga Stream bridge area.	Where present without overlying ashes, silica sinter has relatively better engineering characteristics than much of higher ground. Shear vanes typically 5-80 kPa but obvious problems due to nature of substrata. Sensitivities to disturbance range from 2-8, typically 4. Equivalent CBRs range 1-5, typically CBR=2.	Ground conditions at absolute limit of conventional geomechanics applications. Potential for collapse of sinters, especially around existing developed area

For practical purposes most structures were designed to 15 kPa safe bearing with seismic bearing capacity up to 30 kPa. Areas of loose collapsible acid leached ground overlain by silica sinter adjacent to the Pohutu Geyser had a design safe bearing of 12 kPa which included an overlay of shotcrete prior to placement of pumped concrete bag foundations. The design approach taken was that if we were able to safely walk on areas of the site, then we could build some form of trackway, provided we were able to keep bearing stresses similar and cut and fill level changes to a minimum.

Other design considerations included aggregate chemistry. Compacted pumice was used in some track areas to minimise geothermal decay from acidic gases and groundwater. The effect of soil chemistry and temperature on geotextiles was also considered. In practice these were only used on relatively low temperature parts of the site and were designed to act as a separation layer rather than to provide significant subgrade reinforcement.

The majority of constructed track was 60mm concrete block pavers overlying subgrade CBR dependent depths (typically 150 to 400mm) of compacted GAP 65 rhyolite. Maximum track gradients were set at 12% downslope and 10% upslope.

Paved track design was based on plate load, in-situ CBR and scala penetrometer test results carried out over the whole track alignment. Basecourse depths were also based on design charts and output from software supplied by the manufacturers of the block pavers as well as methods given by Giroud and Noiray (3), and guidance from AUSTROADS (4) and Transit New Zealand (5), although the latter two references essentially deal with typical road design rather than the trackway design loadings. Where excess pavement deflections occurred from construction traffic, additional undercutting and compacted rhyolite backfilling was undertaken. Cement stabilisation was not considered a viable option due to the expense and accessibility for stabilisation equipment.

Around 400 metres of suspended timber track was built on the southern part of the track alignment and adjacent to the Pohutu geyser either on geothermally affected and altered ground or on low scale active features.

The timber structure was bolted onto 600 mm wide by 300 mm deep fabriform bags filled with pumped concrete. Initially lightweight foamed concrete was specified but major construction difficulties were encountered in being able to pump this material - the foamed concrete compressed within the lines and airlocked itself, then tended to explode out of the lines when individual pipes were disconnected in an attempt to clear the airlocks. Eventually a standard pump mix was used with 10% micropoz silica added to increase concrete durability. The majority of bags were laid on the ground surface although the underlying ground was benched and shotcrete used to 'lock' the fabriform bags in where uneven ground was encountered. Elevated track structures were designed for safe bearing loads of 12 to 15 kPa.

Two small glulam timber bridges were also built as part of the suspended track. The ten metre long Lakes Bridge spans a narrow isthmus between two lakes at around four metres height, while the Mudpools Bridge (12 metres long) spans a narrow ridge with active mudpools on either side. Piles were installed on the Lakes Bridge to account for seismic bearing capacity and possible liquefiable ground conditions. The Mudpools Bridge foundations were designed to be fully compensating due to a 1.5 metre cut to achieve track gradients in this area. Both bridge foundations have grouting ports installed to allow for compensation grout relevelling should this be required.

Areas of inferred shallow sinter crust revealed by the ground radar survey were spanned using four metre wide, 100 mm thick 35 MPa concrete reinforced with either fibreglass rods or galvanised steel reinforcement. These bridges were up to 38 metres in length and designed to safely support the people mover loads over a ten metre long collapse zone.

5. BRIDGES

The 3.4 metre wide, 78 metre long Helicopter Gully Bridge spans from the helipad tunnel portal to the building terrace area (Figure 1). Apart from the abutment positions, there are two piers within the gully area, located 28 metres apart. Ground conditions at these piers consisted of a veneer of airfall ashes underlain by 5 to 6 metres of hydrothermally altered silts, muds and clays, underlain in turn by silicified Huka Group sediments.

Due to high lateral loads at ground level under static and seismic loading conditions (up to 400 kN thrust and only 20 kPa safe bearing available on unimproved ground at the surface) four, 600 mm diameter piles at each bridge pier were auger drilled through the surficial soils, then percussion hammer drilled to create rock sockets at least 1.5 metres deep into the silicified Huka Group materials. Bridge abutments were also piled to approximately seven metres depth although loadings did not require rock sockets. The Helicopter Tunnel was also designed to impart significant seismic lateral resistance to the bridge structure, being tied into the bridge abutments.

Due to extreme ground conditions encountered in the central pier pile holes (venting steam, groundwater >90°C, pH 2-3, and 400-600 ppm sulphate), and mudpools adjacent to the eastern pier, duracem cement, a blend of cement and ground granulated blastfurnace slag, was specified for all piles and bridge footing concrete.

The Puarenga Stream Bridge spans some 33 metres from the southern portal of the Puarenga Stream Tunnel across the Puarenga Stream. Foundations were designed to 15 kPa and piled to depths of between 1.7 and 5.0 metres to found on competent materials and provide additional seismic lateral capacity.

The existing glulam beam timber bridge adjacent to the Pohutu Geyser was also widened by 2.2 metres to allow access for both the people mover and pedestrian traffic.

6. TUNNELS

Three moderately shallow cut and cover tunnels (up to eight metre deep inverts) were constructed from 2.5 metre high by 3 metre wide post tensioned box culverts, primarily to ease approach and exit gradients from some of the bridges. Tunnel designs incorporated seismic and soil lateral loads and were also used to provide lateral seismic bracing for adjacent bridge structures. Based on initial fieldwork results, polystyrene backfill was specified at some depths to allow for soil swelling pressures. In practice, cuts were open for up to 3 months and it was considered that any swell present would have occurred over this period. In addition, cut batter stability was carefully considered, but in practice, soils performed better than borehole records suggested.

Tunnel invert subgrade conditions were however quite variable, ranging from equivalent CBR=4 to considerably less than 1. Up to 800 mm of compacted rhyolite GAP 65 hardfill on Polyweave HR geotextile was placed under all tunnel sections with selective undercutting and further backfilling with hardfill where soft ground was encountered during construction

Tunnel cuts were backfilled with hardfill and natural soils following section placement and post tensioning. Backfill materials were graded to limit internal soil erosion, together with draincoils to control groundwater.

7. RETAINING STRUCTURES

Three flexible gabion basket retaining structures up to eight metres high incorporating terramesh panels were constructed to retain filled ground above tunnel portals. PVC coated galvanised mesh gabions were used, backfilled with 150/80 gap graded rhyolite fill. The lowest gabion course was buried and backfilled with compacted pumice to provide a shear key for the wall, especially on the Helicopter Tunnel exit. Gabions were founded on either compacted GAP 65 rhyolite or compacted pumice, depending on ground conditions.

Several areas of the track required cuts of up to 3.5 metres to achieve track gradients and transitions between paved track and structures. Cut slopes were battered back to 60° for slopes up to two metres high, and 50° for slopes greater than two metres in height. Punga logs are currently laid over the batter slopes to prevent erosion.

Battering versus fully retaining slopes with crib walls or similar was seen as a risk-cost trade off as well as limiting environmental impacts. Initial cost savings in leaving steeper batters may be offset by the future risk cost of having to repair any instabilities that may occur. Equally there were also issues of public safety in leaving slopes steeper than ideal, although to some extent these will be offset by regular detailed safety surveys and maintenance/observation procedures put in place to monitor track conditions.

8. CONCLUSIONS

The Whakarewarewa track project has proven how difficult civil construction in active geothermal areas can be as well as the variance that can often occur between "as investigated" and "as encountered during construction" ground conditions. Many of the ground conditions present were initially considered to be well beyond the range of conventional geotechnical engineering design solutions and generally proved to be as such during construction. Despite an extensive geotechnical investigation, there was an expectation that ground conditions would vary from what the project designers had assumed and design values for geotechnical aspects of the buildings, structures, and the roadways were based on anticipated worst case conditions, imposing minimum practical construction loads.

The adopted "observational method" approach allowed designs to be modified during construction by finding the best way out, or implementing design alternatives when unexpected ground conditions were encountered.

One area we had initially overlooked was lateral loads on bridge foundations as initial design discussions had considered simple post and beam type construction. Other problems encountered were loose saturated pumice silts exposed in the Lakes Bridge footings requiring piles to prevent foundation failure during an earthquake (liquefaction). Other problems related to the constructibility of some of the details specified and trying to explain to contractors and clients the relatively unique nature of many of the ground conditions present and the rationale for the design recommendations adopted.

A commissioning process is currently ongoing (June 1998). While excessive structure deformation is not expected, there may be some structure or foundation deformation or pavement distress that will occur as the facility "beds down" and becomes operative. Contingencies for problems include compaction grouting through grout ports installed in bridge footings, regrading of pavements and additional retaining structures.

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