SECOND ANZ YOUNG GEOTECHNICAL PROFESSIONALS CONFERENCE

# PROCEEDINGS OF THE SECOND AUSTRALIA-NEW ZEALAND YOUNG GEOTECHNICAL PROFESSIONALS CONFERENCE

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# THE SECOND AUSTRALIA-NEW ZEALAND YOUNG GEOTECHNICAL PROFESSIONALS CONFERENCE

UNIVERSITY OF AUCKLAND 29 Nov - 2 Dec 1995

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Maurice Fraser
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# WELCOME

Welcome to the *Second Australia-New Zealand Young Geotechnical Professionals Conference* being held at the University of Auckland, Auckland, New Zealand. This Conference builds on the very successful First Conference held in Sydney 1994 and is supported by both the New Zealand and Australian Geomechanics Societies.

Our Regional Conferences follow several similar ones held in Europe over the past decade and are designed to foster the following:

- an increased understanding between young geotechnical engineers and geological professionals,

- develop close links between those in academic research and others working in the consulting industry and

- provide an appreciation of the range of work undertaken by others of a similar age and experience

Additional benefits from these conferences are the opportunity to meet and get to know your peers throughout the region and to develop an active interest in the regions geotechnical affairs.

Considerable thanks are due to Garry Mostyn and his team from Sydney for organising the first conference and providing a framework for subsequent events; to EQC for supporting our Guest Lecturers, assistance for South Island participants and help in publishing conference proceedings; to the New Zealand and Australian Geomechanics Society for taking up the challenge of running these conferences; to the management and staff of Foundation Engineering and Woodward Clyde who have assisted in the organisation and conference sponsorship; and finally to your employers for having the foresight to invest in the future by sponsoring your attendance.

Welcome to Auckland for the Second Young Geotechnical Professionals Conference.

Maurice Fraser
on behalf of the Organisation Committee

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# COMPUTER MODELLING AND ROOF SUPPORT DESIGN FOR LARGE DIAMETER ROAD TUNNELS IN SYDNEY HAWKESBURY SANDSTONE

D.C. BEAUMONT

Douglas Partners Pty Ltd
96 Hermitage Road, West Ryde, NSW, 2114, Australia

SUMMARY

This paper looks at the design of the roof support systems for the M2 tollway twin tunnels located at Epping in Sydney, Australia. The tunnels pass through Hawkesbury Sandstone which is subject to high insitu horizontal stresses. Making use of a finite difference computer modelling program, the tunnel excavations were simulated on computer and the results predicted that differential movements could occur along horizontal bedding planes in the sandstone, immediately above the tunnel crown. These movements, caused by the high horizontal stresses became an important factor in the design of roof rock bolts.

## INTRODUCTION

Construction of the M2 Motorway in Sydney, Australia included a short underground section tunnelled through Hawkesbury Sandstone, by Peabody Resources (Australia) Pty Ltd. Douglas Partners Pty Ltd, designed the support systems for the tunnel. The writer was involved in computer modelling of the tunnels as part of the design process.

## THE M2 TWIN TUNNELS

### Location and Dimensions

The M2 tollway, under construction in 1995 runs from Epping Road to Old Windsor Road in Sydney, Australia. At North Epping, the tollway crosses under Norfolk Road and Epping Oval, through twin 11.7m wide tunnels. The tunnels, were commenced in September 1995 and were cut through Hawkesbury Sandstone at a depth ranging from 8.0 to 22.0 m below ground surface. The tunnels are 460 m long and are separated by 6.8 m. The arch shaped tunnels have a height of 7.8 m at their crown. Figure 1 shows the location of the tunnels.

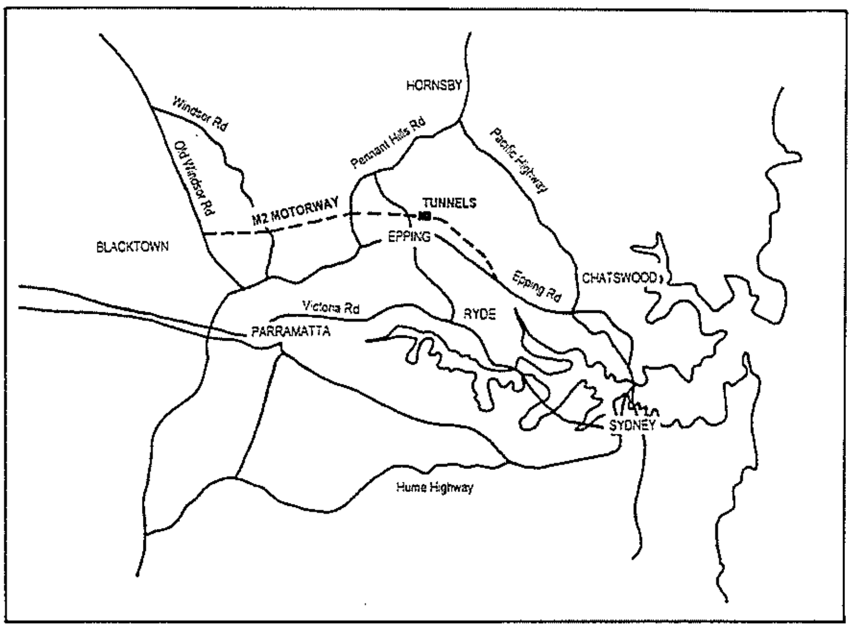



Figure 1. Location of the M2 Twin Tunnels

Geology

Test bores were drilled along the length of the tunnels by others. These indicated that the tunnels are generally located in medium strength, slightly weathered, medium grained sandstone. Features of the rock mass include horizontal bedding partings, some clay filled, spaced at 1.5 to 3.0 m and two subvertical joint sets with strike NNE and ESE, spaced an average distance of five or more metres. The tunnel itself is oriented approximately East West.

INSITU STRESSES IN HAWKESBURY SANDSTONE

The Hawkesbury Sandstone of Triassic age is characterised by high locked in horizontal stresses. The measured horizontal stresses in the rock are far in excess of those that would be expected from the weight of the rock alone, and have been measured at more than two and a half times the vertical stress. As a result, any excavations in this material involve some inward movements of the side walls caused by the elastic response of the material as it is relieved of these compressive stresses. Where bedding planes are present, these movements are concentrated along these planes of weakness. This phenomena has been observed on sites involving deep excavations in the Sydney C.B.D., and when not predicted and allowed for has resulted in disastrous effects. Parts of buildings have been literally crushed by the inwards movements of the walls of the excavation pressing against the structure. Other buildings have been literally pulled apart by stress relief that has occurred when deep excavations have been opened in adjacent blocks.

The value of horizontal stress $\sigma_h$ adopted by Douglas Partners Pty Ltd for design of the M2 twin tunnels was

$$\sigma_h = 0.5MPa + 2.5\sigma_v \qquad (1)$$

where $\sigma_v$ is the vertical stress and is equal to the overburden pressure:

$$\sigma_v = \gamma d \qquad (2)$$

where $\gamma$ is the unit weight of sandstone, and $d$ is the depth below ground surface.

The equation was based on near surface measurements obtained by over coring at the World Square site in the Sydney C.B.D. by D.J. Douglas & Partners in 1988 and measurements from other sites around Sydney. The results of the tests were plotted on a graph and a visual line of best fit applied to formulate the equation.

COMPUTER MODELLING

Computer modelling of the tunnel cross section was carried out using a commercially available program called FLAC [3]. FLAC is a two dimensional non linear explicit finite difference program that is used for stress analysis of a continuum. The program is used for rock or soil masses with a range of material behavioural models and can include interfaces such as joints or bedding planes and structural elements such as rock bolts or tunnel linings.

The computer modelling was used to model the staging of the tunnel excavations and the installation of rock bolts. As the stress distribution is path dependant, the staging of the excavation was simulated as closely as possible, with a central top heading being taken out, then the roof bolts installed before excavating out the lower bench in each tunnel. The tunnel geometry is modelled by developing a finite difference grid, an example of which is shown in Figure 2.

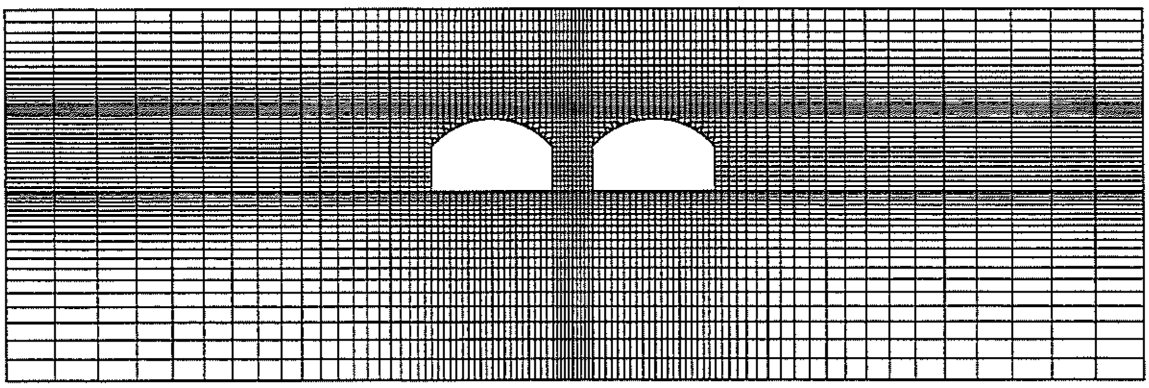


Figure 2. Typical finite difference grid of tunnel geometry used for FLAC modelling

Output from the program included the stress distribution around the tunnels and predictions of the tunnel movements and the expected loads on the rock bolts. Various models were tested that looked at different combinations of tunnel geometry and joint and bedding plane locations.

The modelling work was carried out in parallel with other analytical work to formulate the final tunnel design. This work included the use of rock mass classification systems such as the Q System [1], and the Rock Mass Rating system [2], and a beam analysis to determine the minimum bolt lengths.

## Material Properties

The use of FLAC required the input of a set of material properties for the sandstone and properties of the joints and bedding planes, as well as the insitu stresses previously mentioned. The sandstone was judged to be uniformly medium strength and behaving according to a Mohr Coulomb material model. The properties used for the rock mass are summarised in Table 1. They were chosen based on conservative values and were obtained from UCS and point load tests on the rock and on the properties used for the modelling of World Square by D.J. Douglas & Partners in 1988. These properties were verified by back analysis, comparing predictions of movements with actual measurements. The properties used for the bedding planes and vertical joints are listed in Table 2.

Table 1. Assumed properties of medium strength Hawkesbury Sandstone

| PROPERTY | |
|---|---|
| Unconfined Compressive Strength | 15 MPa |
| Elastic Modulus | 2000 MPa |
| Poisson's Ratio | 0.2 |
| Density | 2300 kg/m$^3$ |
| Cohesion | 3 MPa |
| Angle of friction | 45° |
| Tensile Strength | 1 MPa |
| Shear Modulus | 0.83 GPa |
| Bulk Modulus | 1.1 GPa |

Table 2. Assumed properties for discontinuities

| PROPERTY | CLAY FILLED HORIZONTAL BEDDING PLANES | VERTICAL JOINTS |
|---|---|---|
| Normal Stiffness | 2 GPa | 20 GPa |
| Shear Stiffness | 0.5 GPa | 5 GPa |
| Tensile Strength | 0 | 0 |
| Cohesion | 10 kPa | 0 |
| Angle of Friction | 25° | 45° |

## MODELLING RESULTS

The results of the FLAC modelling indicated that the tunnels would basically be self supporting in medium strength sandstone, however consideration was given to the possibility of unfavourable joint orientations or bedding plane locations causing the formation of loose blocks in the roof. For this reason, a decision was made to install 3 m long bolts in a 2.0 m grid in the roofs of the tunnels.

The models indicated that horizontal compressive stresses as high as 4 MPa could be expected in the tunnel crown and vertical stresses up to 2 MPa in the pillar between the tunnels, so there was little likelihood of large scale stress failures in the rock.

Inwards movements of the side walls of up to 8 mm and of the roof and floor of 3 mm were predicted. An interesting consequence of these movement predictions is their effect on the portal design. The design of the portals by others included a rigid steel reinforced concrete arch to be installed around the inside of the rock cutting at the portal. The curved part of the arch would be cast insitu on completion of the top heading only, then the bench would be excavated and the walls of the arch cast, however, the FLAC modelling predicted that once the arch was installed, after the bench was completed, a further 5 mm inwards movement on each side of the arch could be expected. As this movement would be enough to crush the arch, the portal had to be redesigned.

A summary of the important results is given in Figure 3.

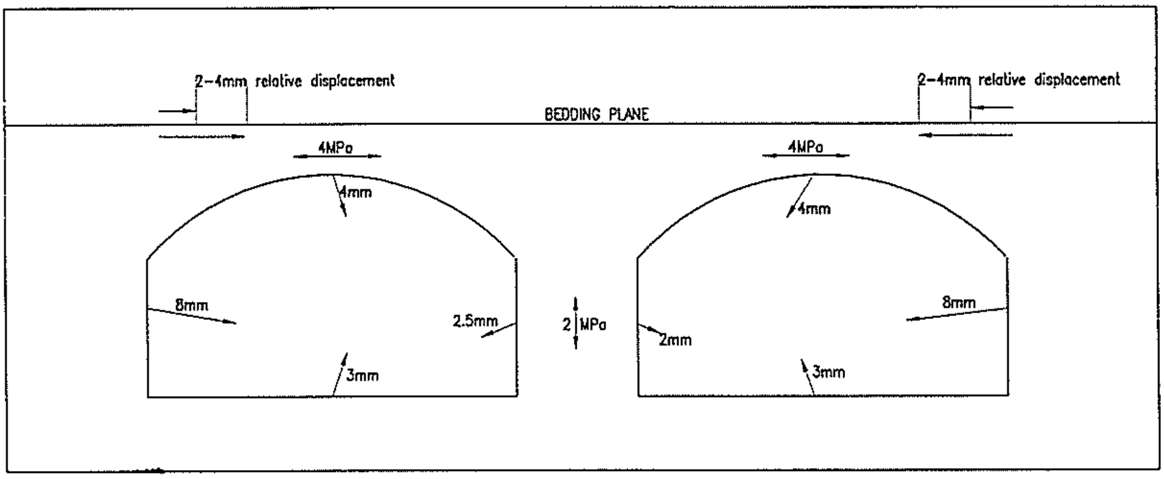



Figure 3. Summary of results from FLAC modelling

### Bedding Planes

Of particular interest was the behaviour of the tunnel when there was a horizontal bedding plane above the roof of the tunnels. The log from a bore drilled near the eastern portals indicated the presence of a clay filled bedding plane only 1.6m above the tunnel crown. Modelling of the tunnels with this bedding plane indicated that the bedding plane acted as a stress "barrier", concentrating horizontal stresses that were redistributed around the top of the tunnel into the rock between the tunnel crown and the bedding plane . As a result there was a significant drop in stress immediately above the bedding plane of the order of 1 MPa. This would be likely to cause some small differential movements of the bedding plane. The rock below the bedding plane was predicted to compress some 2 to 4 mm more towards the centre of each tunnel, than the rock above the bedding plane. This should not cause any stability problems for the tunnel, but could have disastrous consequences for fully grouted rock bolts that intersect the bedding planes. Modelling indicated that these movements could cause the rock bolts to fail in shear where they intersected the bedding plane.

### Design of rock bolts

The original design of the rock bolts was to install standard fully grouted passive dowels using 24 mm high strength bar. However, analysis of the possible bedding plane movements indicated shearing of the bolts. Rather than prevent the movements, it was decided to design the bolting system to allow for the movements while still supporting the rock. The bolts were required to be fully grouted for corrosion protection. Two options were considered, namely:

1) Fully grouted passive dowels with a compressible bandage adjacent to bedding planes:

After drilling the hole for the bolt, the hole would be spoon tested to detect jointing. If the presence of any clay filled joints was detected in the first two metres, then, the dowel would be wrapped in 'Denso' tape for 0.5m either side of the joint and the bolt grouted in as normal. The 'Denso' tape would form a compressible bandage of approximately 1.5 to 2 mm around the bar, between the grout. 'Denso' tape is a grease impregnated material, and this material would be compressible enough to allow some lateral movement of the bar. If there was 2 to 4 mm movement along a bedding plane, the grout would be cracked by the shearing, but the bar would be able to bend over the one metre length of 'Denso' taping. As well as allowing lateral movement, the 'Denso' tape would maintain corrosion protection for the steel, even after cracking of the grout.

2) End anchored dowels:

The majority of the movements associated with the relief of the horizontal stresses are caused by the elastic response of the rock, and as such, they are relatively instantaneous. The bolts would be initially installed with galvanised end anchors and not grouted. As the face of the tunnel advanced away from the location of the bolt, the horizontal stresses would be distributed around the tunnel and any slipping of the bedding planes would occur. The diameter of the bolts would be 24 mm, inside a 41 mm diameter hole, leaving a 17 mm gap to accommodate the movement. After the tunnel face had advanced at least two tunnel diameters away, the bolts could be safely grouted. By then the majority of movement would have occurred.

The first option, while having less cost in materials, would require more labour and installation time, and for this reason the second option was chosen.

## CONCLUSIONS

The Hawkesbury Sandstone of the Sydney Basin is subject to high insitu horizontal stresses, and excavations in it involve various rock movements associated with the stress relief. Computer modelling of the M2 twin tunnels predicted sliding of bedding planes of sufficient magnitude to shear or over stress normal fully grouted rock bolts. Foreseeing the movements allowed for an alternate bolting design to be used that should allow for the movements without failing the bolts. The prediction of movements also affected the design of the tunnel portals, as the magnitude of the expected movements would have been large enough to crush the original reinforced concrete arch.

References


1. Barton N., Lien, R. & Lunde. J. 1974 Engineering classification of rock masses for the design of tunnel support, *Rock Mechanics.*, 6(4): 189-236.

2. Bieniawski, Z.T. 1976 Rock mass classifications in rock engineering, *Proc. Symp. on Exploration for Rock Engineering*, Johannesburg, 1: 97-106.

3. Itasca 1992. *FLAC - Fast Lagrangian analysis of continua, version 3.2; User Manual.* Itasca Consulting Group Inc., Minneapolis

# GROUNDWATER INVESTIGATIONS FOR AUCKLAND CITY COUNCIL

J.P. BURR

Riley Consultants Ltd., P.O.Box 100-253, NSMC, Auckland, New Zealand

## SUMMARY

The 1994 Auckland region drought prompted an investigation into the groundwater resources of the isthmus. Auckland City Council formed a team to identify and develop these resources for both potable and non-potable supplies. Large volumes of groundwater are potentially available from the volcanic formations on the isthmus during a drought. Following the lifting of the drought the schemes were re-examined under normal economic and environmental constraints. Only those potable supplies able to comply with these parameters were selected to continue. Three schemes were chosen to be taken to the Resource Consent application stage. These schemes propose to supply a total of 15 000 $m^3$/day of potable water to the council's reticulation network. The applications are currently being reviewed by the Auckland Regional Council.

## INTRODUCTION

In 1994 the Auckland region suffered a drought. Reservoir storage levels fell to approximately 30% of total storage in mid 1994 and further low rainfalls over the 1994 winter period could have resulted in drastic action being required during the 1994/1995 summer and possibly beyond. In mid 1994 Auckland City Council (ACC) set up the Water Alternative Supply Action Team (WASAT) to find and develop both potable and non-potable water supplies within the boundaries of the city. Riley Consultants Ltd were engaged by Council to provide geological and geotechnical expertise to the WASAT programme.

Many potential water sources were identified within the Council boundaries. These were generally associated with the areas of volcanic activity on the isthmus. Several of these sources were developed as non-potable water supplies while planning took place for the more complicated potable schemes.

In late 1994 the drought was lifted by high spring rainfalls. The WASAT schemes were re-examined by the Council and a decision was made to cease the non-potable supplies but continued to progress the potable municipal supply schemes under normal economic and environmental constraints.

This paper will give a general overview of the potential water sources in the isthmus as well as providing details on the schemes chosen to proceed.

## GROUNDWATER IN AUCKLAND ISTHMUS

Figure 1 is a plan of the Auckland Isthmus showing the locations of the known volcanoes and associated geological activities.

This paper will focus on groundwater found in the volcanic features in Auckland, however further groundwater can be found in the sedimentary Waitemata strata elsewhere. The groundwater from the volcanic areas is easier to locate and abstract and higher volumes are available, hence its greater popularity.

Groundwater is available from two sources within each volcanic area. One source is the scoria cone itself. The volcanic activity creates a relatively impermeable tuff ring around the volcano resulting in essentially a large reservoir within the scoria cone. Many of these cones are large in diameter and capable of holding significant volumes of water, each equivalent to existing reservoirs.

During the drought virtually all of the volcanic cones in Figure 1 were studied to assess their viability as water supply sources. A sustainable yield of approximately 40,000 $m^3$/day and a twelve month emergency yield of approximately 110,000 $m^3$/day were estimated using available data. Some of these schemes would have significant environmental effects and were only added in case of an emergency drought situation occurring. It has been estimated that the total volume of groundwater stored in these cones could be approximately 100 million $m^3$, a similar volume to the existing reservoir capacity of the region. Table 1 gives these possible yields for each volcanic cone.

Table 1. Volcanic cone estimated yields

| Site | Description | Sustainable Yield ($m^3$/day) | 12 mth Emergency Yield ($m^3$/day) | Comments |
|---|---|---|---|---|
| Three Kings | Scoria Cone | 2500 | 15000 | Winstones Quarry |
| Mt Eden | Scoria Cone | 1000 | 5000 | |
| Teachers College | Eruption Centre | ? | ? | |
| Mt Albert | Scoria Cone | 1000 | 5000 | |
| Mt Roskill | Scoria Cone | 1000 | 5000 | |
| Domain | Tuff Crater | 1000 | 2000 | |
| Magistrate Court | Scoria Crater | ? | ? | Centre of Auckland |
| Mt Hobson | Scoria Cone | 1000 | 5000 | |
| Mt St John | Tuff Crater | 500 | 2000 | |
| Little Rangitoto | Scoria Cone | 500 | 2000 | |
| One Tree Hill | Scoria Cones | 1000 | 10000 | Fully Allocated |
| South Epsom | Scoria & Tuff | 1000 | 1000 | |
| Mt Wellington | Scoria Cone | 1000 | 15000 | |
| Purchas Hill | Scoria Cone | - | - | Quarried Away |
| Mt Smart | Scoria Cone | 1000 | 5000 | Fully Allocated |
| Taylors Hill | Scoria Cone | 1000 | 5000 | |
| Mt Richmond | Tuff Crater | 1000 | 5000 | Fully Allocated |
| Sturges Park | Tuff Crater | 500 | 3000 | |
| Rangitoto | Scoria Cone | 25000 | 25000 | DoC Park |
| **TOTALS** | | **40000** | **110000** | |

The main areas of basalt aquifers are shown in Figure 1. These are:-

1. Western Springs
2. One Tree Hill/Onehunga
3. Mt Wellington
4. McLennans Hill

Both the One Tree Hill/Onehunga and the McLennans Hill aquifers are essentially fully allocated to existing users. The Western Springs and Mt Wellington aquifers were seen as the best areas to investigate further.

The second source is the basalt flows that were produced by the volcanic activity. These lava flows generally followed the ancestral valley systems in the existing Waitemata Series formations. The groundwater also flows along these pathways from the volcanoes to the harbours. The basalt is very fine grained and impermeable, however the highly fractured nature of the rock combined with more scoriaceous layers and occasional lava tunnels produce an overall geological feature with high permeability and good groundwater storage characteristics.

Of the two sources the basalt flows provide greater volumes of sustainable groundwater due to the larger surface areas open to rainfall recharge. While the scoria cones contain large volumes of water their comparatively small catchments restrict the yield available on a sustainable basis.

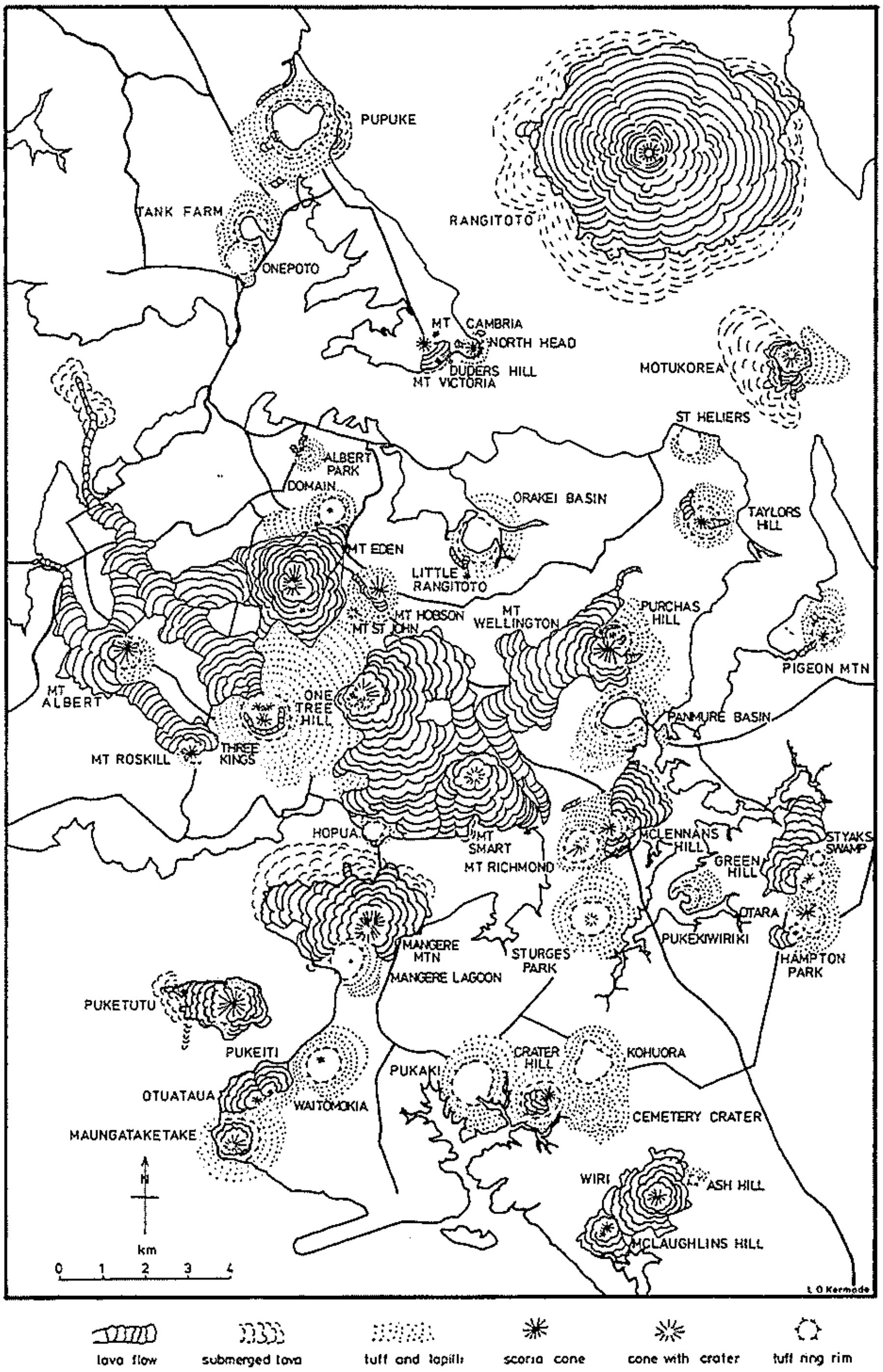



Figure 1. Map of Auckland Volcanoes. (Source: Searle. 1981 [4]).

## GROUNDWATER INVESTIGATIONS

At the height of the drought Resource Consent Applications were submitted for the seven most feasible schemes. In addition, to these a further opportunity arose to use groundwater from the proposed dewatering of the Three Kings quarry, located in the scoria cone. As the prospect of a drought decreased these were eventually culled to four schemes:

1. Western Springs Aquifer
2. Mt Wellington Aquifer (Beside Southern Motorway)
3. Mt Wellington Cone
4. Three Kings Quarry

Scheme 2 was the first to be physically investigated. This scheme was abandoned following adverse water quality results. Its location downstream of an old established industrial area resulted in high levels of iron and zinc in the water. It was found to be uneconomic to treat this water to the required standard.

The loss of the Mt Wellington aquifer scheme in combination with the potential difficulties apparent in drilling in the public reserve on the Mt Wellington cone led to the decision to relocate the Mt Wellington cone scheme. The well locations were repositioned to abstract groundwater from the eastern edge of the aquifer beside Mt Wellington itself. This allowed the utilisation of a large catchment created by the dewatering of the nearby Mt Wellington Quarry.

Three boreholes were drilled in this area to provide both further information on the local geology and for possible use as future abstraction wells. Figure 2 is a plan of the area of the proposed scheme. Using this geological data and hydrological information gained from pump tests of the holes the effects of proposed abstraction scenarios were estimated. This information is given in detail in Riley Consultants Ltd. report on the scheme [2]. A volume of 5000 $m^3$/day of groundwater was chosen as the most economically and environmentally appropriate abstraction rate for this site.

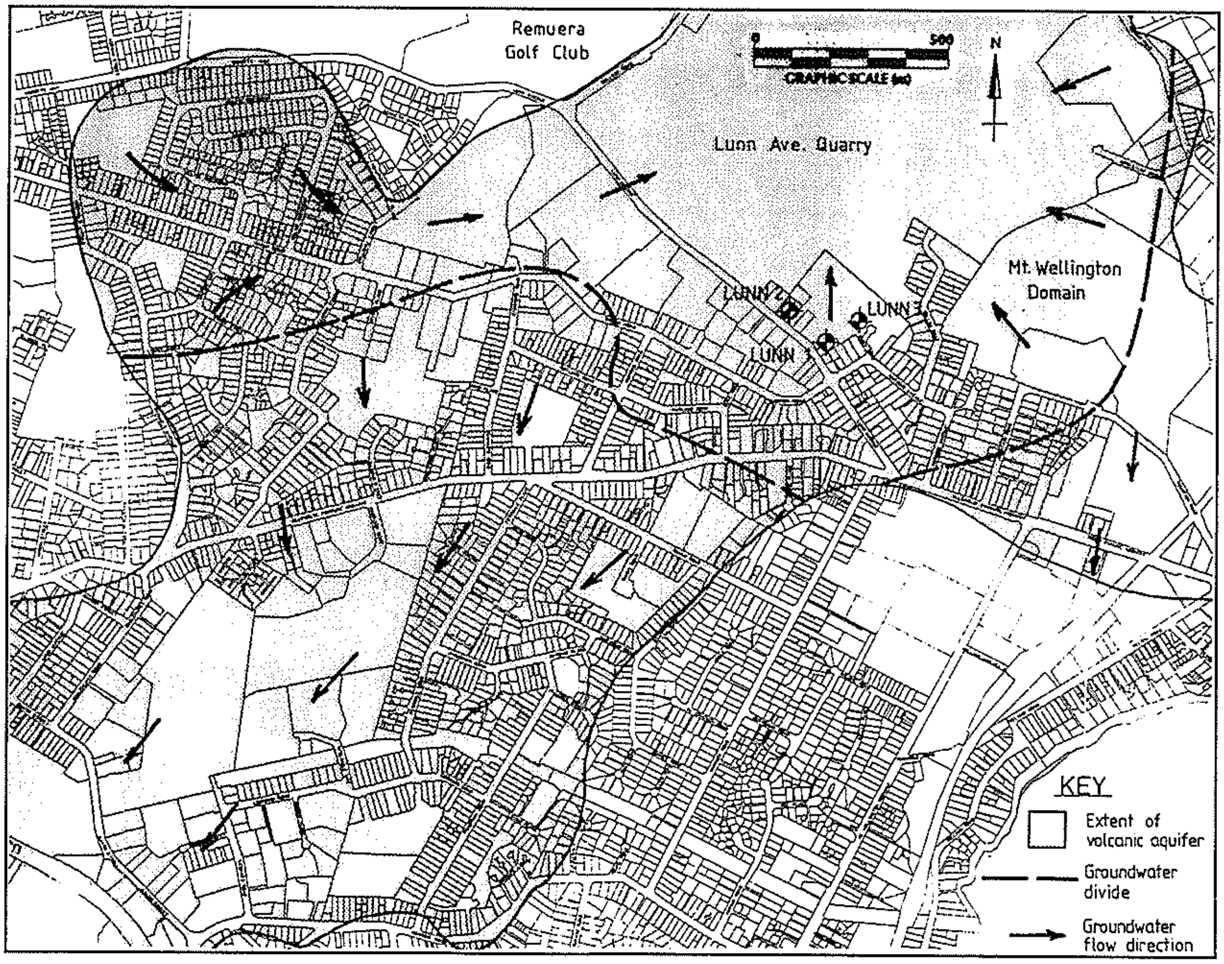



Figure 2. Plan of Mt. Wellington scheme area

The Mt Wellington and One Tree Hill/Onehunga aquifers have undergone hydrogeological investigations previously due to their high usage and proposed developments in their regions. However, the Western Springs aquifer is relatively unused and has not been the subject of any significant studies into its behaviour.

The team were therefore required to undertake a full groundwater investigation in this area to provide the Auckland Regional Council (ARC) with enough information to justify the proposed abstraction rate. The investigation consisted of a ten day (1000 $m^3$/day) pump test on each of the three proposed wells with observation of the effects recorded at the two unpumped wells, three new observation bores, Western Springs lake, the Zoo well and both Meola and Motions Creeks. Figure 3 is a plan of the general area of the scheme.

From these tests a model of the lower section of the aquifer was developed using the finite difference programme MODFLOW. This model was calibrated against the recorded data and the proposed groundwater abstraction scenarios were then modelled with the programme.

The model was able to predict the effects of pumpage on the downstream creeks as well as Western Springs lake. This information was vital to the study on the effects on biota being performed by a subconsultant.

It was determined that abstraction rates of 5000 $m^3$/day to 7000 $m^3$/day were feasible from both technical and environmental viewpoints. A decision was made by the ACC to apply for 5000 $m^3$/day for municipal supply as this would allow all three schemes to use the same water purification works design as well as being a long term economically viable rate. Riley Consultants Ltd. [3] report on the proposed scheme contains details of the investigation and modelling.


Figure 3. Plan of Western Springs scheme area.

The Three Kings scheme was developed in conjunction with the quarry operators, Winstone Aggregates Ltd. Much of the required information for this scheme was already available from their previous geological studies on the site. The proposed abstraction rate is influenced by the quarry requirements to dewater the cone. It is envisaged that an initial rate of 5000 $m^3$/day will be abstracted to lower the groundwater level. This should take approximately 17 years to complete. Thereafter the scheme will reduce to abstracting the sustainable volume of approximately 2500 $m^3$/day to keep the groundwater level in the cone constant.

The quality of the raw groundwater in the three final schemes was good. As the catchments are generally in established urban areas there was some initial concern that they may have been adversely effected by the activities directly above them. However, only in 2 of 141 potential contaminant tests performed on each of the 7 wells were the 1995 Drinking Water Standards [1] Maximum Allowable Values (MAV) were exceeded. These two values are not expected to affect the final potable water produced by the treatment plants, and the schemes will have an A rating in accordance with the 1995 Drinking Water Standards.

The total of 15000 $m^3$/day from all three schemes will provide approximately 15 % of Auckland City Council's municipal supply. The above abstraction rates have been calculated for the average rainfall recharge of the aquifers. During a 1 in 200 year drought (the current Watercare Services Ltd design drought) the available volume of potable water from the three schemes combined is estimated at approximately 12,500 $m^3$/day compared with 15,000 $m^3$ in an average day. These rates will be subject to ARC regulation during such a drought. It has been estimated that this volume will defer the need for a new regional water supply scheme for 2 to 3 years.

Applications for Resource Consents have been made for all three schemes. These are currently being processed by the ARC. It is anticipated that if the consents are obtained construction of some or all three schemes will take place in the 1996/1997 ACC financial year.

## CONCLUSION

Large volumes of groundwater are potentially available in the volcanic formations in the Auckland isthmus. The numerous volcanic cones in the area contain huge volumes of stored water while the extensive basalt flows are capable of producing significant sustainable volumes of groundwater. Four potable water schemes were identified by the WASAT as worthy of feasibility studies. Following these initial investigations three of the potable schemes were further investigated with geological, hydrological and environmental data gathering, collating and finally the production of reports on the Assessment of the Effects on the Environment. It is proposed to abstract a total of 15 000 $m^3$/day of groundwater for use as municipal supply from the three schemes. Resource Consents have been applied for and these are currently being processed by the Auckland Regional Council.

## ACKNOWLEDGEMENTS

The author gratefully acknowledges the assistance of the Auckland City Council and the Auckland City Development Consultancy in this project. In particular Mr Greg Paterson of the Development Consultancy as the team leader.

## REFERENCES

1 Ministry of Health. 1995. *Drinking-Water Standards for New Zealand 1995.* Wellington.

2 Riley Consultants Ltd. 1995. *Report on the Groundwater Resources of the Mt. Wellington Catchment.* 94158/WEL, Auckland.

3 Riley Consultants Ltd. 1995. *Report on the Groundwater Resources of the Western Springs Catchment.* 94158/CHA, Auckland.

4 Searle, E J 1981. *City of Volcanoes.* Longman Paul, Auckland.

# THE RIVER RODING PROJECT
# DIFFERENCES IN GEOTECHNICAL ENGINEERING IN AUSTRALIA AND THE UK

G.W. CAMERON
Woodward-Clyde Pty Ltd
Melbourne, Australia

## SUMMARY

This paper presents a review of the geotechnical site investigation for a tide retention barrage across the River Roding, a tributary of the River Thames, London. An overview of the project is given together with a description of the parties involved and their respective objectives and roles. In particular the inter-relationship between the geotechnical engineer employed by the project Engineer and geotechnical engineer employed by the site investigation Contractor is discussed. Finally the paper discusses the potential pitfalls of the system where the geotechnical engineer carrying out the field works is removed from having a comprehensive knowledge of the project and its objectives.

## PROJECT OVERVIEW

The London Borough of Barking and Dagenham examined the feasibility of constructing a tidal barrage (weir) across the River Roding, which is a tributary of the River Thames in eastern London. The aim of the barrage was to retain tidal waters behind a barrier and thus create a large expanse of water. An open area of water was seen to be a way of increasing the amenity value of the area and produce a focal point for the nearby town center.

The barrage project was seen by the London Borough of Barking and Dagenham as being a primary impetus in changing the nature of the area close to the city center from mainly supporting heavy industry to one increasingly focused on the residential aspects of the area.

Robert West and Partners were commissioned to expand upon the pre-feasibility study and produce a detailed feasibility study and costing for the barrage. As part of the feasibility study a geotechnical investigation was to be carried out at the structure's preferred location and in the immediate vicinity of the barrage. An amount of UK£40,000 (A$80,000) was allowed for the investigation. A perspective view of the barrage is shown in Figure 1 below.

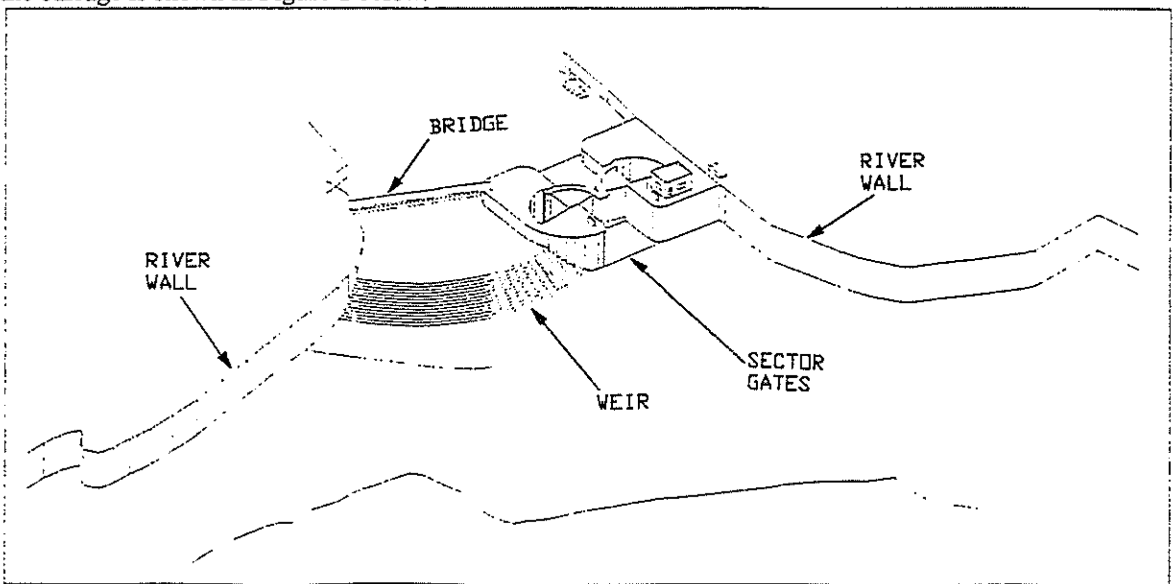



Figure 1. Perspective view of the River Roding barrage

INTER-RELATIONSHIP OF THE VARIOUS PARTIES

The relationship between each of the receptive parties plays an key part in the efficient running of any project. Long or cumbersome lines of communication have the potential to slow the information exchange process to such an extent that in a dynamic project, such as a geotechnical site investigation, important information may be lost or simply not recorded.

Figure 2 below is a diagrammatic representation of the parties involved in the project and their relationships. The downward flow of information from the Principal ultimately to the person in the field is indicated together with the upward flow of information in the form of Reports gained as a result of the investigation.

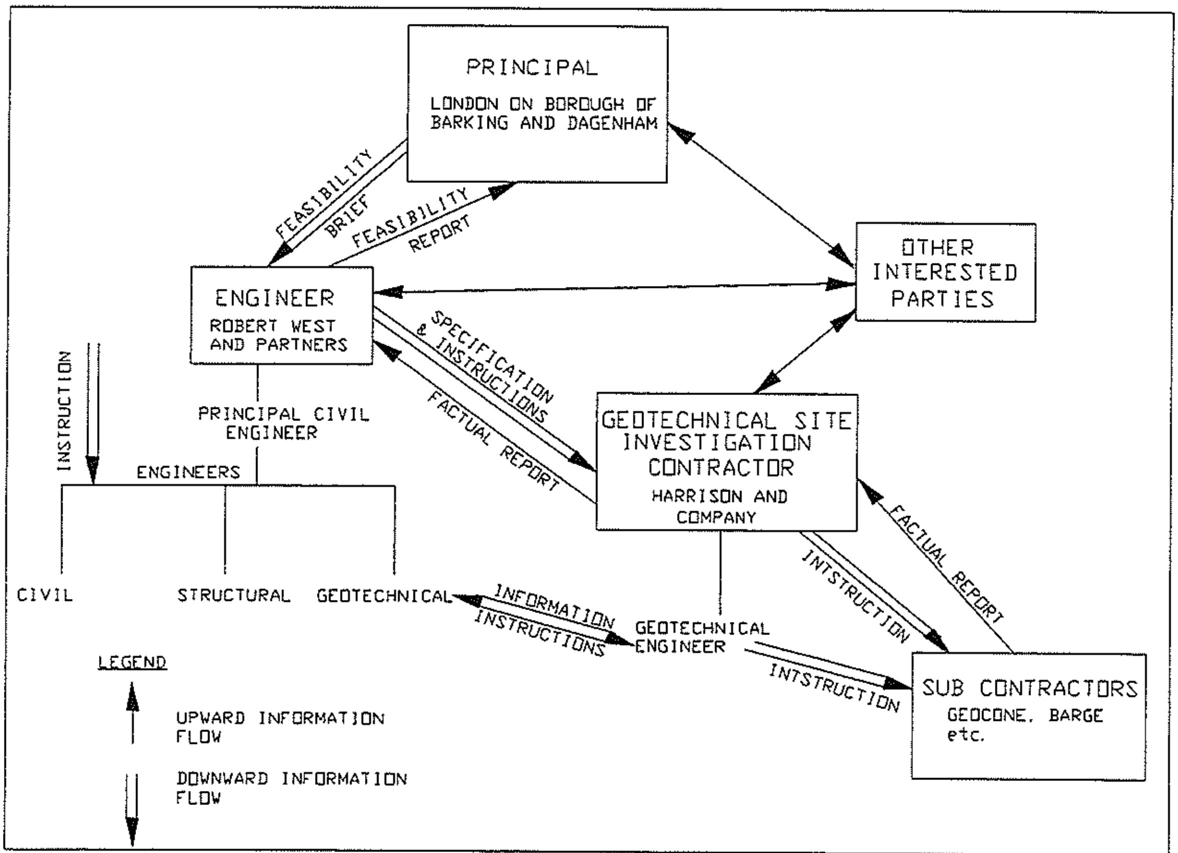



Figure 2. The parties involved in the River Roding project and their inter-relationship

The London Borough of Barking and Dagenham was the Principal for the project and they provided the funding and overall project definition. Robert West and Partners were the project Engineers and operate within the brief for the feasibility study. They were responsible for the design of the structure within the parameters set by the Borough. For the purposes of the geotechnical investigation Contract, Robert West operate as the nominated Engineer's representative with the Borough nominated as the Engineer for contractual and financial reasons. Robert West and Partners employed a geotechnical engineer (the author) to provide specialist advice. The specialist geotechnical site investigation Contractor was Harrison and Company which also employed a field geotechnical engineer. The geotechnical engineer employed by Harrison and Company effectively manages the Contract including site supervision, management of sub-contractors and initial preparation of the Factual Report . Other parties indirectly involved are land and riparian owners, river users and specialist interests groups.

It is important to note that the typical system of geotechnical site investigation in the UK makes use of a Contract and associated detailed specification prepared by the Engineer and issued to a number of specialist geotechnical site investigation Contractors. Typically the Contract documentation details the procedures to be followed and frequency of the physical on-site works, logging, sampling, testing (field and laboratory), daily reporting and the production of a "Factual Report". The site investigation company would supply the drill rig(s), field based geotechnical engineer (who acts as site supervisor) and may sub-contract out other specialist parts of the contract (such as electric friction cone testing). In a limited number of cases the contractor may be asked to carry out interpretation of the results, though in the author's experience this is only rarely the case.

While this does not totally set the investigation "in stone", it does make any changes during the site works cumbersome. In the first instance the contract must be flexible enough to allow changes to be made as the field investigation proceeds. It then requires the Contractor's geotechnical engineer to recognise the significance of an aspect of the investigation, which is often difficult from "cold" documentation. The Contractor's geotechnical engineer then communicates this to the Engineer's geotechnical engineer, who will then liaise with the structural engineer and/or client, receive instructions and then pass these back to the Contractor's geotechnical engineer.

This is somewhat contrasts with the typical Australian geotechnical site investigation where the geotechnical engineer (often a consultant engaged directly by the owner or Engineer) would organise and implement the investigation to achieve the aims set in the site investigation design. More often that not, some parts of the site investigation will be modified in light of preliminary findings with the aim of gaining more information about a particular feature. This is particularly true when there is only limited information available prior to the on site investigation commencing.

Although the system used in the UK adds only one (or two) levels to the chain of communication it was the author's experience that messages can often be lost (due to time pressures of those involved) or can become distorted; it is often difficult to describe something over the phone/on paper that may not have been sighted in the field. While the answer to this may seem be for the Engineer's geotechnical engineer to be on site all/part time, this does introduce a degree of duplicity and it would seem that this does not often occur in reality. The other major disadvantage of this system is that the Engineer's geotechnical engineer may lose a certain amount of the "feel" for the conditions at the site.

## INFORMATION TRANSFER FROM THE GEOTECHNICAL INVESTIGATION

In this type of investigation, information transfer is firstly <u>to</u> the Engineer's geotechnical engineer who needs to gain an understanding of the project, and particularly the structure proposed. In what is often an iterative process a number of structures may be discussed by the Principal or those in the Engineering firm, and for each part of the structure broad options and footing types may be examined. Following this it would be usual to carry out a detailed desk study to both confirm the to options developed previously and assist in the detailed planning of the investigation.

In the case of the River Roding, investigation the needs of a number of type of structures and associated footing systems were examined leading, to a number of more likely options targeted for further investigation. The desk study for the River Roding barrage sought information on the geology, hydrogeology, topography, hydrography, hydrology, riparian information, land ownership and access details. The author believes this was a valuable exercise (as is usually the case) and added greatly to the understanding of the area and gave a good impression of what could be expected during the on site works.

The information gathered was then distributed to the civil/ structural engineer and the Principal. The overall approach of the site investigation was then agreed upon in terms of information sought, scope and a review of the expected cost.

From this information exchange, the geotechnical engineer then designed a detailed investigation and produced the associated Contract, Specification and a Bill of Quantities that was distributed to a number of selected Tenderers.

It is not until this document is distributed that the person actually carrying out the investigation gains any information pertaining to the site. Unfortunately, the typical documentation for a geotechnical site investigation Contract will detail the physical works to be carried out <u>and not the specific information objectives of the works to be carried out.</u> This is where there is a discontinuity introduced into the flow of information for the project. The author did not encounter any situations where the complete desk study information was given to a Contractor, or where the contractor received more than a cursory briefing of the objectives of the investigation.

In the author's view this tends to inhibit the contractor from carrying out of the project with the most efficiently. The Contractor is also likely to have objectives of his own that are not tightly bound in with the current project such as operating to the letter of the Specification, scheduling of future work and (importantly) making a profit. An "Australian" geotechnical investigation is still likely to be placed under the same pressures but they would also be balanced to a certain extent by the information gathering objectives and needs.

After an evaluation of the Rover Roding site investigation tenders, Robert West and Partners informed the London Borough of Barking and Dagenham of their preferred tenderer, who was appointed in accordance with the recommendation.

The on-site works commenced after the award of the Contract. It was here that the differences and possible pitfalls with the UK contractual became apparent.

The field engineer is operating to a proscribed field investigation which details borehole locations and frequency of testing which have been determined as a result of the desk study. To a certain extent, the field engineer is discouraged from making variations to the specified testing routine due to unforeseen circumstances for to inter-related reasons:

- the lack of knowledge of the particular objective of that test location; and,
- the inability to judge where additional of differing testing/sampling would be better able to meet the objective at that location.

By way of example the field engineer may see the primary objective of a certain borehole as being the taking one undisturbed sample per metre to a depth of 5 metres, then conducting a SPTs at the interface with sand and one every metre thereafter to a target depth of 10 metres. Conversely the geotechnical engineer who designed the investigation may see the objective of the investigation as locating a suitable stratum in which to found an end bearing pile and verify that there are no underlying loose layers. What would occur if at a depth of 6 metres a 0.5m sand layer that was much easier to penetrate was struck?. One would hope that a further SPT would be taken, however, this is less likely to occur if the field geotechnical engineer were not aware that end bearing piles are proposed for the structure. Further if the design engineer was not aware of this occurring at this hole he may not be able to modify the next hole so that the SPT were taken in what may be a looser sand layer. What would happen in this situation? Possibly the sand layer that was easier to penetrate would be noted in the logs, but with not give a quantitative measurement on which future design review could occur.

It is also the case that the field geotechnical engineer is not aware of the significance of some seemingly minor features as the intrinsic knowledge gained as part of the desk study has not been effectively passed on. How often has a seemingly insignificant part of the geology (water bearing sand stingers and the like) revealed themselves to be critical in later design of a structure?

Much of the problems described above can be partially remedied by informal discussion between the respective geotechnical engineers. However, in many cases the author is aware of this has traditionally not been the case.

## PROFESSIONAL DEVELOPMENT ISSUES

In the author's view one on the major detrimental effects of the system is that a partitioning of field and design engineers may occur. Where the geotechnical engineers associated with a project are located primarily in the field or in the design office is that there is the possibility that:

- the design geotechnical engineer can be quite removed from the realities and intricacies of the investigation and not fully understand the inevitable problems that are often routine with most site works such as sufficient access to test sites, the need for clear logging, an understanding of how the drilling/testing is actually carried out and how to overcome the difficulties of "problem ground"; while,
- the field engineer may not have a clear understanding of the overall project objectives and the information sought at each individual test situation.

This has the potential to create an investigation where the aims and objectives are not fully reconciled with the desired design output objectives. This system also fosters the concept of "field engineers" and "design engineers" which the author believes is counter-productive to efficient and fully rounded investigations and professionals.

## CONCLUDING REMARKS

The author believes that the structure of a typical geotechnical site investigations in the UK is different to that in Australia. One major result of this is that in the UK, a geotechnical engineer carrying out the field investigation is removed from the aims and objectives of that investigation. This has the potential to create loss of continuity within the design process.

In contrast , a typical Australian geotechnical investigation would not be burdened with many of the potential difficulties highlighted in this paper. The author believes that, in the Australian context, an interactive flow of information from conceptual to the final design would be established. This would benefit the design process as a whole.

While the author notes that the UK Institution of Civil Engineers requires a specialist geotechnical engineer to gain both field and design experience, a general acceptance of the broad philosophy behind this requirement will benefit both the individual and the practicing geotechnical fraternity.

RIPPABILITY ASSESSMENT FOR THE PROPOSED OPEN PIT GOLD MINE AT GLOBE-PROGRESS, REEFTON, NEW ZEALAND

PHILIP CLARK

Department of Geological Sciences
University of Canterbury, Christchurch, New Zealand

## SUMMARY

There are many methods used to predict the rippability of a site, most of which involve using a particular system to rate the rock mass. A preliminary rippability evaluation of a site may be performed by determining the seismic velocity variation within the rock mass. Seismic velocities are used as an estimate of rippability because they are influenced by geological and physical properties of the rock mass. However, a complete rippability evaluation requires rock mass and rock material characterisation as well as data on possible ripping machinery to be used. If the rock mass is rated as being very good rock, then ripping may be difficult. Likewise, if the bulldozer is underpowered, its productivity may be too low and the ripping rate will be slow.

Six seismic refraction traverse lines were surveyed at the proposed open pit gold mine at Globe-Progress, near Reefton using a single channel seismograph, then interpreted using the Generalised Reciprocal Method. Combining the seismic velocities found and data on Komatsu's D575A-2 bulldozer, it is estimated that 85% of the pit is rippable, and a further 7.5% is marginal. This means that 7.5% of the open pit area is unlikely to be rippable. However, further seismic refraction surveys, as well as a complete rock mass classification, need to be done to provide a three-dimensional site model and to clearly identify the rippable, marginal and non-rippable zones.

## INTRODUCTION

Globe-Progress is located in an area of Early Ordovician turbidite sequences of alternating mudstones and sandstones, known as the Greenland Group (figure 1). In the Late Ordovician and Silurian, folding and metamorphism resulted in ore fluid generation and gold mineralisation in the Reefton area. Mining activity started at Globe-Progress in 1878, and by 1920 the ore reserves had been exhausted. By then, 418,343 ounces of gold had been extracted from 1,062,727 tons of crushed quartz at an average grade of 12.2 g/t. In the early 1980s, CRA Exploration Limited acquired exploration licences over most of the Reefton Goldfield and began a regional exploration programme that resulted in 39 holes being drilled on Globe Hill. The drilling defined an area of disseminated gold mineralisation adjacent to the quartz veins. In 1991, Macraes Mining Company Limited took over the licences for the area. They are now proposing to develop an open pit mine at Globe-Progress based on a resource of 8.94 Mt at 2.69 g/t (Barry, 1993).

To extract the overburden from the pit, either bulldozer ripping, drill and blast or a combination of the two methods will be used. To evaluate the rippability of the site, rock mass and rock material characteristics are required, as well as information on possible ripping machinery to be used.

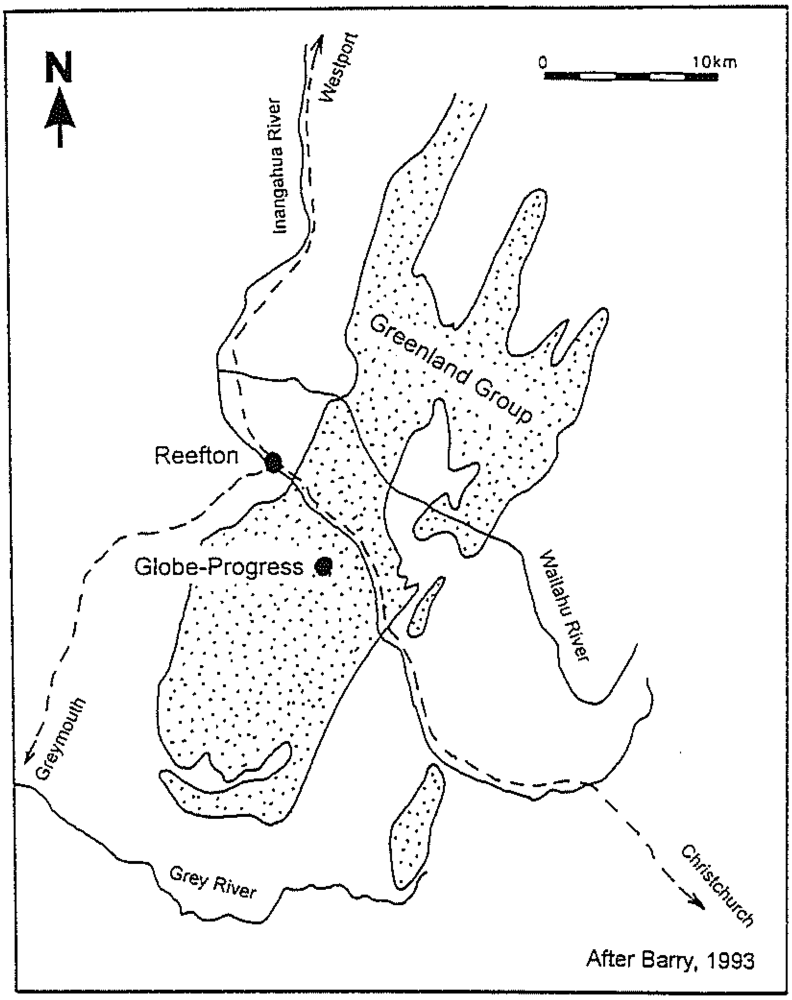



Figure 1: Location of the proposed open pit mine in Greenland Group sediments at Globe-Progress.

## RIPPABILITY EVALUATION

There have been many methods produced that attempt to predict the rippability of rock. One of the first was by Weaver (1975) and is based on Bieniawski's 1973 rock mass classification. It uses Bieniawski's RMR and the seismic velocity of the rock mass. Subsequent studies have altered Weaver's method by trying to predict the productivity of the ripping bulldozer (Braybrooke, 1988; MacGregor et al, 1994; Minty and Kearns, 1983) or by correlating rippability with rock strength and discontinuity spacing rather than seismic velocity (Braybrooke, 1988; Pettifer and Fookes, 1994). The most recent study (MacGregor et al, 1994) recommends prediction of productivity and identification of locations or conditions that will be difficult to rip, and this approach seems to be the most appropriate.

Seismic velocities are used to estimate rippability because they can be related to geological and physical properties of rocks such as density, compaction, cementation, fracturing, anisotropy, mineralogy, porosity, weathering, water saturation and rock elasticity (Palmer, 1980; Weaver, 1975). However, only bulldozer manufacturers use seismic velocity alone to determine the rippability of a site. Weaver (1975) states that seismic velocities should be used to indicate possible methods of excavation and the equipment that could be used, and this approach has been followed in this situation.

A total of 307m was surveyed over six seismic refraction traverse lines using a single channel seismograph and a sledgehammer as the energy source. The Generalised Reciprocal Method was used to analyse the seismic refraction data as this method can tolerate dips in the topography or the refracting surface of up to 20°. An example of the travel time graphs along SR3.1-SR3.2 is shown

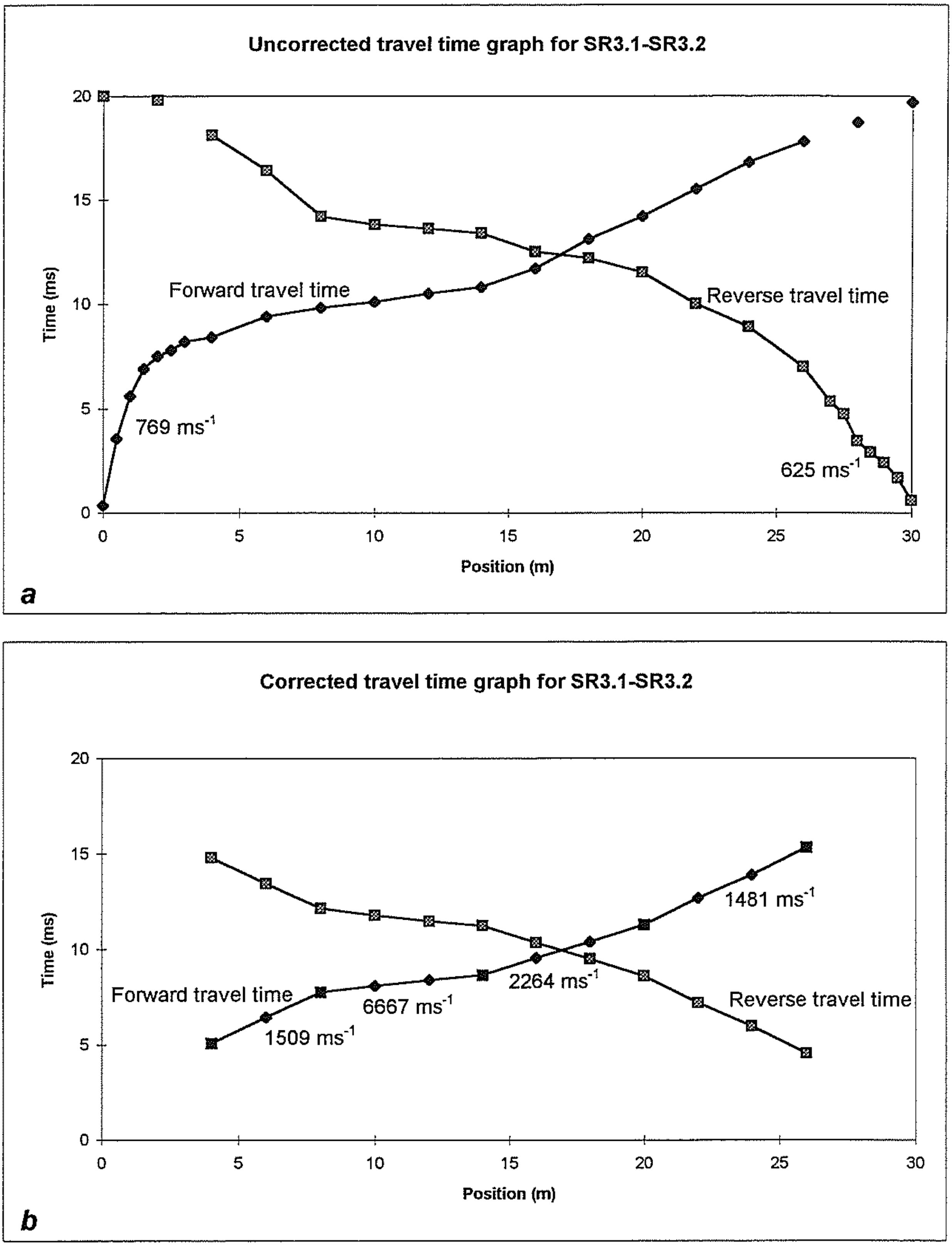



Figure 2: Uncorrected (*a*) and corrected (*b*) travel time graphs for SR3.1-SR3.2

in figure 2. The velocities were sorted into ranges of $500ms^{-1}$ and plotted in a normal distribution (figure 3). The data is very finely skewed, indicating that most of the velocities found are in the lower velocity zones (less than 3000 $ms^{-1}$). On some of the traverse lines the data are slightly scattered, which indicates the presence of defects and/or minor changes in the lithology.

Rippable, marginal and non-rippable zones for Komatsu's D575A-2 bulldozer (the bulldozer recommended by Komatsu; John, 1994) were superimposed onto the normal distribution plot to form a preliminary estimation of areas of rippable, marginal and non-rippable zones (figure 3). Approximately 85% of the velocities fit into the rippable zone - implying that 85% of the pit area will be rippable - and 92.5% of the velocities fit into the rippable and marginal zones, indicating that 7.5% of the open pit is unlikely to be rippable. However, bulldozer manufacturers tend to be overly optimistic in the rippability capabilities of their machines (Braybrooke, 1988; MacGregor et al, 1994) so the non-rippable zones may total more than 7.5 % of the proposed open pit area. An initial study undertaken by Komatsu (John, 1994) suggested that at least 80% of the open pit area was likely to be extracted at a rate of 850 $Bm^3/hr$ and my results therefore, compare favourably with Komatsu's assessment.

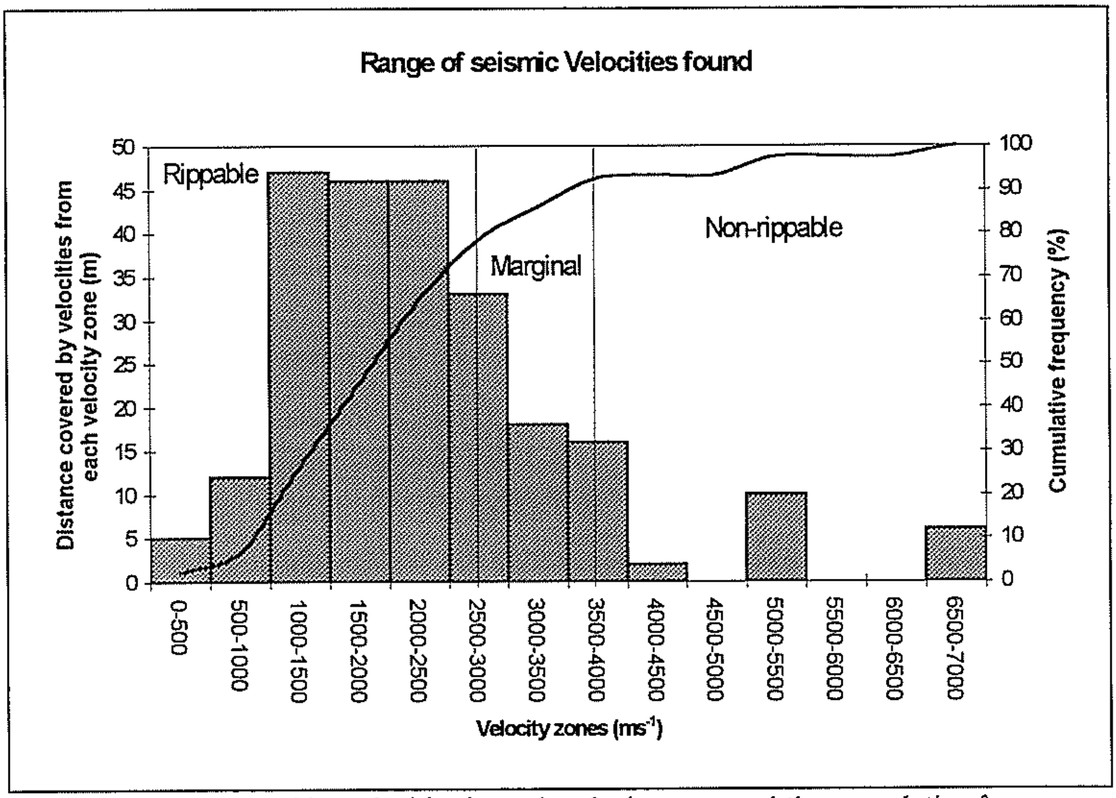



Figure 3: Distances covered by velocities in each velocity zone and the cumulative frequency. The rippable, marginal and non-rippable zones for shales, sandstones, siltstones and claystones - rock types likely to be found in the open pit area - for Komatsu's D575A-2 bulldozer are also marked (Data from John, 1994).

## ROCK MASS CHARACTERISATION

Rock mass characteristics from road outcrops need to be correlated with rock mass characteristics from drill core, so that a three-dimensional geotechnical model of the open pit area can be developed. This will be done by comparing RMR values and strength determinations from road outcrops and drill core. Rock mass velocities range between $750ms^{-1}$ and $6650ms^{-1}$, with a mean of $2300ms^{-1}$, and the velocities from rock material range between $3300ms^{-1}$ and $4700ms^{-1}$, with a mean of $4150ms^{-1}$. The rock mass velocities are for massive sandstone units or for alternating sandstone and siltstone beds, while the rock material velocities are all from sandstone drill core.

Rock mass velocities are normally less than rock material velocities because of the presence of defects that may be infilled with water, air, soil or alteration products, and which reduce the average velocity of the rock mass.

Irregular lump point load testing has been performed on samples from road outcrops, and porosity and density characteristics have also been determined on selected sandstone core samples. Point load strength indices depend on the weathering of the sample and vary considerably. Unweathered rock samples yielded a mean $I_{s(50)}$ value of 5.06 MPa from 59 samples whereas slightly weathered rock samples had a mean $I_{s(50)}$ value of 2.68 MPa from 21 samples. Moderately and highly weathered samples were not tested as their strength would be expected to be minimal. Also determined from limited drill core samples are the mean dry density (2695$kgm^{-3}$), the mean saturated density (2710$kgm^{-3}$), and the mean porosity (1.30%). The density values are relatively high for sandstones, and the porosity is relatively low because the grain size is medium to fine and the rocks have a metamorphic overprint that has compacted the rock, resulting in low void (pore) volume. The implications of this are that the rock from within parts of the open pit area may be very hard to rip and may even require blasting. Therefore, more than 7.5% of the rock mass may be non-rippable.

## CONCLUSIONS

- Seismic velocities can be used to estimate rippability because the seismic velocity of a rock is dependant on geological factors such as density, porosity, cementation and weathering.
- A preliminary rippability evaluation, combining seismic refraction data and data from Komatsu, indicates that it may be feasible to rip 80-85% of the rock mass from the open pit and a further 7.5% should be marginal to rip.
- A complete rippability assessment needs to use geological factors such as rock strength, hardness, weathering, density, defect spacing, defect persistence and defect orientation, which may be correlated with the productivity of the bulldozer to be used. Once this data is obtained, a complete rippability evaluation will be done and may alter the preliminary estimation presented here.

## REFERENCES


Barry JM, 1993. *The History and Mineral Resources of the Reefton Goldfield,* Resource Information Report 15, Energy and Resources Division, Ministry of Commerce, 59 pages.

Beetham RD; Richards L, 1995, *Globe-Progress Mining Project. Assessment of open pit geometry,* Unpublished Report for Macraes Mining Company Limited, Reefton, 26 pages.

Bieniawski ZT, 1973, Engineering classification of jointed rock masses, *Transactions of the South African Institute of Civil Engineers,* volume 15, pages 335-344.

Braybrooke JC,1988, The state of the art of rock cuttability and rippability prediction, *Proceedings of the Fifth Australia New Zealand Conference on Geomechanics,* pages 13-42.

John I, 1994, *Site visit report, Globe Progress Development near Reefton, New Zealand,* Unpublished report for Macraes Mining Company Limited by Komatsu, 13 pages.

MacGregor F; Fell R; Mostyn GR; Hocking G; McNally G, 1994, The estimation of rock rippability, *Quarterly Journal of Engineering Geology,* volume 27, pages 123-144.

Minty EJ; Kearns GK, 1983, Rock mass workability, *In* Knight MJ; Minty EJ; Smith RB (editors) *Collected Case Studies in Engineering Geology, Hydrogeology, Environmental Geology: Geological Society of Australia Special Publication,* volume 11, pages 59-81.

Palmer D, 1980, *The Generalised Reciprocal Method of Seismic Refraction Interpretation,* Society of Exploration Geophysicists, 104 pages.

Pettifer GS; Fookes PG, 1994, A revision of the graphical method for assessing the excavatability of rock, *Quarterly Journal of Engineering Geology,* volume 27, pages 145-164.

Weaver JM, 1975, Geological factors significant in the assessment of rippability, *The Civil Engineer in South Africa,* volume 17, pages 313-316.

**HARDSTAND LOAD TESTING FOR HEAVY LIFT CRANES**

S.J. CLARKE
Coffey Partners International Pty Ltd, Sydney, Australia

SUMMARY

Large capacity plate load tests were used to predict settlement of heavy lift cranes. The rear counterweight crawlers of the heavy lift cranes will have a mass of 2200 tonnes, and will apply a track bearing pressure of 490 kPa. The load testing program involved applying a 1000 tonne load over a 6 m long by 2.4 m wide area to result in a maximum applied bearing pressure of 680 kPa. Piezocone penetration testing and magnetic extensometers were used to assess deformation properties of the different soil units encountered at the site. This paper describes the field component of the load test program, the procedure for the assessment of soil deformation properties, and the predicted settlement behaviour for the heavy lift cranes.

INTRODUCTION

Two heavy lift cranes will be used to lift large prefabricated sections onto a large structure. Each of the heavy lift crane comprises a front crawler supporting a 140 m long boom, connected with a 35 m long strut to a rear counterweight crawler weighing 2200 tonnes.

Large-scale load testing was required to check the ground deformations which would occur in the hardstand area during operation of the heavy lift cranes. Preliminary analyses based on borehole data and conventional plate load tests indicated that settlements beneath the crane tracks of 250 mm to 290 mm may occur.

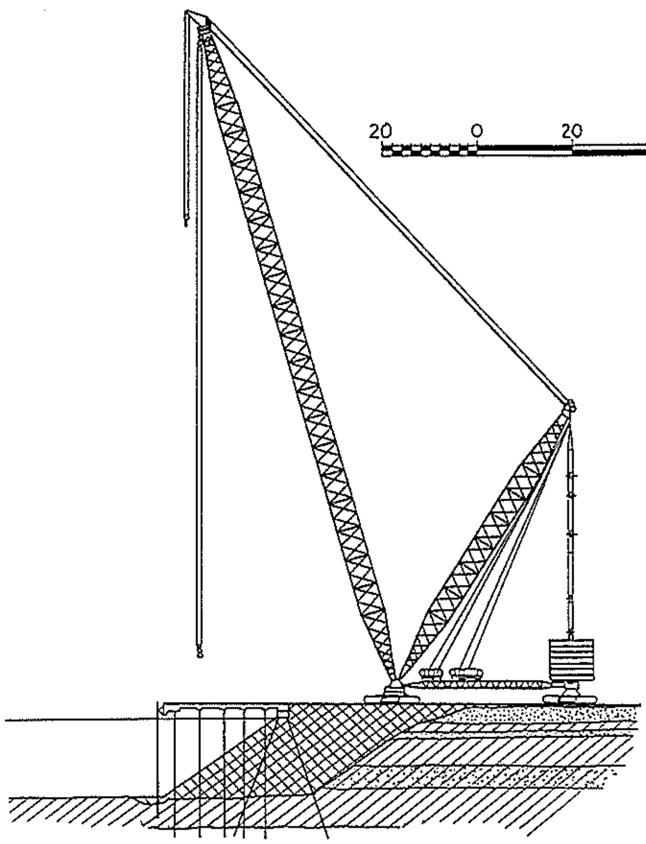



Figure 1 Heavy lift cranes on hardstand area.

This paper discusses the following aspects of the load test program:

- Additional site investigations performed to better define sub-surface conditions
- Installation of magnetic extensometers to measure vertical movement at various depths beneath the ground surface
- Load test methodology and results
- Assessment of soil deformation parameters
- Predicted settlement behaviour of the heavy lift crane

## SITE CONDITIONS

The hardstand area is approximately 90 m by 60 m in area, and situated about 60 m from the shoreline of Port Kembla Inner Harbour. Site investigations performed prior to 1995 comprised four wash-bored boreholes, with undisturbed tube samples taken of clay soils, Standard Penetration Tests (SPT) performed in sands, and 15 in-situ vane shear tests performed in the soft clay soil.

Generally less than a metre thickness of blast furnace slag gravel occured over the surface of the site. Underlying the slag fill was a 2.5 to 6 m thick layer of medium dense to dense sand and silty sand. Beneath the sand a very soft to soft high plasticity estuarine clay unit was found, which was up to three metres thick. Loose sand and another soft clay layer occured below this depth, though these were less important with respect to crane settlement behaviour than the soils nearer to the surface.

## LOAD TEST PROGRAM

The load test program involved additional geotechnical investigation of the hardstand area and the installation of instrumentation in preparation for the load tests. Three large scale load tests were subsequently performed at locations chosen based on the results of the additional investigation. Large scale tests were necessary in order to simulate as closely as possible the proposed crane loading, so that the same magnitude of stresses were developed in the soil profile, and non-linear stress-strain behaviour identified.

### Piezocone Penetration Tests

Four boreholes drilled previously had shown variability in the near surface soils. Piezocone penetration testing was performed at ten locations within the hardstand area to more accurately assess the soil unit boundaries and to better assess (compared with SPT results) the relative density of the near-surface sand soils. The use of the piezocone (rather than the more common electric friction cone) enabled greater confidence in soil identification, with pore pressure behaviour highlighting the occurrence of clay layers. The piezocone test results were also used as a basis for the correlation of elastic parameters with cone resistance.

The ten tests were performed to depths of up to 16 m from the back of a conventional truck-mounted drilling rig. The use of a drill rig enabled cemented slag gravel on the surface to be easily pre-drilled using conventional augers, and enabled quick testing and retrieval of the probe when testing the soft clay soils. The same drill rig was also used to install the magnetic extensometers.

The piezocone test results indicated that the soft clays which underly the site occured in two distinct layers, with a 1 to 3 m thick layer of silty sand and sand separating the two clay layers. Total cone resistance, $q_t$, was used to assess undrained shear strength of clays using the following relationship:

$$s_u = \frac{q_t - \sigma_v}{N_{kT}} \qquad (1)$$

where
$s_u$ = undrained shear strength
$q_t$ = total cone resistance
$\sigma_v$ = total vertical overburden stress
$N_{kT}$ = piezocone factor.

The parameter $N_{kT}$ was found to range from 15 to 20, based on correlating it with the field vane test results. This range of $N_{kT}$ is typical of high plasticity clays.

Of the 15 vane shear tests performed in the soft clays, all but one was performed below 6 m depth, and indicated shear strengths in the range 32 to 66 kPa. Using Eq. (1) it was found that the uppermost clay layer had shear strengths as low as 15 kPa, which was significantly less than that of the deeper clay unit, though of similar magnitude to the 19 kPa result from the one vane shear test which was performed above 6 m depth. The vane shear tests had not clearly identified that the shallow clay layer was considerably softer than the deeper clay unit.

The locations of the proposed load tests were based on the results of the piezocone tests, with load tests located in the areas considered to present the most severe conditions for settlement and bearing capacity of the heavy lift crane.

Magnetic Extensometers

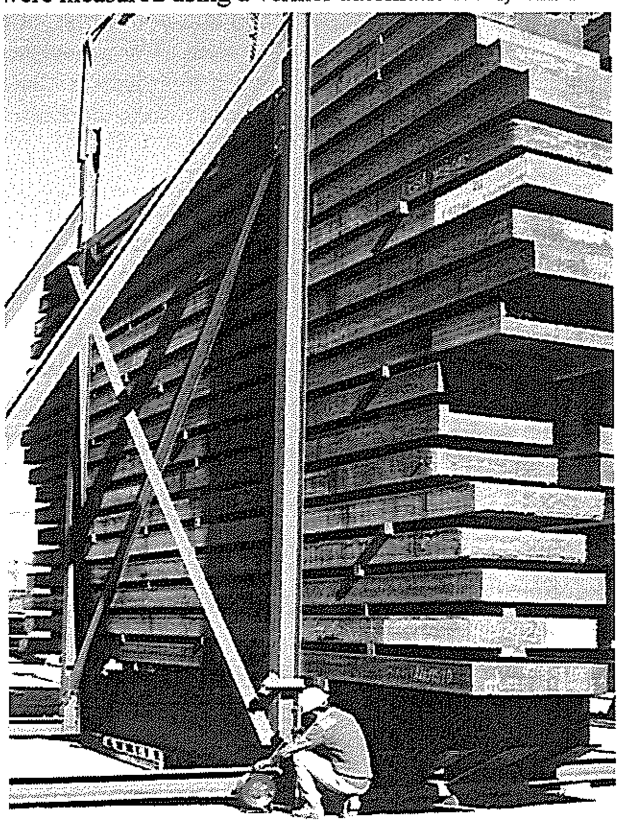
Magnetic extensometers were installed at the location of two of the three load tests. The extensometers allow for the measurement of vertical movements within the soil, thus allowing identification of the soil layers which contribute to the observed surface settlement.

Three extensometers were placed around the perimeter of the proposed location of each of the two load tests. Extensometers comprise magnetic targets within a flexible vertical tube. A magnetic sensor is lowered into the tube, and a reed switch is triggered when the sensor approaches a target, emitting a sound at the ground surface. It was possible to measure the location of magnetic targets to about 1 mm vertical accuracy. Extensometers were installed to depths of about 12 m, with four magnetic targets installed in each extensometer. The spacing of the targets was increased with depth.

Extensometers were installed about one month prior to the load tests being carried out to allow for consolidation of soil around the extensometer, so that the extensometer tubing would deform with the surrounding soil.

Load Tests

The load tests comprised the loading of a purpose-built steel frame with up to 44 steel ingots, each with a mass of 20 to 25 tonnes. The loading frame comprised four large parallel steel beams connected with smaller sections, resulting in a 2.4 m wide platform with a 6 m length in contact with the ground. The steel ingots were placed in layers, with each layer comprising two ingots. A maximum of 22 layers was placed in the frame, corresponding to a load of 995 tonnes, and an average ground bearing pressure of 680 kPa. Surface settlements were measured using a vernier automatic level, which was capable of measuring to an accuracy of 0.1 mm.



The first load test resulted in the least settlement of the three tests, with less than 50 mm observed at the surface. About 1.3 m of compacted slag fill occurred at the first test location, and surface cracks were observed to extend up to 15 m from two corners of the load frame, indicating that the slag fill was capable of transmitting the load over a considerable area. Extensometer data indicated that half of the settlement occurred below 2 m depth, and that the permanent deformation of 30 mm was attributable to soil units above 6 m depth.

The second test was performed at a location half way between the two test locations where extensometers had been installed. Settlement of 100 mm occurred immediately after loading the frame to 680 kPa, with an additional 70 to 100 mm occurring overnight. Permanent deformation of 160 mm was measured. The slag thickness was 0.6 to 0.9 m, significantly less than for the first test, resulting in reduced capacity of the slag to spread the load. As a result surface cracking was over a much smaller area than for the first test.

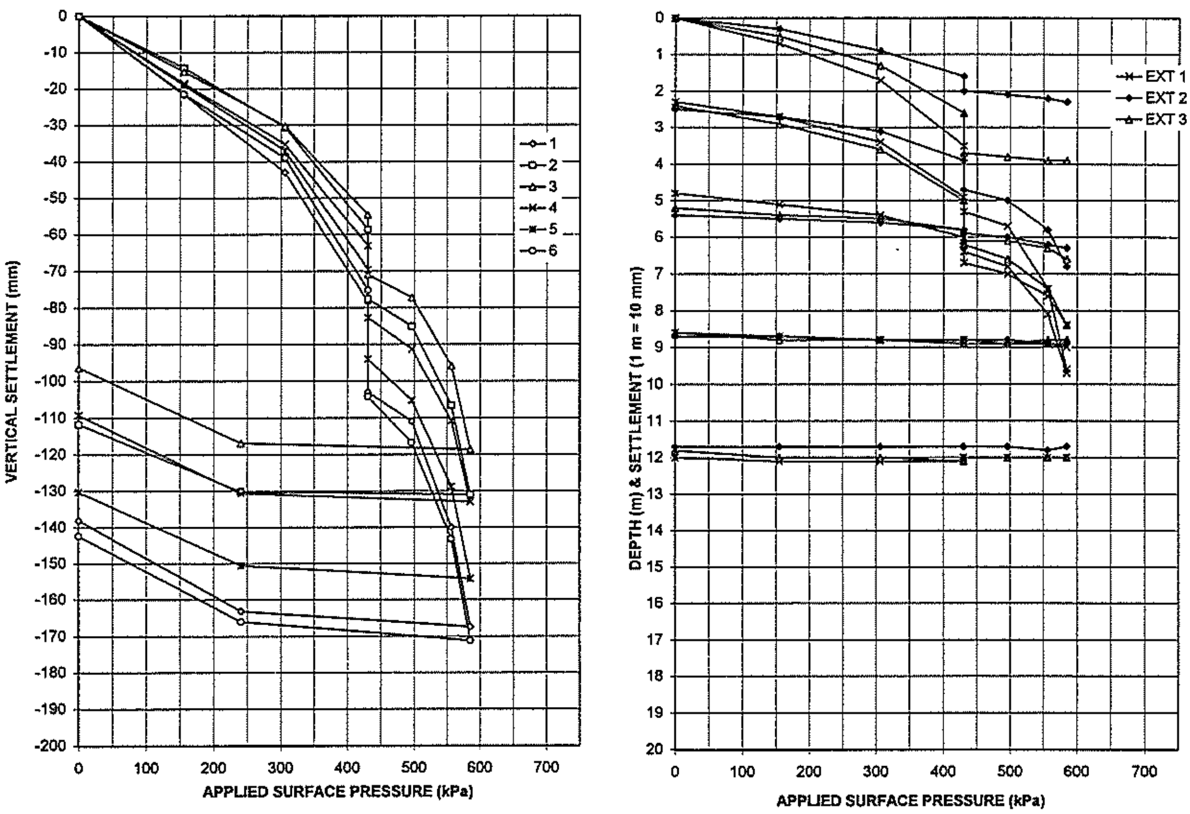



Figure 2 Loading frame settlement and magnetic extensometer settlement for the final load test.

The final test was loaded to only 585 kPa, and resulted in maximum surface settlements of about 150 mm. Extensometer targets indicated that only 60 mm of the total 150 mm settlement was attributable to soil deformation below 2 m depth.

## ASSESSMENT OF DEFORMATION PARAMETERS

The methodology adopted for the settlement analyses resulted in a set of site-specific parameters which could be applied to other locations within the site where load testing has not been performed. In particular, the piezocone parameters total cone resistance ($q_t$) and undrained shear strength ($s_u$) were used to assess undrained and drained elastic moduli for sands and clays, such that a piezocone log could be used to assess soil deformation properties at other locations around the site.

### Methodology

Elastic moduli relationships were assessed based on extensometer and piezocone data, and involved analysis of the load test data. An iterative procedure was adopted for each test which gradually fitted equivalent elastic parameters to the soil layers. The methodology adopted was proposed by Poulos (personal communication), and involved the iterative use of strain influence factors, with actual measured strains used to correct calculated strains. Initially an elastic analysis was performed for a uniform material underlying the applied surface loading, and the vertical strain distribution assessed. Influence factors for each soil layer (whose depth and thickness was defined by the actual location of the extensometer targets) were calculated as follows:

$$I_{e(i)} = \frac{E_{(i)} \cdot \varepsilon_{ave(i)}}{\rho_{surface}} \tag{2}$$

where
$I_{e(i)}$ = strain influence factor for the ith iteration
$E_{(i)}$ = elastic modulus used in the calculation of $\varepsilon_{ave(i)}$ in the ith iteration
$\varepsilon_{ave(i)}$ = average strain of the layer for the ith iteration
$\rho_{surface}$ = total surface settlement

For the next iteration the elastic modulus was calculated as:

$$E_{(i+1)} = \frac{\rho_{surface} \cdot I_{e(i)}}{\varepsilon_{real}} \qquad (3)$$

where $E_{(i+1)}$ = elastic modulus used for the next iteration
$I_{e(i)}$ = strain influence factor calculated for the ith iteration
$\varepsilon_{real}$ = real strain of the layer (based on extensometer results)

The iteration procedure was repeated until the calculated values of E converged for each layer. This usually occured after only a few iterations.

### Deformation Parameters

Once the equivalent soil moduli were assessed for each layer, simple relationships with piezocone data were established. Typical published relationships between elastic modulus, E, and $q_t$, for sandy soils indicate a range from E = (1.5 to 5) $q_t$. For clays, relationships between undrained elastic modulus, $E_u$, and $s_u$ range from $E_u$ = (300 to 1500) $s_u$.

The load tests indicated that deformation of the sand units had a time-dependent component, possibly due to dissipation of excess pore pressures within thin silty or clayey lenses in the sand unit. Adopted settlement parameters for the sand soils gave an initial "undrained" secant modulus of $E_u = 2.3\ q_t$, with a (24 hour) drained modulus of $E' = 1.4\ q_t$. The 24 hour "drained" modulus presented is not necessarily the same as the normal drained modulus, since it was unclear whether full dissipation of excess pore pressures had occurred. However, this approach was considered to be appropriate for crane loadings of up to one day duration.

Time-dependent deformation was not observed for the clay soils during the 24 hour duration of the load tests, probably due to the very low permeability of the clay units. Only the undrained secant modulus was assigned; $E_u = 900\ s_u$.

For the surface slag fill, the secant modulus was found to vary, depending on the thickness of the slag fill at the particular test location. This was thought to be as a result of localised overstressing and failure of the slag particles, and also the breaking of the cementation which develops in compacted blast furnace slag over time. Thinner slag layers were also subjected to much greater tensile stresses at the base of the layer than thick layers, resulting in earlier loss of any tensile strength which the slag may have developed. The slag moduli was assessed to range from 15 MPa for 0.6 m thick slag to 85 MPa for 1.3 m thick slag. The value of 15 MPa was considerably less than would normally be expected, and was likely to be due to the non-linear behaviour of the material combined with the high stresses the material had been subjected to in the load tests.

### Layer Equivalencies

When using continuous penetrometer data for assessment of elastic soil parameters it was important to use the harmonic mean, rather than a simple average, since deformation is inversely proportional to the elastic modulus. Therefore thin compressible layers had a much greater effect in reducing the equivalent modulus of a soil profile compared to the effect of thin incompressible layers increasing the equivalent soil modulus. The equivalent elastic modulus was calculated as:

$$E_{equiv} = \frac{t}{\sum_{i=1}^{n} (t_i/E_i)} \qquad (4)$$

where $E_{equiv}$ = equivalent elastic modulus for the soil layer whose thickness is t
t = soil layer thickness
$t_i$ = thickness of soil sub-layer i
$E_i$ = elastic modulus of soil sub-layer i
n = number of soil sub-layers within the soil layer

Where soil sub-layer thickness is uniform, such as when using digital data from electric penetrometer tests, the harmonic mean was simply calculated using a spreadsheet program, and Eq. (4) can be simplified to:

$$E_{equiv} = \frac{n}{\sum_{i=1}^{n} (1/E_i)} \qquad (5)$$

## PREDICTED HEAVY LIFT CRANE SETTLEMENT BEHAVIOUR

Based on the back-analysed parameters presented in the previous section, the initial settlement of the rear crawler of the heavy lift crane was calculated to be between 85 and 100 mm, with the settlement over one day expected to increase to between 110 and 150 mm.

Bearing capacity was addressed using the methodology presented by Meyerhof [6]. Effective friction angle of the upper sand soils was assessed based on a relationship presented by Lunne and Christoffersen [5] which uses the piezocone parameter $q_t$. Back-analysis of the load tests indicated that an effective friction of at least 33° was appropriate, since bearing capacity failure had not occurred. A design effective friction angle of 35° was adopted.

Bearing capacity analyses indicated that Factors of Safety of less than 1.0 would occur for the heavy lift crane where the very soft clay layer occurred at only three metres depth. Various remedial foundation treatments were designed, based on the soil parameters derived in the load test program.

## CONCLUSIONS

Large scale plate load tests were used to assess soil deformation properties when subject to high vertical stresses. Magnetic extensometers provided a useful tool in assessing deformation properties of different types of soil, and, when combined with piezocone test data, allowed site-specific parameters to be established, such that a piezocone log could be used to assess soil deformation properties at other locations around the site.


## ACKNOWLEDGMENTS

The author gratefully acknowledges the technical contributions and support of other Coffey personnel involved with the project, in particular the help of Proffesor Harry Poulos.


## REFERENCES


1. Coffey Partners International Pty Ltd 1992. *Geotechnical Investigations - Phase 2.* Report No. S9710/2-BU. Sydney.
2. Coffey Partners International Pty Ltd 1995. *Outfitting Berth Crane Runways, Foundation Design.* Report No. S9710/12-AG. Sydney.
3. Coffey Partners International Pty Ltd 1995. *Outfitting Berth Hardstand, Load Testing - Phase 1.* Report No. S10352/1-AB. Sydney.
4. Coffey Partners International Pty Ltd 1995. *Outfitting Berth Hardstand, Load Testing - Phase 2.* Report No. S10352/1-AE. Sydney.
5. Lunne, T and Christoffersen, H P 1983. Interpretation of cone penetrometer data for offshore sands, *Offshore Technology Conference.*
6. Meyerhof, G G 1974. Ultimate bearing capacity of footings on sand overlying clay, *Canadian Geotechnical Journal* 11(2): 223-229.

# WAIKAREMOANA POWER SCHEME - PIRIPAUA PENSTOCK ENGINEERING GEOLOGY STUDY AND RISK ASSESSMENT

R.M. DAWSON

## SUMMARY

ECNZ wished to have an engineering geology and risk assessment study undertaken of the Piripaua penstock slope, part of the Waikaremoana Power Scheme. This paper presents the method used for geological and statistical analysis and the conclusions made. Aerial photograph interpretation as well as field mapping of the geology was carried out to determine the likely failure mechanisms. Three dimensional analysis of wedge failures was performed with variation of input parameters such as strength and earthquake magnitude to allow probabilities of failure to be evaluated.

## INTRODUCTION

Piripaua Power Station is the third in a line of three stations on the Waikaretaheke River, draining Lake Waikaremoana. The lake is 610 metres above sea level and the valley below it falls over 448 metres within 8 km, making it ideal for hydro-electric purposes.

The scheme uses Waikaremoana's water by firstly carrying it through a tunnel then two pipelines to Kaitawa power station into Lake Kaitawa. The water then passes through another tunnel to a surge chamber, and down penstocks to Tuai station and is discharged into Lake Wakamarino. A further tunnel leads to a surge chamber and then through penstocks to Piripaua station. The water finally discharges into the Waikaretaheke River.

The combined power output of the three stations is 132 MW.

## STUDY REQUIREMENTS

ECNZ Tokaanu Hydro Group wished to have a study undertaken of the Piripaua penstock area. The brief was to undertake and complete a study of the engineering geology of the Piripaua surge chamber, penstocks and powerhouse area to investigate slope stability hazards and assess possible failure scenarios and their impacts upon structures. The study was to consider the risk probabilities of the possible failure scenarios and to identify engineering options that would mitigate the impacts of the failures.

## ENGINEERING GEOLOGY

### Regional Geology

The regional geology of the area is well understood considering the studies that have been undertaken on the massive landslide that formed Lake Waikaremoana approximately 2000 years ago, and the construction activities which have taken place with the Waikaremoana Power Scheme. The Huiarau and Ikawhenua Ranges, west and northwest of the lake respectively are formed from axial-range greywacke which is part of the mountain chain that extends from eastern Bay of Plenty, southeast to Cook Strait. The lake is surrounded by younger sedimentary rocks of Tertiary age.

The rock in the immediate area of Piripaua is Tertiary aged sedimentary consisting of interbedded sandstone, siltstone and mudstone. The sedimentary sequence has been uplifted and tilted gently at dips up to 20° to the southeast.

## Aerial Photograph Interpretation

Vertical aerial photographs from 1952 and 1965 were available from NZ Aerial Mapping for the area. In addition, a set of nine aerial photographs were borrowed from ECNZ in Tokaanu, taken during construction of the Piripaua power scheme (1942).

Aerial photograph interpretation suggested that two historic landslips were present in the immediate area of the Piripaua penstocks and powerhouse, as shown on Figure 1. The construction photographs showed that one of these slips was linked to construction of the tunnel leading to the surge tank, as the photo shows that the tunnel spoil was dumped on the north facing side of the ridge leading to the west from the surge chamber. Some of this spoil appears to be pushed over the ridge line and has slid down the slope towards the powerhouse.

## Field Mapping

Detailed field mapping of the area surrounding the penstock slope and powerhouse was carried out. Two days were spent walking over the penstock slope and surrounding countryside to develop a localised geological model. Rock exposures were logged in the hills behind the penstocks, in the Piripaua Stream bed and cuttings for SH38. A large amphitheatre was also inspected from the left bank access road leading from Piripaua to Tuai.

## Rock Mass Description

At the surge tank the rock mass consists of dark grey soft weak slightly to moderately weathered thinly interbedded mudstone and siltstone with minor interbedded sandstone. This lithology extends downstream on a farm track at or above the surge tank level. Bedding, dipping 22°, N110° is the most well developed defect in this lithotype with defect sets orthogonal to bedding prominent. Upstream of the surge tank unweathered light grey to grey fine, weakly commented, moderately strong sandstone appears to be the dominant lithology outcropping on the surge tank access road and on SH38 below the old hydro village. The more erosion resistant sandstone may form the prominent ridge to the north of the site.

Defect surveys were carried out at the surge tank and at nearby road exposures. The defect orientations are summarised on Table 1 and Figure 2. The main defect set is bedding (set F) with the orthogonal defect sets C, D and E.

A slickensided bedding plane shear surface was observed on the amphitheatre to the northwest of Piripaua. The defect was filled with 5 to 10 mm of gouge and indicative of movement in the past. The amphitheatre may have been formed by a wedge failing on the bedding plane shear surface.

Table 1. Rock mass strength estimation

| SET | DIP | DIRECTION | CONTINUITY | WAVINESS | SEPARATION | SPACING |
|---|---|---|---|---|---|---|
| A | 70° | 350° | 2+ m | planar | <0.1 mm | 0.3-1.0 m |
| B | 70° | 100° | 5+ m | planar | <0.1 mm | 1.0-2.0 m |
| C | 70° | 270° | 2+ m | planar | <0.1 mm | 0.3-1.0 m |
| D | 70° | 240° | 2+ m | planar | <0.1 mm | 0.3-1.0 m |
| E | 75° | 210° | 2+ m | planar | <0.1 mm | 0.3-1.0 m |
| F | 75° | 310° | 20+ m | planar | <0.1 mm | 0.05-0.2 m |

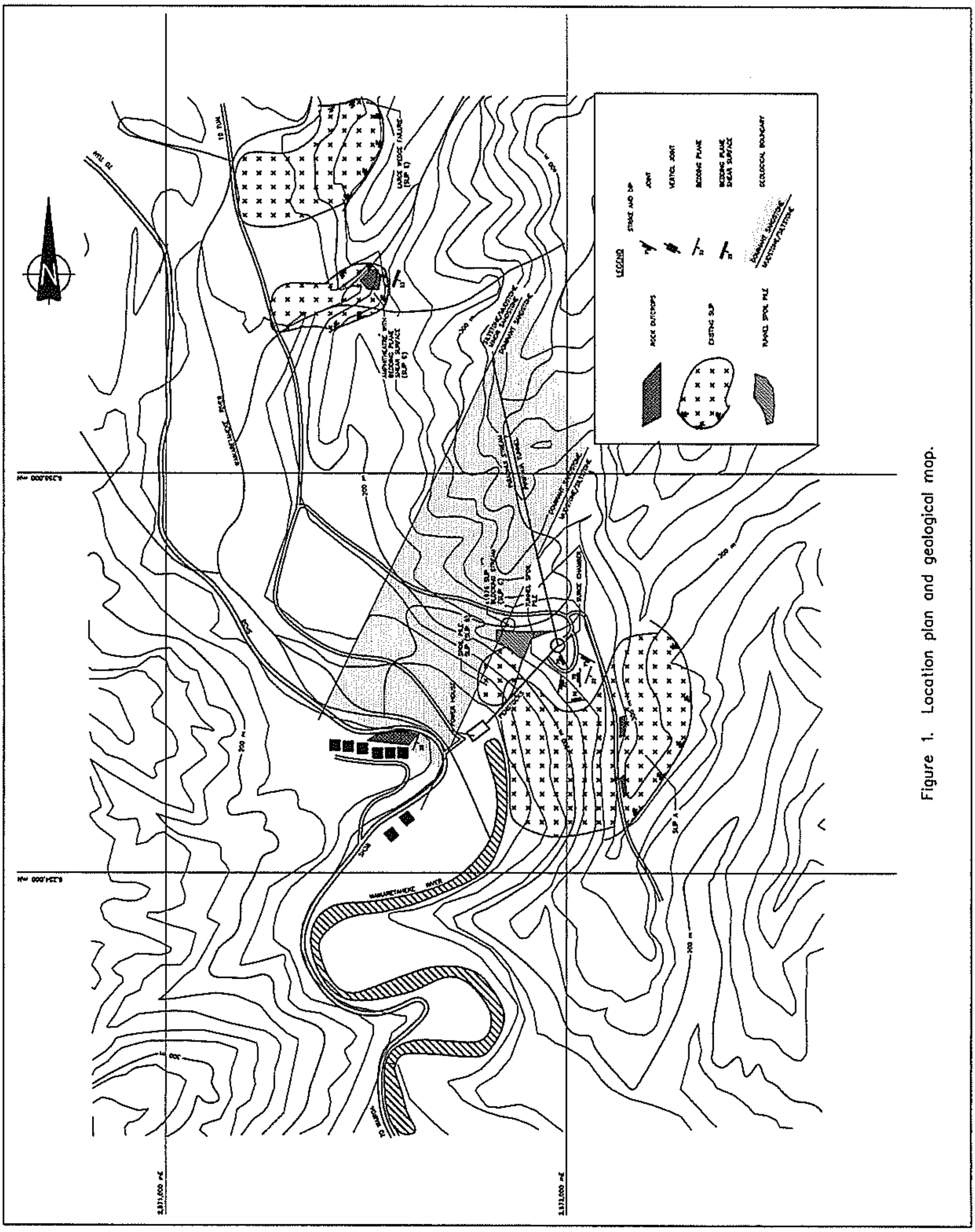


Figure 1. Location plan and geological map.

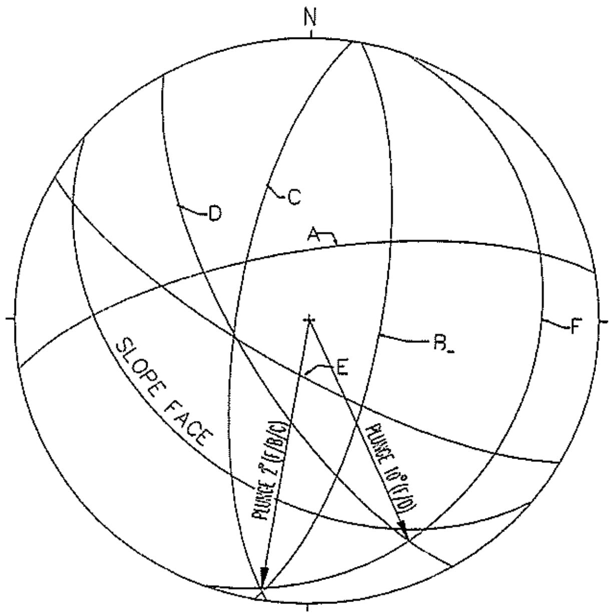



Figure 2. Design stereoplot data.

### Rock Defect Strength Parameters

For use in the risk analysis, the 5th and 95th percentile limits for a non-linear Mohr Coulomb envelope were estimated, based on the method presented by E Hoek et al [1]. Table 2 presents the values selected using Hoek's method.

**Table 2. Rock mass strength estimation.**

| FACTOR | 5% VALUE | 5% SCORE | 95% VALUE | 95% SCORE |
|---|---|---|---|---|
| Intact Rock Material Strength | 3 MPa | 1 | 10 MPa | 2 |
| Drill Core RQD | 50% | 8 | 75% | 17 |
| Joint Spacing | 0.3 m | 15 | 1 m | 25 |
| Joint Condition | | 12 | | 25 |
| Ground Water | | 7 | | 7 |
| TOTAL | | 43 | | 76 |
| GSI | | 43 | | 76 |
| CONDITION | | VERY BLOCKY | | BLOCKY |
| $m_b/m_i$ | | 0.16 | | 0.40 |
| S | | 0.003 | | 0.062 |
| a | | 0.50 | | 0.50 |
| Em | | 9000 | | 40 000 |
| $\vartheta$ | | 0.25 | | 0.20 |
| GSI | | 48 | | 75 |
| $m_i$ | | 6 | | 19 |

The shear strength versus effective normal stress curve was plotted for the 5th and 95th percentiles, and power curves fitted to them. The mean and standard deviations of $c'$ and $\phi'$ were developed and are presented in equations 1 to 4 below.

$$\varnothing'_{MEAN} = \frac{ATAN(8.480.\sigma_n^{-0.2821}) + ATAN(3.789.\sigma_n^{-0.3220})}{2} \quad ...1$$

$$\varnothing'_{STD} = \frac{ATAN(8.480.\sigma_n^{-0.2821}) - ATAN(3.789.\sigma_n^{-0.3220})}{4} \quad ....2$$

$$c'_{MEAN} = \frac{3.332.\sigma_n^{0.7179} + 1.799.\sigma_n^{0.6780}}{2} \quad ....3$$

$$c'_{STD} = \frac{3.332.\sigma_n^{0.7179} - 1.799.\sigma_n^{0.6780}}{4} \quad ....4$$

The above curves are presented in Figure 3.

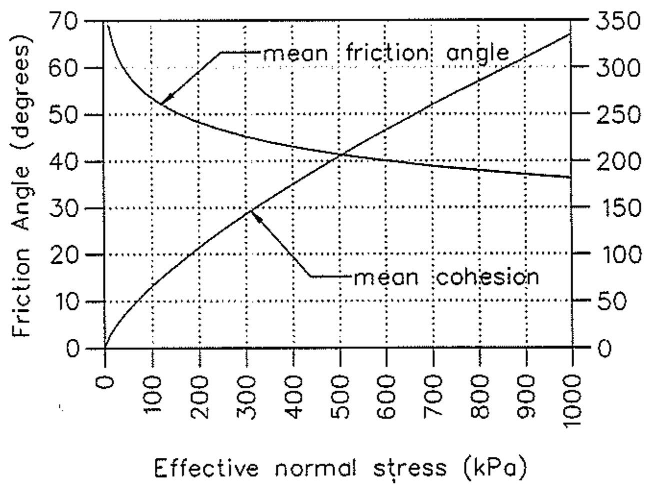



Figure 3. Mean Strength Parameters.

### Likely Failure Mechanisms

Based on interpretation of the defect data, kinematically the most likely failure mechanism involves wedges formed by defect sets: F & C, F & B, F & D and C & D with rear release joints A or E. Large wedge failures have the most significant consequence of failure in that they would be likely to cause considerable damage to the penstocks and possibly to the power house, surge chamber and feed tunnel. Evidence of large wedge failures are reasonably common in the area, with the landslide which formed Lake Waikaremoana being an example.

Large wedge failures are likely to be formed by bedding plane defects in combination with two or more other joint sets. Ground water pressures reduce the frictional component of strength. The large wedges are likely to be stable under static conditions, aided by discontinuous joints, and require a trigger such as a very large seismic event to cause failure. The energy involved in such an event would cause massive damage to the affected area.

## RISK ASSESSMENT

### Methodology

Various failure mechanisms were investigated and the critical mechanism was established as a three dimensional wedge failure. To allow evaluation of the wedge failure mode a spreadsheet was developed to calculate the factor of safety of a wedge based on the method developed by Hoek & Bray [2] using various plane geometries, material parameters and groundwater conditions. All of the combinations of defect orientations were evaluated to find the critical combination.

Risk modelling was carried out on the critical failure geometry using the risk analysis and modelling software "@RISK" in conjunction with the "LOTUS 123" spreadsheet. The upper and lower non-linear Mohr Coulomb envelopes previously developed were adopted as the 5th and 95th percentile values of a normal probability distribution. The mean value (relative to effective normal stress) for cohesion and friction angle was taken as the median of the upper and lower strength envelope, while the standard deviation was taken as one quarter of the range of upper and lower values. A power function was fitted to the resulting curves and non-linearity with respect to the normal effective stress was incorporated into the spreadsheet.

The second parameter with a possible variation was the horizontal seismic acceleration. A cumulative distribution function was evaluated based on the probability of a certain level of earthquake shaking occurring within one year. The probability distribution was programmed into the spreadsheet. The resulting model allowed normal variation of cohesion and friction angle, with the mean and standard deviation dependent upon the calculated effective normal stress, and variation of earthquake magnitude. The program "@RISK" was then used to run 1000 simulations of the model, randomly choosing variable values from their appropriate probability distribution. The factor of safety was calculated for each simulation and recorded for statistical analysis. Using this method, the probability of failure for various size wedges could be evaluated.

## Results of Risk Analysis

The critical failure mode was found to be a three dimensional wedge formed by the defect sets F and C with a tension crack formed by defect set A. Failures essentially wholly within the 40° penstock slope were found to be most likely. The height and hence dimensions of the wedge were varied with mean factor of safety calculated under static conditions presented in Table 3 below.

**Table 3. Mean static factor of safety for critical wedge geometry.**

| Wedge Height (m) | Factor of Safety |
|---|---|
| 5.0 | 8.07 |
| 10.0 | 6.60 |
| 15.0 | 5.88 |
| 20.0 | 5.42 |
| 30.0 | 4.84 |
| 40.0 | 4.46 |
| 50.0 | 4.20 |

As shown in Table 3 wedge failures become more critical with depth. Failures were limited to 50 m height by the height of the penstock slope.

Using "@RISK" in conjunction with "LOTUS 123" a full probability analysis was carried out allowing for variability in cohesion, friction and PGA. The results of this analysis are summarised in Table 4.

**Table 4. Seismic probabilistic analysis of 3D wedge stability.**

| Wedge Height (m) | Probability of Failure |
|---|---|
| 5.0 | 0.17 |
| 10.0 | 0.22 |
| 15.0 | 0.20 |
| 20.0 | 0.40 |
| 30.0 | 0.40 |
| 40.0 | 0.30 |
| 50.0 | 0.47 |

To further highlight the effect seismic acceleration has on the probability of failure (failure is defined as a factor of safety less than 1.0) a relationship between PGA and probability of failure was developed. The results are presented in Table 5 and plotted in Figure 4 for large volume slips (i.e. full wedge height).

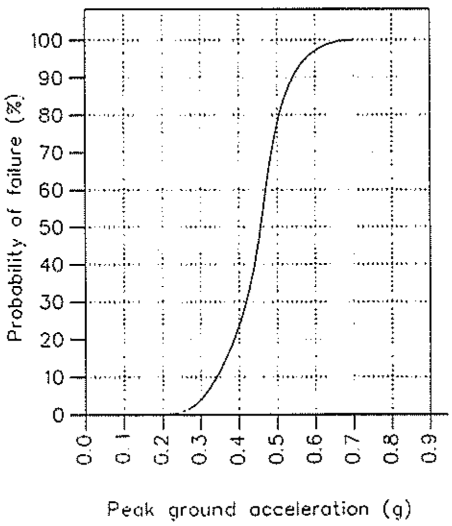



Figure 4 Effect of seismic acceleration.

Table 5. Relationship between PGA and probability of failure for a large volume slip.

| PGA (g) | P(FOS<1.0) (%) |
|---|---|
| 0.0 | 0.0 |
| 0.1 | 0.0 |
| 0.2 | 0.0 |
| 0.3 | 0.6 |
| 0.4 | 21.4 |
| 0.45 | 41.5 |
| 0.5 | 89.6 |
| 0.6 | 99.6 |
| 0.7 | 100.0 |

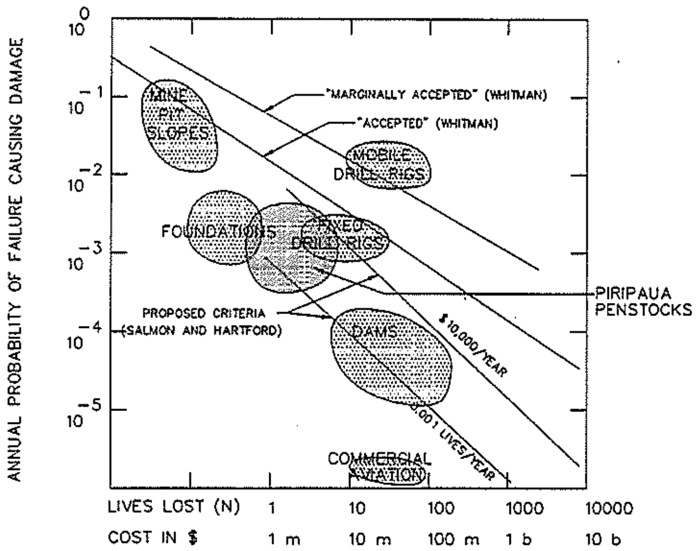



Figure 5. Risks for selected engineering projects.
(Whitman, 1984 and Salmon and Hartford, 1995)

### Comparison With Acceptable Risks and Mitigation Measures

The risk of a large wedge failure with current assumed ground water conditions (full saturation) is approximately 0.5% for any one year. Figure 5 shows some examples of normally acceptable risks for selected engineering projects as suggested by Salmon and Hartford. As shown by the area shaded for the Piripaua project, the present risk is generally within the lower range of commonly accepted risk criteria.

The critical failure analysed is a large wedge failure that is limited by the height of the slope. Due to the volume of rock involved conventional methods of reinforcement such as rock bolting or anchoring are not feasible. The only feasible option for reduction in the degree of risk is drainage of the penstock slope.

The degree of risk was calculated on the basis of fully saturated conditions, which is consistent with field observations. If the level of risk was considered to be unacceptably high, the first course of action would be to verify the groundwater regime by monitoring groundwater levels in the penstock slope. Analyses have been carried out for a range of groundwater conditions expressed as $r_u$ ($r_u$ is the ratio of water pressure head at the slip surface to the weight of soil/rock above: $r_u$ of 0.42 is full saturation). The results are summarised in Table 6 and show that a significant improvement in risk level can be achieved by small reductions in $r_u$.

**Table 6. Variation in risk with groundwater conditions.**

| $r_u$ | Probability of Failure (%) |
|---|---|
| 0.0 | <0.005 |
| 0.04 | <0.005 |
| 0.08 | <0.005 |
| 0.13 | <0.005 |
| 0.17 | 0.02 |
| 0.21 | 0.05 |
| 0.25 | 0.08 |
| 0.29 | 0.14 |
| 0.33 | 0.25 |
| 0.38 | 0.30 |
| 0.42 | 0.47 (present condition) |
| 0.63 | 2.86 (confined aquifer) |

If groundwater levels proved to be high after monitoring, two main options would be available for drainage of the slope. As a relatively inexpensive option, horizontal bored drains could be installed in the slope. A drainage adit would be more effective but would involve considerably more cost.

## CONCLUSIONS

The main conclusions from the study were:

- The toe of a landslip on the south eastern side of the penstocks has been eroded by the Waikaretaheke River (Slip A). The slip was likely to have been caused through undercutting and over-steepening of the rock mass by the Waikaretaheke River resulting in a defect controlled release surface. The slope now appears to be in equilibrium with slip debris at the toe of the slope and providing a buttress which will reduce the likelihood of further failures.

- The major source of risk to the scheme consists of large wedge failures triggered by seismic events. With the current assumed high groundwater conditions, there is an approximate risk of failure within any one year of 0.5%. This is within the lower range of commonly accepted risk/cost relationships. If the risk was considered to be unacceptable the groundwater assumptions should be verified by installation of piezometers. If saturation is proved by monitoring, significant improvements in risk level could be achieved by slope drainage.

## ACKNOWLEDGEMENTS

I would like to acknowledge ECNZ for their hospitality during our stay in the Waikaremoana area and their permission to use this material in this paper.

## REFERENCES

1. Hoek, E 1994. Strength of Rock and Rock Masses, *International Society for Rock Mechanics News Journal*: 4-16.

2. Hoek, E and Bray, J W 1977. *Rock Slope Engineering*. The Institution of Mining and Metallurgy.

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# LIQUEFACTION ANALYSIS FOR FOUNDATIONS OF MOTORWAY BRIDGES IN HAWKE'S BAY

DAVID DENNISON

Works Consultancy Services Limited
Private Bag 6019, NAPIER

## SUMMARY

The results of in-situ Standard Penetration Testing and Cone Penetration Testing were used to assess the soil liquefaction potential at the bridge sites for the proposed extension of the Napier - Hastings motorway. Published correlations between the in-situ tests and field performance of sites subjected to earthquake shaking were used to identify liquefiable soil layers and to estimate the intensity of ground shaking that is likely to cause liquefaction.

A seismic hazard study was compared with the results of the liquefaction analysis to produce a quantitative estimate of the risk of soil liquefaction. This data can be used in an economic assessment of the various options to mitigate the effects of soil liquefaction on the performance of bridge structures in earthquakes.

## INTRODUCTION

The first section of a two lane arterial motorway linking Napier and Hastings was opened in 1970, followed by extensions in 1973 and 1975. Investigation and design work is currently underway for additional extensions to the motorway that will require construction of up to five major bridge structures.

The motorway is located within the Heretaunga Plains in Hawke's Bay; a sediment filled depression formed by tectonic subsidence. The Hawke's Bay area is seismically active, and evidence of soil liquefaction, such as sand boils and lateral spreading, has been reported during past earthquakes.

Current design criteria do not explicitly state recommended methods of assessing the potential for soil liquefaction or the possible effects of liquefaction on bridge structures.

## CAUSES OF LIQUEFACTION

The upward propagation of shear waves through the ground in an earthquake induces repeated cycles of loading and unloading within the subsoils. These repeated cycles of stress often result in progressively increasing magnitudes of excess porewater pressure within sandy soils. If the porewater pressures build to a magnitude equal to the confining stress, the effective stress is reduced to zero, at which point the granular soil loses its strength and essentially flows like a liquid, hence the term liquefaction.

Even if the induced cyclic porewater pressure does not reach the confining stress, the reduction of effective stress within granular soils and consequent reduction of shear stress can result in significant strains, even in dense dilative sands. This phenomenon is known as cyclic mobility. Although the soil does not suffer a complete loss of strength, the deformations caused by the earthquake may be greater than the structure can tolerate.

In this paper the term liquefaction is used in its generic sense to include cases of complete loss of shear strength and instances of excessive deformation resulting from cyclic mobility.

Some potential effects of liquefaction are sand boils, decreased lateral soil stiffness, landslides, lateral spreading of embankments, settlement or tipping of shallow foundations, ground cracks and buoyancy of buried structures.

The main factors affecting the liquefaction potential of a soil deposit are:

- Intensity of ground shaking;
- Duration of ground shaking;
- Soil type;
- Initial confining pressure;

- Relative density or void ratio.

Soils must also be at or near saturation to experience the porewater pressure increase that leads to liquefaction.

Soils most susceptible to liquefaction are loose, uniform, fine grained sands. Liquefaction potential decreases as the density or the fines content increases. Soils with high plasticity are not likely to liquefy.

## SEISMIC HAZARD ANALYSIS

A probalistic seismic hazard evaluation was carried out by the Institute of Geological and Nuclear Sciences (IGNS) [2] for the motorway project area. Some points noted by IGNS in their report are:

- Hawkes Bay lies in one of the most seismically active regions of New Zealand
- At least 11 seismically active faults/folds are within 50 km of the bridge sites
- A magnitude 7.8 earthquake occurred in 1931 along the Napier-Hawke Bay fault. This is considered the Maximum Credible Event and has a return period of 5000 years.

Intensity of ground shaking is related to the level of cyclic shear stress induced by an earthquake. Induced shear stress in the ground is commonly estimated as a function of the peak horizontal ground acceleration (PGA). The return period versus peak ground acceleration relationship determined by IGNS for the project area is presented in Figure 1.

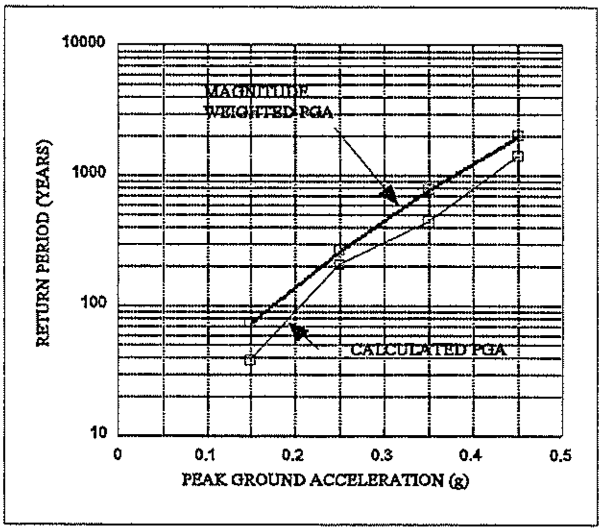



Figure 1: Seismic Hazard Analysis [2]

Other earthquake parameters, such as duration of shaking and frequency content, have significant influence on the potential for soil liquefaction. Generally, earthquakes of higher magnitude are more damaging for a given peak ground acceleration. Magnitude-weighting factors were applied to the peak ground accelerations to account for the difference in various magnitude earthquakes. The magnitude-weighting factor normalizes the PGA with respect to a magnitude of 7½. This magnitude was chosen because most of the field data regarding liquefaction of sands and silty sands is for magnitudes of approximately 7½. Figure 1 also shows the relationship between magnitude-weighted PGA and return period.

## ANALYSIS OF LIQUEFACTION POTENTIAL

### Grain Size Criteria

Many soils at the bridge sites are described as clay, clayey silt or clayey sand. Both laboratory tests and field performance data have shown that most clayey soils will not liquefy during earthquakes. According to criteria reported by Seed *et al.* [4], soils may be considered non-liquefiable if they meet any of the following conditions:

- Percent finer than 0.005 mm > 15%
- Liquid limit > 35%
- Water content < 0.9 x liquid limit

The limits in the particle size distribution curves proposed by Iai *et al.* [1] may be used as alternate criteria to decide if a soil has a gradation that is susceptible to liquefaction.

### In-Situ Tests

The adopted method of evaluating the liquefaction potential at the motorway sites is to compare the critical intensity of ground shaking which is likely to cause liquefaction to the expected intensity of ground shaking for various levels of risk. The critical intensity of ground shaking in the local soils is determined by comparing results of in-situ tests, such as the Cone Penetration Test (CPT) and Standard Penetration Test (SPT), with field observations of the performance of sites subjected to earthquakes in the past where similar tests were carried out.

Seed and De Alba [3] compared the SPT 'N' values at several sites that suffered liquefaction with those that did not liquefy during strong earthquakes. They used these data to derive relationships between the stress ratio (a function of the PGA) causing liquefaction and corrected 'N' values for sands and silty sands shown on Figure 2.

Compared with the SPT, the CPT produces a more continuous profile of subsurface conditions, is less susceptible to variations in test procedures and is quicker. The disadvantages are that no sample is retrieved and fewer data are available to compare the liquefaction resistance of soils to CPT results.

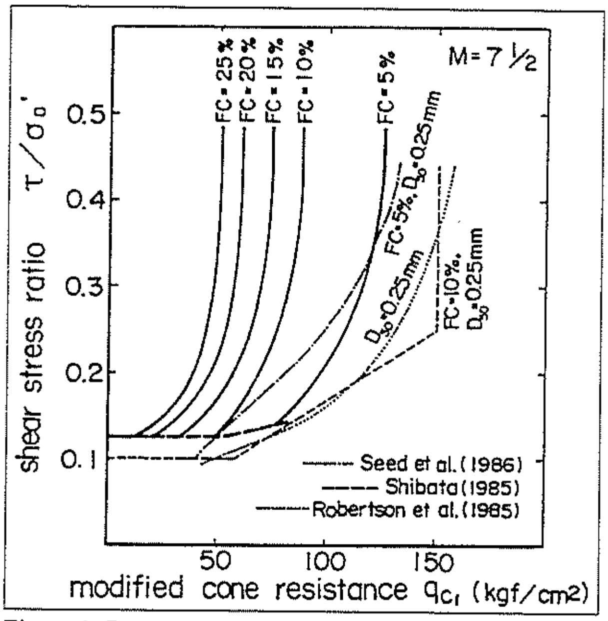



Figure 3: Proposed Correlation Between Liquefaction Resistance of Sands and Cone Resistance [5]

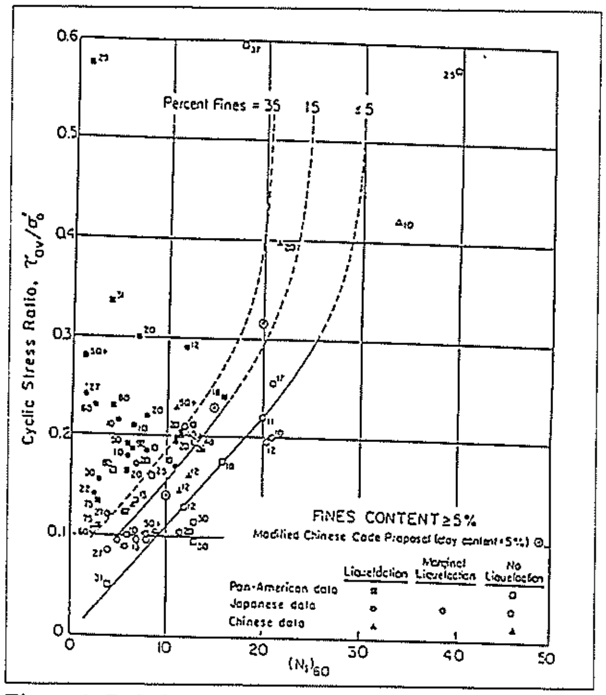



Figure 2: Relationships Between Stress Ratio Causing Liquefaction and $N_1$ -Values for Silty Sands for M = 7½ Earthquakes [3]

Seed and De Alba [3] used published correlations between SPT and CPT results to convert the SPT-Liquefaction correlation to a CPT-Liquefaction correlation. Sugawara [5] proposed a correlation between liquefaction resistance of sands and CPT cone resistance based on CPT field performance data of previous earthquake sites, and cyclic triaxial testing. Sugawara's correlation, shown on Figure 3, is in good agreement with that suggested by Seed and De Alba and was adopted for assessing the liquefaction potential for the motorway bridge sites.

The adopted procedure for assessing liquefaction resistance at the motorway sites is as follows:

- Eliminate tests in clayey soils and those above the water table
- Calculate the effective overburden pressure at test depth. Correct the measured SPT result (N), or CPT point bearing resistance ($q_c$), to equivalent values at effective stress of one ton/sq. ft. (95.8kPa) by multiplying by a factor, $C_N$,

$$C_N = \frac{(\sigma_v' + 0.07)}{0.17} \quad (\sigma_v' \ in\, MPa) \tag{1}$$

- Determine the critical cyclic shear stress ratio to cause soil liquefaction, *CSR*, from Figure 2 for SPT values and Figure 3 for CPT values. Use the curve appropriate for the fines content of the soil being tested.
- Calculate the critical PGA causing liquefaction from the following equation:

$$PGA_{crit} = CSR \times \frac{\sigma_o'}{\sigma_v} \times \frac{1}{0.65} \times \frac{1}{r_d} \tag{2}$$

where:

$$r_d = 1 - \frac{depth^2}{1486} \quad depth\ in\ metres,\ valid\ above\ 30\ m \tag{3}$$

- Compare the calculated critical PGA with the magnitude-weighted PGA on Figure 1 to find the risk of liquefaction for the soil represented by the SPT or CPT test.

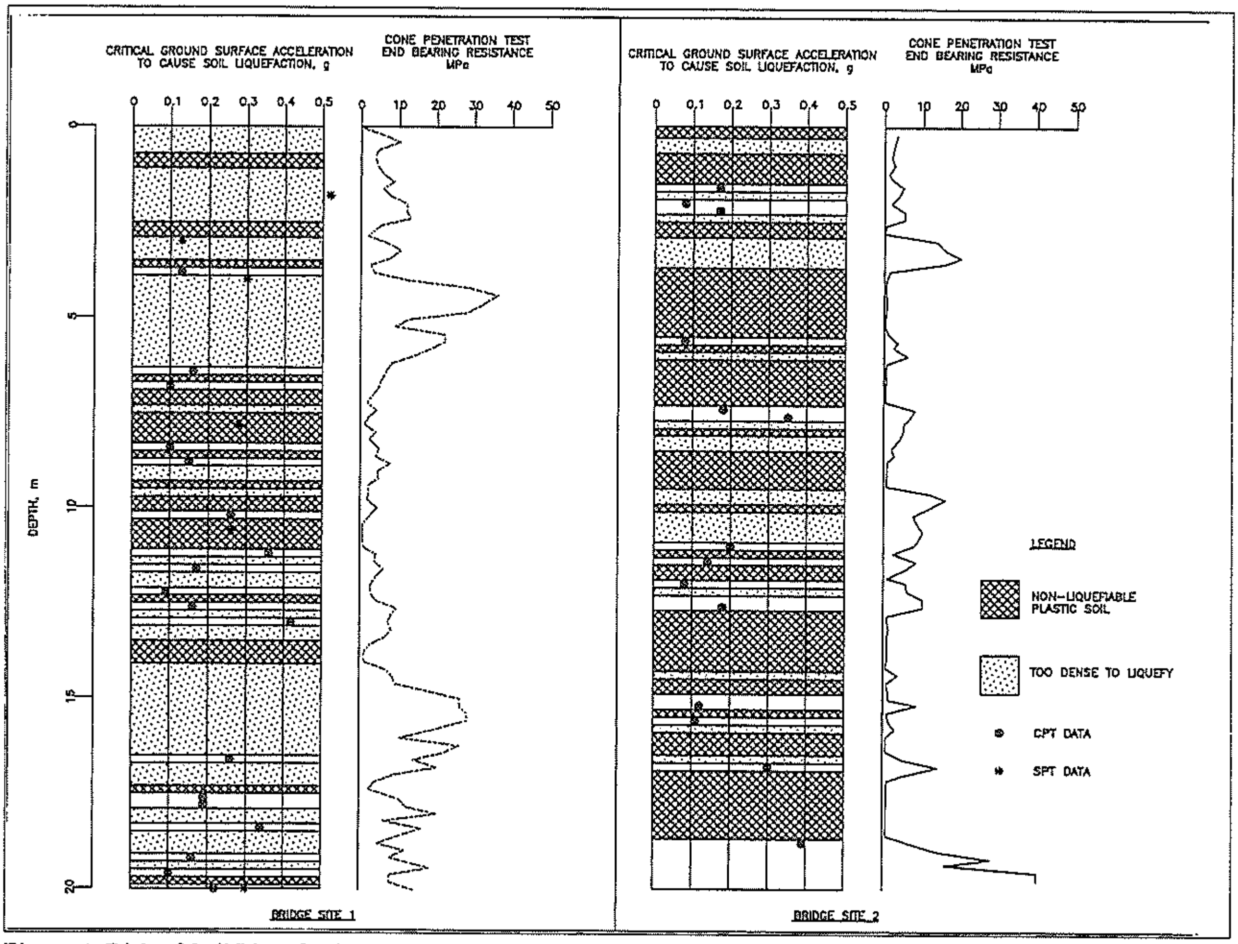



Figure 4: Risk of Soil Liquefaction at the Sites of Two of the Proposed Bridges at the Napier Hastings Motorway

Figure 4 presents the results of the liquefaction assessment for typical data at the motorway bridge sites. Both examples show several layers of soil that are potentially liquefiable at peak horizontal ground accelerations of between 0.08g and 0.42g. This corresponds to a return period for liquefaction of between 120 years and 1500 years.

The measured CPT resistance in thin layers of sandy soil may be influenced by adjacent layers of soft cohesive soils. Vreugdenhil, Davis and Berrill [6] used elastic theory and experimental data from CPT calibration chamber experiments to investigate this effect. They showed that a thin layer could be incorrectly classified as susceptible to liquefaction. Many thin zones of cohesionless material at the motorway sites (some are only 200 mm thick or one CPT measurement) are likely to have higher critical peak accelerations than calculated using the above method. The results of the liquefaction analysis should be interpreted accordingly.

## IMPLICATIONS FOR MOTORWAY STRUCTURES

The liquefaction assessment described above is a screening process, based on empirical data, to identify potential liquefaction problems under design earthquake conditions. It does not explicitly determine earthquake soil parameters, such as strength, stiffness or settlement, nor does it predict the consequences of soil liquefaction. Potential effects of soil liquefaction on the motorway bridge structures are described below.

### Negative Skin Friction on Piles

Liquefaction of soils at the bridge site would tend to densify the affected soils, causing some surface settlement. The downward movement of the soil induces downdrag on the pile that tends to reduce the usable pile capacity. This condition would be temporary and apply only until the excess porewater pressures within the soil have dissipated after the earthquake.

### Reduced Pile Capacity

The rise in pore water pressures decreases the effective confining stress within the soil, which in turn decreases the available shear strength. Analysis methods to estimate the residual strength of liquefied soils are not well developed. A conservative approach is to assume zero shear strength and the lateral stiffness in soil layers identified as liquefiable under design earthquake conditions.

### Approach Embankment Failure

Earthquake-induced deformations or excessive settlements in the approach embankments could take bridge structures out of service temporarily until they could be reestablished following the earthquake. In addition, slumping of approach embankments could impose unacceptably large loads on the bridge structure itself.

Liquefaction is not anticipated in the embankment fills since they are above the water table and the embankment fill can be compacted to high densities. Earthquake-induced failures will more likely take the form of lateral spreading or slumping because of liquefaction in the foundation soils.

### Lateral Loads

Lateral loads on abutment retaining walls would be increased during earthquake shaking because of inertial loads of the retained soil. Liquefaction of the retaining wall's foundation would lower the resistance to sliding and may result in a bearing capacity type failure.

## GROUND IMPROVEMENT

Several methods can be considered to modify the properties of the in-situ soils to reduce the potential for liquefaction and/or reduce its damaging effects. The goal of the ground improvement programme is to increase the liquefaction resistance of the ground by either densifying the susceptible soils or providing additional shear strength that is not reduced by earthquake shaking.

### Preloading

Ground treatment by preloading involves placing a precompression load, usually earth fill, on top of the ground to be treated before the construction of the proposed structure. Preloading treatment may be recommended to reduce the post construction settlement of the approach embankments, and for densifying the foundation soils to improve their liquefaction resistance.

The effect of preloading is felt mainly in the near surface soils where the applied pressures are the highest. This method therefore has limited effect on performance of pile foundations.

### Heavy Tamping

This technique consists of dropping a heavy steel or concrete weight repeatedly on the surface of the ground to be treated. The strain waves generated by these impacts rearrange the soils into a denser state, which may be expected to have a greater resistance to liquefaction. Using conventional lifting equipment, silty sands and sands may be treated to depths of 10 to 12 metres.

### Vibro Processes

Several types of vibrators are available which are inserted in the ground to densify the in-situ soils. Vibrating probes, combined with water jetting can reach significant depths of penetration. Variations of this technique (known as vibro-replacement or stone column methods) include techniques in which soil washed out during the jetting are replaced with coarse materials that are then compacted as the probe is withdrawn.

### Blasting

Deep compaction of saturated sands has been achieved by detonating buried explosives. No generally accepted theoretical design procedures are available for densification by blasting, and field trials are usually carried out before production blasting.

### Injection and Grouting

This class of ground improvement includes techniques such as cement injection or deep lime mixing and compaction grouting. These techniques create chemical bonds to provide additional shear strength in the soil, or, act as a radial hydraulic jack to compress the surrounding soil.

### Embankment Reinforcement

The risk of lateral spreading of the embankment fills could be reduced by installing structural elements within the fill, such as plastic geogrids, which impart some tensile strength to the fill. Additional benefits from use of embankment reinforcement may also be realized, such as reduced batter angles. In abutment areas, where embankment failure could affect structural components of a bridge designing the reinforcement to encourage failure in a direction that will not affect the structural components may be possible.

## CONCLUSION

Empirical methods of comparing the results of in-situ CPT and SPT results to field performance of sites that have undergone liquefaction have been used to assess the risk of soil liquefaction at the sites of several bridge structures at the proposed extension to the Napier to Hastings motorway. Soil layers that have the potential to liquefy in an earthquake have been identified.

Correlations between the results of in-situ tests and the field performance of sites subjected to earthquakes are often used to determine a profile of critical SPT or CPT results for given design earthquake parameters. This is plotted against measured SPT or CPT values to assess the potential of the soil to liquefy under those earthquake conditions.

For the present study, these correlations were used to find critical peak ground acceleration required to cause liquefaction at each bridge site. A quantitative estimate of the risk of liquefaction at each site can be made by comparing the critical peak ground acceleration to the magnitude-weighted peak ground accelerations calculated in seismic hazard assessment. This can then be used in a risk based economic assessment of the available options to mitigate against soil liquefaction.

## REFERENCES


[1] Iai S, Tsuchida H, and Koizumi K, 1989. A Liquefaction Criterion Based on Field Performances Around Seismograph Stations, *Japanese Society of Soil Mechanics and Foundation Engineering, Soils and Foundations*, 29(2): 52-68.

[2] Institute of Geological and Nuclear Sciences Limited, 1995. Evaluation of Uniform Hazard Response Spectra for Ngaruroro and Ahuriri Bidge Sites, Client Report 33495B.20 - and Supplemental Letter to Works Consultancy Services Limited, 29 May 1995.

[3] Seed H and De Alba P, 1986. Use of SPT and CPT Tests for Evaluating the Liquefaction Resistance of Sands, Use of In Situ Tests in Geotechnical Engineering, *Proceedings of In Situ '86, American Society of Civil Engineers, Geotechnical Special Publication No. 6*, pp. 281-302.

[4] Seed H, Idriss I M, and Arango I, 1983. Evaluation of Liquefaction Potential Using Field Performance Data, *American Society of Civil Engineering, Journal of Geotechnical Engineering*, 109(3): 458-483.

[5] Sugawara N, 1989. Empirical Correlation of Liquefaction Potential Using CPT, *Proceedings of the XIIth International Conference on Soil Mechanics and Foundation Engineering*, pp. 335-338.

[6] Vreugdenhil R A, Davis R O and Berrill J B, 1995. Liquefaction Potential and Piezocone Response, *Bulletin of the New Zealand National Society for Earthquake Engineering*, 28(2):106-112.

# THE APPLICATION OF NATURAL HAZARD MAPPING BY TERRITORIAL REGULATORY AUTHORITIES

N C ELDRED
Woodward-Clyde (NZ) Ltd

## SUMMARY

Natural hazard mapping is a commonly used method for defining land areas prone to processes such as slope instability and subsidence. Recent changes in New Zealand legislation require Regulatory Authorities to control the potential effects of natural hazards. Natural hazard mapping provides one method by which the Regulatory Authorities can meet their responsibilities.

This study developed a natural hazard classification system for Waitakere City, an area located to the west of Auckland City on the North Island of New Zealand. Waitakere City is characterized by a diverse topography and geology and has a history of natural hazard issues. Integral to the development of a natural hazard classification system for the Council was an assessment of how the system could be included as part of the proposed Waitakere City District Plan and assist in the control of natural hazards. A range of proposals were developed as part of the study for the inclusion of the hazard classification system in the Plan.

## INTRODUCTION

Under the Resource Management Act (1991) [1] District Councils in New Zealand are required to control any potential effects of the use, development, or protection of land. Included in this is the avoidance or mitigation of natural hazards. This paper presents a case study of an investigation undertaken for Waitakere City Council, Auckland, New Zealand to assist the council in addressing these Resource Management Act requirements. This study proposed the use of hazard mapping as one available tool for regulatory control of land. The Council requested that the study consider both the development of a hazard classification system and ways of applying the system to regulate land use.

Waitakere City forms the western part of the Auckland area and has a diverse topography and geology, including both inland and coastal environments. This has frequently created a range of inland and, in particular, coastal stability issues in relation to the development of land. Continued expansion of the City is also resulting in the development of generally more geotechnically challenging areas.

In the past natural hazard issues have had an uneven treatment in the City with an ad hoc approach to the definition of areas potentially prone to hazards. In addition, the enforcement of these issues has lead to what has been perceived to be unreasonable demands on developers and builders with no apparent sound justification for requirements, such as detailed geotechnical investigations. Therefore, technically robust and cost effective District Plan provisions, using suitable implementation techniques, were required. The study addressed these issues by providing a technically sound classification system for identifying natural hazards within Waitakere City. The developed classification system was then applied to produce a series of hazard maps to assist the Council in performing its regulatory responsibilities.

## STATUTORY RESPONSIBILITIES

The Council has a number of responsibilities under the Resource Management Act 1991 [1], the Building Act 1991 [2]and other legislation for avoiding or mitigating natural hazards. Hazards include erosion, slippage, subsidence and coastal flooding.

Under the Resource Management Act (the Act), the Council has as one of its functions the requirement to control any actual or potential effects of the use, development, or protection of land, including for the purpose of the avoidance or mitigation of natural hazards. Furthermore, under the Act, the Council cannot grant a subdivision consent if it considers that any land in respect of which the consent is sought is likely to be subject to damage by natural hazards or, if any subsequent use that is likely to be made of the land is likely to accelerate, worsen or result in material damage to the land or other land by natural hazards.

Under the Building Act, the Council shall refuse to grant a building consent if the land upon which the building is to be located is subject to, or likely to be subject to, natural hazards as defined above. In addition, the Council shall also refuse to grant a consent if the building work is likely to accelerate, worsen or result in natural hazard impacts.

For both the Resource Management Act and the Building Act the onus is on the applicant to satisfy the Council that the effects of natural hazards can be avoided, remedied or mitigated.

It can be seen that the maintenance of a suitable database of information on natural hazards is critical to the implementation of the Councils functions under these various legislative requirements. In addition, any database must be technically robust and the Council must be able to demonstrate that the information was collected using sound, technically defensible methods. Waitakere City Council proposed an approach where a natural hazard database would not form part of the District Plan but will be referred to in the Plan. This will allow the database to be updated and refined as additional data becomes available.

## NATURAL HAZARDS AND THEIR CONTROLLING FACTORS IN THE WAITAKERE CITY AREA

### Topography and Geology

Waitakere City is characterized by its diverse topography. The study area is shown on Figure 1. The east, north east and south east sections of the city border the upper reaches of the Waitemata Harbour and the Manukau Harbour and typically consist of a subdued gentle topography crossed by a series of streams. Much of the existing urban area of Waitakere City is located in these areas. Towards the west, the landscape rises to the Waitakere Ranges, a series of rugged hills which occupy much of the western part of the city. Much of the recent and proposed development is along the foothills of these ranges. The main body of the Waitakere ranges predominantly fall within the Auckland Centennial Park and were not considered in this study as they are not available for development.

Waitakere City is bordered to the west by the Tasman Sea, the Manukau Harbour to the south and the upper reaches of the Waitemata Harbour to the east and north east. The coastal environment varies from steep, high rugged cliffs with low angle beaches along the west cost and a mixture of steep cliffs, low angle estuaries, faceted lowlands and low angle beaches along the harbour coastlines.

Much of the low lying eastern and north eastern parts of the city, adjacent to the Waitemata Harbour, are underlain by sediments of the Tauranga Group consisting of interbedded muds, sands and gravels with some beds of peats and lignites. Rock of the Waitemata Group underlie the low lying eastern and north eastern parts of the city and also much of the foothills of the Waitakere Ranges. These rocks are mantled in part by the Tauranga Group sediments in low lying areas. The Waitemata Group rocks consist of alternating sandstones, muddy sandstones and mudstones with occasional grit beds. The upland Waitakere Ranges are predominantly underlain by volcanoclastic sandstones of the Waitakere Group. Within the City, a number of areas of unsupervised fill are also known to exist consisting of either a mixture of reworked materials or refuse.

### Controlling Factors

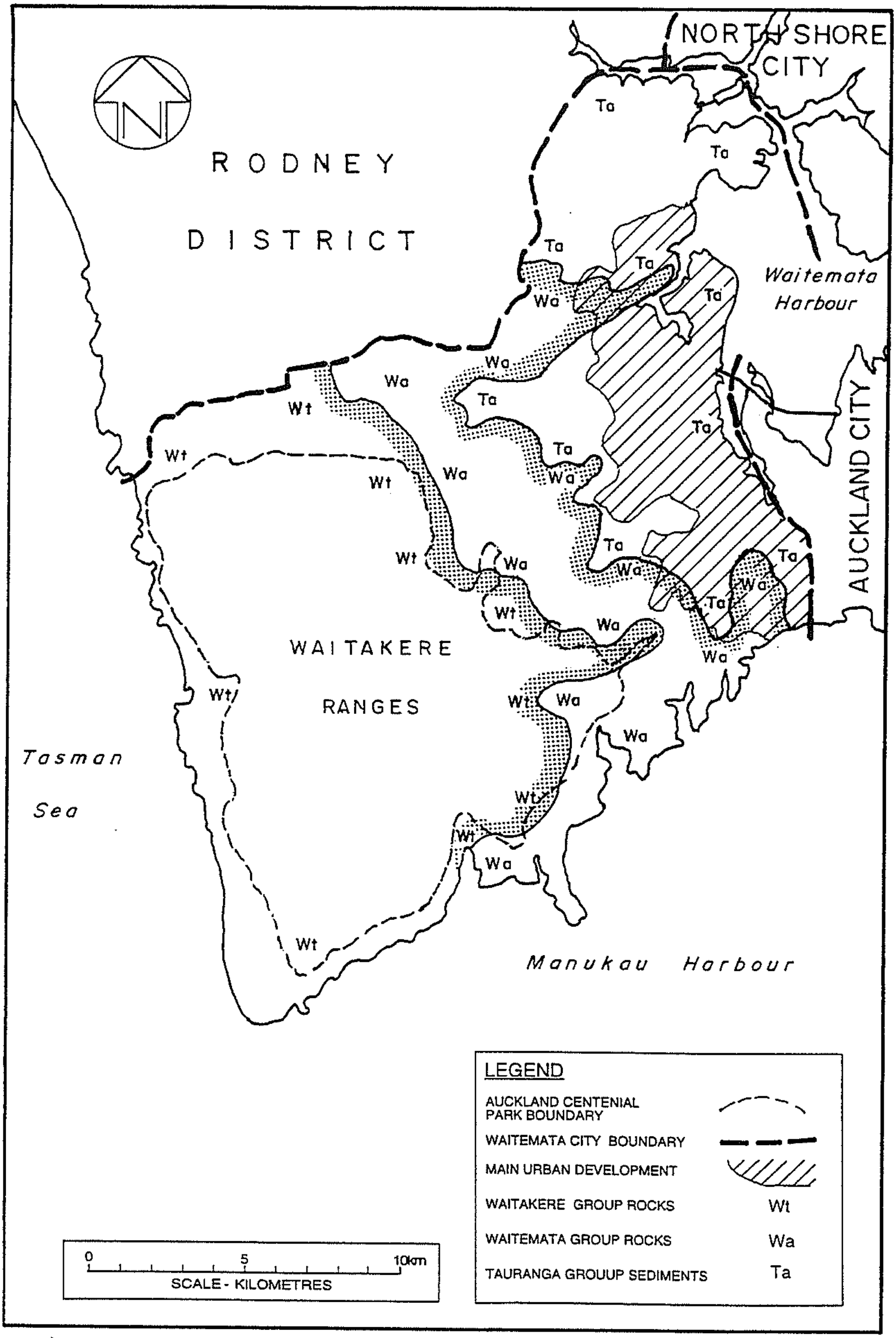
In developing a classification system for natural hazards within the City, the first step was the identification of the nature and significance of hazards along with their controlling factors. Natural hazards were considered under two headings:

Inland Hazards

Areas susceptible to **erosion, slope instability** or **subsidence**

Coastal Hazards

Areas susceptible to **erosion, slope instability** and **inundation**




Physical and Geological Plan of Waitakere City

Figure 1

Inland areas susceptible to flooding were specifically excluded at the request of the Council. Seismic hazards were also not considered as part of this study.

For each of these hazards, the factors which predispose the susceptibility of a given area were defined. For example, the susceptibility of land to **slope instability** is dependent on a number of factors. These include, but are not limited to:

Slope angle; geology; groundwater; landform; and process (both natural and human).

The characteristics of the geological materials within the Waitakere City area are a key factor in the predisposition of natural hazards. Weathering of the Waitemata and Waitakere Group rocks results in a loss of structural integrity and a significant drop in strength. Slope failures associated with weathered materials are well documented throughout the Auckland region. These are typically characterized by a shallow failure plane along the boundary between highly and moderately weathered material, although deep seated failure can and do occur. The Tauranga Group sediments are often indistinguishable from completely weathered Waitemata and Waitakere Group rocks and display similar failure forms, although their generally greater thickness can result in deeper seated failures. The Tauranga Group may also be susceptible to **subsidence** where organic clays and peat units occur. Other areas susceptible to **subsidence** include zones of unsupervised fill.

Another significant attribute of the Waitemata group rocks is their behavior at the coastal margin. The Waitemata rocks appear from beneath a mantle of Tauranga Group rocks along much of the Waitemata Harbour and Manukau Harbour coastlines. While these rocks are typically competent in the unweathered state, exposure and weathering at the coastal margin results in a significant loss of strength and integrity, particularly within the more clay rich units. Relatively low energy wave and current action may result in **erosion** of material. Coastal **erosion** is of particular concern around the densely populated Waitemata and Manukau Harbours where land **erosion** rates can be as high as 1 m / 5 years.

## ESTABLISHMENT OF A CLASSIFICATION SYSTEM

Based on the natural hazards identified for the Waitakere City area, the following Classification System was developed for the City. The system follows a similar methodology to classification schemes developed for other areas of New Zealand and abroad.

Four categories (Categories 1 to 3 and I) have been defined for classifying both inland and coastal hazards.

**Category 1 : Potential Hazards**

Areas where factors indicate a susceptibility to **instability, erosion** or **subsidence** but do not show evidence of relic or active hazards.

**Category 2 : Active Hazards**

Areas which either:

- display evidence of **instability, subsidence** or **erosion** events, either recent or historic, and have been identified either during development or on an "undisturbed site"; or
- existing detailed engineering reports for sites within the area have identified stability issues which have required, or will require, special measures for their development such as bulk earthworks, retaining structures and drainage

**Category 3 : Continuously Active Hazards**

Areas which display on-going **instability, subsidence** or **erosion** problems and require a proactive approach to development or on going mitigation of hazards.

**Category I : Potential Inundation**

Areas where factors provide a susceptibility to coastal **inundation** or where coastal **inundation** events are known to have occurred.

Within each category the type of hazard has been broadly defined as either slope failure (inland), **subsidence** hazard (inland), cliff failure/**erosion** hazard (coastal), beach or coastal margin **erosion** hazard or sand dune migration/**erosion** hazard. Coastal **inundation** was considered under a separate category (Category I) as it represents a somewhat different form of hazard.

It should be noted that areas of Waitakere City which do not fall within one of the above categories are not necessarily stable or free from the potential for hazards. They may simply represent zones where the methodology used in this study to apply the Classification system has not identified either existing hazards or factors which predispose potential hazards.

## APPLICATION OF THE CLASSIFICATION SYSTEM

A section of the Waitakere hazard map prepared during this study is shown in Figure 2. The Council had a restricted budget available for the production of these maps. Therefore, this necessitated a desk top approach to application of the classification system with limited resources available for field visits. It was recognized by the council that these plans represent an initial classification of the City. It is envisaged that as further data is collected over time, these plans will be added to and amended as appropriate. The methodology used to develop these maps is presented below:

### Category 1

Different methods were used to define the range of natural hazards for Category 1.

#### Inland **Instability**

Of the range of controlling factors for inland **instability**, slope angle is considered to be critical and a parameter that can be readily defined from a desk top investigation. **Instability** features are generally associated with weaker materials within the Waitakere City area, either residual weathered material or more recent deposits, that cover nearly all the land surface area. All these materials display a generally similar range of characteristics with regard to **slope instability**. Variations that do occur would generally result from changes in critical slope angle with respect to **instability** of one or two degrees. It was not considered practical to vary the selected critical slope angle given the subtle changes in geology and geotechnical characteristics and the scale of topographic maps available for the area. Therefore, a conservative slope angle of 7 degrees was selected as an angle below which **instability** features have not generally been identified in any of the materials present within the Waitakere City area.

Category 1 zones were defined based on the topographic plans available for the City, namely the 1:50,000 and 1:25,000 topographical plots produced by Department Of Survey and Land Information (DOSLI). It was recognized that the scale of these plans is such that small areas with slope angles in excess of 7 degrees may occur outside categorized zones but may not be shown due to the map scale.

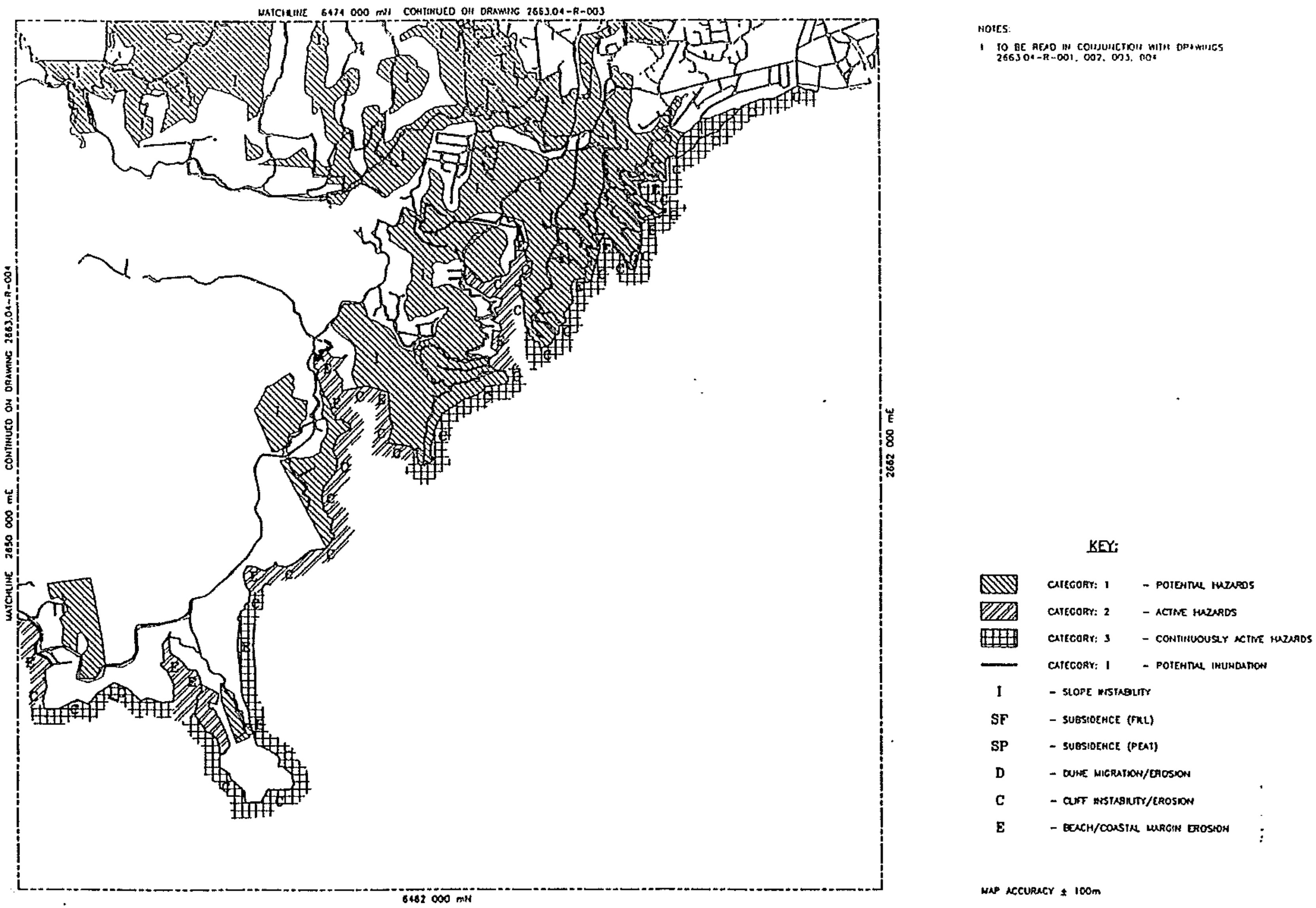
An example of this is small water courses that cross otherwise relatively level sites. The combination of steep slopes immediately adjacent to water courses and the **erosion** activity of the water course could present potential problems for development immediately adjacent to channels. It was recognized that such features are of special consideration and should be classified as either Category 1 or 2 zones. Conversely, small areas of level ground may also fall within zones categorized as potentially unstable. These issues are addressed later in this paper.

#### Inland **Subsidence**

Areas potentially susceptible to **subsidence** were defined based on: geological maps and records of unsupervised fill; anecdotal records from senior engineering office's for Waitakere City and other experience in the area. By its very nature, materials susceptible to **subsidence** are often difficult to identify without detailed investigation and a relatively conservative approach was adopted to define these areas.

#### Coastal **Instability** and **Erosion**

All areas of the coast which fall outside Category 2 and 3 zones were considered to be inherently within Category 1 . Based on the processes operating at the coast and the geological materials present, it was established that **erosion** and **instability** could potentially occur, or be induced to occur, anywhere along the coast.




**Example of Natural Hazards Classification**

**Figure 2**

Category 2 and 3 - All Hazards

Category 2 and 3 were defined on the basis of the following data for both coastal and inland hazards: 1:25,000 stereo aerial photographs and 1:10,000 vertical aerial photographs; 1:250,000 and 1:50,000 geological maps; anecdotal information from Council offices; experience within the area; Geotechnical Reports for the area; and limited field visits to selected sites.

Category I - Potential For **Inundation**

Category I was defined on the basis of the following data: coastal elevation; slope angle of coastal margin; and likely maximum storm surge events; and tsunami events.

## APPLICATION OF THE HAZARD CATEGORIZATION SCHEME

Hazard mapping has been used extensively throughout New Zealand and abroad for categorizing land for a wide range of purposes. However, the practical application of these schemes under various regulatory methods has been sporadic and somewhat ad hoc. The second part of the brief from Waitakere City Council was the development of methodologies which enable the City Council to regulate land use activities in relation to natural hazards. With respect to the natural hazard classification system developed for Waitakere City Council as part of this study, the following is one possible method for controlling the effects of development in relation to hazards.

With regard to the subdivision consent process, District Pan rules could require differing levels of geotechnical investigation (e.g. to establish safe and stable building platforms) and engineering design. The methods would depend upon the zoning of land being subdivided and its categorization under the hazard system outlined above.

With regard to building consent applications, the subdivision consent process may already have identified suitable building sites during subdivision. However, in previously developed areas, circumstances may arise where land is redeveloped in such a way that could contribute to and/or be affected by a particular hazard. In these circumstances, use should be made of the classification system and associated geotechnical investigation and design measures outlined below if no site specific geotechnical report is available for the original subdivision.

### Rural Zones

In some circumstances only parts of a given land parcel may fall within one of the identified hazard category areas. If it can be demonstrated that the proposed building development is outside of the categorized area, no specific geotechnical investigation should be required under this scheme. However, if the proposed development falls within one of the hazard category zones, the proposed investigation and design criteria outlined below should be followed.

### Urban Zones

Where a subdivision is proposed, a geotechnical investigation of some form will be required to determine the location of suitable building sites on each lot. Where a proposed subdivision falls wholly or partly within one or more of the identified hazard zones, the investigation and design criteria outlined below should be followed.

Category 1 Areas - Where land is identified as being prone to **Instability** (either coastal or inland) it may be possible to identify building platforms that meet the following criteria within the Category 1 areas: slope angles less than 7 degrees; the site is outside the coastal environment; and the site is not crossed by, or immediately adjacent to a stream or river channel where steep and potentially unstable banks and/or bank **erosion** occur.

In these circumstances geotechnical investigations could require, as a minimum, a walkover survey to confirm that building platforms meeting these criteria were available. However, where building sites: are proposed in **instability** areas that do not meet these criteria; are within the Coastal Margin; or are in areas that have been identified as potentially prone to **subsidence**, a more thorough geotechnical investigation would be required which specifically addresses these hazards to the satisfaction of the Council It should be noted that these are minimum requirements. Any subdivision will require some form of geotechnical investigation.

Category 2 Areas - In general, development within Category 2 areas should require a detailed geotechnical investigation that addresses the specific hazard(s) identified. It is envisaged that the design of the development would include measures to mitigate potential hazard issues.

Category 3 Areas - Category 3 areas are generally confined to the coastal margin. It is envisaged that development would be either discouraged in these areas or very carefully controlled. Furthermore, a proactive approach may be required in some areas to mitigate the effects of hazards on existing development.

Category I Areas - It is envisaged that development within areas of the coastal environment designated as potentially prone to **inundation** would be discouraged. Where development is proposed, any engineering investigation and design should incorporate measures to mitigate potential **inundation** effects.

## OTHER REGULATORY METHODS

Other regulatory methods available for controlling or mitigating natural hazards based on the hazard classification system include:

- District Plan rules or bylaws could set minimum floor levels for new residential development in areas subject to flooding and a "coastal protection yard" in coastal areas in general .

- District Plan rules could require a land consent for certain activities, the effects of which have the potential to contribute to natural hazards. For example, vegetation clearance, and earthworks that exceed a specified volume. Such rules could be targeted at areas identified as being affected or potentially affected by particular natural hazards and the Council could grant consents if satisfied that adverse effects would be controlled.

- Undeveloped coastal and inland areas identified as being affected by significant natural hazards could be zoned "rural" or "open space" rather than for future urban development.

- In areas of existing urban development identified as being affected by significant natural hazards District Plan rules could limit the density of development. For example, higher density infill housing may have a greater potential to contribute to and/or be affected by hazards such as slippage or coastal flooding.

- District Plan provisions for the creation of esplanade reserves and esplanade strips can be used to mitigate natural hazards at the coastline. For example, the legal instrument creating an esplanade strip can prohibit the willful damage or removal of any plant from the strip and control public access over the strip (including the use of recreational vehicles). Such controls can be used to mitigate coastal **erosion.**

## ADVANTAGES AND DISADVANTAGES

The development of hazard classification schemes and their application under appropriate regulatory methods can ensure that the adverse effects of certain activities in terms of natural hazards are adequately controlled. This may not otherwise occur if, for example, the avoidance or mitigation of natural hazards was solely reliant upon the provisions of information and public education.

However, the disadvantages of a regulatory approach is that perceived private property rights may be curtailed, with limitations placed on the ability of landowners to make their own choices about the use of their land. Categorization of property may also impact upon property values Furthermore, the cost of making a land use consent application to the Council for a particular activity can be high. In addition, the method relies upon adequate and accurate information so that controls target only areas of concern, a particularly difficult requirement when dealing with natural hazard identification without spending comparatively large sums of money.

## CONCLUSIONS

This approach to the application of hazard mapping systems provides a useful tool to assist the Territorial Regulatory Authorities in meeting their requirements under various legislation. Furthermore, the location of the hazard database outside the District Plan allows the data to be updated as and when further information becomes available.

REFERENCES

1 The Resource Management Act (1991), New Zealand

2 The Building Act (1991), New Zealand

# DESIGN OF DYNAMIC COMPACTION ON LANDFILLS

D L FELLOWS
Woodward-Clyde (NZ) Ltd

## SUMMARY

Dynamic compaction is a ground improvement technique currently used to reduce void space, increase density and reduce long term settlement in soils. It has been used with varying success as a treatment for landfill deposits. Thirteen case studies have been evaluated to assess the success of dynamic compaction on unengineered landfills.

The results suggest that current dynamic compaction design practices tend to overestimate the treatment depth achieved. A damping of input energy due to a cohesive component in the landfill as well as the presence of groundwater in the treatment zone are presented as possible explanations. Based on back analysis of available data, modified design constants are suggested for unengineered landfill deposits.

Relationships between four characteristics associated with dynamic compaction in landfill (i.e. landfill age, depth, input energy, induced settlement) are also explored and alternative design equations are proposed based on these relationships.

## INTRODUCTION

The increasing scarcity of good building land coupled with the need to redevelop inner city areas and reclaiming industrial waste lands has increasingly led to the development of landfill sites. Of particular engineering concern in the redevelopment is the engineering properties of the landfill. A landfill is generally weak, highly compressible and exhibits low bearing capacity and large long term settlements. A range of ground improvement techniques are available to improve the engineering properties of landfill. Such techniques include: vibro compaction, stone columns, dynamic compaction, dewatering, preloading or surcharge and drainage.

This paper examines the effectiveness of dynamic compaction (DC) in treating unengineered landfills by examining a detailed case study from a site in the UK and makes comparisions with 12 published examples. The validity of the design equations presently used are reviewed in light of the available data and modifications to the present design practices are suggested. Furthermore, the relationships of four characteristics associated with DC on landfills are explored and correlations between these characteristics lead to alternative design equations presented in this paper. It is not the intention of this paper, to present a detailed discussion on the DC technique, or the problems associated with DC on landfills.

## PRINCIPLES OF DYNAMIC COMPACTION

Dynamic compaction is a ground improvement technique based on the improvement of weak soils by controlled high energy tamping. The technique was pioneered in the early 1970's by Louis Menard [16, 17]. This saw the increased application of this technique for ground improvement. Following on, other workers considered the behaviour of different soil types to DC ie. clay fills [2, 24]; loose deposits [14]; and granular fills [7].

Several papers summarise the 'state-of-the-art' of dynamic compaction, namely Mitchell [18], Mitchell & Katti [19], Greenwood & Kirsch [12], and more recently Slocombe [22]. There are also numerous published accounts of DC for the construction of airports, roads, and building foundations.

The technique requires repeated surface tamping using a heavy steel or concrete weight. Typically, the tamper weighs between 6 to 20 tonnes dropping in free fall from a height of up to 20 metres. A common approach is to divide the soil to be treated into three layers; deep, middle and shallow. The first tamping pass is aimed at treating the deepest layer by adopting a relatively wide grid pattern and a suitable number of drops from the full height of the crane. The middle layer is next treated by tamping on an intermediate grid with a reduced number of drops and drop height. The final (surface) layer is treated by a continuous tamp of a small number of drops from low height over the entire surface[22]. This is often termed a rolling pass and smooth wheeled vibrating rollers can be used instead of the tamper.

In the design of DC a number of factors should be considered. The main factors are outlined below:-

1. Type of material to be treated:- is of fundamental importance in the design of the DC programme. Granular and cohesive soils behave differently when subjected to high energy impacts. In granular soils DC reduces the void ratio, increases relative density, and improves load bearing and settlement characteristics. When load is applied to a clay soil, it exhibits consolidation settlement as a result of the expulsion of pore water from between the clay particles causing a reduction in volume, and an increase in strength over a long period of time[27]. The presence of soft layers or groundwater within the treatment depth has a damping influence on the dynamic forces. Therefore it is very important to have a detailed knowledge of the soil constituents. Landfill is a combination of coarse and fine grained materials which can include soil, domestic refuse, timber, bricks etc. The importance of a thorough ground investigation prior to the design of a DC programme cannot be stressed too strongly.

There is some contention in the published literature as to the suitability of DC as a ground improvement technique for landfill [12, 22]. General consensus is that DC is less successful in fine grained or variable soils. It is believed that a maximum of 25% to 30% fines (<0.02mm) in a soil is the maximum before DC becomes ineffective. In granular soils DC is very successful [15].

2. Depth of treatment: is a function of the size of equipment adopted ie. impact energy, soil type and the degree of improvement of the soil. The depth of treatment (Z) can be calculated from a number of different relationships depending upon the soil type. The total energy input per square metre (E) is the major requirement and is given in Eq. 1.

$$E = \frac{No. of\ Drops\, x\, D\, x\, W}{A} \tag{1}$$

Current design methods are related to the empirical rule [17]:-

$$Z = \sqrt{(WxH)} \tag{2}$$

Where:
H = Thickness of layer to be improved (m)
E = Impact energy (Tonnes/m$^2$)
W = Tamp weight (T)
D = Drop height (m)
A = Treatment area (m$^2$)

This relationship has been modified to take into account the treated soil type by using a varying constant multiplier, d. These are summarised in Table 1 below.

Table 1 Summary of Design Equations used for Dynamic Compaction

| Soil Type | Treatment depth Relationship | Reference |
|---|---|---|
| Coarse Granular Soil | 0.5√ (WxH) | 13 |
| Coarse Granular Soil | 0.65 to 0.80√ (WxH) | 14 |
| Coarse Granular Soil | 0.375 to 0.7√ (WxH) | 19 |
| Loose Fills | 0.4√ (WxH) | 6 |
| Stiff Clay Fill | 0.35√ (WxH) | 4 |
| Domestic Refuse | 0.4√ (WxH) | 4 |
| Sand Fill | 0.5√ (WxH) | 4 |

It should be noted that Eq. 2 and its derivatives do not take into account other factors which influence the depth of treatment. Such as the fact that soils are rarely homogeneous, and are commonly layered and the presence of soft cohesive soils causes a damping influence on the dynamic forces. This is the situation within landfill sites. Initial strength and groundwater conditions also influence the result.

Equation 2 evaluates the soil properties such that the depth of treatment is inversely proportional to the dynamic resistance (q) of the soil skeleton to compression [1].

$$Z = \frac{WgD}{qB^2} \qquad \text{or} \qquad Z = \frac{WkD}{B^2} \tag{3}$$

where:
q = Dynamic resistance of soil skeleton
g = Acceleration of gravity (m/sec$^2$)
B = Plan dimension of tamp (m)
k = g/q

Since the soil parameter q is rarely known, it is usually necessary to adjust the initial design following preliminary trials. The value of K typically lies in the range 0.1 to 0.16 [25].

Both Eq. 2 and 3 are used commercially in the UK and USA to produce design charts for DC.

3. Amount of induced settlement: The net volume imprint ie. the total imprint minus the amount of heave, is a measure of the total void reduction beneath a tamping position. Unfortunately, the distribution of the void reduction below the ground surface is not known and is likely to be different for every soil type. The initial imprint is used as an initial guide to the amount of compaction obtained. The average depression of the site after levelling gives the cumulative volume change. Therefore imprint volume relative to the number of blows should be monitored. Total volume change depends on the initial state of the soil but 5% to 10% is usual. Substantial depressions must be formed if compaction is to be achieved, especially in fine grained or cohesive soils.

## USE OF DYNAMIC COMPACTION ON LANDFILL

In the UK and USA DC is used on landfill sites prior to their redevelopment with varying success. Data available from thirteen case studies have been evaluated to assess the effectiveness of DC on a non engineered landfill. The published case studies present various amounts of detail regarding the design and success of the treatment used. A case study made available by the Department of Transport in the UK (DoT) provides the most detailed information on the use and success of DC on a landfill [9]. In particular, the study has been able to evaluate the depth of treatment achieved by comparison of pre and post treatment cone penetration tests. Detailed on site measurement of impact craters has also provided data to calculate average induced settlements and volume reduction.

## DISCUSSION

### Existing Design Equations

The data set currently available on the effective depth of treatment provides an opportunity to assess the suitability of the design equations currently used and comment on their applicability to landfills. The relationship between the amount of energy used and the effective depth of treatment is of paramount importance in design of DC. Figure 1 presents this relationship with superimposed design lines whose gradient is equivalent to the constant d. Using the effective depth of treatment it is possible to back analyse the data to calculate more appropriate constant values for use in the design equations. A summary of these results is presented below in Table 2.

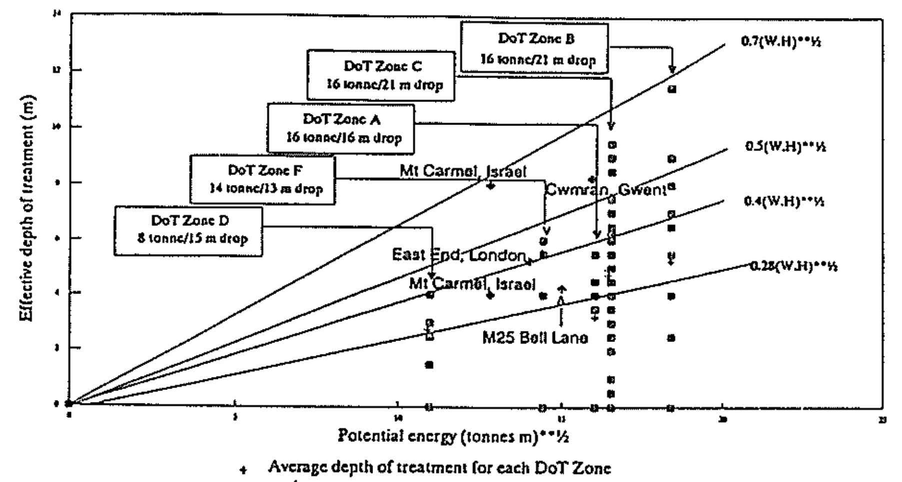



Figure 1 Effective Depth of Treatment against Potential Energy

Table 2 Summary of back calculated constants, d and k, for each treatment site

| Case Study Reference | Design Constant d | | | Design Constant k | | |
|---|---|---|---|---|---|---|
| 9 UK | Minimum | Maximum | Average | Minimum | Maximum | Average |
| Zone A | 0.22 | 0.34 | 0.26 | 0.06 | 0.09 | 0.07 |
| Zone B | 0.00 | 0.63 | 0.33 | 0.00 | 0.12 | 0.08 |
| Zone C (east) | 0.00 | 0.66 | 0.24 | 0.00 | 0.16 | 0.07 |
| Zone C (west) | 0.09 | 0.62 | 0.33 | 0.20 | 0.14 | 0.09 |
| Zone D | 0.00 | 0.37 | 0.38 | 0.00 | 0.16 | 0.07 |
| Zone F | 0.00 | 0.42 | 0.32 | 0.00 | 0.13 | 0.12 |
| Average of Data | 0.00 | 0.50 | 0.28 | 0.00 | 0.13 | 0.08 |
| 8 UK | 0.41 | | | - | | |
| 20 UK | 0.28 | | | 0.23 | | |
| 3,4 UK | 0.39 | | | 0.45 | | |
| 10[1] Israel | 0.31 | | | - | | |
| 10[1] Israel | 0.62 | | | - | | |

Footnote: 1. This site reported DC trials and the difference in depth of treatment is due to a difference in energy input.

There is a considerable range of design constant (d) values varying from a minimum of zero for areas of no improvement to a maximum of 0.66. Similar back analyses have been carried out to calculate the constant, k, used in Eq. 4. The values obtained are summarised on Table 2 and range from a minimum of zero to a maximum of 0.45. All the data show a considerable scatter in terms of effective depth of treatment and the back calculated design constants. However, the general trend indicates that the design constants currently used in commercial practice are too high. The difference in values obtained must be related, in some way, to the behaviour of the fill and its composition. The data available does not allow for the further differentiation of constant d values based on fill composition. Based on the data reviewed in this discussion values of d = 0.3 and k = 0.08 are considered to be more appropriate for landfill materials in design Eq. 2 and 5.

## Landfill and DC Treatment Characteristic Relationships

Four characteristics unique to each site are considered which provide the basis for comparison of the data:-

1. landfill depth
2. landfill age
3. amount of input energy used during DC
4. the amount of settlement induced by DC

Although a data set of thirteen (Refs. 3,4,5,8,9,10,11,15,20,23,26,28,29) is not large enough to provide unequivocal empirical correlations, approximate relationships between each of the above characteristics can be shown .

The nature of the design of DC is such that the amount of input energy used is related to the depth of the landfill to be treated. Figure 2 shows a wide spread of data with deeper landfills requiring a higher input energy for treatment. The best fit line through all data could be used as an initial DC design curve, such that input energy can be identified. Once this is obtained it then only requires the identification of satisfactory combination of weight, drop height, number of blows and grid spacing to maximise the amount of induced settlement.

There are six data points which fall outside the general cluster. These are: Case Studies in Refs. 8,15,20, Zones A, B, D in Ref. 9. The amount of cohesive material in the Zones A and B may account for the damping effect. It is considered possible that the case studies in Ref. 8 and 15 may also have a high fines content. The dampening effects in the Ref. 20 results are probably due to groundwater within the treatment zone.

If the data is considered in terms of the fill content two linear best fit lines can be drawn, one for granular type refuse and the other for landfill with a probable fines content of greater than 25% (Figure 2). Regression analysis on these two data sets has enabled the following relationships to be defined.

For Granular Refuse:
Input energy I ($Tm/m^2$) = 9 x Landfill depth (m) (4)

For material with a cohesive content > 25%:
Input energy I ($Tm/m^2$) = 18 x Landfill depth (m) (5)

or alternatively
$$Z = \frac{W\,x\,D\,x\,No.of\,blows}{A} x \frac{1}{\tau} \tag{6}$$

Where $\tau = 9.44$ for granular refuse $\tau = 18$ for fill with cohesive content >25%

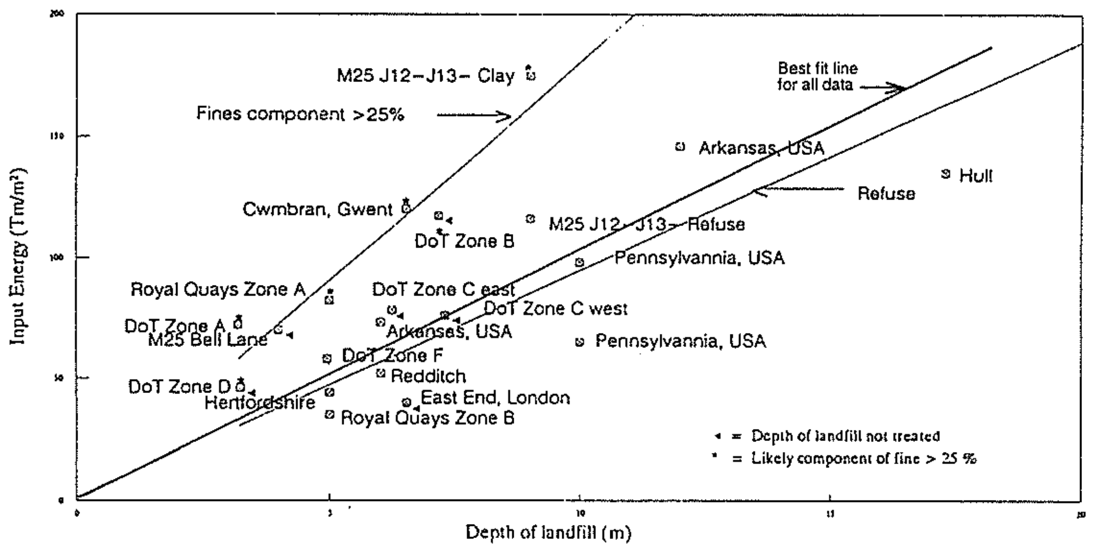



Figure 2 Graph of Input Energy against Depth of Landfill from published case studies.

The difference between Eq. 4 and 5 reflects the damping effect of energy in fine grained soils. Equation 2 should be applied to landfill with a cohesive component less than about 25% and Eq. 5 to landfill with a cohesive component in excess of about 25%.

It is expected that the amount of volume reduction induced is proportional to the input energy used. This relationship is shown on Figure 3 and in general there appears to be a linear relationship. However, there is a wide spread in the data and a possible reason for this may be that the amount of volume reduction achieved by DC is also a function of fill density. The older fills are denser whilst younger fills will be less dense. Comparison of the ages of the landfill at the time that DC are shown on Figures 3. The older landfills (10 to 40 years) show only 10% or less volume reductions, whilst the younger landfills exhibit much higher volume reductions. Unfortunately, it is not known whether any compaction was carried out as part of the infill process at each site. However, consideration of volume reduction in terms of the fines content gives two possible relationships as shown on Figure 3. Linear regression of the two data sets (less than and greater than 25% fines) enabled the definition of the following expressions:-

For granular refuse:
Input energy I ($Tm/m^2$) = 5 x Volume reduction (%) (7)

For material with a cohesive content >25%
Input energy I ($Tm/m^2$) = 10 x Volume reduction (%) (8)

or alternatively
$$\delta = \frac{W\,x\,D\,x\,No.of\,blows}{A} x \frac{Z}{\beta} \tag{9}$$

Where $\beta = 545$ for granular refuse $\beta = 989$ for fill with a fines content >25%

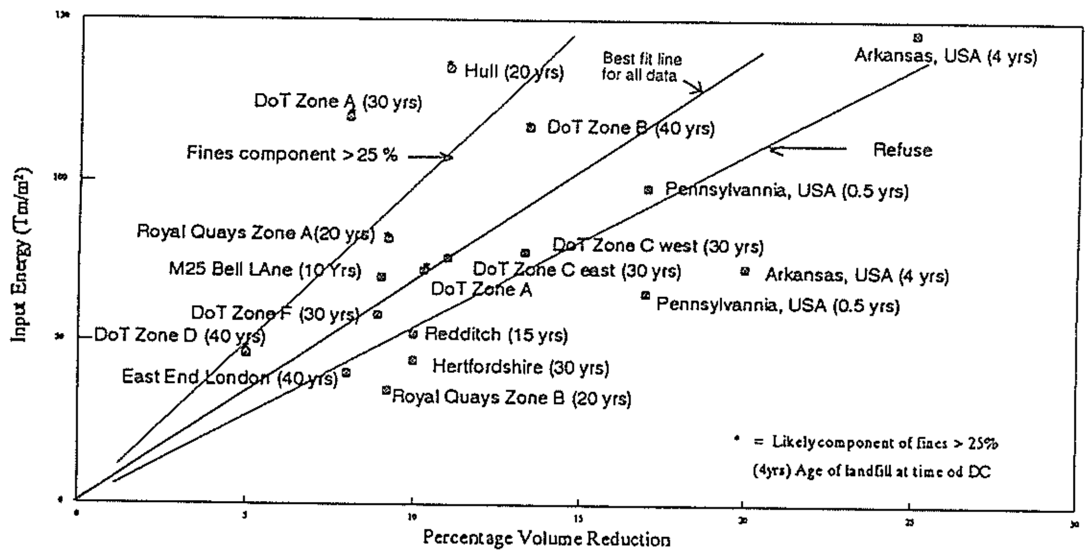



Figure 3 Graph of Input Energy against Percentage Volume Reduction from published case studies.

All of the expressions given above can be considered as alternative design formulae for DC on landfill. These expressions allow the energy required to be assessed for any given landfill depth or requisite amount of settlement. For that particular energy it only remains for the appropriate combination of weight, drop height, number of blows and grid spacing to be established. However, the main difference between Eq. 4 and Eq. 3 [1] is the way in which the input energy is expressed. Equation 6 considers the number of blows used to calculate the input the energy, whilst Eq. 3 does not.

## CONCLUSIONS

A landfill is recognised as being neither homogeneous nor inelastic and hence its behaviour as a result of DC is not well predicted by current practice. Twelve published examples and a further detailed case study have been analysed to assess the suitability and appropriateness of DC on landfill using the current standard practise. The results show that existing design equations overestimate the effective depth of treatment achieved by DC in landfills. Possible reasons for this are:

1. The landfill material is heterogeneous, and that soft cohesive layers or a high component of fine grained material has a damping influence on the energy transmission through the soil.

2. Perched or natural groundwater tables within the zone of treatment also tend to damp energy transmission.

3. The landfill may be denser than initially considered, (ie. it may have been compacted at the time of emplacement), hence the expected results for a specific amount of energy input are not achieved.

The implication of the underestimated treatment depth is that there remains within the landfill zones of material that has not been treated which may still have the capacity for large amounts of long term settlement. The design equations currently used in industry to calculate effective depth of treatment are clearly inappropriate for landfill. Analysis of the case studies suggest that the commonly used design equations for landfill materials be modified, as detailed below.

$$Z = 0.28\sqrt{(W \, x \, H)} \qquad (10)$$

$$Z = \frac{0.08MH}{B^2} \tag{11}$$

Furthermore, four characteristics of a landfill have been shown to be important in the evaluation of DC. These are: depth of landfill, age of the deposit, input energy used in DC and the amount of enforced settlement achieved. The data available suggest that linear relationships can be defined between the depth of landfill and the energy used for treatment and the amount of enforced settlement achieved. These relationships are given in Eq. 6, and 9. In all cases assessed in this paper, it has been assumed that the landfill was not compacted significantly on deposition. Thus the correlations discussed apply only to non engineered landfill sites.

## ACKNOWLEDGEMENTS

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## REFERENCES

1. Billam, J 1979. Depth of treatment in dynamic compaction *Engineering Behaviour of Industrial and Urban Fill. Symposium Proceedings 23-25th April 1979.* Midland Geotechnical Society. pp. E138-E140.

2. Charles, J A 1978. Methods of treatment of clay fills. *Clay Fills. Proceedings of the Conference held at the Institution of Civil Engineers, 14-15 November 1978.* The Institution of Civil Engineers, London, General Report pp. 315-321.

3. Charles, J A 1979. Field observations of a trial of dynamic consolidation on an old refuse tip in the East End of London. *Engineering Behaviour of Industrial and Urban Landfill. Proceedings of the Symposium held at the University of Birmingham. 23-25th April 1979.* pp. E1-E7.

4. Charles, J A 1984. Settlement of fill. Attewell P B and Taylor R K (eds.). *Ground movements and their effects on structures.* Surrey University Press. pp. 26-42.

5. Charles, J A; Burford, D; Watts, K S. 1981. Field studies of the effectiveness of "Dynamic Consolidation". *Proceedings of the 10th International Conference of Soil Mechanics and Foundation Engineering, Stockholm.* Vol. 3 pp. 617-622.

6. Department of Transport 1987. *Specification for Highway works.* 6th Edition 1987. HMSO. Department of Transport 1991. Earthworks. Design and preparation of contract documents. Department of Transport Highways Safety and Traffic Departmental Advice Note HA 44/91.

7. Dobson, T; Slocombe, B 1982. Deep densification of granular fills. *The proceedings of the second Geotechnical Conference and exhibit on Design and Construction. Las Vegas Nevada.*

8. Downie, A R; Treharne, G 1979. Dynamic consolidation of refuse at Cwmbran. *Engineering behaviour of Industrial and Urban Landfill. Proceedings of the Symposium held at the University of Birmingham.* 23-25th April 1979. pp. E15-E24.

9. Fellows, D L 1993. *The design and specification of Dynamic Compaction on Landfill.* Unpublished MSc Thesis, University of Surrey, UK.

10. Frydman, S; Baker, R. 1987. Construction of a bus parking station on a waste deposit site. *Building on Marginal and Derelict Land.* Thomas Telford Ltd. pp. 255-266.

11. Galante, V N; Eith, A W; Leonard, M S; Finn, P S 1991. An assessment of deep dynamic compaction as a means to increase refuse density for an operating municipal waste landfill. *The Planning and Engineering of Landfills. Proceedings of the Conference organised by the Midland Geotechnical Society, University of Birmingham 10-11 July 1991.* pp 183-191.

12. Greenwood, D A; Kirsch, K 1984. Specialist ground treatment by vibratory and dynamic methods. *Proceedings of Conference on Piling and Ground Treatment.* Thomas Telford, London. pp. 17-45.

13. Leonards, G A; Cutter, W A; Holtz, R D 1980. Dynamic Compaction on Granular Soils. *Journal of Geotechnical Engineering Division of American Society of Civil Engineering.* 106 pp. 35-44.

14. Lukas, R G 1980. The densification of Loose deposits by pounding. *Journal of Geotechnical Engineering Division of the American Society of Civil Engineers.* 106 GT4 April 1980. pp. 435-446.

15. Mapplebeck, N J; Fraser, N A 1993. Engineering landfill by Dynamic Compaction to support Highways and Buildings. *Proceedings of Conference on Engineered Fills 93.* Paper 39, p. 492-504.

16. Menard, L 1974. *La Consolidation dynamique des sols de foundation Conferences.* ITBTP.

17. Menard, L; Broise, Y 1976. Theoretical and practical aspects of dynamic compaction. *Ground treatment by deep consolidation. Institution of Civil Engineers, London.* pp. 3-18.

18. Mitchell, J M 1981. Soil improvement - state of the art report. *Proceedings of 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm.* Vol. 4 pp. 509-559.

19. Mitchell, J M; Katti, R J 1981. Soil improvement - state of the art report. *Proceedings of 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm.* Vol. 4 pp. 567-575.

20. Perelberg, S; Boyd, P J H; Monutague, K N; Greenwood, J R 1986. M25 Bell Lane Pit: Ground improvement by dynamic compaction. *Building on Marginal and Derelict Land.* Thomas Telford Ltd. pp. 267-280.

21. Schlosser, F; Juran, I 1979. General report - Design parameters for artificially improved soils. *Proceedings of VII European Conference on Soil Mechanics and Foundation Engineering. Brighton.*

22. Slocombe, B C 1993. Dynamic Compaction. Moseley M P (ed). *Ground Improvement.* Chapman & Hall. pp. 20-39.

23. Swain, S; Holt, D N 1986. Dynamic compaction of a refuse site for a road interchange. *Building on Marginal and Derelit Land.* Thomas Telford Ltd. pp. 339-357.

24. Thomson, G H; Herbert, A. 1979. Compaction of clay fills in situ by dynamic consolidation. *Clay Fills. Proceedings of the Conference held at the Institution of Civil Engineers, 14-15 November 1978.* The Institution of Civil Engineers, London. pp. 197-204.

25. Vibroflotation Ltd 1993. *Method Statement for Contract on Confidential Site.*

26. Welsh, J P 1984. Dynamic compaction of sanitary landfill to support superhighway. *Proceedings of the 8th European Conference on Soil Mechanics and Foundation Engineering. Helsinki. 1983.* Balkema, Rotterdam. Vol. 14 No. 1 pp 31-36.

27. West, J M 1976. The role of ground improvement in Foundation Engineering. *Proceedings of Conference on Ground Improvement by Deep Compaction, Institution of Civil Engineers,* 1975, London. pp. 71-78.

28. Williams, P J 1983. Preparing Motorway Junctions. *Consulting Engineer.* October pp 18-19

29. Willams, P J 1984. Construction Over Water Filled and Back Filled gravel Pits. *The Journal of the Institution of Highways and Transportation.* pp. 26-30.

# THE SHEAR BEHAVIOUR OF ROCK JOINTS

B. FLEUTER

Department of Civil Engineering
Monash University, Vic, Australia

## SUMMARY

The methods of prediction of rock joint behaviour have traditionally relied on empiricism or over-simplified theoretical approaches. In addition only limited boundary conditions are considered in many cases. Recent advancements into rock socketed piles have resulted in a theoretically based roughness model for the shear behaviour of concrete-rock interfaces. As there are many similarities between rock joints and the concrete-rock interface of socketed piles it is likely that the new theories are also applicable to rock joints. This paper describes the beginning of a project to expand socketed pile research to rock joints.

## INTRODUCTION

The behaviour of a rock mass is generally controlled by the strength of its weakest components. These are usually any discontinuities present within the rock mass such as joints, faults and bedding planes rather than the intact rock. When a rock mass is subject to loading from foundations or unloading from excavation and tunnelling, movement at the discontinuities is possible. For the purpose of this project only discontinuities that are mated (no displacement) will be considered. Such discontinuities are known as joints. Under opening a joint is assumed to have zero tensile strength while under closing the intact strength of the rock is usually taken. It is therefore the shear behaviour of a joint that is of most concern to the rock engineer.

To date researchers have tackled this problem by adopting over-simplified theoretical approaches or have relied heavily on empiricism. Recent work at Monash University has resulted in an experimentally based theoretical model for the shear behaviour of concrete-rock interfaces of rock socketed piles. This problem has many similarities to rock joints so it is likely that these models are also applicable to rock joints.

This paper will provide a background to the area of rock joints, a brief description of the models developed at Monash for socketed piles and the current project on the behaviour of rock joints.

## ROCK JOINTS

The shear behaviour of rock joints is complex and depends on several factors including roughness, rock strength, boundary conditions and presence of infill. A brief description of the effect of these factors is given below.

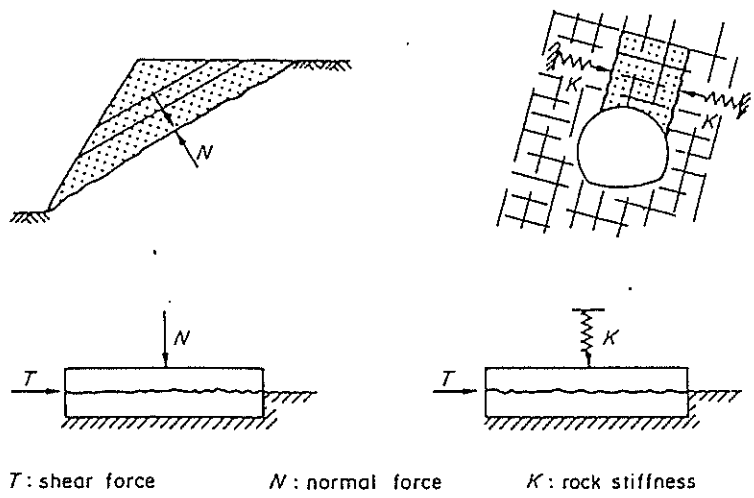


Figure 1 : Boundary conditions for rock joints (after Leichnitz, [6])

*Boundary Conditions*

There are basically two different boundary conditions under which a joint will be subject to. These are known as the constant normal load (CNL) and constant normal stiffness (CNS) conditions, Fig. 1. The laboratory shear test approximations are also shown.

The CNL condition is applicable to rock slopes where sliding may occur along a joint under the constant load of the overlying rock block. The CNS condition is applicable to rock socketed piles and underground excavations where sliding along a rough joint causes a dilation against the surrounding rock mass. Since the rock has a stiffness, additional normal load is applied to the joint. In CNS cases a load usually acts on the joint prior to shear displacement. This is known as the initial normal stress, $\sigma_0$.

*Roughness*

The effect of roughness is of major importance to the shear behaviour of rock joints, Seidel and Haberfield [9]. Joint roughness has a direct influence on the strength of the joint and on the amount of dilation developed during shear displacement. Roughness can be considered as a parameter that increases the friction angle above the base friction angle by an amount $i$, where $i$ is the "asperity" or "dilation" angle, (see Eq. 1). Seidel and Haberfield, [9], also describe the scale dependence of roughness. For example, small scale roughness may be present on a larger scale roughness. Under small shear displacements peak shear strength will be governed by small scale roughness and vice versa.

$$\tau = \sigma_n \tan(\phi_b + i) \tag{1}$$

where $\tau$ = shear strength
$\sigma_n$ = normal stress
$\phi_b$ = base friction angle
$i$ = angle of asperities

*Infill*

The presence of infill affects the frictional resistance of the sliding surfaces. Infill generally reduces the friction angle however some infills can bond the joint making it stronger than the parent rock. It has been suggested by Goodman, [2], that where the depth of infill is 1.5 times the asperity height the shear behaviour of the joint may be governed entirely by the shear strength of the infill material. For the purpose of the current project infill will not be considered. The joints are assumed to be mated and clean.

## PREVIOUS RESEARCH

Previous research has followed two separate paths. The first is to reduce the problem to simple theoretical models. The second approach is to rely on empirical methods. Key examples of these approaches will be described below. It should be noted that much of the previous research has tended to concentrate on CNL boundary conditions. The importance of CNS boundary conditions has been recognised by many researchers only quite recently.

*Ladanyi and Archambault Model [5]*

Ladanyi and Archambault used an energy approach to model joint behaviour. In this model the total shear stress, $\tau$, is given by Eq. 2.

$$\tau = \frac{\sigma(1-a_s)(\dot{v} + \tan\phi_\mu) + a_s(\sigma \tan\phi_0 + S_0)}{1-(1-a_s)\dot{v}\tan\phi_f} \tag{2}$$

where, $\dot{v}$ = rate of dilation at failure
$\phi_\mu$ = angle of frictional sliding resistance
$\phi_f$ = average value of sliding friction
$\phi_0$ = internal angle of friction
$S_0$ = intact rock strength
$a_s$ = area ratio

It can be shown that under high or low stress this model can be reduced to the appropriate sections of the bilinear model proposed by Patton [7], which is derived from statics. While an improvement on Patton's model, there are several limitations. These include the assumptions that shear occurs on a horizontal plane, the normal stress acts equally on all asperities and asperities are rigid.

*Barton Model [1]*

Barton's original model was based on the results of extensive shear tests on natural joint samples of varying strength and roughness. The empirical expression shown in Eq 3 is the result of this work.

$$\tau = \sigma_n \tan [JRC \log_{10} (JCS/\sigma_n) + \phi_b] \tag{3}$$

where $\tau$ = peak shear strength
$\sigma_n$ = effective normal stress
JRC = joint roughness coefficient
JCS = joint wall compressive strength
$\phi_b$ = basic friction angle

JRC is a measure of roughness determined by tilt tests or by visual observation with standard profiles and ranges from 0 for planar surfaces to 20 for very rough surfaces. It can be seen that the Barton model is the same as Eq. 1 but with an empirical expression for $i$ ($i$= JRC $\log_{10}$ (JCS/$\sigma_n$). The factor $\log_{10}$ (JCS/$\sigma_n$) is introduced to account for normal stress suggesting that as the normal stress increases the dilation angle decreases. While the use of this model is very popular there are several shortcomings. Although the basic model has been retained, later publications have added various empirical correction factors to account for effects such as scale. In addition, the determination of JRC by visual observation can be difficult and there is no cohesive component to account for shearing of asperities.

## RESEARCH AT MONASH UNIVERSITY

Since the mid 70's the Geotechnical Group at Monash have conducted extensive research into the behaviour of concrete piles socketed into rock. The construction process of a socketed pile results in a rough interface between the concrete and rock. When the pile is loaded and displaced vertically, dilation occurs against the surrounding rock mass, Fig. 2. Socketed piles are therefore governed by the CNS condition. The concrete-rock interface of socketed piles has many obvious similarities with rock joints. As the purpose of this project is to apply the advancements in socketed pile models to rock joints it is necessary to briefly describe the history and current stages of development into socketed piles.

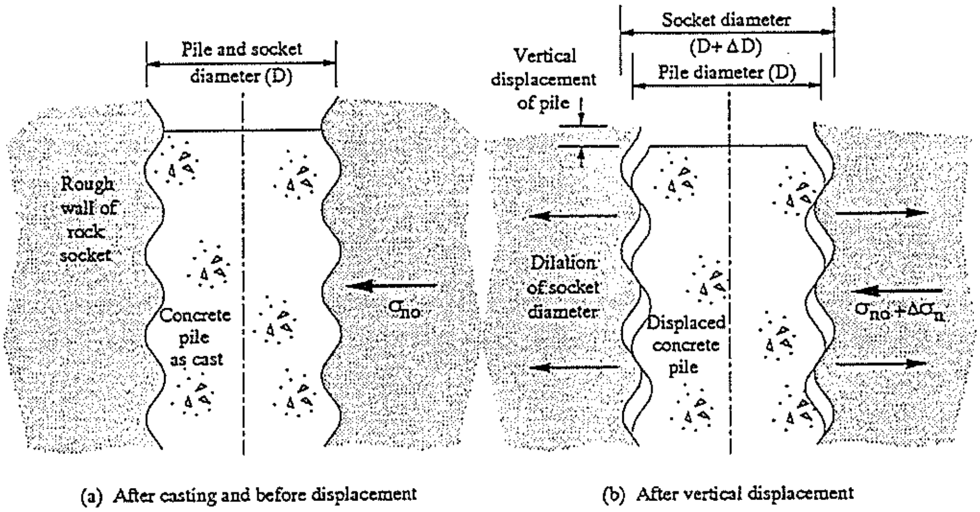


Figure 2 : Rock socketed piles (after Johnston and Lam, [4])

The work into socketed piles is the result of several researchers' efforts culminating, so far, with the work of Seidel, [8]. Early work evolved from rock joints by representing joint roughness as regular triangular asperities.

Later work was extended to include elasticity effects and irregular triangular asperities. While these models enabled the basic failure mechanisms of single asperities to be established they still did not represent the behaviour of natural rock joints adequately. Seidel [8], used the concepts of fractal geometry to generate profiles with a more realistic roughness. To make practical use of these profiles Seidel developed methods to relate the fractal dimension (the fractal dimension of a rough line is between 1 and 2) of the profile to roughness statistics such as chord angle and height. The fractal model provides a basis to verify if the mechanisms established from regular triangular asperities can be used to predict the behaviour of more complex and realistic profiles.

The work of Seidel has been incorporated into a Windows based computer program called Rocket. Rocket is able to predict the full response of piles socketed into weak rock. While the program is specifically designed for Melbourne mudstone it has been found to be accurate for a wide variety of rock types and strengths. Another feature of Rocket is a prediction of CNS direct shear tests on which the testing program was based. It is this function that will be used to compare with the testing on rock joints and modified as necessary.

## CURRENT PROJECT

As mentioned previously the main objective of this project is to extend the successful research into rock socketed piles to rock joints. Since the socketed pile work originated from rock joint research the extension of the new models back to rock joints is natural, however some differences between the two cases are expected. The basis of this investigation will be a series of direct shear tests. Analysis and modifications to the model will be undertaken after the completion of testing.

The experimental testing program is being conducted on two types of sedimentary rocks, Melbourne mudstone and Hawkesbury sandstone. The rocks are of significant economic importance in the Melbourne and Sydney areas. Due to the difficulty of obtaining intact samples of mudstone a reconstituted and reconsolidated mudstone - known as "Johnstone", [3] - will be used. This provides a sample that is homogeneous, reproducible and has well established properties. Natural sandstone will be used. The unconfined compressive strengths of Johnstone (8MPa) and sandstone (17MPa) provide a reasonable range of rock strengths.

The most recent direct shear device at Monash provides an opportunity to test larger samples sizes. A maximum sample size of 600×200mm, significantly greater than other devices, is possible. This allows a large number of segments that are needed for fractal profiles to represent natural behaviour. Hydraulic actuators of 250kN capacity apply the shear and normal loads. The device is capable of both CNS and CNL boundary conditions and is fully computer controlled.

Regular Triangular Asperities - 10 degree

Fractal Profile - 10 degree mean angle

Figure 3 : Profiles (vertical scale exaggerated)

The direct shear tests will be used to determine the effects of the following on joint behaviour :

- joint roughness
- joint normal stress
- joint normal stiffness
- joint scale

The shear tests are to be performed on Johnstone and sandstone rock samples with artificial profiles comprising both regular triangular asperities and fractal profiles. Profiles are cut by waterjet techniques. As with previous research, regular triangular profiles will be used to establish the basic failure mechanisms. In addition, time lapse video images will be utilised to observe the failure mechanisms. Angles of 5, 10 and 20 degrees have been adopted for the regular triangular asperities with base lengths of 16 and 48mm. Three fractal profiles with mean absolute angles of 5, 10 and 15 degrees and base lengths of 16mm have been chosen. Figure 3 gives an example of a regular profile and a fractal profile. While the testing program is yet to be completed some results on regular triangular profiles are presented in Figures 4 and 5.

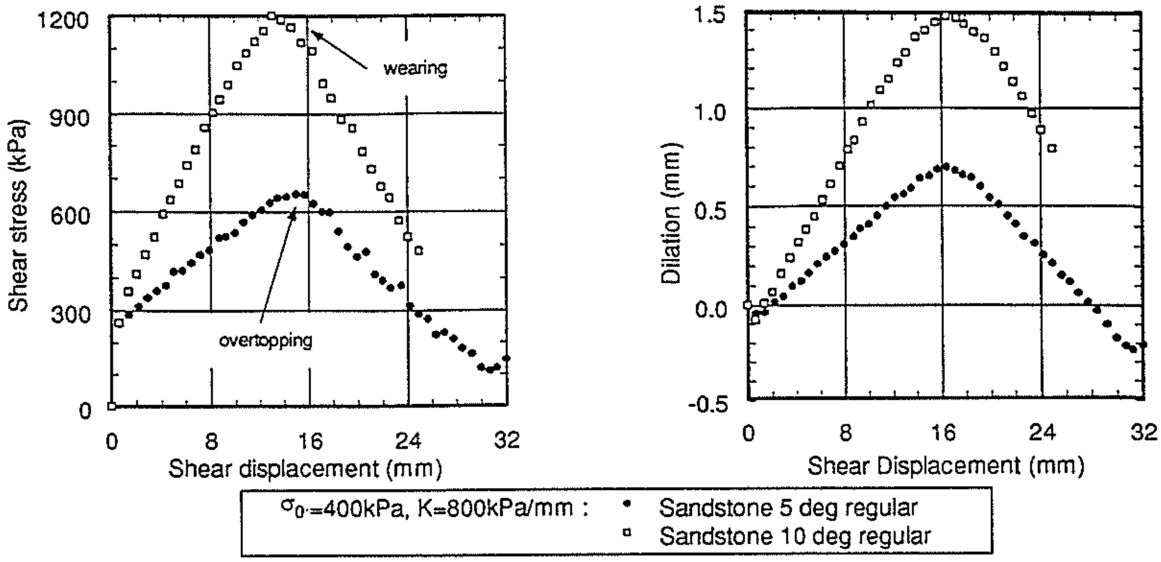


Figure 4 : Typical results - effect of roughness

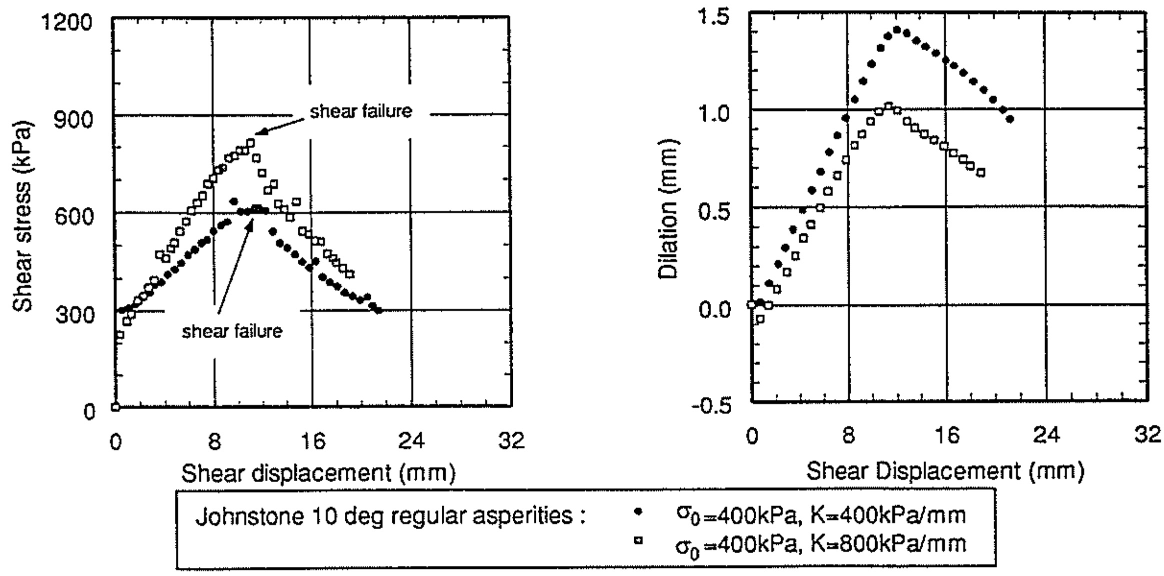


Figure 5 : Typical results - effect of CNS boundary conditions

Figure 4 shows a comparison of typical test results for sandstone samples with 5 and 10 degree regular triangles with 16mm base lengths. The initial normal stress and normal stiffness are identical at 400kPa and 800kPa/mm respectively. The 10 degree sample can be interpreted as being a "rougher" profile. From the shear stress and dilation versus shear displacement graphs it can be seen that the greater angle has a significant influence on the shear stress and dilation developed. The greater shear strength experienced by the 10 degree sample is due to two effects. Firstly the greater asperity angle produces a greater friction angle, ($\phi_b$ + $i$). Secondly as the tests were carried out under CNS conditions the greater dilation of the 10 degree sample increases the normal stress acting on the joint. Therefore as both the friction angle and normal stress acting on the joint are greater for the 10 degree profile the shear resistance is also greater.

Figure 5 shows a comparison of typical test results for Johnstone samples with identical profiles of 10 degree regular asperities. The initial normal stresses are also identical however, one has a greater stiffness. As shearing progresses the dilation of the sample with higher stiffness is more restricted however the increase in normal load due to stiffness is still greater resulting in a greater shear stress. It can be seen that both roughness and boundary conditions have a significant influence on the shear behaviour of rock joints.

Comparisons between Johnstone and sandstone are also seen in Figures 4 and 5. For the tests on the 10 degree profiles at an initial normal stress of 400kPa and stiffness of 800kPa/mm the sandstone sample achieves a much greater shear strength. This is a reflection of the higher strength and friction angle of the sandstone.

Comparisons of the completed shear tests to the existing concrete-rock model using the Rocket program have been performed. Currently the Rocket program consistently overestimates the shear strength of the rock joint tests. This is to be expected since the Rocket program was modelled on concrete-rock interfaces. As one surface (concrete) is much stronger than the other, dilation of the joint is greater and compression of the sample is less, hence normal stress increases, leading to greater shear strengths (for CNS conditions). This is one obvious modification necessary to the Rocket program which will need to be incorporated in the near future. However, it is observed from time lapse photography that the fundamental mechanisms of failure for Johnstone samples are similar, suggesting that the models are valid. Further investigation of the sandstone samples will be necessary. A full review of the models will take place after completion of the testing program.

## CONCLUSIONS

This paper describes the beginning of a research project into the shear behaviour of rock joints. The project is an expansion of the related work into rock socketed piles that has achieved significant advances in recent years. The basis of investigation is a series of direct shear tests on two types of sedimentary rocks currently underway. The results will be used to adapt the existing model to predict the shear behaviour of mated rock joints under CNS and CNL boundary conditions.

## REFERENCES


1. Barton, N. and Choubey, V. (1977). The Shear Strength of Rock Joints in Theory and Practice. Rock Mechanics 10, 1-54.

2. Goodman, R.E. (1970). The deformity of joints. In Determination of the in-situ modulus of deformation of rock. ASTM Special Publication, No. 477, pp. 174-196.

3. Johnston, I.W. and Choi, S.K. (1986). A synthetic soft rock for laboratory model studies. Geotechnique 36, No. 2, 251-263.

4. Johnston, I.W. and Lam, T.S.K. (1989). Shear behaviour of regular triangular concrete/rock joints - analysis. ASCE Jnl. of Geotech. Engg., Vol. 115, No. 5, May 1989, 711-727.

5. Ladanyi, B. and Archambault, G. (1970). Simulation of shear behaviour of a jointed rock mass. Proc. 11th Symp. on Rock mechanics. Rock Mechanics, Theory and Practice, 105-125.

6. Leichnitz, W. (1985). Mechanical Properties of Rock Joints. Int. J. Rock Mech. Min. Sci. and Geomech. Abstr. Vol. 22, No. 5, 313-321.

7. Patton, F. D. (1966). Multiple modes of shear failure in rock. In: Proc., 1st Congress Int. Soc. Rock Mechanics, Lisbon, 1, 509-513.

8. Seidel, J. P. (1993). The analysis and design of pile shafts in weak rock. PhD Dissertation, Dept. of Civil Engng., Monash University.

9. Seidel, J.P. and Haberfield, C.M. (1995). Towards an understanding of Joint Roughness. Rock Mech. Rock Engng. 28 (2), 69-92.

# GROUND TREATMENT IN RECLAMATION DESIGN & CONSTRUCTION - THREE HONG KONG CASE HISTORIES

STEPHEN L. GAMBLE

Frankipile Australia Pty Limited,
Sydney Australia.

SUMMARY

Most reclamation projects involve the forming of land over soft or loose near-shore or shallow water sediments, which often require some form of ground treatment to minimise post-construction settlement. Each reclamation project has a unique set of conditions, which will determine which of the available ground treatment techniques are appropriate for achieving the required performance improvement Three case studies are presented which illustrate some of the different ground treatment techniques used, and the reasons for their choice is discussed.

## INTRODUCTION

The construction of new land by reclamation is not a new process. It has been carried out throughout history where swamps have been drained, coastal lagoons filled or shorelines extended. These operations would typically be slow and the occupant of the resultant land would need to be able to tolerate the ensuing settlement. In modern times our reclamation technology allows us to construct reclaimed land quicker, and of a better finished quality, than these humble beginnings. The ability to quickly produce a good quality reclamation arises directly from the advances made in ground treatment. These include techniques to improve the condition of both cohesive and granular material. The techniques of ground treatment more commonly used in reclamation construction include pre-loading with a surcharge, inducing additional soil drainage, improvement by introducing additional material, and by compaction. These techniques are implemented using a variety of methods. For ease of discussion, the various ground improvement methods will be introduced and described under four group headings in the first part of this paper.

Nowhere is the advancement of reclamation technology more apparent than in Hong Kong. Ever since the island of Hong Kong was ceded to Great Britain in 1841, the pressure of development in the colony has demanded a continual expansion of the available building space which has continually outstripped the supply of available building sites. The first reclamation shown on the current geological mapping of the colony [1] is dated 1863, and it is seen that further reclamation projects have been carried out intermittently since that time. In recent times, spanning the lead-up period to the handing over of sovereignty from Great Britain to the Peoples Republic of China, the pace of development in Hong Kong has increased dramatically. With container port traffic growing approximately 20% every year, the construction of a replacement international airport with road and rail links, together with the requirement for space to accommodate the burgeoning commercial and residential development; the areas of reclamation recently completed, under construction, or in design stage add together to form a brobdingnagian amount of construction activity. All of this work is carried out under very tight programmes, with the occupants of the newly created land expecting to be able to build on it immediately after hand-over. To further increase the geotechnical difficulties, these reclamations are built in areas where the existing geological sequence includes a soft near to normally consolidated marine silty clay, which is highly compressible. Three case histories of recent reclamation projects in Hong Kong will be discussed in the second part of this paper. These three cases illustrate some different geotechnical problems that have occurred, and the ground treatment techniques that were adopted to address these problems.

## GROUND TREATMENT METHODS

The ground treatment methods used in reclamation design and construction, generally aim to improve the strength and the load-settlement behaviour of the reclaimed land, and although it is recognised that some projects will have other special requirements, this paper shall focus on these two main aims. The various methods for ground treatment are grouped for ease of discussion under the following four main categories.

- Soil Compaction Methods
- Soil Replacement Methods
- Accelerated Consolidation Methods
- In-Situ Soil Column Improvement Methods

These are briefly discussed in turn in the following section, giving details on mode of implementation, special applications, and limitations of the techniques.

### Soil Compaction Methods

The ground treatment methods included in this group includes vibratory compaction, dynamic compaction, and compaction grouting, however the latter is rarely applied to reclamation construction and hence will not be discussed further. Both the vibratory and dynamic compaction methods aim to increase the density of the soil by realigning the individual soil grains with vibration or dynamic impact energy.

With the vibratory method, a probe is vibrated into the soil, sometimes aided by water jets (depending on the probe type). Upon reaching the required depth the probe is vibrated and lifted with series of cyclic lifts and re-insertions that ensure that each level of the sub-surface profile is sequentially compacted. The compaction probing is usually carried out on a regular grid, with spacing chosen on the basis of site trials that match the probe system being used, to the soil type. The various systems are usually crane mounted and depending on site conditions, produce results which are both fast and economical. The plant used for vibratory compaction generally falls into one of two categories: horizontal amplitude (immersion) probes which have the vibrator at the tip of the probe, or vertical amplitude probes where the vibrator is mounted at the top of a probe which is often fitted with fins or wings. The noise and vibration levels associated with vibratory compaction are generally low but will obviously vary according to a combination of vibration energy, frequency, soil conditions, and the sensitivity of the nearby structures. Experience [2] has shown that vibrating plant of current day energy levels can be safely used 15m from sound structures, with some instances of vibroflots used significantly closer.

To be successful, vibratory compaction requires the soil to be predominantly sandy, with a fines content of less than about 15%. If the fines content of the soil is too high, the increase in pore pressure resulting from the re-arrangement of the grains cannot dissipate quickly and there is a tendency for liquefaction of the soil. Several authors [3 & 4] have published criteria based on the particle size distribution of the soil for judging the suitability of the soil for vibratory compaction.

Dynamic compaction employs impact energy to cause the re-alignment of soil grains. This impact is achieved by dropping a heavy weight, usually from a crane, onto the ground surface. The dynamic compaction is usually carried out in 3 phases, consisting of the primary phase which compacts the deeper layers in the soil, the secondary phase which compacts those upper layers missed in the primary, and lastly the ironing phase where material is added and compacted in the depressions created in the first two phases and as the name implies, the surface is ironed out by a general pounding. As with vibratory compaction dynamic compaction is carried out on a regular grid with the spacing defined by site trials, which are also used to determine the appropriate size of pounder to be used. Most soil types other than soft silts and clays can be successfully compacted using this technique and the results achieved are quick and cheap. The compactive effort penetrates beneath the water table and in layered soil conditions. Fills contaminated with boulders or rubble can be successfully compacted using dynamic compaction.

The dynamic nature of this compaction technique gives rise to its main disadvantage, in that the shock-waves from the dropped pounder, have the potential for causing problems on site. As with other construction activities that cause ground vibrations, the public perception of the vibrations are often more serious than any physical damage to nearby structures, either way the problem is a real one. The depth of penetration of the dynamic compaction effort depends on the size of the pounder and the height of the drop, and the size of available cranes is often the limiting factor. The effectivness of this treatment as well as the vibro compaction, can be confirmed by before and after CPT soundings.

### Soil Replacement Methods

The methods for ground treatment bracketed in this group include vibratory replacement, dynamic replacement and driven stone columns. Each method involves the construction of a column of granular material, usually gravel sized, using either vibration or dynamic impact energy. The construction of the stone column aims to improve the bearing capacity and the drainage characteristics of the soil. The performance of each of the three methods can be checked by conducting load tests, and by monitoring settlements in the treatment area.

Vibratory replacement uses an immersion type vibrating probe already briefly described in the above section on vibratory compaction. The probe is advanced into the ground to the desired depth, and then successive charges of gravel (usually crushed rock) are added around the probe as it is lifted and re-inserted. This operation is continued cyclically, so that a column of compacted stone is built up to the surface. The introduced gravel displaces the in situ soil horizontally; and the size of the column, which largely depends on the strength of the in situ soil, is calculated from the volume of gravel used. The compactive effort is measured by monitoring the current drawn by the electric motor, or in the case of a hydraulic system the oil pressure in the vibrator. Vibratory replacement is usually carried out where an increased surface bearing capacity is required, usually in soft clays and silts where compaction methods are not appropriate. The installation of stone columns using vibratory replacement, is usually carried out on a regular grid, depending on the extent of the structural load on the ground surface. If the area of treatment is large, the stability of the stone columns is generally very good. If however the treated area is narrow or small in size, or the columns are constructed in very soft layers of significant thickness, some checking of column stability is required, since they have little shear capacity.

Dynamic replacement employs the same equipment as the dynamic compaction described above. The stone columns constructed by dynamic replacement are formed by pounding successive charges of crushed rock into the soft ground. The ideal conditions for dynamic replacement are for a surface layer of sandy soil overlying a soft silt or clay, which in turn overlies a firm/dense founding layer. The stone column is constructed through the soft layer and is founded on the firm/dense layer. This method of ground treatment is only effective for a limited thickness of the soft layer, which depends on the size of pounder and the height of its drop (thus by the size of crane available). If the capacity of the crane/pounder is reached, there is a risk that the stone column will not fully penetrate the soft layer. This may be overcome by pre-drilling, however this would markedly increase the cost of the ground treatment.

Driven stone columns have a final result similar to the two preceding soil replacement methods, however their mode of construction is markedly different. They are installed using a rig usually used for installing driven cast-in-place (Franki) piles. The stone column is constructed by a driving a temporary liner into the ground using a bottom drive drop hammer. When the liner has reached the required level, which can be checked by the blow-count of the hammer, successive charges of crushed rock are driven out of the bottom of the temporary liner as it is withdrawn. This forms a column of compacted stone similar to those described above. The distinct advantage this system offers is one of confidence. With the ability to monitor the penetration rate of the diving tube, and also the energy imparted to each charge of gravel as the tube is withdrawn, this method of installation gives greater reliability to the quality of the resultant stone column. It must be recognised that the method is slower, and also requires the mobilisation of a specialist piling rig, which as a consequence makes this method marginally more expensive.

### Accelerated Consolidation Methods

Accelerated consolidation treatment aims to speed up the settlement due to soil consolidation by increasing the overburden pressure and/or reducing the length of the drainage path within the soil. These methods are applied to reclamations with clayey and silty layers, where the inherent impermeability of these gives rise to consolidation settlements.

Pre-loading using some form of surcharge increases the overburden pressure acting on the consolidating soft layer, thereby causing early consolidation of the soil during the construction period. The surcharge is usually a temporary fill embankment placed on the ground surface, however the surcharge load can also be achieved by de-watering or by some other form of temporary load. Pre-loading is mostly used in conjunction with other ground treatment, but in suitable conditions it is used by itself. Ground treatment with surcharge is expensive, and extra resources of earthmoving plant and fill must be included in the construction programme, as well as the time required for sourcing, placing, removal and disposal of the surcharge material itself.

The method for accelerating the consolidation by reducing the length of drainage paths involves the introduction of drains into the soil, and thus decreases the time taken for consolidation of the soil layer. This well established method of treatment is particularly applicable to reclamation construction, which often involves thick layers of soft impermeable sediments being buried under the reclamation fill. The drains are generally installed vertically into the soil and are usually made from a free draining material such as sand or a synthetic drain fabric. The sand drains are simply a column of suitably sized free draining sand placed in a hole that is drilled or driven into the soil. The size of sand drain is usually 150-250mm diameter, which balances the installation cost against the ability of the drain to resist necking, clogging or some other mode of failure. The alternative, more modern form of vertical drain is the band or wick drain. These are usually made from a geotextile wrapped, shaped strip that allows free draining along grooves, dimples or similar relief in the strip's surface. The drain itself is usually between 2 to 10mm thick and 100mm wide. These drains have the advantage of a longer service life which is attributed to their ability to remain free draining after considerable compression of the surrounding soil, and their resistance to clogging. The band drain is usually installed within a hollow mandrel which is pushed into the ground by a hydraulically operated installation rig, the mandrel is withdrawn leaving the drain behind in the soil. Both types of drain are relatively inexpensive, however they require purpose-built crane or excavator based installation equipment. The treatment method is time consuming, because an extended period of typically 12 to 18 months consolidation time is required on top of the time for the drain installation, which in itself comparatively rapid. The alternative however, is to leave the ground untreated which may entail consolidation periods extending for perhaps decades. Usually vertical drain treatment is used in conjunction with surcharge treatment. Geotechnical instrumentation plays an important part in carrying out this treatment, since it is only by monitoring the settlement behaviour of the reclamation can a judgement be made on its effectiveness. This would usually require the monitoring of settlement with buried plates and magnetic extensometers, together with pore pressure monitoring using standpipes and piezometers.

### In Situ Soil Column Improvement

The technique of in situ column improvement involves the injection or mixing of additives to the soil to improve its performance under load. The two main methods in this group are jet grouting, and lime columns. These techniques are less frequently used in reclamation construction, because of their specialist requirements.

Jet grouting involves the mixing and partial replacement of the in situ soil with cement slurry injected under very high pressure. A grout tube is installed into a hole either pre-drilled, or self bored to the required depth. The jetting operation involves the grout being pumped under high pressure through jets at the base of the grout tube which are aimed in a horizontal direction into the soil layer. Where a cylindrical grout body is required, the grout tube is rotated during the grouting and withdrawal. Tabular shaped bodies can also be formed by halting the rotation. Jet grouting is ideally suited to granular soils, but can be successfully used in a fairly wide range of soil types however cobbles and larger sized particles significantly effect the shape of the grout body, and purely cohesive soils tend to substantially reduce the jet penetration and hence the size of the grout body. Jet grouting is a relatively expensive ground treatment method that is not often applied in reclamation construction, it is used more frequently in other specialised applications.

Lime columns are constructed by deep mixing unslaked lime ($CaO$) with soft clay or silt layers. The effect of this mixing is to cause a change in the soil structure and chemistry, giving it increased strength and permeability. The lime column is constructed using a powerful top-drive drill rig or auger rig, on which is mounted a hollow kelly with a mixing head. After a hole is drilled to the required depth, lime is blown down the kelly and is well mixed with the soft clay/silt while the head is gradually withdrawn. A column of lime improved soil is constructed through the soft layer, which will have (depending on initial condition) an improvement in shear strength of 10 to 50 times [5], and significantly improved drainage characteristics. The treatment is not applicable to all clay types, and soils with a high organic content are unsuitable. This technique has not seen wide application in reclamation construction, but with more exposure it may see an increased utilisation

## THREE HONG KONG CASE HISTORIES

Three case histories of recent reclamation projects are briefly presented in the following section. Each case offers some insight into the way the ground treatment techniques have been used in reclamation construction in Hong Kong, and a few key features are noted for each.

### The Third Industrial Estate - Tseung Kwan O

With available building sites at a premium, the Hong Kong Industrial Estates Corporation was allocated an area in the planning of Tseung Kwan O (Junk Bay) development on which to establish Hong Kong's 3rd industrial estate. The area of approximately 40ha, was to be reclaimed adjacent to the eastern side of Tseung Kwan O. The soil profile beneath the seabed was found to be fairly typical of the near shore sequence in Hong Kong, and can be characterised as a very soft to soft, near normally consolidated marine silty clay ("marine mud") of 7 to 15m thickness, underlain by a firm to stiff overconsolidated alluvial clay of approximately 5m thick, over a weathered granite profile grading to solid rock.

Several key conditions were set as part of the design for the reclamation construction as follows:

- The soft marine silty clay was to be treated in situ as part of the reclamation construction.
- The consolidation settlements should be substantially complete on hand-over
- The tight programme for construction was to include an early completion of part of the site.
- A resource of suitable sand fill sourced from the seabed in nearby Tathong Channel had been identified, and would be allocated by the Hong Kong Government to the project.

The soft marine mud was found to have a coefficient of compressibility of approximately $C_c = 0.6$, and a coefficient of consolidation for vertical drainage of approximately $c_v = 1.2$ m$^2$/year. These properties indicated that the marine mud layer would undergo a large amount of compression, which would take a long time to complete. Clearly treatment with vertical drains was required. These were assumed in the design to be the synthetic type band drain and would be installed as soon as possible after the reclamation fill rose above sea level. The design specified drains installed on a triangular grid with a spacing of 1.5m. In order to provide time for roads and drains construction in the early hand-over area, the allowable time for consolidation was restricted to 12 months, and surcharge embankments placed over these services areas were included in the construction programme. Since a further 6 months was scheduled for the roads and drains work, the consolidation in the remaining part of the early hand-over area was able to catch up to the surcharged areas. The rest of the reclamation which had a scheduled completion at the end of the contract, was treated with vertical drains but not surcharged. The justification for not pre-loading the building areas was the design assumption that all buildings would be piled and hence not increase the overburden pressure on the marine mud.

As with most reclamations designs involving ground treatment measures and staged sectional completion of the works, the construction strategy adopted for the Third Industrial Estate had to be tailor-made to suit the specific requirements of the reclamation, and the assumptions made about the consolidation behaviour of the clay had a direct impact on the construction programme, as well as the expenditure on ground treatment. It can also be seen that the geotechnical engineer had to be aware of the site-specific construction and planning issues, and had to address these in a realistic programme of construction activities in order to arrive at a workable reclamation design.

### The Island Reclamation for the HKCEC (Wanchai Reclamation Phase I)

The existing Hong Kong Convention and Exhibition Centre (HKCEC), which was built in the late 1980's, has proved to be a very successful venue for conventions and exhibitions, and within a short time after opening, a requirement for additional exhibition hall space was identified. This expansion was in line with the long-term planning for a large reclamation spanning the Central and Wanchai districts on the northern coast of Hong Kong Island, however the HKCEC's need was immediate and the Central & Wanchai Reclamation had not passed the planning stage. Therefore an island reclamation, that would be incorporated into the future Central & Wanchai scheme was proposed, and approved by the Hong Kong Government.

The island reclamation was to be located in Victoria Harbour a short distance offshore from the existing waterfront promenade in front of the HKCEC. A series of drillholes and cone penetration tests (CPT) revealed that ground conditions beneath the seabed included a varying sequence comprised of very soft marine mud (similar to that found in Tseung Kwan O, discussed above) , over marine and alluvial sands and alluvial silty clay, which in turn were underlain by a weathered granite profile grading to bedrock. The ground investigation showed that the thicknesses and lateral extents of the various surface layers in the sequence varied, and this was thought to be due to the more turbulent water currents in this fairly narrow section of the harbour. This variability of ground conditions required rather more care to be devoted to the geotechnical aspects of the reclamation design.

The reclamation was to be purpose built for a low-rise exhibition hall building of some 3 to 4 storeys, which would occupy most of the reclamation area. The only other occupant of the reclamation would be a ferry pier and its associated bus access and terminal, both with only minor infrastructure requirements. The geotechnical design assumed that the exhibition hall was to be a piled structure with a single level basement slab suspended off the structure. This configuration of building meant that no ground treatment measures were required beneath the building, however the services connections and the on-grade access road would require a careful assessment of the consolidation behaviour of the sub-reclamation silty clay layers, and also of the settlement of the reclamation fill itself.

Since by good fortune, the thickness of the soft marine mud was generally found to be less than 2m beneath the reclamation area outside the building footprint, consolidation settlements were considered unlikely to cause problems for the on-grade roads and services. Thus ground treatment to accelerate the consolidation using band drains was considered unnecessary. However the settlement of the hydraulically placed sand-fill was recognised as a potential problem for these structures. Vibratory compaction was therefore specified to be carried out on a grid to cover the whole area of reclamation not within the footprint of the future exhibition hall building. The spacing of the grid for vibratory compaction was to be defined by site trials, with the results confirmed by before and after CPT soundings. Vibratory compaction was chosen for its well demonstrated ability in improving the density of the marine sand fill, both above and below the water table.

This project illustrates that with the good communication between the geotechnical and structural engineers associated with this project, significant (and expensive) extra ground treatment was avoided. The key assumption used in the geotechnical design, which was subsequently adopted in the structural design, of a piled building structure with suspended basement slab, meant that most of the reclamation, which incidentally included the thickest layer of problem soil, was able to be left untreated.

West Kowloon Expressway/West Kowloon Reclamation

The West Kowloon Reclamation (WKR) is a major new reclamation project that was commenced in the early 1990's on the western shore of the Kowloon peninsula. The primary planning function of the new reclamation, was to provide a much-needed transport corridor, combining the West Kowloon Expressway and the Lantau and Airport Railway mass transit rail link. These transport links, were both included in the Hong Kong Government's Airport Core Programme (ACP), which is the overall project management scheme which was set up to control programming for all the key infrastructure projects associated with the construction of Hong Kong's replacement airport.

The reclamation works for the WKR was split into several different construction contracts which were each to be completed at different stages. This approach was necessary from a construction management view, due to the sheer size of the whole reclamation, and also because of its location next to one of the worlds most densely populated urban areas. In order to meet the tight construction programme that involved a complicated scheme of sectional completion, and to accommodate a variety of different major structures, the WKR construction works included ground treatment programmes that resembled a patchwork quilt. The soft marine mud on the seabed, was either dredged out or left in place to be subsequently treated with vertical drains, depending on the time available for consolidation. The application of pre-loads using surcharge fill was scheduled in many parts of the reclamation, whilst certain areas of the reclamation fill were treated with vibro compaction, to avoid the later settlement of the sand fill. With all these activities being carried out at different times and places in different parts of the site, problems would appear to be inevitable. Through good project management, these were largely avoided.

In one section of the reclamation works, where partial dredging of the marine mud had been specified in the design, the sand fill had been contaminated with lenses of the soft marine mud during placement. This phenomenon is not unusual where sand fill is placed on the marine mud, and is largely due to a displacement of the mud by the sand. The problem is often referred to as mud waves, due to the appearance of wave-like mounds of displaced mud near the leading edge of a reclamation. Part of the West Kowloon Expressway was located in this section of reclamation, and the presence of mud lenses was discovered during the vibro compaction works associated with the expressway construction contract. The vibration of the vibroflot caused liquefaction of the sand/mud mixture, and the compaction treatment was rendered ineffective in those areas. A programme of CPT soundings was instigated and these confirmed some contamination of the reclamation fill. The reclamation area containing the mud contamination was to accommodate the expressway on-grade, together with multi-cell drainage culverts, and major fill embankments for several of the expressway slip-roads, and the remedial treatment of these lenses appeared necessary to avoid future settlement problems. A combination of vertical drains and surcharge treatment was assessed, with an alternative of stone columns installed using the vibro replacement method. The former treatment would require a massive mobilisation of drain installation plant, and the time taken for drain installation, and the placing, holding and removal of the surcharge mounds, would cause serious project delays. With the stone column option, some of the requisite plant was already on site, and additional vibroflots could be mobilised if required. Field trials had been carried out on the site, and the performance of these indicated that this would be an relatively expedient method for treating the problem areas, but would likewise cause delays. As might be expected, the subsequent discussion of the project delays resulting from either of these remedial measures was not well received by the ACP project managers, and further investigation into the nature and expected behaviour of the mud lenses was investigated using vibrocore sampling. This further investigation showed that the mud lenses were rather more sandy, and far less extensive than had been previously thought, avoiding much of the expensive ground treatment that had earlier been considered. To the relief of all those involved!

This case history illustrates how ground treatment may be used in reclamation construction, in a variety of ways, including remedial works. It also demonstrates the importance of care in the filling operation, and also highlights a slightly grey area on a CPT interpretation chart, namely identifying the true nature of soft/loose sand/silt/clay mixtures.

## CONCLUSIONS

The modern techniques of ground treatment allow reclamations to be rapidly built and to a high finished standard. The construction environment that exists in Hong Kong illustrates this very well. However, it is seen that when the reclamation construction is completed under a tight programme, a high quality of geotechnical investigation data is essential, otherwise there is a very real risk of significant and expensive delays. To arrive at a workable and efficient reclamation design including ground treatment works, the geotechnical engineer needs to be aware of a broad scope of planning and construction issues.

## REFERENCES

1 Hong Kong Geological Survey. 1986. *1:20,000 Solid and Superficial Geology Sheet 11, Hong Kong & Kowloon*. Series HGM20.

2 Greenwood, D A and Kirsch, K 1983. Specialist Ground Treatment by Vibratory and Dynamic Methods, *Piling and Ground Treatment - Proceedings of the international conference on advances in piling and ground treatment for foundations. ICE London March, 1983.* Thomas Telford Ltd., London.

3 Brown, R E 1977. Vibroflotation Compaction of Cohesionless Soils, *Journal of the Geotechnical Engineering Division*, ASCE Vol. 103, No. GT12, 1437-1451.

4 Mitchell, J K and Katti, R K 1981. *Soil Improvement State-of-the-Art Report*, Proc. 10th Int. Conf., ICSMFE, Stokholm 1981

5 Broms B B 1993. Lime Stabilisation, *Chapter 4 - Ground Improvement Ed. M P Mosely*, Blackie Academic & Professional, Glasgow 1993

# THE GEOLOGY OF THE QUEEN STREET AREA, AUCKLAND, NEW ZEALAND

A.K. GEORGE

WORKS Consultancy Services Ltd
9th Floor AA Building, PO Box 5848, Auckland, New Zealand

## SUMMARY

The geology along Queen St Auckland, consists of underlying Miocene marine deposited flysch and associated soils. This material is overlain by Quaternary volcanics, alluvium and recent fill, all of variable geotechnical character. Of geological importance is a lava flow which flowed down the paleovalley underlying present day Queen St, from the corner of Victoria St east. The original paleovalley gives clues as to the extent of this flow however more information is required to determine its terminal extent north and lateral extent east. The course of the Ligar stream last century is useful to determine the extent of reclamation.

## INTRODUCTION

The Queen St area of the Auckland central Business district is econmically important to the wider Auckland region. The geolgy tha t underlies Queen St is in places complex. To be able to interpret the information found from limited site investigations it is important to have an understanding of the wider geology that is present.

This paper presents a summary of the geology along Queen St, highlighting areas where information is sketchy, and where further research is required to build a definitive picture of the geology.

## GEOLOGICAL SETTING

The geology of the area around Queen Street Auckland, consists of an underlying "bedrock" of Miocene-aged marine deposited sandstones and mudstones of the Waitemata Group, with an overlying layer of associated weathered soils. Much of the original geomorphology has been modified and is currently covered with buildings. What can be deduced from available information is that before Quaternary volcanism, the area corresponding to what is now the central business district (CBD) featured moderately elevated hills comprised of Waitemata Group rock and residual soils. Slopes were steep and the terrain was dissected by numerous small streams in deep valleys, including what was to become known as the Ligar Stream. The Ligar Stream flowed north down the "Queen St gully", a paleovalley that runs beneath the corner of Queen and Wellesley St's at RL -1m (Datum M.S.L today) and passes beneath Customs St between Fort Lane and Queen St at approximately RL-10m, and crosses below Queens Wharf in a northwesterly direction at RL-14m, to eventually join the ancestral Waitemata River bed[1] at Rl -30m beneath the Waitemata Harbour (Figure 1).

Approximately 60 000 yrs. ago the Albert park volcanic centre erupted explosively onto higher ridges east and south of the centre, depositing ash, lapilli, and ash/lapilli tuff. Later a small lava flow from this centre flowed down Victoria St East passing northwest under the corner of the AMP building. The flow then travelled northwards down the Queen St gully. The western edge of the flow has been traced passing under buildings along the western side of Queen St as far as the southern corner of Queen and Wyndham St's[2] where the flow passes back to the northeast under the western edge of Queen St following the old "Queen St gully" towards what is now the Fort Lane end of Fort St.

The flow itself although relatively short, is thick (10-12m at the corner of Durham and Queen St's), and effectively dammed the Queen St gully. Behind the lava dam, reworked ash/lapilli and tuff materials were deposited with other alluvial sediments and vegetation washed in from the hill slopes behind, building an alluvial flat that extended upstream as far as Myers St today. Subsequently the Ligar Stream continued to flow along the western edge of the lava flow, issuing from the coastline at the corner of Fort and Queen St's onto marine mud flats.

Last century extensive reclamation of bays along the foreshore of Auckland resulted in fill material being emplaced from Fort St (formerly Foreshore St) out to Quay St. This fill derived for the most part of reworked local Waitemata Group materials (soil and rock) directly overlies insitu marine sediments as far as Custom St East. Below QEII Square a combination of marine mud and hydraulic fill was emplaced upon Waitemata Group Rock and capped with the same Waitemata Group derived fill material as found further up Queen St.

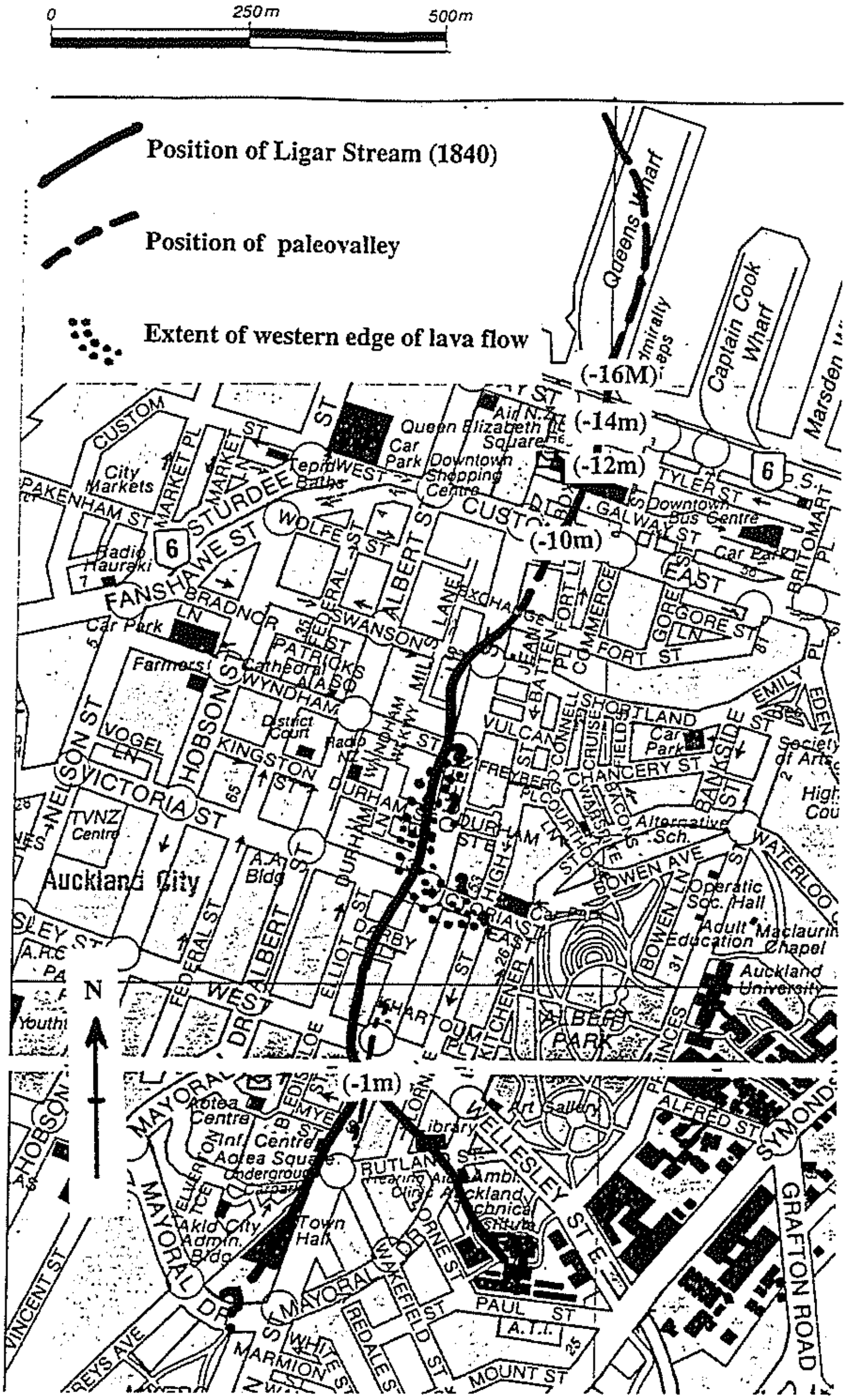


Figure 1: Sketch plan showing the bottom of the "Queen St gully" paleovalley (dashed line) in relation to present day streets with approximate RL depths in brackets (datum is the mean sea level today). The solid line represents the position of the Ligar Stream last century which was displaced from the paleovalley by the lava flow approx. 60 000 yrs BP (shown in dots), and which flowed onto marine sediments at the corner of present day Queen and Fort St's.

GEOLOGY ALONG QUEEN STREET

A geological long-section has been plotted along a section-line drawn from the old postoffice on QEII square, up Queen street to approximately 200m southwest of Mayoral Drive (Figure 2).

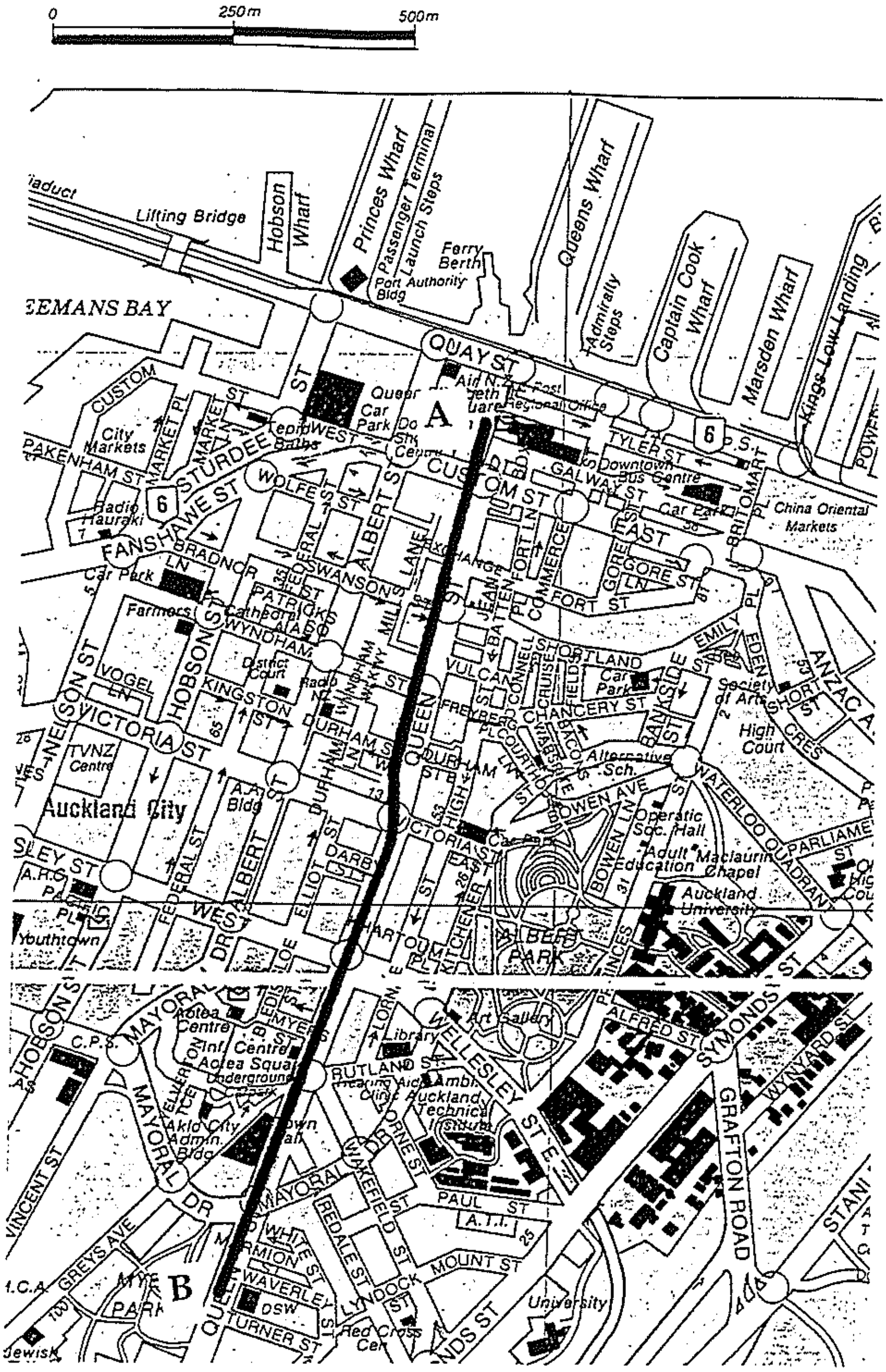


Figure 2: A plan showing the section-line, represented by the solid line, from which is drawn the the longsection. It is plotted from the old postoffice on QEII square up Queen St to approximately 200m southwest of Mayoral Drive.( see figure 3).

Geological information plotted on the long-section and the summary of the nature of the material, is based on a general knowledge of these materials, and drillhole information from the data files of WORKS Consultancy Services Ltd (WCS) and the Auckland City Council (ACC). Ground conditions at each position are inferred and extrapolated from this existing data drawn from specific projects at isolated localities along Queen St. Of note is the likely lateral variation of ground conditions across Queen Street which cannot be assessed from this study. Specific investigation is required to verify any extrapolation, especially any lateral variation.

**0-100m:** We encounter 3-8m of very soft-firm clay and medium dense sand (hydraulic fill), overlaid by 3-4m of very soft to firm clayey/sandy SILT. Below the hydraulic fill is 3m of soft-firm sandy clay, silt, and clay (marine sediments), overlying 1-3m of firm-stiff clayey sand, silt and clay (HW-CW Waitemata Group) and Waitemata Group rock.

**100-250m:** We find soft-firm clayey sand, silt and clay (marine sediment), overlying 0.2-3m of firm-stiff clayey sand, silt and clay (HW-CW Waitemata Group). At 250m Marine sediments directly overly very weak-weak interbedded mudstone and sandstone (Waitemata Group Rock) before pinching out against weathered Waitemata Group soils.

**250-425m:** We find stiff-hard clayey sands, silts, and clay (SW-HW Waitemata Group rock) 0-8m thick. Underlying this is Waitemata Group rock.

**425-625m:** We pass through weathered basalt consisting of loose-medium dense scoria/basalt fine-coarse gravel. Below this from 0-3m is moderately strong/strong highly fractured basalt, to approximately 19m below the surface at 485-490m metrage(RL -10m). The basalt overlies Waitemata Group rock.

**625-700m:** We have 1-2m of stiff sandy, clayey SILT (alluvium and reworked tuff) overlying stiff sandy SILT, clay, fine scoria, basalt gravels, and peat (alluvium and reworked volcanics). This material sits above firm-stiff clayey sands, silts, clays of HW-CW Waitemata Group soil which in turn overlie Waitemata Group rock 3-5m underneath.

**700-975m:** We encounter 3m of firm-stiff silts and clays (ash/tuff), overlying alluvium consisting of firm-stiff sandy silt, clayey sand, clay, reworked basaltic gravels and rare thin layers of organic clay and peat. From 850-975m the alluvium directly overlies Waitemata Group rock.

**975-1275m:** We find firm silts and clays (weathered ash/ash tuff) becoming shallower in depth (6-1m)approaching Mayoral Dr. This sits on top of approximately 7m of firm-stiff clayey sands, silts and clays (HW-CW Waitemata Group). While below this lies Waitemata Group rock.

## AREAS REQUIRING FURTHER RESEARCH

No information was made available to enable determination of the eastern boundary of the lava flow or to determine the position at which the lava flow terminates. More information on the boundaries of the paleo-valley down which the lava flowed may give an estimate of the dimensions of the flow in addition to borehole information along the eastern side of Queen St and other streets immeadiaetly to the east. Of further interest may be the influence the lava flow has on the local hydrogeology, as no information of this nature was available.

## CONCLUSION

The geology of the Auckland central business district is complex. It includes a wide variation of both rocks and soils. Rocks range from soft weak sandstone s and siltstones to moderately strong basalt. Soils are soft-very stiff sands, silts and clays. Understanding the spatial relationship of each material to the other is tied in with an understanding of the local geology at each individual site. An understanding of the overall geology will provide an important framework in which to plan an appropriate site investigation.

## ACKNOWLEDGEMENTS

WORKS Consultancy Services Ltd were commissioned to undertake a desktop study summarising the geology of the Auckland central business district in view of a proposed light rail network servicing this area. Information was gathered from WORKS Consultancy Services files and Auckland City Council records. Of this latter resource it is acknowledged the contribution of other engineering consultancy companies over the years.

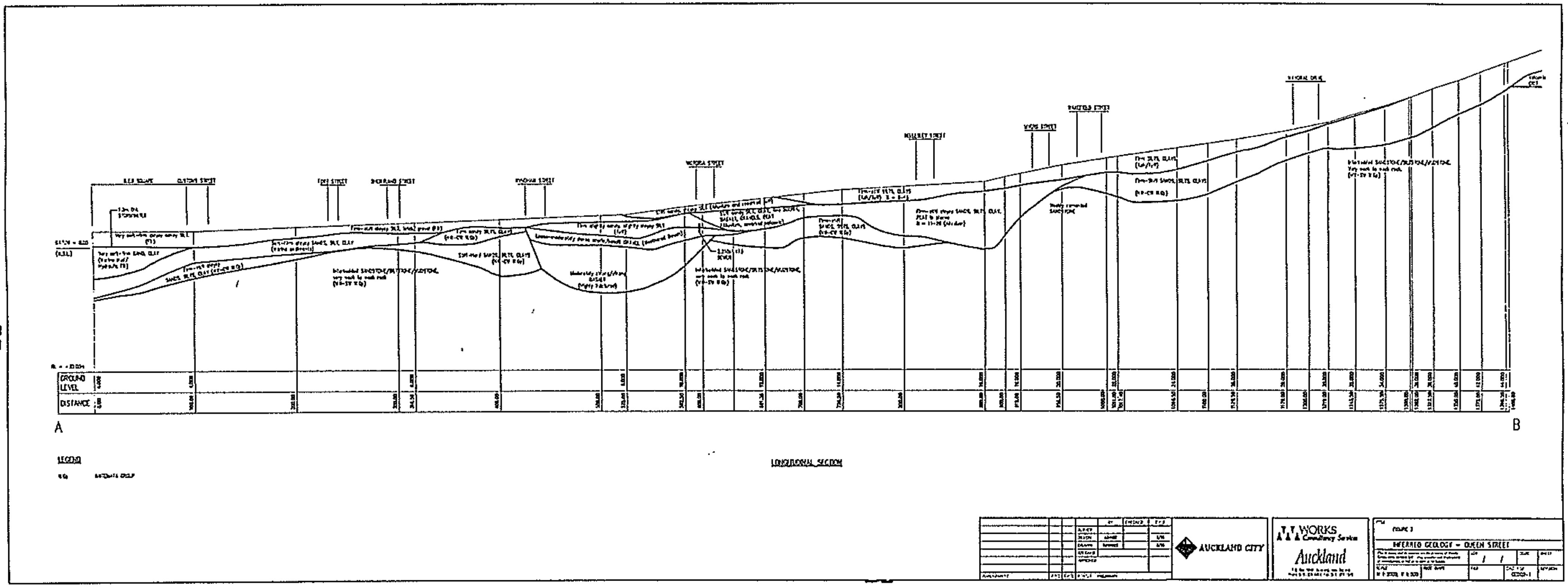


Figure 3: Geological longsection along Queen St. Geological information is based on a general knowledge of these materials and drillhole information from the data files of WORKS Consultancy Services and Auckland City Council. Boundaries are inferred and extrapolated from this existing data which was from specific projects at isolated localities. Of significance in this context is the likely lateral variation of the ground conditions across Queen St. Specific investigation is required to verify this extrapolation.

REFERENCES

1 Searle, E J 1964. *City of Volcanoes - A Geology of Auckland.* Paul's Book Arcade.

2 Taylor, PW, Callander, RA, Entrician, GC, Tonkin, RM, Beca, GS and Taylor, PW 1967. *Building Foundations in Auckland City.* Fifth Australia - New Zealand Conference On Soil Mechanics and Foundation Engineering.

# GEOTECHNICAL ISSUES ASSOCIATED WITH METHANOL STORAGE TANK DESIGN

K.M. HILLS

Works Consultancy Services Ltd Hamilton

## SUMMARY

When completing a geotechnical assessment of a tank site, the importance of observation and factual data should be realised. Data from adjacent tank sites can give a more representative picture than data from laboratory testing of how the tank will perform, especially when considering tank settlement. One dimensional oedometer test data often gives conservative data for settlement estimation, leading to over conservative foundation design. This is especially true for over consolidated soils.

## INTRODUCTION

Two steel tanks were constructed for Methanex New Zealand Ltd for the purpose of methanol containment on a site comprising complex soils of volcanic origin. The tank site was selected by Methanex and Works Consultancy Services Ltd were commissioned to provide a geotechnical assessment and design the foundations and civil works for the site. The primary geotechnical issues considered as part of the project were:

- Bearing capacity
- Settlement
- Methanol containment

## SITE DEVELOPMENT

Two steel methanol storage tanks, 58m in diameter and 17.5m high were constructed in September 1994 at the southern end of the Omata Tank Farm in New Plymouth (see location plan, Figure 1).

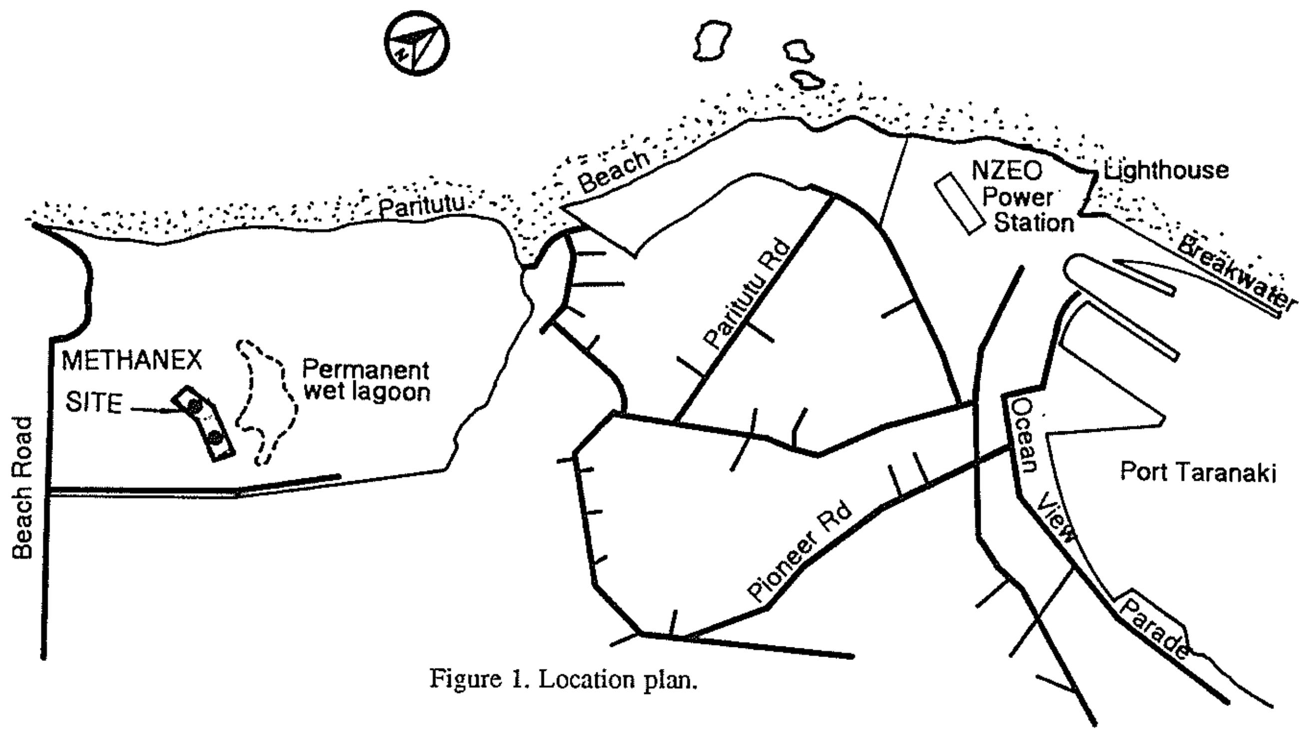


Figure 1. Location plan.

Under normal operational conditions there is a fluctuating level of Methanol in the tanks, up to 13.5m deep. The tanks were filled with water to 15.8m above the tank base during tank commissioning.

Construction of the foundation platform for the two tanks involved cut to waste, with minor excavation on the north side of the site rising to about 5m on the south side. The tank foundations comprise a concrete annular ring beam placed on a ring of compacted fill. The tanks are constructed over a HDPE liner and are surrounded by a 3m bund for the purpose of methanol containment in the case of tank leakage.

## SITE INVESTIGATIONS AND LABORATORY TESTING

Investigations completed at the Methanex site comprised the following:

- Two boreholes to a depth of 45m
- Sixteen Cone Penetrometer Tests (CPTs) to refusal
- Three test pits excavated to a depth of 3.6m

Standpipe piezometers were installed in Borehole 1 and CPTs 2 and 12. The positions of the site investigations completed at the Methanex site are shown on Figure 2. Laboratory testing of samples taken from Borehole 1 and Test Pits 1 and 2 was completed.

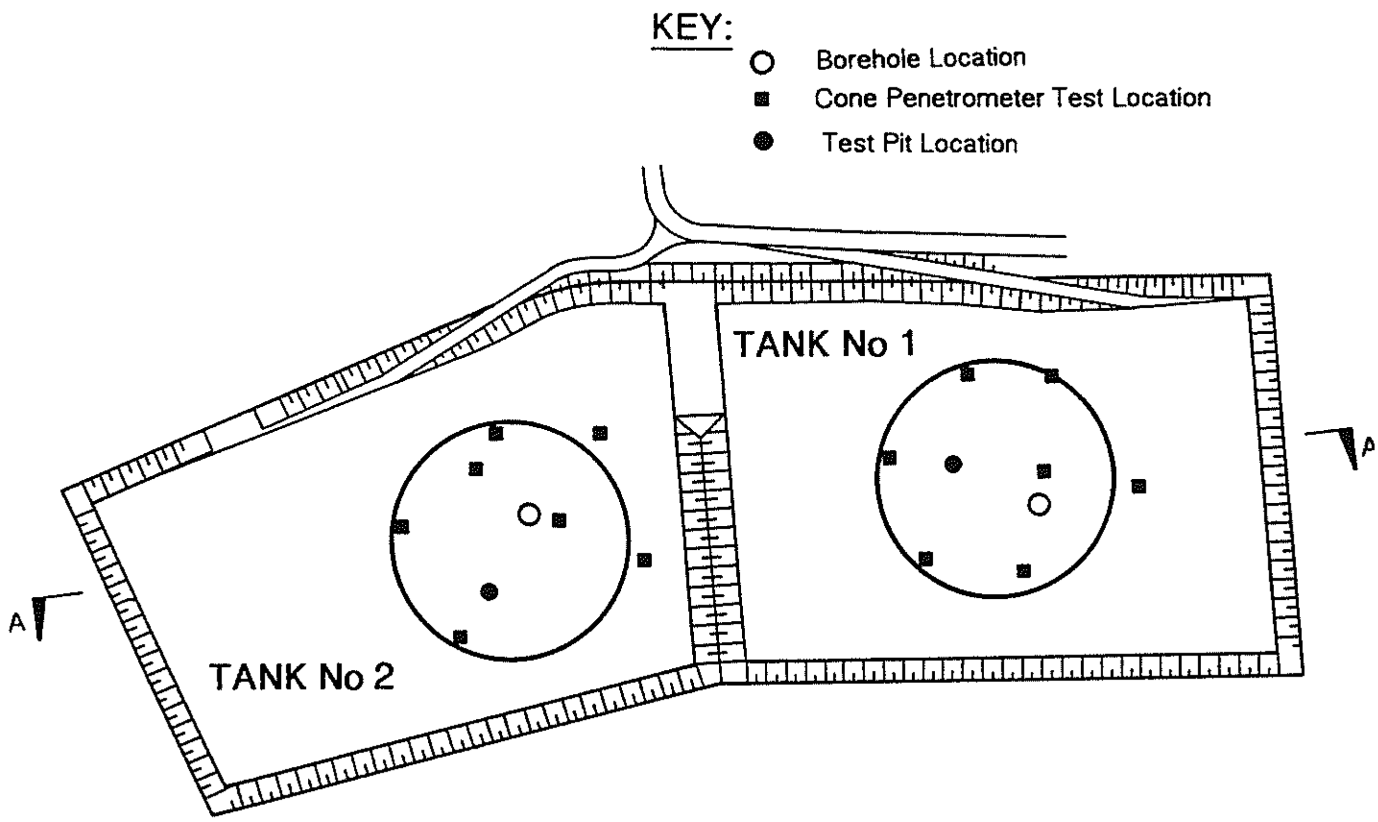


Figure 2. Site plan.

Results from site investigations and laboratory testing for an adjacent tank in the Omata Tank farm were used for the Methanex tank assessment.

## SITE DESCRIPTION

The Omata Tank Farm is situated on the northern flank of Mt Egmont, south-west of New Plymouth City (Figure 1). Prior to site development the Methanex site was grassed, and sloped at a grade of approximately 1 in 20 to the north.

## GEOLOGY

Material covering the site is summarised as follows:

| | |
|---|---|
| 0 - 20m | Ash deposits (a soft SILT layer was identified at 8m to 10m) |
| 20 - 37m | Variable gravels and organic sand silt mixtures |
| 37 - 45m | Boulders and cobbles in a gravelly silty sand matrix (Laharic agglomerate) |

Within these primary material units there is some variability as demonstrated in a typical cross section through the site presented as Figure 3. A soft SILT layer which exists at a depth of 8m to 10m was identified within Borehole 1 and CPT testing indicated the layer was persistent across the site.

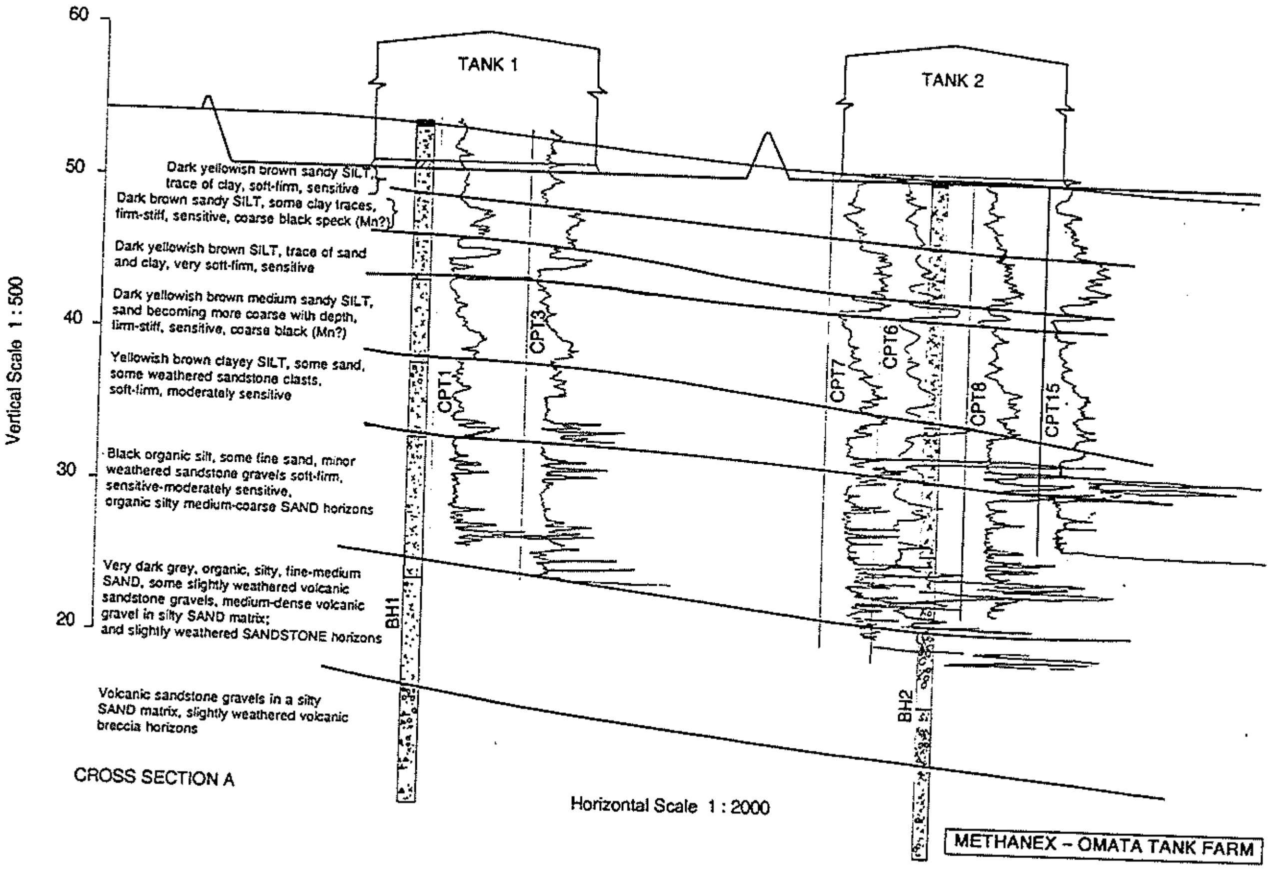


Figure 3. Cross section A-A.

The ash deposits at the site have been correlated to the New Plymouth ash group and are considered to have their origin from the Pouakai Range, to the south of the site. Generally they can be described as an orange-brown, coarse andesitic ash with a predominantly clayey silty sand texture.

A summary of material properties is given in Table 1 below.

Table 1. Summary of Material Properties

| Depth (m) | Material description | Shear strength (kPa) | Atterburg results | | | | Oedometer results for stress range of $\sigma_o$' to $\sigma_o$'+160kPa | Permeability (m/s) | |
|---|---|---|---|---|---|---|---|---|---|
| | | | w/c | LL | PL | PI | | Water | Methanol |
| 0 - 8 | Volcanic Ash | 100 (min) | 80 | 90 | 65 | 30 | $m_v$ = 0.072 - 0.17$m^2$/MN<br>$c_v$ = 62 - 78$m^2$/yr | 2.3x$10^{-8}$ | 3.1x$10^{-8}$ |
| 8 - 10 | Volcanic Ash (soft SILT layer) | 80 (min) | 100 | 110 | 55 | 50 | $m_v$ = 0.3 - 0.4$m^2$/MN<br>$c_v$ = 17 - 25$m^2$/yr | | |
| 10 - 20 | Volcanic Ash | | | | | | $m_v$ = 0.058 - 0.082$m^2$/MN<br>$c_v$ = 27 - 44$m^2$/yr | | |
| 20 - 37 | Variable gravels and organic sand silt mixtures | | | | | | $m_v$ = 0.16 - 0.27 $m^2$/MN<br>$c_v$ = 58 - 72$m^2$/yr | | |

## BEARING CAPACITY

Bearing capacity was assessed using the method of Duncan and D'Orazio [4] to check for bearing capacity failure modes under the base and edge of the tank. An average undrained shear strength value of 80kPa was adopted for all the soils underlying the tank. Edge shear was found to be critical. The following Factors of Safety for edge shear were given by analysis:

| | |
|---|---|
| Product loading (107kPa) | = 4 |
| Hydrotest (158kPa) | = 2.7 |
| Seismic loading (210kPa) | = 2 |

The Factors of Safety given all satisfied named engineering criteria and the clients brief.

A total stress analysis using Bishops Method was used to check stability of a circular failure surface associated with the tank loading. The proposed levelling of the site meant that the depth of the soft SILT layer would vary across the site. Two situations were analysed:

Soft layer 5-7m under Tank Base
Soft layer 3-5m under Tank Base

Shear vane testing within this soft layer, completed during the drilling of Borehole 1, gave an undrained shear strength value of 82kPa. A correction for the plasticity index, as given by Bjerrum [2] for this layer (PI = 50 as given by laboratory testing), was then applied, reducing the undrained shear strength to 64kPa.

For the material overlying the layer, an undrained shear strength of 80kPa was used. This value was adopted conservatively from shear vane results within the overlying layer, where refusal was met in all tests. The hydrotest loading of 160kPa (16m water head) was applied with a minimum Factor of Safety of 2.5.

An effective stress analysis was also performed using Bishop's Method for long term loading using effective stress values given by triaxial testing. Soil parameters of effective friction angle of 37° and effective cohesion of 0kPa were adopted for the analysis which gave a Factor of Safety greater than 5.

## SETTLEMENT

Estimated settlement at the site was assessed on the basis of oedometer testing, CPT interpretation, and field observations from an adjacent tank. Settlement was calculated for both the hydrotest and product loading under the centre of the tanks and under the tank edges. Settlement was calculated using (lower bound) coefficient of volume compressibility values ($m_v$) given by oedometer testing at representative stresses for each layer. Six layers were adopted coinciding with representative oedometer test results for each layer ($m_v$ values varied between $1x10^{-7}$ and $8x10^{-8}$ $m^2/N$). The method described by Duncan and D'Orazio [5] was used to calculate total settlements assuming full consolidation under sustained loads.

To assess the differential settlement potential around the edge of Tank 1 and Tank 2, the method of Belshaw [1] was used to calculate settlement using CPT data. The effect of a possibly compressible organic layer indicated by borehole data was used to adjust settlement predicted by Belshaw (ie difference between settlements for CPT 1 calculated using Belshaw and settlements calculated using oedometer results for Borehole 1, situated adjacent to CPT 1).

This method of calculating settlement assumed that the organic layer below 30m (limit of CPTs) is consistent across the site which is indicated to be a fair assumption by Boreholes 1 and 2. This method also assumed that settlement calculated using one dimensional Terzaghi consolidation theory using oedometer results is more or less equal to total settlement as given by Burland et al [3].

Measured settlement values at an adjacent tank during a month long hydrotest period (waterhead of 16m for 11 days) were much smaller that those calculated for the tank. The actual observed settlements for this tank were approximately 35% of the settlements calculated to occur during the hydrotest and approximately 55% of the settlement calculated to occur during the product loading. Survey information also demonstrated that total settlement did not alter significantly (±3mm) over the 8 year period after the end of the hydrotest. It was anticipated that similar behaviour would occur at the Methanex tank site.

Test results for the soils underlying the Methanex tank site demonstrated very low permeability and a slow rate of consolidation. The model adopted for the analysis assumed an average value for $c_v$ of $42m^2$/year over the entire layer, a 20m compressible layer and one free draining boundary. The calculated time for consolidation was 8 years. If the layer was assumed to be free draining from both boundaries (eg if a gravel blanket was placed under the tank) the time for consolidation was calculated to be 2 years. Total settlements calculated during hydrotest loading were not expected to occur during the hydrotest period, as the period of testing was not long enough for the soil to consolidate this amount. Total settlements calculated for product loading were therefore more appropriate.

The product loading over time is a variable which was not defined by Methanex. Because of this it was necessary to design on the basis of continuous full product loading. As the site will not be fully loaded at all times, predicted product settlements were also not expected to occur. This behaviour has been observed at the adjacent tank where surveying indicated no significant settlement during product loading over the 8 years since hydrotesting. 60% of the settlements calculated under product loading were therefore recommended by WCS for design purposes.

Actual settlements have been monitored at the two Methanex tanks using eighteen survey points and three settlement tubes at each tank. Actual maximum settlement monitored are summarised in Table 2 below, along with design settlements:

Table 2. Actual settlement for the Methanex tanks.

| Tank | Actual Tank Centre Settlement | Design tank centre settlement | Actual tank edge settlement | Design tank edge settlement | Maximum settlement |
|---|---|---|---|---|---|
| 1 | 170mm | 170mm | 15mm | 70mm to 150mm | 191mm (38m across tank) |
| 2 | 70mm | 220mm | 26mm | 130mm to 210mm | 82mm (23m across tank) |

## METHANOL CONTAINMENT

A requirement of the project was that the tank was founded on a platform relatively impermeable to methanol. Site investigations demonstrated that volcanic ash forms a thick, low permeability blanket across the whole Methanex site.

Permeability tests in fixed ring permeameters have shown that the permeability of methanol ($k_{methanol}$) can be $10^3$ greater than $k_{water}$ (Mitchell et al [7]). Similar testing in flexible membrane permeameters indicate $k_{methanol} = 10$ $k_{water}$. It appears that the soil structure changes with methanol as a result of damage/shrinkage of the soil in which situation the fixed ring permeameter test is unlikely to provide a representative result (ie there may be possible leakage along the sample in the permeameter). Methanol concentrations less than approximately 80% have little effect on permeability (Mitchell et al [7]).

Permeability testing (method of Head [6]) completed for the Methanex Project indicated the permeability of recompacted Taranaki ash to water and methanol to be very similar. A sample of Taranaki ash from a depth of 1-2m was firstly tested for water permeability then methanol permeability. The sample was slightly less permeable to methanol and no erosion or deterioration of the soil after methanol permeation was observed.

## DISCUSSION

The most important point gained from the geotechnical assessment completed for the Methanex tank site is the importance of observation and factual data. The tank adjacent to the Methanex site was of similar size and loading and the site investigations demonstrated similar materials types underlying both sites. It was therefore considered appropriate to reduce the design settlements at the Methanex site to a similar level monitored at the adjacent tank site.

It is realised that factual data such as observations from an adjacent tank is not always available when completing a geotechnical tank assessment. The loading pattern of the tank, expected consolidation of the underlying soils, and stress history of the underlying soils should then be taken into account when estimating settlement. Oedometer testing completed for the Methanex site indicated low permeability, overconsolidated soils.

The oedometer test is one dimensional in both strain ($k_o$ conditions) and drainage and this test condition is not strictly applicable to field conditions. Sample disturbance may also lead to laboratory results that are not representative of field conditions.

Laboratory testing completed at the site indicated the thick ash layer overlying the site would provide a robust containment medium for the tank compound and the natural ash permeability satisfied the client's brief. Allowing for some soil damage on exposure to the methanol the expected flow rate through the ash could increase, however the penetration by methanol would be restricted by the need to displace soil pore water (hence the soil permeability with water would dominate) and also by the fact that the deposit of low permeability ash across the site is very thick.

## CONCLUSIONS

1) The prediction of performance in stratified volcanic soils is complex and difficult.

2) Field observations of previous performance can enable significant improvements in the prediction of future performance.

3) Environmental aspects of containment prevent a further challenge in the definition and understanding of geotechnical performance.

## ACKNOWLEDGEMENTS

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## REFERENCES

1 Belshaw T 1973. Use of the deep sounding static penetrometer in the Napier District, New Zealand, NZ Engineering, May 1973: 125-133.

2 Bjerrum L 1973. Problems of soil mechanics and construction on soft clays, Proc. Eighth Int. Conf. Soil Mech. Fdn. Engng, State of the art report, Session 4, Moscow: 109-159

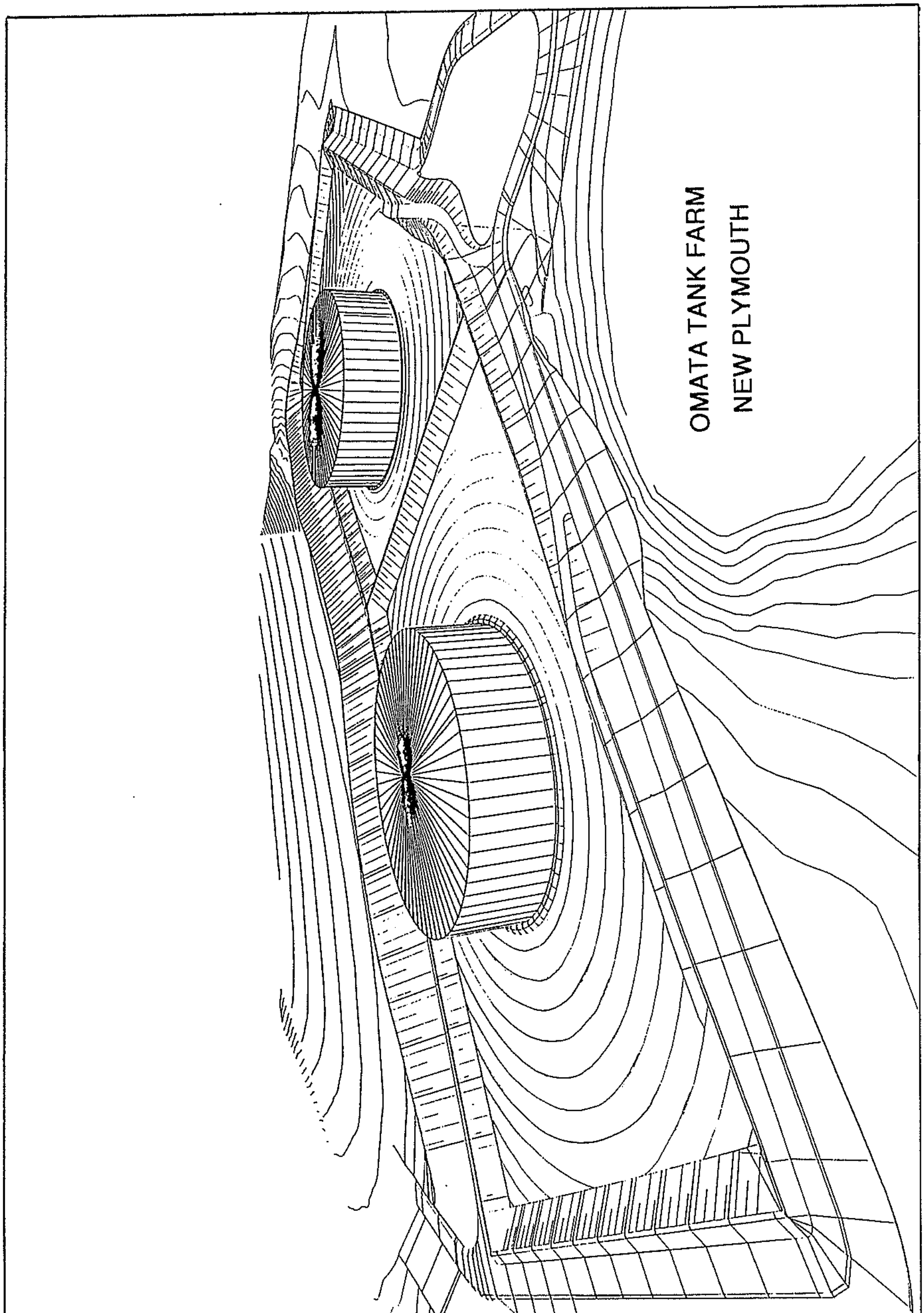
3 Burland JB, Broms BB and DE Mello VFB 1977. Behaviour of foundations and structures, 9th International Conference SMFE: 495-546.

4 Duncan JM and D'Orazio TB 1984. Stability of steel oil storage tanks, ASCE J Geotech Eng, Vol 110 No.9: 1219-1239 and Vol 114 No.3 (1988): 353-356.

5 Duncan JM and D'Orazio TB 1987. Differential settlements in steel tanks. ASCE J Geotech Eng, Vol 113, No.9; 967-983 and Vol 115, No.7: 1032-1035.

6 Head KH 1986. Manual of soil laboratory testing, Volume 3: Effective stress tests, ELE International Ltd.

7 Mitchell JK and Madsen FT 1987. Chemical effects of clay hydraulic conductivity.


OMATA TANK FARM
NEW PLYMOUTH

# BEHAVIOUR OF BORED PILES IN EXPANSIVE CLAY

A. G. KILSBY

Structural Materials and Assemblies Group
University of South Australia, SA, 5095, Australia

## SUMMARY

Four reinforced concrete piles, of the same length and shaft diameter, but having different shaft-soil interface conditions, were constructed in late 1993. The test piles were located at an extremely reactive clay site where the potential surface movement exceeds 120 millimetres. Observations were made, with time and changing moisture regimes, of the soil movements, soil properties, pile movements and pile strains.

Computer modelling has commenced in order to simulate and better understand the pile behaviour. The types of numerical analysis being investigated include an elastic thermomechanical model (that is, no pile-soil slip) and a simplified boundary element model which simulates the soil as an elasto-plastic continuum (that is, pile-soil slip is possible). The paper summarises the experimental observations to date and provides preliminary results of the theoretical modelling.

## INTRODUCTION

Reactive clay soils shrink upon drying and swell upon wetting. Trees, landscaping, and plumbing leaks, in addition to seasonal changes, all affect the moisture condition of the soil. The volumetric changes of the soil can lead to differential foundation movements of lightly loaded structures. Unsightly cracking, jamming doors and windows and structural damage can occur in buildings that have been constructed upon the moving surface. Expansive clays are found in many parts of the world, particularly in semi-arid environments, including Australia, Egypt, Israel, South Africa and the United States. Damage to housing due to differential soil movements is a costly concern in these parts of the world. Foundation engineers are faced with the challenge of designing economical and effective footing systems that can withstand the extreme soil conditions.

Piled foundations are common in areas where the characteristic site soil movement is calculated to be in excess of 100 millimetres. The floor slab is suspended above the soil by piles which are founded at a stable depth as recommended by AS2870 [4]. Due to the nature of expansive clays, bored reinforced concrete piles, sometimes with underreaming, are generally the most economical type of piling. However, the piles must still resist the actions of the surrounding soil and are susceptible to uplift and tensile forces due to the swelling clays.

One type of piled footing being developed is the Tri-Ped footing. The design is based on a reinforced concrete floor slab suspended above the moving soils by three piles. The Tri-Ped has been developed over the past four years by the University of South Australia in collaboration with the South Australian Housing Trust. The SA Housing Trust has used the footing for houses in its normal building program at highly reactive sites. A prototype footing was successfully constructed and test loaded in November 1992 [6]. The prototype house construction was completed in February 1993 after which it was tenanted. Since then twenty eight houses have been built using the Tri-Ped concept. Other piled foundations that have been used include pile and beam with timber floors, and pile-and-beam with concrete floors using lost cardboard formwork or steel tray decking.

As part of the research and development program undertaken at the University of South Australia, methods of isolating the piles from the soil have been trialed. The investigation into the effectiveness of different pile-soil interfaces began in late 1993 with the construction of four piles at a highly reactive clay site.

## THE EXPERIMENT

### Piles

A known reactive site in the north-eastern suburbs of Adelaide, South Australia was chosen to conduct an investigation into pile behaviour in heaving clays with varying soil-pile interface conditions. The site has a characteristic site soil movement of 120mm, and therefore, is classified as an extremely reactive site [4]. Four piles were constructed during November 1993. All of the piles were bored, cast in-situ reinforced concrete, five metres long with a 0.5 metre diameter shaft. Each of the piles consists of different combinations of soil-pile conditions and anchorage (that is, underream) conditions as shown in Figure 1.

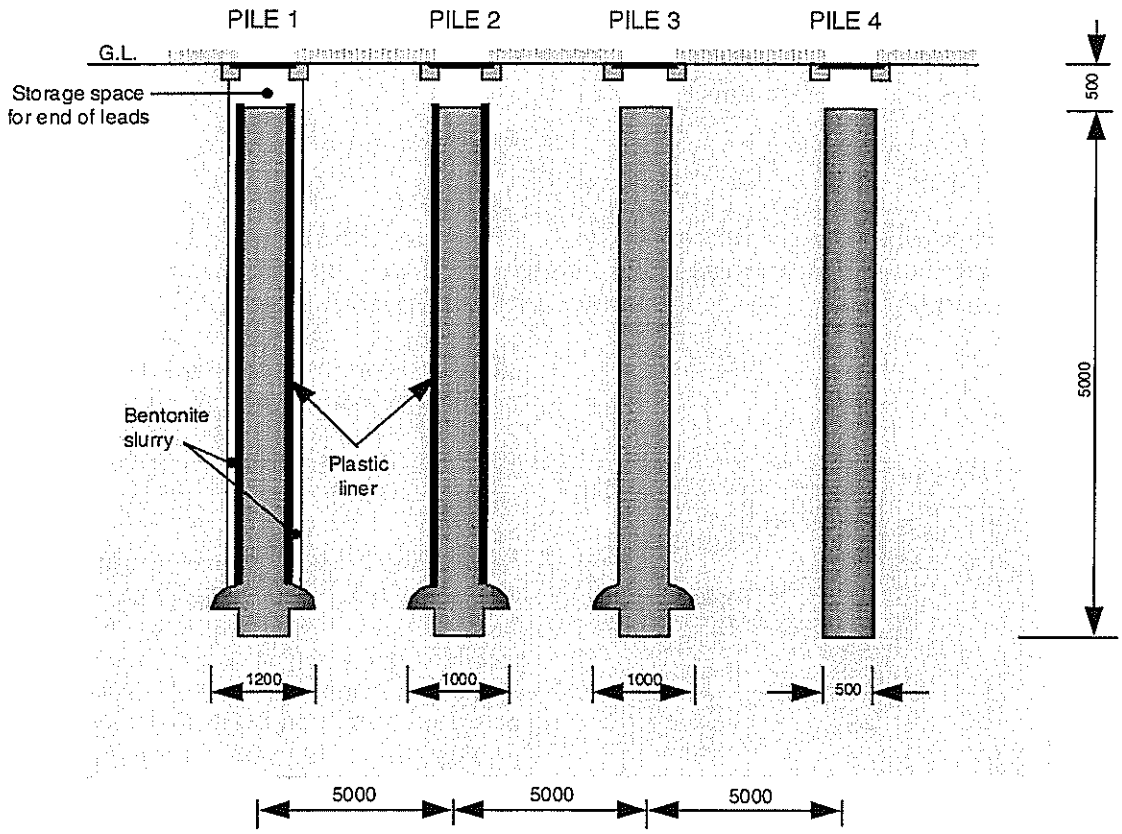



Figure 1 - Elevation of piles

**Pile 1** was designed to be independent from the surrounding soil movements. The shaft of the pile was formed inside a 500 mm diameter smooth plastic liner and a 50 mm annulus of bentonite slurry was placed between the liner and the soil to the depth of the underream. The bentonite slurry consisted of pure bentonite powder mixed with water at a ratio of 1:8 by mass. In the hydrated state, bentonite exhibits negligible shear strength, but also prevents water from migrating to the base of the pile. The liner is a spirally wound, ribbed plastic with the ribs on the inside. This configuration is aimed to eliminate all shaft friction. The borehole of 600 mm diameter was underreamed to form a 1200 mm diameter base. This type of pile was used for the prototype Tri-Ped footing.

**Pile 2** was constructed with the plastic liner directly between the soil and the pile to the depth of the underream with the aim to reduce shaft friction. The borehole diameter was 500 mm and the outside diameter of the liner was 490 mm which resulted in a neat fit against the side of the bored hole.

**Pile 3** was constructed as a standard underreamed pile with no special pile-soil interface. This type of pile was used for all Tri-Ped footings except the prototype.

**Pile 4** was constructed as a standard straight pile with no special pile-soil interface and acts as a control pile.

The piles are reinforced with eight Y20 reinforcing bars, W6 ties at 150 mm centres and 60 mm cover. The steel area of 1.3% of concrete area is significantly greater than the minimum steel of 0.5% of gross concrete area recommended by the piling code [5]. Since the pile with the bentonite surround (pile 1) has no lateral support from the soil, the pile is required to be designed as a column and therefore minimum steel required is 1% of gross pile area. The concrete was specified as compressive strength ($f'_c$) of 20 MPa, 20 mm aggregate and 80 mm slump. The concrete was mechanically vibrated to the full depth of the pile. Laboratory testing showed a compressive strength of 18 MPa at 28 days.

The piles were instrumented with embedded electrical strain gauges in order to monitor the stresses developed in the piles due to the soil movements. Each of the 120mm gauges was precast into a 'dog bone' shaped concrete block to ensure a strong clean bond with the gauge. The blocks were then fixed at the appropriate locations to the prefabricated steel reinforcement cages ready for construction of the piles. Each pile has 12 gauges, that is, two at each of six depths along the length. One of the piles was also instrumented with temperature gauges in order to correct the strain gauge readings for errors due to temperature changes. A dummy gauge next to each of the piles, wrapped in compressible rubber foam, was also buried at the depth of the shallowest gauge in the pile for control purposes.

### Soil

Soil sampling was taken at 0.5 metre intervals to a depth of 5.0 metres, using hydraulically pushed, thin-walled sample tubes. Laboratory tests were undertaken to determine suction, moisture content, soil modulus and undrained shear strength profiles. Atterberg limits and loaded shrinkage indices were also determined.

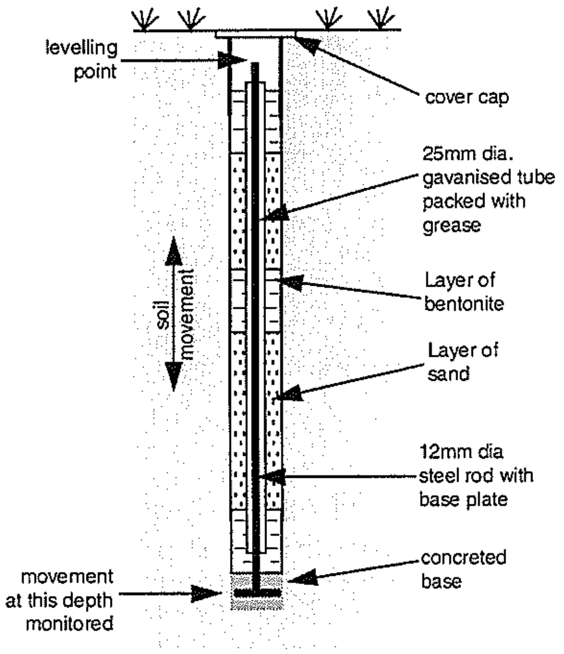



Figure 2 - Soil probe details

Three nuclear meter probe holes were also constructed with the aim of giving instantaneous values of moisture conditions without the need of sampling. The results are not reported within this paper.

Movements have been monitored at the surface and at depths of 0.5, 1.0, 1.5, 2.0, 3.0 and 4.0 metres via level surveys. The movements at depth were measured by two sets of deep probes across the site. The probes consist of a 12 mm diameter steel rod placed inside a 20 mm galvanised pipe filled with packing grease. The base of the rod is welded to a 75 mm base plate which was concreted into the soil. The 100 mm diameter hole was backfilled with layers of sand and bentonite powder which acts as a water sealant and reduces skin friction adjacent to the pipe surrounding the rod as shown in Figure 2.

A benchmark was constructed using the same methods and materials. It is founded at a depth of six metres, and therefore is founded in the Tertiary sands and is anchored below the expansive soil movements.

## RESULTS

### Soil

The soil profile at the site is typical of the north-eastern suburbs of Adelaide where reactive soils have damaged a significant proportion of housing. Table 1 lists the general descriptions and properties of each of the three soil horizons overlaying the Tertiary sands. The clays and silts were deposited by wind and water erosion of the nearby Adelaide Hills during the early to middle Quaternary period.

The layer of fill was placed in 1993 as part of the development of a new subdivision and consists of the shallow soil cut to form wet lands approximately 500 metres from the pile site. The land was previously used for agricultural research.

**Table 1 - Typical borehole**

| MATERIAL | SOIL DESCRIPTION | MATERIAL PROPERTIES |
|---|---|---|
| 0.3m | Fill - mixture of Black Clay and Clay | - |
| 1.0m | Black Clay: dark grey to black, hard structure (dry) sticky, very firm, highly plastic (wet) | $I_{ls}$ = 6.0%<br>LL = 95%, PL = 35% |
| 4.5m | Clay: light grey to olive brown heavy clay, stiff and highly plastic, with yellow to red mottles (Keswick Clay) | $I_{ls}$ = 4.5 - 6.0%<br>LL = 100%, PL = 25%<br><br>LEGEND<br>$I_{ls}$ - loaded shrinkage index<br>PL - plastic limit<br>LL - liquid limit |
| 6.0m | Clayey Silt/Silty Clay: multi-coloured (highly mottled) yellowish brown, brownish yellow, dark red and grey, very stiff (Hindmarsh Clay) | $I_{ls}$ = 0.5 - 2.0%<br>LL = 65%, PL = 20% |
| EOH | Silty Clayey Sands: quartz sands, weakly cemented, some gravel (Tertiary Sands) | $I_{ls}$ = 0.0% |

## Movements

Initial soil moisture conditions at the site showed relatively high (dry) suction values near the surface, grading down to medium suction values at depths greater than 2.5 metres. For the first year, the winter (wet) season was drier than usual and therefore changes in suctions only occurred in the top one metre of soil; very minor soil movements were observed, as is shown in Figure 4.

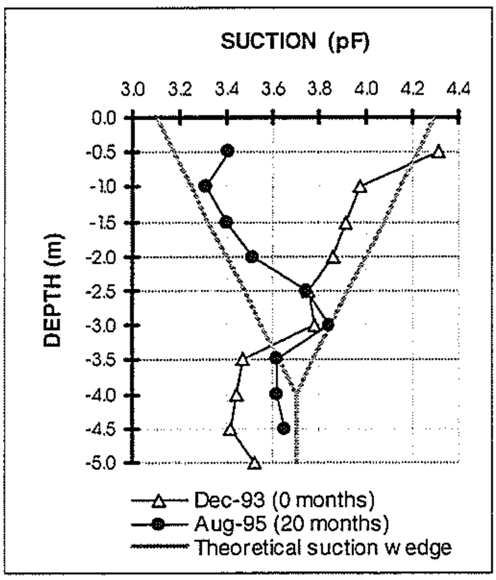



Figure 3 - Suction profiles

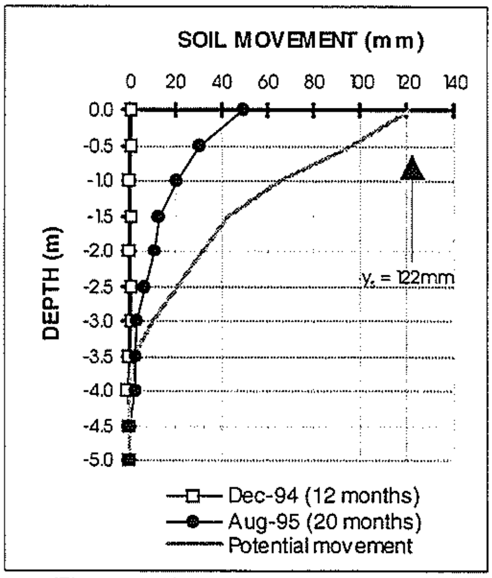



Figure 4 - Soil movement profiles

In order to achieve significant surface movements of the soil over a short period of time, a soakage trench was constructed adjacent to the piles. The trench is located 2 metres from the piles and is 0.3 metres wide and 1.5 metres deep. Below the trench, a series of holes were bored to a depth of 3 metres for deeper water penetration. The trench was first artificially filled with water in January, 1995 and significant soil movements were recorded almost immediately. The soil continued to heave for approximately four months. The top 2.5 to 3.0 metres of soil had considerably decreased in suction over this period and significant soil movements had been recorded to these depths. Figures 3 and 4 show the suction changes and corresponding soil movements from near the time of construction, December 1993 until August 1995.

Since the top 2 to 3 metres of soil were now low in suction, a nearly impermeable barrier was formed against the seasonal rains that fell in the winter months of June to August. Very little watering of the trench was undertaken between April and August 1995. Almost no movements were observed during these months even though this is the expected time for the soil to take in water and heave. Due to the lack of surface movements the trench is now being filled with water regularly in order to obtain further (and deeper) soil movements.

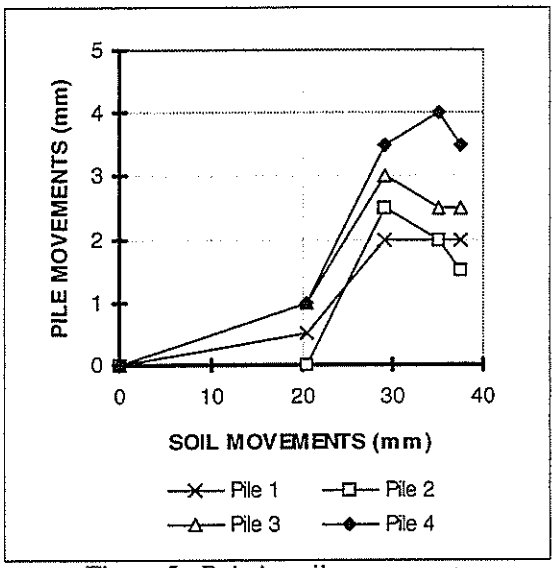


Figure 5 - Relative pile movements

Many more months of natural and artificial wetting will be required to achieve movements of above 100mm at the surface, which is the potential movement in an urban environment. Also, since the tops of the piles are 0.5m below the surface, the soil movements (at the top of the pile) are significantly less, by approximately 25 mm of heave, than at the surface.

The pile movements are taken as an average of two readings either side of the pile and have been taken monthly. Since the soil movements at the top of the pile have only reached 35 mm, theoretical predictions only expect about 4 mm pile movement in the worst case of the straight pile (pile 4). The measurements are taken using a dumpy level and require two change points from the location of the benchmark. Therefore, an accuracy of approximately ±2 mm was achieved. Since the pile movements were all less than 5 mm, a 2 mm error is significant. However, the pile movement results are in agreement with expected values. Figure 5 shows that the straight pile with no shaft lining (pile 4) has moved the greatest and the underreamed piles with the plastic linings (piles 1 and 2) have moved the least.

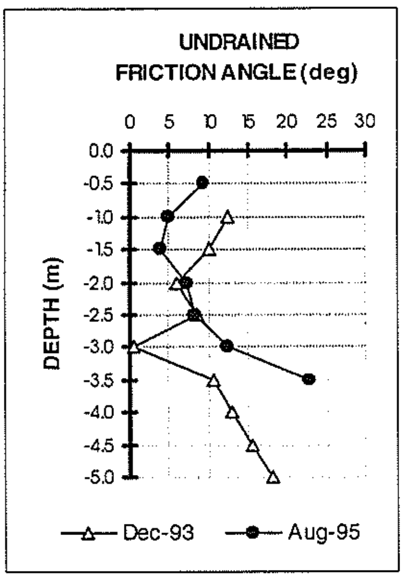


Figure 6 - Friction angle profiles

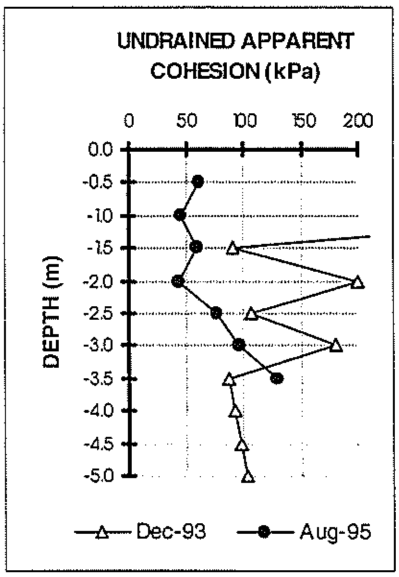


Figure 7 - Cohesion profiles

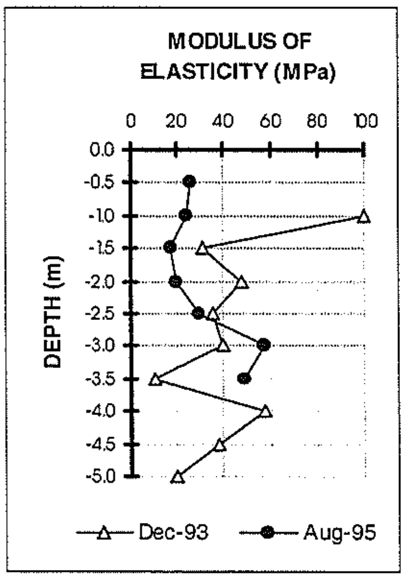


Figure 8 - Soil modulus profiles

Undrained shear strength parameters and Young's Modulus for the change in moisture conditions between December 1993 and August 1995 are shown in Figures 6, 7 and 8. The soil properties are more consistent when the suction values are lower (that is, August 1995). Triaxial testing was conducted on a single 38 mm diameter sample for each depth in a 3-stage test. The ratio of modulus of elasticity ($E_u$) to apparent undrained cohesion ($c_u$) equates to 430 for the August 1995 results. This correlates well with the suggested value for $E_u/c_u$ of 400 for a 38mm diameter sample of stiff clay [2].

### Strains

The strain gauge readings have been taken regularly, however the values obtained are spurious with readings of over 4000 microstrains ($\mu\varepsilon$) on one side of a pile at a specific depth and of less than 100 $\mu\varepsilon$ on the other side at the same depth. These high readings were also obtained before any significant soil heaving. Laboratory tests of concrete samples indicate that approximately 200 $\mu\varepsilon$ of tension is required to initiate cracking. No compressive values of strains have been recorded. The dummy gauges (surrounded by compressible rubber foam) have also all given readings of over 4000 $\mu\varepsilon$. The reasons for these inconsistent readings are still being investigated. Corrosion is one possibility, however, the gauges, leads and connections used were designed to be embedded in concrete. Also, a dummy gauge that has been exhumed shows no sign of corrosion.

### Modelling

The research aims to interpret the experimental observations and provide comparisons with the theoretical simulations. Elastic modelling, that is, no pile-soil slip, was executed using the finite element package STRAND6 [1]. This involved a simple two dimensional, axi-symmetric, heat model. The boundary element analysis program PIES - Piles In Expansive Soils [3], was used to simulate pile-soil slip and non-linear soil properties. The output shows forces induced in the pile by the soil movements and the resultant pile movements. The parameters used in these models are listed in Appendix A. The results of these models are shown in Figure 9 for the straight pile case (pile 4). The fine line indicates the predicted pile movement for each model corresponding the soil movements at August 1995. The non-linear value of 3.1 mm corresponds closely with the experimental results. Further parametric studies are to be undertaken in order to assess the sensitivity of each of the parameters.

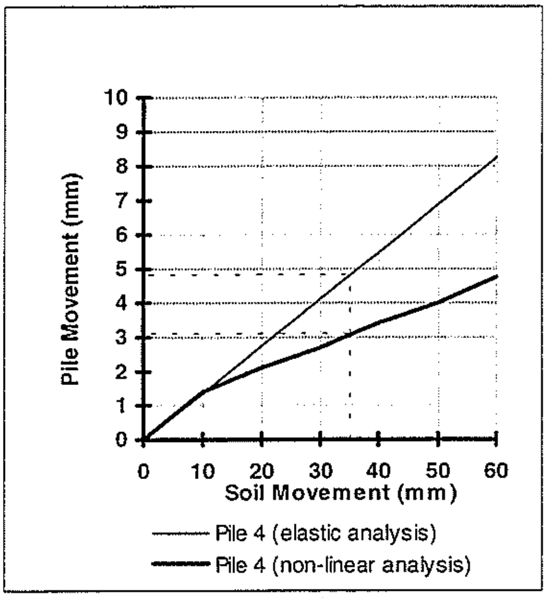


Figure 9 - Theoretical pile movements

It is envisaged that thermo-mechanical, axi-symmetric, finite element modelling will also be undertaken to further understand the relationships between soil and pile behaviours.

## CONCLUSION

More field and modelling data is required in order to reach strong conclusions. Another six months of monitoring should result in further soil heave and therefore pile movements. However, as the soil suction values decrease, the soil absorbs moisture at a decreasing rate, therefore requiring greater time for measurable results. Increased movements will enable further data to compare with the computer modelling.

The finite element and the boundary element computer simulations will be used to demonstrate and facilitate possible design methods with higher precision than the currently available design charts.

## ACKNOWLEDGMENTS

This research project was made possible by funding from the Australian Research Council and the University of South Australia. The assistance and guidance given by Associate Professor Mark Symons and Don Cameron is greatly appreciated. The laboratory staff within the School of Civil Engineering deserve thanks, especially Ray Yerbury for all the soil sampling and testing. Thanks also to the Regent Gardens developers for their ongoing cooperation and the use of the site.

## REFERENCES

1. G + D Computing. 1993. *STRAND6 Finite Element Analysis System - Reference Manual and User Guide*. G + D Computing, Sydney

2. Padfield C J and Sharrock M J. 1983. *Settlement of Structures on Clay Soils.* Construction Industry Research and Information Association. PSA Civil Engineering Technical Guide 38, London

3. Poulos H G. 1989. *Program PIES - Axial Response of Piles in Expansive Soils - Users' Guide.* Centre for Geotechnical Research, Sydney University, Sydney

4. Standards Australia. 1995. *Residential Slabs and Footings.* AS 2870, Sydney

5. Standards Australia. 1995. *Piling - Design and Installation.* AS 2159, Sydney

6. Symons M G, Fanning M P and Kilsby A G. 1993. Pile and Slab Footings for Small Buildings. *XXIst World Housing Congress.* International Association for Housing Science, South Africa, May 10-14

## APPENDIX A

**Table A1 - Parameters for elastic finite element model, STRAND6**

| Pile properties | | | Soil properties | | |
|---|---|---|---|---|---|
| Young's Modulus | ($E_{pier}$) | - 24 000 MPa | Modulus | ($E_{soil}$) | - 20 MPa |
| Poisson's Ratio | ($\mu_{pier}$) | - 0.25 | Poisson's Ratio | ($\mu_{soil}$) | - 0.30 |
| Density | ($\rho_{pier}$) | - 2500 kg/m$^3$ | Density | ($\rho_{soil}$) | - 2500 kg/m$^3$ |
| Coeff of Expansion | ($\alpha_{pier}$) | - 0.00 | Coeff of Expansion | ($\alpha_{soil}$) | - 0.01 |
| Length | ($L_{pier}$) | - 5 m | Soil movement: | | |
| Diameter | ($d_{pier}$) | - 0.5 m | triangular distribution (from 0 at 2.5 m | | |
| No. of elements | | - 10 | deep to swelling movement at surface) | | |

**Table A2 - Parameters for non-liner boundary element model, PIES**

| Pile properties | | | Soil properties | | |
|---|---|---|---|---|---|
| Young's Modulus | ($E_{pier}$) | - 25 000 MPa | Modulus | ($E_{soil\ at\ tip}$) | - 20 MPa |
| Length | ($L_{pier}$) | - 5 m | Modulus | ($E_{soil\ below\ tip}$) | - 100 MPa |
| Diameter | ($d_{pier}$) | - 0.5 m | Poisson's Ratio | ($\mu_{soil}$) | - 0.30 |
| No. of elements | | - 10 | Shaft resistance: | | |
| | | | $\sigma_{peak}$ = 60 kPa, $\sigma_{residual}$ = 45 kPa, | | |
| | | | displacement$_{peak\text{-}to\ residual}$ = 10 mm | | |
| | | | Soil movement: | | |
| | | | triangular distribution (from 0 at 2.5 m | | |
| | | | deep to swelling movement at surface) | | |
| | | | Hyperbolic soil response: | | |
| | | | shaft ($R_{fs}$) = 0.5, base ($R_{fb}$) = 0.9 | | |

# WICK DRAIN APPLICATION FOR ONE DIMENSIONAL CONSOLIDATION, GULF HARBOUR MARINE VILLAGE, WHANGAPARAOA

R.J. KNOWLES

Foundation Engineering Limited
Pakuranga, Auckland

## SUMMARY

During the summer of 1994/95, approximately 80,000 metres of geosynthetic wick drains (Colbond® EX1000) were installed at the Gulf Harbour Marine Village, Whangaparaoa. This was undertaken to accelerate the consolidation settlement of large areas containing up to 9 metres depth of soft to very soft estuarine and alluvial deposits overlying sandstone/siltstone bedrock at depth. This paper outlines the site geology and geotechnical considerations of the project, together with assessing predicted versus actual performance.

## INTRODUCTION

Situated on the southern side of the Whangaparaoa Peninsula, some 45 minutes drive north of Auckland City, Gulf Harbour is one of the largest subdivisional developments in New Zealand. Apart from considerable residential development, it involves the extension of an existing marina and construction of a new boat harbour, marine village and town centre situated around a canal development, hotel, international 18 hole golf course and man-made lakes for irrigation.

As part of the marine village/town centre development, extensive areas of underlying saturated estuarine deposits needed to be rapidly consolidated to allow for programmed construction of residential/commercial buildings within six to eighteen months of the completion of preload earthworks and to reduce the liquefaction potential of the substrata.

Final design levels for the marine village allowed for approximately 2.5 metres depth of compacted clay filling ("structural blanket filling") with varying depths of additional preload to accommodate expected settlement.

## SITE INVESTIGATION AND GEOLOGY

Extensive investigations involving machine drilling and sampling, trial pit excavations and cone penetrometers (CPTs), combined with a detailed geological assessment, review of similar projects overseas and extensive laboratory testing was undertaken throughout 1992 and 1993 as part of the marine village project. Strata beneath the marine village area were categorised into five types as outlined below:

(i) Wind blown sand deposits - loose, fine and uncompacted, up to 1.8 metres thick, with scattered bands of shell.

(ii) Estuarine silts - soft to very soft, generally non-plastic to slightly plastic with some sand, shell and organic inclusions. Shear strengths ranged between 6 and 22 kPa, Mv and Cv values averaging 1.21 $m^2$/MN and 7.6 $m^2$/year respectively under loadings up to 60 kPa.

(iii) Alluvial silts and/or highly weathered residual Waitemata Group soils - soft to very stiff, non-plastic to very plastic orange/brown/blue/grey/green clayey silts and silty clays up to 5 metres thick. Average Mv and Cv values were 0.37 $m^2$/MN and 60.3 $m^2$/year respectively, shear strengths ranging from 18 to greater than 140 kPa.

(iv) Underlying Waitemata Group sandstone/siltstone - very weakly to well cemented, weak to strong, dark grey alternating sandstone and siltstone with unconfined compression strengths from 2 to 25 MPa.

(v) Construction spoil fill - originating from the original marina construction. End tipped or hydraulically pumped, loose, mixture of sands, silts, sandstone and shells.

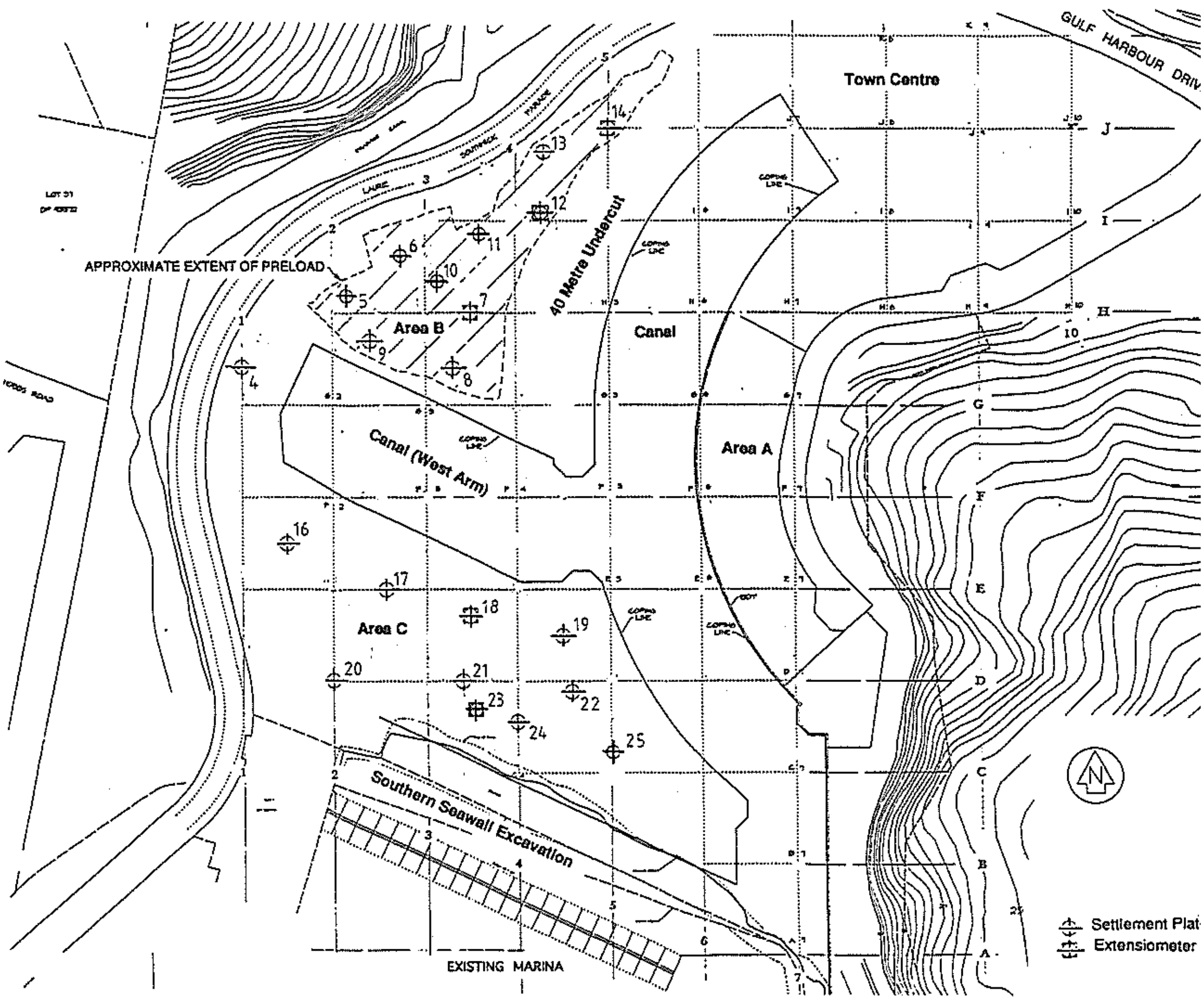



Figure 1: Layout Plan of the Gulf harbour Marine Village

## DEVELOPMENT PROPOSALS AND REMEDIAL OPTIONS

Final development levels for Areas B and C, on the western side of the proposed canal (refer Figure 1), called for approximately 2.5 metres depth of clay filling to be placed over up to 9 metres of type (ii) and (iii) soils. Preliminary scheme proposals for these areas indicated that they were likely to contain two to three storeyed residential buildings and related services, resulting building loads being limited to around 20 kPa by means of raft foundations and careful foundation design.

In order to provide suitable conditions for such development, several remedial and construction options were considered, including:

(a) Deep compaction: This option was discounted due to the very soft and non-granular nature of many of the substrata. It was felt that the compactive effort would have been largely wasted in pushing aside the soft materials rather than providing compaction.

(b) Full Removal of Soft Strata: This option was utilized for one area of the development known as the 40 metre undercut (refer Figure 1), to ensure good foundation conditions for programmed building within a short period, in case preload settlements did not proceed as quickly as expected. It was considered too expensive for large-scale use.

(c) Partial Removal of Soft Strata: The partial replacement of some soft materials with better quality engineering filling was not considered adequate due to the long-term effects of consolidation settlement within remaining strata, particularly in relation to the provision of services and roading accessways.

(d) Foundation Piling: Piling without addressing long term services settlement was considered inappropriate. Additional complications would have been created by negative skin friction acting on the piles.

(e) Wick Drains: As previously shown, the compressibility of types (ii) and (iii) soils was high. Initial calculations showed that some areas could take up to 18 years to reach 90% of total primary consolidation. However, with the provision of vertical wick drains and 1 to 1.5 metres of preload filling (plus structural fill) it was calculated that this consolidation could be achieved within 6 months over Area B and 12 to 18 months over Area C depending upon wick drain spacing.

Finalised wick spacings were 1.7 metres on Area B with 2.2 metre spacings used on Area C. As works progressed it became apparent that Area C could not be preloaded as quickly as expected and spacings were reduced to 1.7 metres to provide for more time flexibility when full preload was achieved.

## CONSTRUCTION

For the Gulf Harbour contract, COLBOND® CX1000, 100mm wide, 6mm thick, wick drain was selected. As with similar synthetic wick drains, it contains a stiff, porous drainage core wrapped in a heavy duty, fine-pored filter membrane, the stiffness of the core being relied upon to provide a competent vertical drainage path, while distorting as settlement occurs.

Installation of the wick drains commenced in mid November 1994 and continued as required until mid January 1995 on Areas B and C. Placement of the drains was carried out using a 27 tonne hydraulic excavator installing two drains simultaneously at the desired spacing on a wide weed bucket.

After installation, the drains were linked together with additional Colbond® laid out on the surface to form a drainage network. Collector drains approximately 300mm deep at 24m centres were constructed over the area, into which the wick drain network was fed. The entire area was then covered with a 300mm to 500mm deep sand layer (Type (i) soil) to help provide a drainage blanket and also to prevent damage from earthmoving machinery.

Along with the installation of settlement monitoring instrumentation, certified filling was won from the nearby boat harbour excavation and other site borrow areas and placed on the affected areas between January and June 1995. Subsequently, as preload material became available from the nearby southern seawall excavation, it was placed throughout June to August 1995 on Area B. However, due to deteriorating weather conditions, certified filling and preload works were not able to be completed during the 1994/95 earthworks season.

## SETTLEMENT MONITORING

Prior to filling, 20 settlement plates and 4 vertical magnetic extensiometers were installed on the site in the locations shown in Figure 1. Although expensive and difficult to install, the extensiometers were expected to provide better quality results than the settlement plates especially in establishing differing settlement rates of Type (ii) and (iii) soil. Settlement monitoring was subsequently carried out on a fortnightly basis, commencing in late November 1994. Vertical extensions to the monitoring devices were constructed as, firstly certified filling and later preload filling were placed.

## RESULTS

Results available at the time of preparation of this paper, indicate that on the whole, both the rate and degree of consolidation settlements occurring in the field are comparable with the predicted results.

Of the twenty four settlement monitoring points installed, five have been either temporarily or irreparably damaged, while a further two have been buried beneath a topsoil stockpile for much of the time. However, results from the remaining sites show that, in the ten months since the commencement of monitoring on Area B, maximum settlements have been in the order of 550 mm, at extensiometer plate 7, the predicted settlement having been in the order of 500mm in this locations for 55 kPa certified fill loading plus 45 kPa surcharge loading. In addition, the settlement curves relating to settlement plates 10 and 13 indicate that much of the primary consolidation has already occurred in the few months since the completion of preloading.

| Area B | | | PLOT | | |
|---|---|---|---|---|---|
| Plate 10 | | | Sink | Rate | |
| N729,116.83 | E302,159.88 | | Total mm | mm/day | Fill RL |
| | Total Days | Plate RL | | | |
| 6/1/95 | 0 | 0.417 | 0 | | |
| 24/3/95 | 77 | 0.279 | 138 | 1.79 | 2.97 |
| 4/4/95 | 88 | 0.270 | 147 | 0.82 | |
| 11/4/95 | 95 | 0.261 | 156 | 1.29 | 3.50 |
| 8/5/95 | 122 | 0.238 | 179 | 0.85 | 3.50 |
| 14/6/95 | 159 | 0.218 | 199 | 0.54 | 3.50 |
| 24/7/95 | 199 | 0.178 | 239 | 1.00 | 4.26 |
| 7/8/95 | 213 | 0.170 | 247 | 0.57 | 4.70 |
| 21/8/95 | 227 | 0.161 | 256 | 0.64 | 4.70 |
| 4/9/95 | 241 | 0.160 | 257 | 0.07 | 4.70 |
| 18/9/95 | 255 | 0.158 | 259 | 0.14 | 4.70 |
| 3/10/95 | 270 | 0.156 | 261 | 0.13 | |

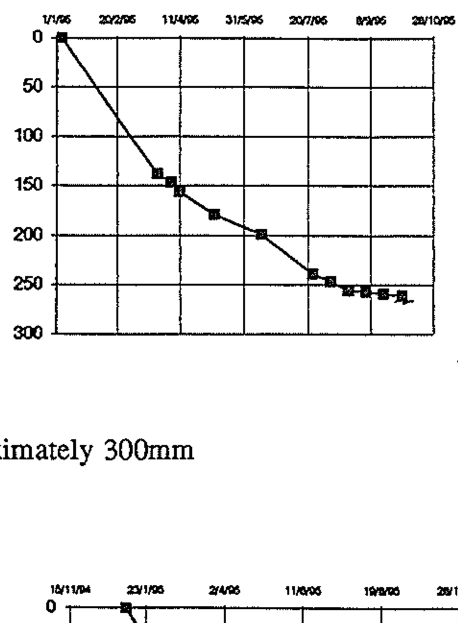



Plate 10: Total Predicted Settlement Approximately 300mm

| Area B | | | PLOT | | |
|---|---|---|---|---|---|
| Plate13 | | | Sink | Rate | |
| N729,185.32 | E302,227.68 | | Total mm | mm/day | Fill RL |
| | Total Days | Plate RL | | | |
| 6/1/95 | 0 | 0.378 | 0 | | |
| 4/4/95 | 88 | 0.203 | 175 | 1.99 | |
| 11/4/95 | 95 | 0.197 | 181 | 0.86 | 3.00 |
| 8/5/95 | 122 | 0.166 | 212 | 1.15 | 3.20 |
| 14/6/95 | 159 | 0.127 | 251 | 1.05 | 3.60 |
| 24/7/95 | 199 | 0.092 | 286 | 0.88 | 4.17 |
| 7/8/95 | 213 | 0.083 | 295 | 0.64 | 4.30 |
| 21/8/95 | 227 | 0.080 | 298 | 0.21 | 4.30 |
| 4/9/95 | 241 | 0.079 | 299 | 0.07 | 4.30 |
| 18/9/95 | 255 | 0.079 | 299 | 0.00 | 4.30 |
| 3/10/95 | 270 | 0.076 | 302 | 0.20 | |

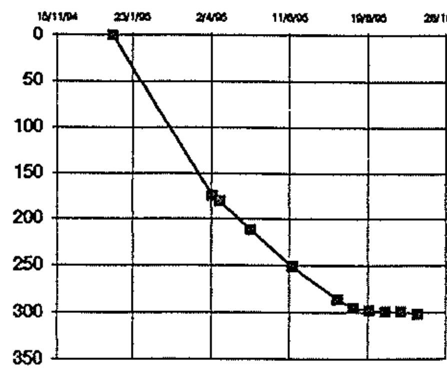



Plate 13: Total Predicted Settlement Approximately 300mm

Given the large degree of error generally associated with consolidation settlement prediction, these results have shown an encouraging degree of consistency and in some areas, settlement has occurred at a faster rate than expected. Several reasons have been mooted for the faster than expected settlements. Firstly, Mv and Cv values were conservatively used for calculations, rather than MH and CH. In addition, horizontal sand lenses within the natural deposits. are likely to have provided shorter drainage paths than those used in design. Thirdly, the lowering of the general watertable as a by-product of the adjacent canal excavation and construction works would have allowed for more rapid consolidation.

Consolidation monitoring of the site will continue during the removal of surcharge and the construction of buildings, services and accessways. This is due to commence on Area B in March 1996. However, as filling and preloading was not completed on Area C during the 1994/95 earthworks season, further work will need to be undertaken during the 1995/96 season to achieve the required settlements.

## CONCLUSIONS

During the construction of the Gulf harbour Marine Village, the use of vertical wick drains in conjunction with preloading to achieve rapid consolidation settlements has been found to be both effective and economical in conditions of soft, wet estuarine silts and clayey silts, overlying competent bedrock at shallow depth. As a result, development times have been reduced, enabling construction date targets to be achieved and resulting in faster capital returns for the developers.

Acknowledgements

The author would like to thank Gulf Harbour Management Limited for permission to write this paper.

Bibliography

1. Hausmann, Manfred R, 1990. Engineering Principles of Ground Modification, McGraw-Hill Publishing Company

2. Van den Elzen, Ir. LWA, Accelerated Consolidation of Compressible, low-permeable subsoils by means of COLBOND® drains, undated

3. Lambe, T.W. and Whitman, R.V., 1979. Soil Mechanics SI Version, John Wiley & Sons, p406 - 422

# FACTORS INFLUENCING DESIGN OF SLOPES IN RESIDUAL SOILS: AN OVERVIEW

S. KRISHNAN
Tonkin & Taylor Ltd., Auckland, New Zealand

## SUMMARY

The key to understanding the stability of slopes in residual soils lies in recognising the influence of the weathered profile, relict discontinuities, shear strength parameters, and groundwater conditions. These factors would need to be carefully considered while preparing a site investigation programme to establish the information required for a stability analysis.

## INTRODUCTION

Cut slopes in residual soils are increasingly encountered in present-day engineering projects, and often prohibitive construction and maintenance costs arise from their instability. Instability manifests itself dramatically as landslips, landslides and mudflows. The inability to assess the stability of these slopes is often attributed to the "wide range of complex factors involved". The factors influencing slope stability in residual soils are:

1. slope height and slope batter;
2. any imposed loads;
3. geology;
4. shear strength parameters;
5. soil suction;
6. groundwater condition;
7. appropriate factor of safety;
8. surface protection of slope.

This paper examines only factors 3 through 7, though some interesting concepts relating to remainder of the factors will emerge.

## GEOLOGICAL FACTORS

A number of studies on the development of natural slopes and slope instability in residual soils have concluded that:

- landslides are the predominant method for the development of natural slopes in tropical regions where deep residual soil profiles occur;
- landslides are associated with the characteristics of the weathered profile;
- maximum landslide activity is associated with periods of intense rainfall;
- shallow slides are the most common mode of instability, but deep slides which involve movement of a block of soil along relict discontinuities are also frequently encountered.

Discussions on the above are presented in the following subsections.

### Characteristics of the Weathered Profile

The weathered profile is the sequence of layers of materials which have developed in place over fresh, unweathered rock due to the weathering processes of chemical decomposition and physical disintegration. These layers possess significantly different strength and permeability characteristics. A typical weathered profile for igneous rocks, proposed by Deere and Patton [2], is shown in Figure 1. Deere and Patton [2] have also dealt with typical weathered profiles in other rock types.

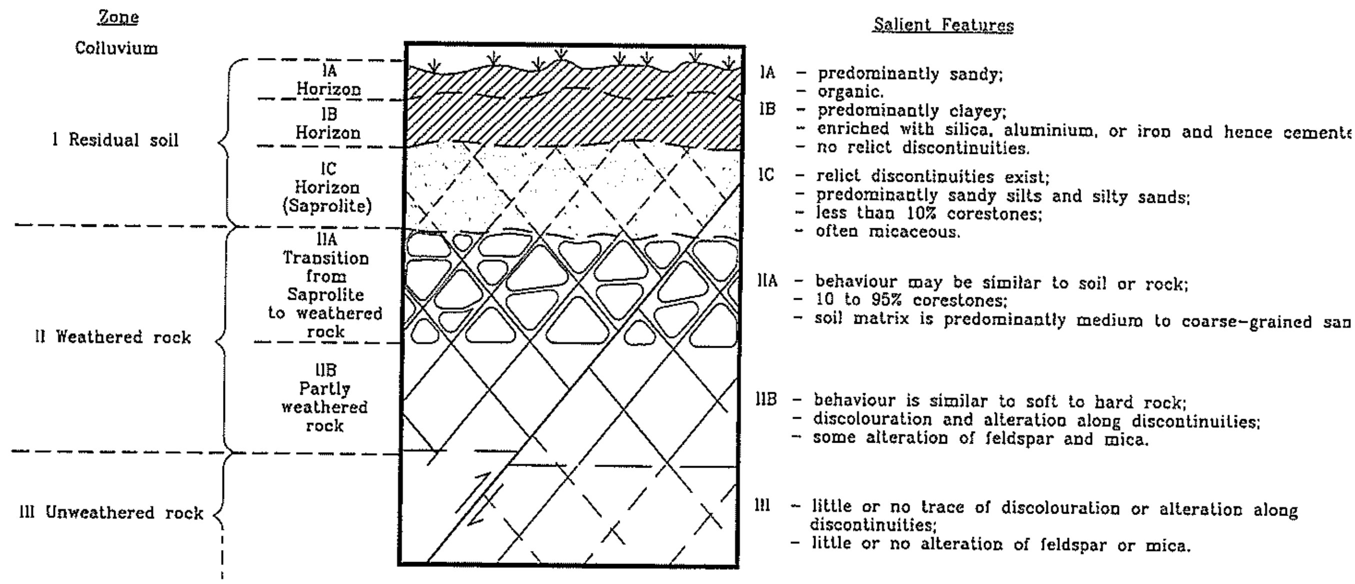



**Figure 1 Typical weathered profile for intrusive igneous rocks (Deere and Patton [2])**

Residual soils are considered to be the products of chemical decomposition and a combination of weathering factors influence the resultant soil profile (Townsend [10]):

- climatic conditions during weathering, with rainfall controlling the supply of moisture for chemical reactions and the leaching of soluble constituents of the minerals, while temperature influences reaction rates;

- properties of the parent rock material and rock mass;

- topography and drainage conditions during weathering, which govern the moisture intake by the parent rock and the leaching of soluble constituents of the minerals;

- period over which weathering has been active.

Obviously, the above factors are optimised in rolling, gently sloping terrain in tropical regions, where deep residual soil profiles (up to 50 m) exist.

The colluvium (or slope wash and slide debris) mantling residual soils is a product of physical disintegration, which involves the processes of mechanical abrasion, erosion, etc.

The weathering processes result in a dramatic alteration in the strength and permeability characteristics of the parent rock slope so as to increase its susceptibility to failure. Relict discontinuities, such as bedding planes, joints, foliations and faults, which are inherited from the parent rock mass further reduce the stability of the slope. Although apparently stable under normal conditions, instability frequently occurs during periods of intense rainfall or during excavation.

Rainfall-landslide relationships have been established in different geological and meteorological settings, but that established in Hong Kong by various authors, including Lumb [6], Brand et al. [1] and Kay et al. [5], are considered as models in many countries.

## Common Modes of Slope Instability

The modes of instability are closely associated with the geological settings, which may be broadly divided into two classes (Deere and Patton [2]):

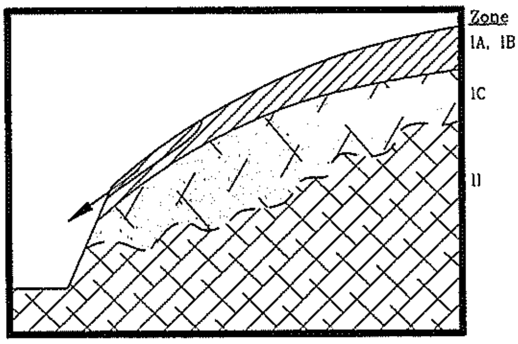
- **massive homogeneous rock**, such as granite, gneiss, thick-bedded sandstone, etc.;
- **anisotropic rock structure**, characterisedby schist, slate, thin or interbedded shale and sandstone, etc.;

As highlighted previously, the residual soil profiles in the above geological settings are different and each may be mantled in colluvium.

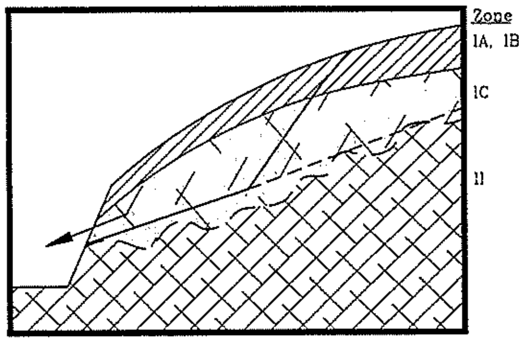
In a geological setting consisting of massive homogeneous rock, adversely orientated relict discontinuities are not frequently encountered. Therefore, the most common mode of instability comprises shallow slides, as shown in Figure 2a. These slides frequently occur during periods of intense rainfall.

By comparison, in areas where anisotropic rock structure predominates, adversely orientated relict discontinuities invariably exist. Therefore, the most common mode of instability comprises block slides along relict discontinuities, as shown in Figure 2b. Such slides may occur as the slope is excavated or during periods of intense rainfall, and are the principal consideration during slope design.

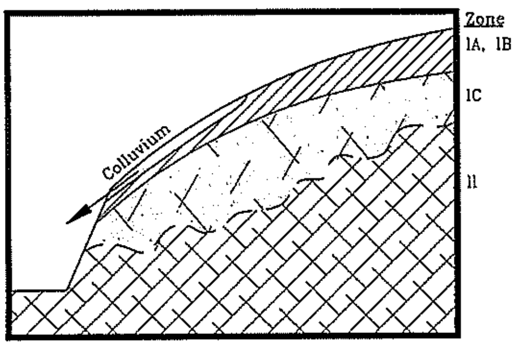
Slides in the colluvium, as shown in Figure 2c, are also common following a period of intense rainfall.




(a) Shallow slide in residual soil




(b) Block slide along relict discontinuities in residual soils




(c) Ravelling of thin colluvium

**Figure 2 Common modes of instability in residual soils and colluvium**

## SHEAR STRENGTH PARAMETERS

A number of studies on the stability of slopes in residual soils and the shear strength characteristics of residual soils have concluded that:

- the only type of stability analysis that is valid is an effective stress analysis;
- rational selection and grouping of samples for effective stress shear strength testing is important;

- effective stress shear strength testing should be undertaken in the stress range appropriate to the particular slope stability problem;

- residual soils are typically partly saturated and the resultant soil suction (i.e. negative pore-water pressure) contributes to an increase in the shear strength and hence slope stability;

- the influence of relict discontinuities on the bulk shear strength of residual soils is important.

Discussions on the above are presented in the following subsections.

**Effective Stress Analysis**

The use of an effective or total stress analysis depends upon whether the stress change likely to cause failure is imposed under conditions which allow only negligible or rapid dissipation of the excess pore-water pressure. The principal factor governing the likely drainage condition is the coefficient of consolidation ($c_v$) which is given by the expression:

$$c_v = k/m_v\gamma_w \qquad (1)$$

where k denotes the coefficient of permeability, $m_v$ denotes the coefficient of volume compressibility, and $\gamma_w$ denotes the unit weight of water.

Since residual soils are the products of leaching under high rainfall, they exhibit relatively high bulk permeabilities, ranging from $10^{-6}$ to $10^{-7}$ m/s (Townsend [10]). They also exhibit relatively low compressibility due to cementation. Therefore, based upon equation (1) residual soils have high $c_v$ values which will result in rapid drainage, and hence only an effective stress analysis is valid.

**Rational Selection and Grouping of Samples**

For the laboratory measurement of shear strength under controlled drainage conditions, we are largely dependent upon triaxial compression tests on undisturbed soil samples. The effective stress shear strength parameters, i.e. cohesion (c') and angle of internal friction ($\phi$'), may be determined from either a consolidated-drained (CD), or a consolidated-undrained test with pore-water measurements (CUP). The CUP test (multi-stage or series of single-stage tests) is commonly used, being a quick test.

It is common to see the test results plotted on a t-s' diagram (or a q-p' diagram) with a large scatter of the results about the best fit failure envelope. The scatter is largely due to the lack of attention to variations in strength in the residual soil profile. The selection of samples for testing and grouping on t-s' diagrams (or q-p' diagrams) should be undertaken based upon strength indicators such as SPT N values, void ratio (or moisture content) measurements, or visual examination (i.e. 'thumb test'). The author has found that the above procedures generally reduce the scatter considerably and typical results are presented in Figure 3.

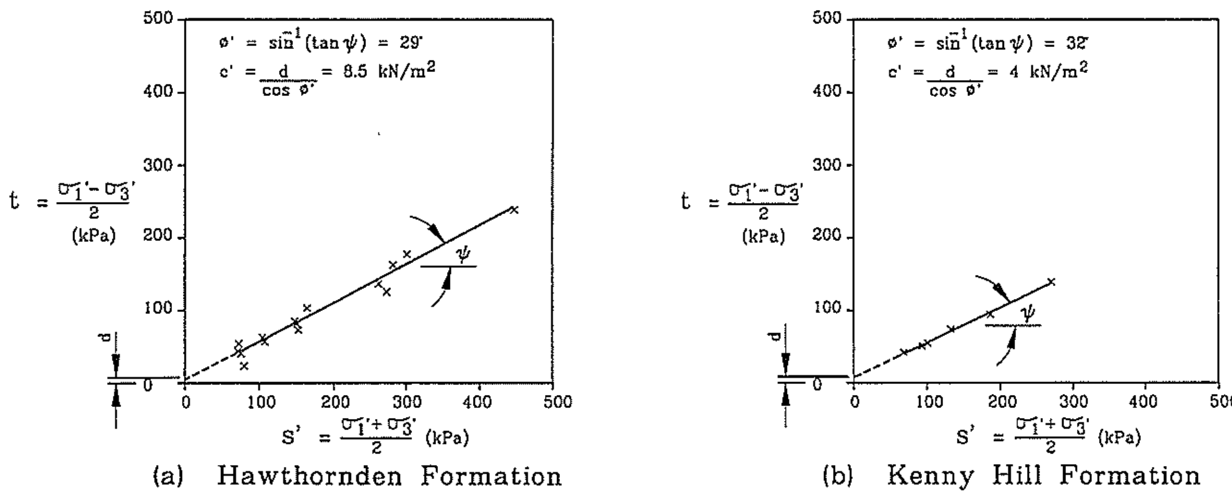



**Figure 3** **Shear strength data for residual soils in Kuala Lumpur, Malaysia, obtained from CUP triaxial tests**

## Stress Range for Testing

An important aspect of effective stress strength testing in relation to slope stability is the choice of the appropriate stress range, due to the non-linear nature of the failure envelope. As highlighted previously, shallow slides are the most common mode of instability, and therefore triaxial tests should be undertaken at low effective confining pressures, representative of those existing along shallow potential failure surfaces. A typical failure envelope for residual soils in Hong Kong is shown in Figure 4, where at low stress levels, $\phi'$ increases but c' reduces. However, since a long extrapolation to the ordinate axis is not required, measured c' values may be regarded as reliable.

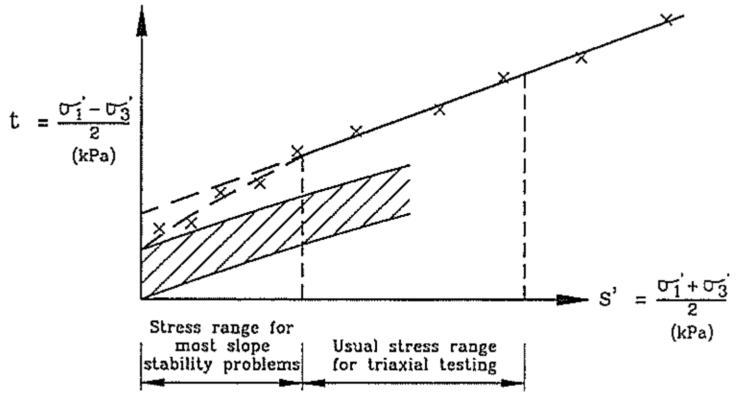



**Figure 4 Appropriate stress range for triaxial testing (Sweeney et al. [9])**

In selection of these stresses for CD or CUP triaxial tests, the slope of the stress path should be taken into account, as shown in Figure 5.

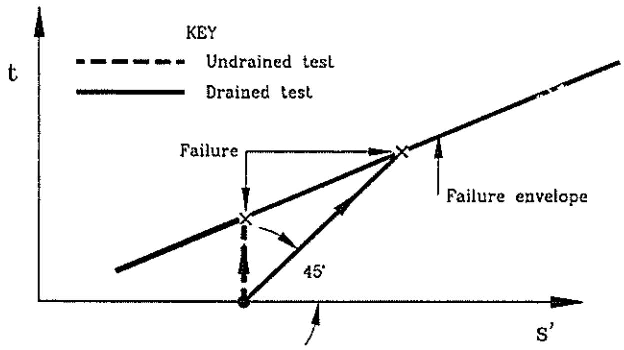



**Figure 5 Stress paths for triaxial tests**

## Soil Suction

Typically, residual soils are partly saturated and the resultant soil suction (i.e. negative pore-water pressure) contributes to an increase in the shear strength of the soil, and hence slope stability. The shear strength equation for saturated soils is given by the expression:

$$\tau_f = c' + (\sigma - u_w) \tan\phi' \qquad (2)$$

where $\tau_f$ denotes the maximum resistance to shear on any plane, $\sigma$ denotes the normal total stress, and $u_w$ denotes the pore-water pressure.

By comparison, the shear strength equation for unsaturated soils, proposed by Fredlund et al. [3], is given by the expression:

$$\tau_f = c' + (\sigma - u_a) \tan\phi' + (u_a - u_w) \tan\phi^b \qquad (3)$$

where $u_a$ denotes pore-air pressure, and $\phi^b$ denotes the angle of internal friction with respect to change in matric suction, i.e. $(u_a-u_w)$.

For convenience, equation (3) may be rewritten by grouping terms independent of normal stress as:

$$\tau_f = C + (\sigma - u_a) \tan\phi' \quad (4)$$

where C denotes $c' + (u_a-u_w) \tan\phi^b$.

Therefore, the influence of soil suction can be considered as an increase in the cohesion of the soil (Fredlund et al. [4]) which in turn contrbutes to slope stability. Soil suction can be determined by field measurements using a tensiometer, or in a laboratory by testing soil samples at their natural degree of saturation.

However, soil suction is generally discounted in stability analysis because the well-developed internal drainage of residual soils is conducive to water infiltration (Townsend [10]), with subsequent reduction in soil suction, soil strength and stability with time. This emphasises the importance of surface protection of slopes against water infiltration by such methods as a layer of "chunam" (soil-cement and lime plaster) and thick vegetation.

### Effect of Discontinuities on Shear Strength

As highlighted previously, relict discontinuities such as bedding planes, joints, foliations and faults are frequently encountered in the residual soil profile, and their influence on the bulk shear strength is important. In a series of tests, Skempton and Petley [8] demonstrated that the formation of discontinuities resulted in the total loss of the cohesive component of shear strength (i.e. $c' = 0$), with the $\phi'$ value remaining unaltered. However, the bulk shear strength may exhibit a considerable cohesive component, although the magnitude would depend upon the spacing of discontinuities.

It should be noted that where past shear deformation has occurred along a discontinuity, the angle of internal friction assumes a residual value.

## GROUNDWATER CONDITION

As highlighted previously, the only type of stability analysis that is valid is an effective stress analysis. To undertake an effective stress analysis, it is essential to obtain a best estimate of the equilibrium value of the pore-water pressure which will act on potential failure surfaces, which in turn requires an appreciation of:

- the change in groundwater table due to excavation;
- the rise in groundwater table during periods of intense rainfall;
- perched water table.

Discussions on the above are presented in following subsections.

### Change in Groundwater Table Due to Excavation

To evaluate the stability of cut slopes, it is essential to estimate the drawdown in the groundwater table resulting from the excavation of the slope, and after steady state conditions are achieved. General solutions for estimating the drawdown are not available, but a solution for incompressible material with drainage occurring from the voids within the soil as the groundwater table lowers, has been proposed by Nguyen and Raudkivi [7]. The soil properties required are the coefficients of permeability (k) and voids storage ($S_v$).

### Rise in Groundwater Table

Pore-water pressures in slopes are likely to change substantially with changes in rainfall if the $c_v$ value of the soil exceeds about 0.1 $m^2$/day (Wesley [11]), which is so for residual soils. Typically, the groundwater table is low during dry season, but rises significantly during periods of intense rainfall. A rise in the groundwater table influences slope stability by:

- increasing the pore-water pressure on potential failure surfaces, and consequently reducing the effective stress and shear strength (significant effect);
- reducing the soil suction, and consequently shear strength (potentially significant effect);
- increasing the weight of the sliding mass due to an increase in the bulk unit weight (minor effect).

The combination of factors results in slope instability, and therefore, it is essential to predict the rise in groundwater table due to the "maximum possible precipitation" (or "probable maximum precipitation") during the design life of the relevant project. This requires an appreciation of the following hydrogeological factors:

- the residual soil profile and variations in permeability;
- the intensity and duration of rainfall;
- the regional topography;
- surface protection of slope.

### Perched Water Table

Considerable variations in permeability are frequently encountered within the residual soil profile. The sequence of a relatively high permeability layer overlying an impermeable layer frequently results in a local build up of pore-water pressure, or perched water table, and consequently instability. However, the scale of instability is likely to be small.

For example, the colluvium is often more permeable than the underlying layers, resulting in perched water tables during periods of intense rainfall, and consequently the ravelling of colluvium.

## APPROPRIATE FACTORS OF SAFETY

In slope design, the factor of safety is defined as the ratio of the actual strength available and that mobilised, with the mobilised shear strength being equal to the destabilising force. Conventionally, this is incorporated in design as a single factor of safety. The purpose of a factor of safety is to take account of significant uncertainties and the appropriate value for design depends upon:

- the nature of the site investigation programme to establish the characteristics of the weathered profile, probable mode of instability, appropriate shear strength parameters along potential failure surfaces, and groundwater conditions;
- risk of reduction in shear strength due to an increase in pore-water pressure after intense rainfall;
- risk of increase in the destabilising force due to removal of the slope toe (either by geomorphological, or man-made processes), or "live loading" at the head of the slope (by construction plant, earth fill stockpile, etc.), or seismic loading;
- necessity to limit ground deformation;
- limitations of existing methods of analysis.

Two distinct approaches frequently adopted in determining an appropriate factor of safety are:

1. **Most probable** shear strength parameters, groundwater conditions and loads are selected, and a generous factor of safety, between 1.3 and 1.5, is considered;
2. **Most unfavourable conceivable** shear strength parameters, groundwater conditions and loads are selected, and a lower factor of safety is considered.

Regardless of the approach adopted, a sensitivity study to develop an appreciation of the relative importance of the various factors influencing slope stability is essential.

## CONCLUSION

The advent of computers has made methods for the analysis of potential circular and non-circular failure surfaces very simple. However, there are many problems associated with the application of these methods for the design of slopes in residual soils due to significant uncertainties regarding the characteristics of the weathered profile, the probable mode of instability, appropriate shear strength parameters along potential failure surfaces, and groundwater conditions. This paper has examined the above factors which would need to be carefully considered while preparing a site investigation programme for slope design. Important points include:

- a sound understanding of the geological setting is required to establish probable modes of instability;
- the site investigation must determine the variation in the strength and permeability characteristics within the residual soil profile;
- the site investigation must identify any relict discontinuities within the residual soil profile;
- the only type of stability analysis that is valid is an effective stress analysis;
- effective stress shear strength testing should be undertaken in the stress range appropriate to the problem;
- the influence of relict discontinuities on the bulk shear strength of residual soils is important;
- groundwater observations to develop an appreciation of hydrogeological influences is important;
- a sensitivity study to develop an appreciation of the relative importance of the factors influencing slope stability is essential.

## REFERENCES


1 Brand, E W 1984. Relationship between rainfall and landslides in Hong Kong, *Proceedings of the Fourth International Symposium on Landslides*, Toronto, 1: 377-384.

2 Deere, D U and Patton, F D 1971. Slope stability in residual soils, *Proceedings of the Fourth Pan American Conference on Soil Mechanics and Foundation Engineering*, San Juan, Puerto Rico: 87-170.

3 Fredlund, D G, Morgenstern, N R and Widger, R A 1978. The shear strength of unsaturated soils, *Canadian Geotechnical Journal* 15: 313-321.

4 Fredlund, D G 1979. Appropriate concepts and technology for unsaturated soils, *Canadian Geotechnical Journal* 16: 121-139.

5 Kay, J N and Chen, T 1995. Rainfall-landslide relationship for Hong Kong, *Proceedings of the Institution of Civil Engineering: Geotechnical Engineering* 113: 117-118.

6 Lumb, P 1975. Slope failures in Hong Kong, *Quarterly Journal of Engineering Journal* 8: 31-65.

7 Nguyen, V U and Raudkivi, A J 1983. Analytical solution for transient two-dimensional unconfined groundwater flow, *Hydrological Sciences Journal* 28(2):

8 Skempton, A W and Petley, D J 1967. The strength along structural discontinuities in stiff clays, *Proceedings of the Geotechnical Conference*, Oslo, Norway, 2: 29-46.

9 Sweeney, D J and Robertson, P K 1979. A fundamental approach to slope stability problems in Hong Kong, *Hong Kong Engineer*: 35-44.

10 Townsend, F C 1985. Geotechnical characteristics of residual soils, *Journal of Geotechnical Engineering (ASCE)* 111(1): 77-94.

11 Wesley, L D 1994. Slope stability in mountainous tropical regions, *Proceedings of Geotropika '94: Regional Conference in Geotechnical Engineering*, Malacca.

# EFFECTS OF INTERPARTICLE AND INTRAPARTICLE MATERIAL ON THE GEOMECHANICAL PROPERTIES OF LIMESTONE

GLENN McINTOSH*, VICKI MOON, CAMPBELL NELSON

Department of Earth Sciences, University of Waikato, Private Bag 3105, Hamilton
*Present address: Coal Corporation of New Zealand Ltd, Private Bag 502, Huntly

## SUMMARY

Interparticle and intraparticle material within limestone is that material deposited between skeletal (and other) grains and within the cavities of skeletal grains, respectively. Combinations of the distribution and quantity of these carbonate materials, and whether they are sparite cement (>0.02mm) or fine micrite matrix, form important controls on the geomechanical behaviour of limestone.

Interparticle material is correlated with durability/abrasion and dissolution rate parameters. Geomechanically, these involve surficial processes, suggesting that surface degradation is influenced most significantly by the material deposited between grains in limestone. Micrite (muddy) materials are more durable than sparite materials, and intersparite is also responsible for higher rates of dissolution.

Intraparticle material is correlated especially with sonic velocity and various strength properties. These areas of geomechanics are concerned primarily with the body of the rock, suggesting that internal breakdown of limestone under stress is controlled particularly by the amount and type of material infilling pore spaces in skeletal grains. An increase in intraparticle material improves the geomechanical performance of limestone. Intramicrite appears to be particularly conducive to higher limestone strengths.

## INTRODUCTION

Little published information exists about the geotechnical properties of limestones. This paper contributes to this short-coming, and suggests some likely compositional controls on limestone behaviour under different physical and chemical test environments. Eight texturally diverse limestones were collected from quarries in the North Island, New Zealand (Appendix Table 1). The compositions of the limestones were determined by petrographic (microscopic) analysis, and a wide range of field and laboratory geotechnical measurements were made on the specimens. We report here a subset of the statistical correlations that link the petrographic features of the limestones to their geomechanical properties. We recognise the limitations of using only eight cases in the statistical evaluation, but trust the results will encourage further research into the relationships postulated.

## TEST PARAMETERS

Limestones are sedimentary rocks, normally comprising more than 50% $CaCO_3$. A special feature of limestones compared to terrigenous rocks, such as mudstones or sandstones, is that their particles consist mainly of the carbonate fossils or skeletons of various organisms involved in the sedimentation process, and that many of these skeletons include internal cavities and pores. Thus, during conversion of an unconsolidated carbonate sediment into a limestone, precipitated carbonate and other material is deposited both *within* and *between* the skeletal grains, here referred to as *intraparticle* and *interparticle* material, respectively (Fig. 1). This material may include calcite spar cement (or sparite >0.02mm) or an infiltered very fine matrix of calcite material, known as micrite (sometimes including terrigenous silt and clay) [11]. The abundance of each of these materials, and of the skeletal and terrigenous framework grains, was determined by point-counting under a petrographic microscope, identification of the limestone constituents being aided by the photographic plates in Adams et al. [1] and Scholle [12].

The following geomechanical tests were undertaken on the limestones: sonic velocity - a measure of the velocity of high frequency soundwaves passing through cored specimens; unconfined compressive strength (UCS) - a measure of compressive strength characteristics of cored specimens; pointload strength - a measure of (largely) tensile strength characteristics of small irregular blocks of limestone in the field; slake durability and Los

Angeles abrasion - measuring the effects of surface degradation of rock aggregate under alternate wetting and drying cycles, and under purely dry conditions, respectively; dissolution rate - an index measure of the rate at which small limestone disks undergo dissolution over a two hour period in distilled water. Apart from the dissolution rate test, which was designed specifically for the project [10], all geomechanical tests follow standard methods [2, 3, 5, 6].

The abundances of skeletal (and other) grains and of the interparticle and intraparticle material in the limestones are summarised in Appendix Table 2, along with the geomechanical properties of the specimens.

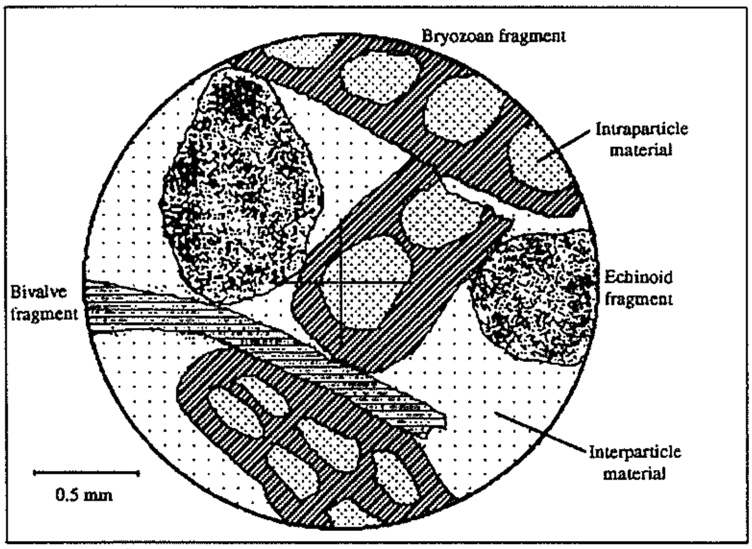



Figure 1: Sketch of a portion of a thin-section of a limestone under a petrographic microscope showing three common types of skeletal fragments and the distribution of interparticle and intraparticle material.

## GEOMECHANICAL CORRELATIONS WITH INTERPARTICLE MATERIAL

Interparticle material influences the behaviour of limestone aggregate tested in the Los Angeles abrasion machine. Abrasion loss is more substantial with larger amounts of intersparite (Fig. 2), whereas considerable resistance to abrasion loss is demonstrated with intermicrite materials (Fig. 3). Knill [8] discovered that the surface polishing of roadstones was significant in material containing sparite, and that materials comprising mainly a fine matrix were most resistant to polishing. The results of the Los Angeles abrasion test (involving similar processes) on the limestone support these observations.

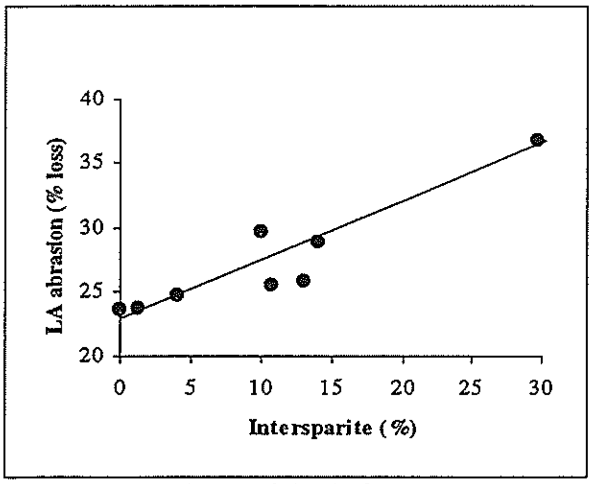
Lees and Kennedy [9] and Collis and Fox [4] attributed higher amounts of surface degradation associated with spar cemented materials to the softness and cleavage typical of large calcite sparite crystals.

Interparticle material also shows correlation with slake durability, despite the small range of recorded values (Appendix Table 2). Durability of the limestones is reduced by increasing amounts of interparticle material (Fig. 4), and correspondingly increased with larger numbers of grains, suggesting cement-rich limestones may be more prone to degradation than grain-rich materials.

The alternate wetting and drying cycles associated with the slake durability test may be partially responsible for lower durability observed in materials with more interparticle material (see the dissolution rate correlation below). Note that the outlier in Fig. 4 is the very sandy, lower carbonate limestone (D), perhaps contributing to the more significant degradation.

The proportion of interparticle material and grains in the limestones influences their rate of dissolution. Larger amounts of interparticle material (and correspondingly fewer grains) produce greater rates of dissolution (Fig. 5) and suggest that cement-rich limestones are more prone to (relatively) more rapid chemical degradation.

Intersparite materials in particular show good correlation with higher dissolution rate. This is supported by the work of Jakucs [7], who determined that dissolution of sparite textures was more evident than dissolution involving dense, uniform textures.



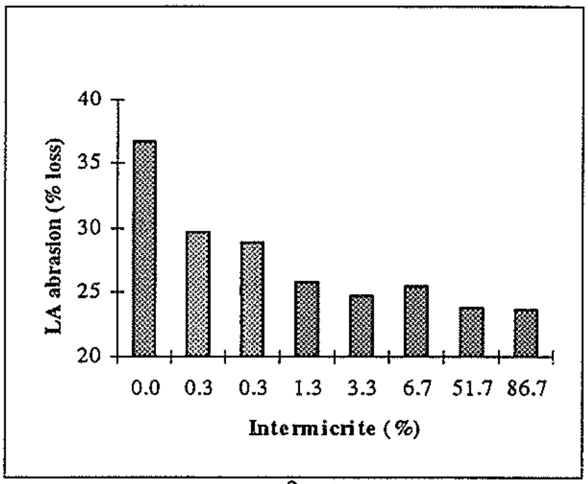
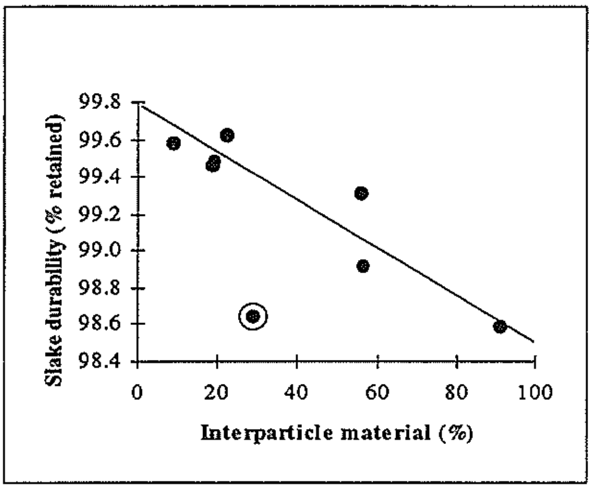




Figure 2 (left): Correlation between LA abrasion and intersparite. $R^2 = 84.6\%$.
Figure 3 (right): Histogram relating LA abrasion and intermicrite content.



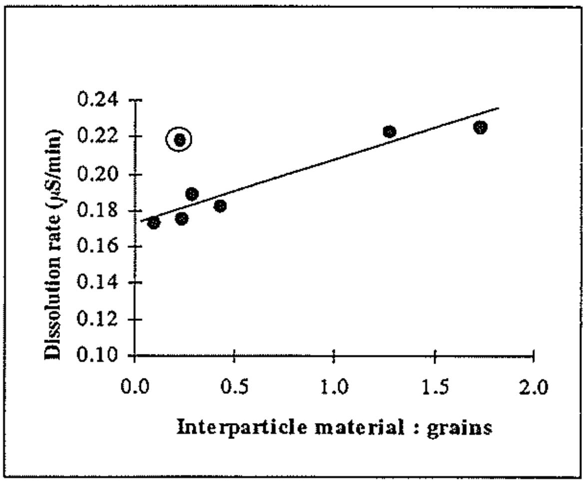



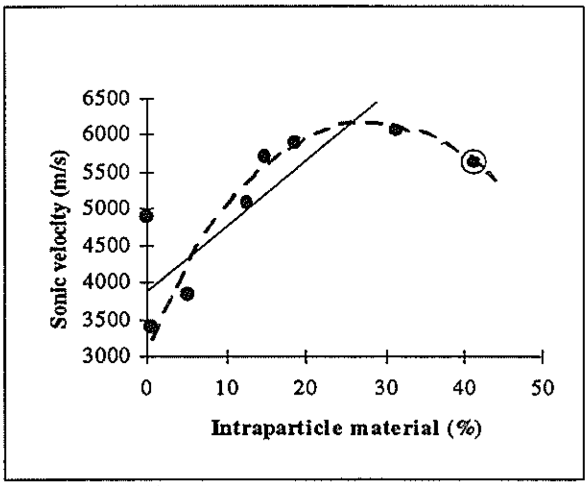

Figure 4 (left): Correlation between slake durability and interparticle material. The outlier is circled and omitted from the regression analysis. $R^2 = 91.2\%$.
Figure 5 (right): Correlation between dissolution rate and interparticle material:grains ratio. The outlier is circled and omitted from the regression analysis. $R^2 = 93.5\%$.

## GEOMECHANICAL CORRELATIONS WITH INTRAPARTICLE MATERIAL

The sonic velocity parameter is influenced by the presence of intraparticle material. Higher soundwave velocities occur with larger quantities of intraparticle material (Fig. 6), and notably with larger amounts of intramicrite (Fig. 7). An explanation for these observations is the increase in grain 'rigidity' produced by the infilling material, as well as a simple reduction in porosity, both of which are likely to improve soundwave propagation through the material. Scanning electron microscopy (SEM) has shown the boundary between intraparticle material and the associated cavity wall to be extremely secure [10], further promoting rigidity of the grain and infilling material as a whole.

Intraparticle material correlates with pointload strength of the limestones. Larger amounts of intraparticle material produce higher pointload strengths (Fig. 8), and again intramicrite appears particularly conducive to producing more competent specimens (Fig. 9). Pointload testing involves 'splitting' the specimen (a measure of tensile strength), and intraparticle material appears to provide effective resistance to the splitting of individual grains as the specimen fails. As mentioned earlier, SEM has highlighted the secure bond between intraparticle material and the cavity walls [10].

UCS values also correlate with intraparticle material, and again an increase in intraparticle material (particularly intramicrite) produces higher compressive strengths (Figs. 10 and 11). The positive influence of intraparticle material on limestone under tension (above) is likely also to be important during compression of the specimens.



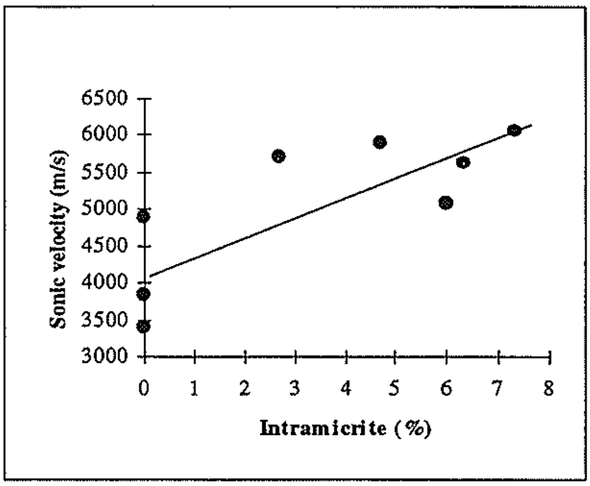
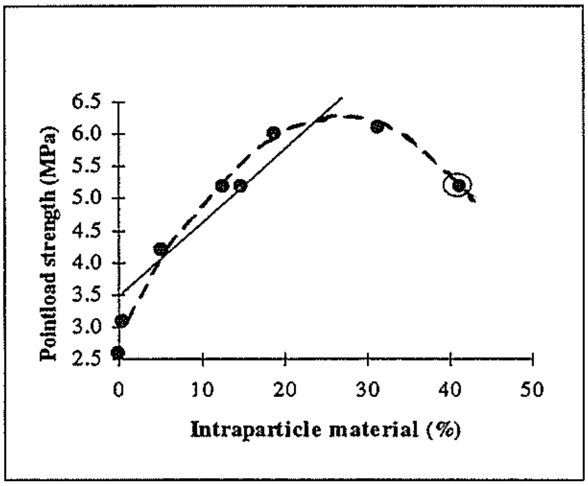




Figure 6 (left): Correlation between sonic velocity and intraparticle material. The outlier is circled and omitted from the regression analysis. $R^2 = 69.2\%$. The dashed line is discussed below.
Figure 7 (right): Correlation between sonic velocity and intramicrite. $R^2 = 62.7\%$.



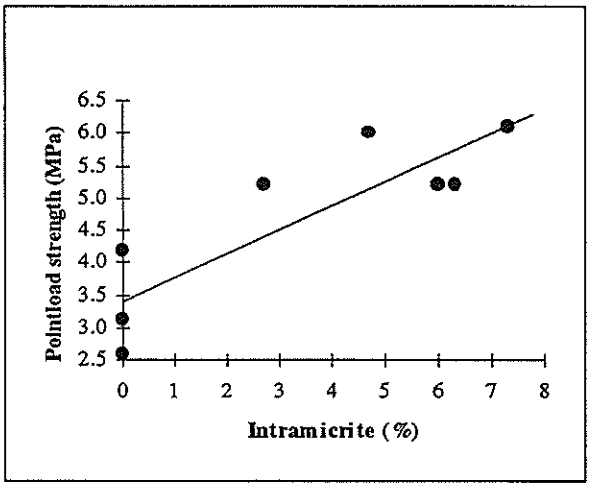
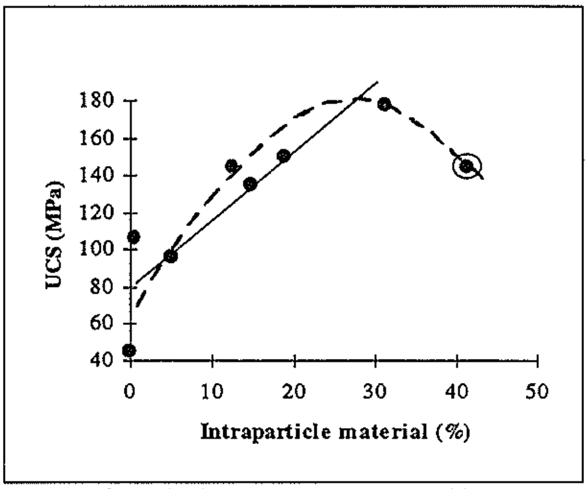




Figure 8 (left): Correlation between pointload strength and intraparticle material. The outlier is circled and omitted from the regression analysis. $R^2 = 83.5\%$. The dashed line is discussed below.
Figure 9 (right): Correlation between pointload strength and intramicrite. $R^2 = 72.4\%$.



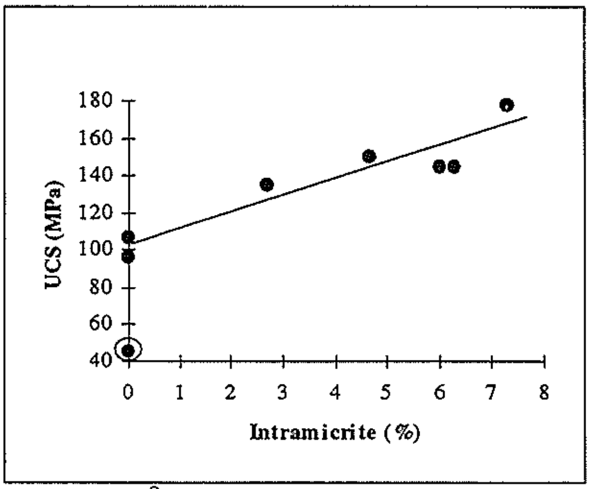




Figure 10 (left): Correlation between UCS and intraparticle material. $R^2 = 79.5\%$. The dashed line is discussed below.
Figure 11 (right): Correlation between UCS and intramicrite. The outlier is circled and omitted from the regression analysis. $R^2 = 86.8\%$.

The alternative (curved/dashed) best fit lines shown in Figs. 6, 8 and 10 suggest that the respective geomechanical parameters may approach a maximum at about 25 - 30% intraparticle material, after which velocities or strengths reduce. Although more data are needed to confirm this relationship, it is a recognised phenomenon in other geomechanical applications (e.g. the optimum quantity of lime added to road foundations to maximise consolidation and stability, above which additional added lime is detrimental to the overall product).

## CONCLUSIONS

The content of interparticle and intraparticle material in a variety of New Zealand limestone types appears to correlate with the results of a number of geomechanical tests, suggesting that the presence (or absence) of the materials, and their types, influence the behaviour of the limestones during testing. The proportion of these materials to grains within the limestones is also shown to be important.

Interparticle material affects the geomechanical behaviour of the *surface* of the limestone, as is shown by correlations with Los Angeles abrasion, slake durability and dissolution rate tests. Intersparite materials are responsible for higher amounts of physical degradation and higher rates of dissolution, whereas intermicrite materials show significant resistance to physical degradation. Conversely, intraparticle material affects the geomechanical behaviour of the *body* of the limestone, demonstrated by correlations with sonic velocity, pointload strength and UCS tests, which impose forces through the specimens (not just at their surface). Intraparticle material, and particularly intramicrite, is conducive to higher strengths and sonic velocities. No significant correlations were observed between interparticle material and strength or sonic velocity tests. Similarly, no significant correlations were observed between intraparticle material and abrasion, durability or dissolution rate tests. This reinforces the conclusion that different types of material influence the behaviour of the limestones during different physical processes.

The correlations described are useful where limestone is to be considered for a particular geotechnical application. During the earliest stages of an investigation, where only simple field textural observations or perhaps published textural information are available, some appreciation of the likely behaviour of the materials may be gauged, prior to rigorous geomechanical laboratory appraisal. Applications involving surface degradation and weathering, such as roading, may best utilise intermicrite-rich limestones, whereas foundation materials, in which structural soundness is more important, might better utilise limestones with an abundance of skeletal grains infilled with intraparticle material. In many applications combinations of both these requirements are essential; the selection of an intermicrite, grain/intraparticle material-rich limestone, or perhaps blends of intermicrite-rich and intraparticle material-rich aggregates, may then be appropriate.

## ACKNOWLEDGMENTS

The permission and cooperation of staff at Golden Bay Cement Quarry, Wilsonville Quarry, Paradise Quarry, McDonald's Lime Quarry, Cook County Quarry and Tangoio Bluff Quarry is gratefully acknowledged.

Staff at the Materials Testing Laboratory, Works Consultancy, Hamilton are thanked for there cooperation and assistance. Raewyn Towes is thanked for her assistance during the project and review of this manuscript. Coal Corporation is acknowledged for its financial support, and endorsement of the presentation of this work.

## REFERENCES

1. Adams A E; Mackenzie W S and Gulsford C 1984. *Atlas of sedimentary rocks under the microscope*. Longman Group Ltd, England.
2. Bieniawski Z T 1975. The pointload test in geotechnical practice, *Engineering Geology* 9: 1 - 11.
3. Brown E T 1981. *Rock characterisation testing and monitoring - ISRM suggested methods*. Pergamon Press, worldwide.
4. Collis L and Fox R A (eds.) 1985. Aggregates: Sand, gravel and crushed rock aggregates for constructional purposes, *Geological Society*, Engineering Geology Special Publication 1.
5. Hawkes I and Mellor M 1970. Uniaxial testing in rock mechanics laboratories, *Engineering Geology* 4(3): 177 - 285.
6. ISRM 1985. Suggested method for determining pointload strength, *Int. J. Rock Mechanics and Mining Sciences and Geomechanical Abstracts* 22(2): 53 - 60.
7. Jakucs L 1977. *Morphogenetics of karst regions*. Akademiai Kiado, Budapest.
8. Knill D C 1960. Petrographic aspects of the polishing of natural roadstones, *N.Z. J. Applied Chemistry* 10: 28 - 35.
9. Lees G and Kennedy C K 1975. Quality, shape and degradation of aggregates. *Q. J. Engineering Geology* 8(3): 193 - 209.

10. McIntosh G D 1994. *Relationships among the geomechanical and petrographic properties of a textural range of limestones, North Island, New Zealand.* Unpublished MSc Thesis, Department of Earth Sciences, University of Waikato.
11. Reijers T J and Hsu K J 1986. *Manual of carbonate sedimentology: a lexicographical approach.* Academic Press, worldwide.
12. Scholle P A 1978. *A colour illustrated guide to carbonate rock constituents, textures, cements and porosities.* American Association of Petroleum Geologists memoir 27.
13. Velleman P F 1992. *Data Desk 4 - statistics guide, vol. 2.* Data Description, Inc.

Appendix Table 1: Field and textural information for the studied limestones.

| Limestone | Name | Locality | $CaCO_3$ (%) | Textural description |
|---|---|---|---|---|
| A | Onerahi Formation | Whangarei | 70 | argillaceous, slightly fossiliferous |
| B | Whangarei Limestone | Whangarei | 98 | crystalline, fine grained, very fossiliferous |
| C | Whangarei Limestone | Whangarei | 97 | crystalline, fine grained, very fossiliferous |
| D | Ruatangata 'Sandstone' | Whangarei | 80 | sandy/terrigenous, slightly fossiliferous |
| E | Otorohanga Limestone | Otorohanga | 99 | crystalline, coarse grained, very fossiliferous |
| F | Otorohanga Limestone | Otorohanga | 99 | crystalline, coarse grained, very fossiliferous |
| G | Patutahi Limestone | Gisborne | 90 | argillaceous, very coarse grained, fossiliferous |
| H | Tangoio Limestone | Napier | 95 | shelly, very porous, very fossiliferous |

Appendix Table 2: General petrographic properties and geomechanical performance of the studied limestones. Interparticle material and grains total 100% in the table, except where limestone porosity is significant. Intraparticle material is counted along with porous grains where appropriate (e.g. Limestone F comprises about 81% grains, the volume of which is made up of approximately half intraparticle material). Not all inter/intraparticle materials are tabulated. Abridged terms are used for interparticle sparite (intersparite), intraparticle micrite (intramicrite), etc. Note that *larger* LA abrasion values and *smaller* slake durability values reflect *less competent* materials.[1]Only one core was tested, hence no standard error. [2]Empirical estimates were used for these values as suitable test specimens were not available. [3]Suitable specimens were not available.

| | *Petrographic Parameters* | | | | | | | *Geomechanical Parameters* | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Limestone** | **Interparticle material (%)** | **Intersparite (%)** | **Intermicrite (%)** | **Intraparticle material (%)** | **Intramicrite (%)** | **Skeletal (and other) grains (%)** | **Interparticle material : grains** | **Sonic velocity (m/s)** | **UCS (MPa)** | **Pointload strength (MPa)** | **Slake durability (% retained)** | **LA abrasion (% lost)** | **Dissolution rate (μS/minute)** |
| A | 91.4 | 0.0 | 86.7 | 0.6 | 0.0 | 8.6 | 10.63 | 3400±190 | 107[2] | 3.1±0.2 | 98.58 | 23.6 | -[3] |
| B | 22.7 | 10.7 | 6.7 | 18.7 | 4.7 | 77.3 | 0.29 | 5900±60 | 150±10 | 6.0±0.1 | 99.62 | 25.5 | 0.189 |
| C | 9.3 | 4.0 | 3.3 | 14.7 | 2.7 | 90.6 | 0.10 | 5700±110 | 135±6 | 5.2±0.1 | 99.57 | 24.6 | 0.173 |
| D | 29.3 | 10.0 | 0.3 | 5.0 | 0.0 | 67.8 | 0.43 | 3840±40 | 96±2 | 4.2±0.1 | 98.64 | 29.6 | 0.182 |
| E | 19.3 | 13.0 | 1.3 | 31.3 | 7.3 | 80.7 | 0.24 | 6060±60 | 178±9 | 6.1±0.3 | 99.48 | 25.7 | 0.175 |
| F | 19.0 | 14.0 | 0.3 | 41.3 | 6.3 | 81.1 | 0.23 | 5642[1] | 144[2] | 5.2±0.3 | 99.46 | 28.8 | 0.218 |
| G | 56.3 | 1.3 | 51.7 | 12.4 | 6.0 | 43.9 | 1.28 | 5080±50 | 145[2] | 5.2±0.2 | 99.31 | 23.7 | 0.222 |
| H | 56.7 | 29.7 | 0.0 | 0.0 | 0.0 | 32.7 | 1.73 | 4900±150 | 45±4 | 2.6±0.2 | 98.92 | 36.8 | 0.225 |

# GEOLOGICAL HAZARD ZONATION AND LAND USE PLANNING ASSESSMENT IN THE SOUTH EASTERN MARLBOROUGH SOUNDS, NEW ZEALAND.

SONIA. T. McMANUS

## 1. INTRODUCTION

The Marlborough Sounds, located at the top of the South Island, New Zealand (Figure 1.), is well known for its scenic beauty. However, the area is less well known for the high degree of weathering and instability associated with the steep slopes. There is a high demand for land in the south eastern Marlborough Sounds due to urban expansion from Picton and Waikawa. Increasingly, the land being selected for house sites is on steeper and potentially more unstable land.

Quaternary climatic influences saw the beginning of the deep weathering profile typical of the Marlborough Sounds today. Periglacial and interglacial periods assisted in the preferential weathering of shear zones and the development of significant amounts of regolith and colluvial material overlying the schistose and greywacke bedrock.

## 2. GEOLOGICAL PROCESSES

*Mass Movement*

Weathering has produced a veneer of unconsolidated surficial material which frequently fails due to the steepness of the slopes and the influence of high antecedent water conditions. Failure types are twofold (Figure 2.). Using the Varnes Classification Scheme, the complex failures in the Marlborough Sounds are either translational/flow failures, or rotational/translational/flow failures. Within thick, >1m, deposits of colluvium and regolith on the lower slopes, the failure types tend to have a rotational headscarp region, the middle of the slide fails in translational motion, while the toe region becomes a flow. On the higher, steeper slopes where the surficial material is <1m failures are predominantly translational with the typical flow features at the base of the slide. Generally, both types of failure occur along the bedrock/surficial material interface which acts as a zone of increased water movement and reduced shear strength.

Several factors are influential in producing the surficial failures readily seen in the Marlborough Sounds. Rainfall is the principal factor which induces failures as high antecedent water conditions are common during the generally wet winters in the Sounds. Failures are most likely to occur when high antecedent conditions combine with the frequent high intensity or long duration rainstorms which occur throughout the year. Additionally, urban development on slopes prone to sliding will often aggravate previous failures or induce new slides. Removal of support at the base of slopes will also promote the development of slope failures. Therefore, roading, cut batters for house sites, and stream bank erosion in gullys all provide potentially unfavourable conditions. Consequently failures are commonly associated with urban expansion and roading. As such, slides often develop below main roads in response to frequent heavy traffic. Large logging trucks are particularly damaging to roads in the area due to their weight and frequent use of the roads. Finally, vegetation removal will generally reduce stability for slopes in excess of 35°, which is approximately the angle of repose for surficial materials. Rain may fall directly onto the ground inducing rilling and additionally no soil moisture is taken up by trees and other vegetation.

*Tunnel Gully Erosion*

In addition to slope failure, the Marlborough Sounds are subject to other geological processes which have hazard potential. Tunnel gully erosion is a widespread feature which is rarely identified. Many tunnels are not obvious and often only become apparent after building has taken place. Tunnels occur generally at the bedrock/colluvium interface, or in relation to tree roots and large rock fragments in thick colluvial material. The collapse of tunnels leads to instability and can also undermine house foundations if they are not properly identified.

*Flooding*

Flooding is also a prominent geological hazard in the south eastern Marlborough Sounds. The Graham River in Whatamango Bay and Waikawa Stream which flows through Waikawa township are both capable of flood events which threaten property. Stream flow responses to high intensity rainfall are sudden with very little lag time identified between rainfall and flood events in the lower reaches.

*Seismicity*

Seismic hazards are important due to the near vicinity of the active Wairau Fault and the occurrence of numerous fault traces within the Marlborough Sounds. However, activity of faults such as the Waikawa Bay Fault has not been determined and therefore, seismic hazards are difficult to quantify.

## 3. HAZARD ZONATION

*Background*

Prior to the work of P.J. Horrey in 1989, the data concerning the geology and soils of the Marlborough Sounds was at large scales (1:50,000) and wholly unsuitable for geotechnical or hazard evaluations. With the introduction of the Resource Management Act and the Building Act in 1991, the requirement for a detailed database of geological or natural hazards became a priority. Local authorities in the Marlborough Sounds saw the need for small scale (1:5000) geological maps identifying the active geological processes and their hazard potential with regard to urban development.

*Methodology*

Geological hazard zonation in the Marlborough Sounds has developed a specific methodology and ultimately leads to decisions regarding land use planning issues. Previous hazard zonation schemes have used the various physical attributes of the land to produce a terrain evaluation. The scale of the final map, therefore, will define the detail of the evaluation. For the purposes of hazard zonation in the Marlborough Sounds, scales of 1:5000 or less require detailed field and limited laboratory analysis of geological processes and materials. Following the GASP system used in Hong Kong, the current hazard zones were determined after analysis of the current natural state of the land. Hazard maps are similar to the GASP physical constraint maps. However, they attempt to quantify the different types of constraints to development rather than just identify them. The maps provide an intermediate level between engineering geological information (Terrain Classification maps) and land use planning maps (GLUM - Geotechnical Land Use Maps). The principal factor in the production of hazard zonation maps is the identification and mapping of geological processes which are deemed potentially hazardous. Hazard assessment is predominantly made on the basis of the time frame between events; the periodicity. Using remote sensing techniques such as air photographic interpretation, detailed field mapping and limited field and laboratory testing, the age and activity of hazardous processes can be estimated .

*Zonation Parametres*

The degree of hazard is subdivided into high, moderate, low and negligible hazards and maps utilise the 'stoplight' method of presentation; red represents high hazard areas while blue identifies negligible hazards. High hazard zones are those areas which have undergone modification by slope movement, flooding, debris deposition or erosion within the last 36 years; the extent of the aerial photography database for the area. Moderate hazard zones, coloured orange, are areas active earlier than 36 years ago. Any unmodified slopes in excess of 35° were also included as being of moderate hazard. Low hazard was allocated to slopes between 15°-35° which had not previously failed, and areas prone to flooding should protection measures fail. Low hazard areas are coloured green. Any areas which are not seen to be affected by hazardous geological processes are allocated negligible hazard.

Additional to the hazard zonations depicted on the maps, additional geological and geomorphic information is included. Features such as headscarps, terraces, runout zones, tunnel gullys and erosion provide evidence of the type of process which presents the greatest hazard. The principal hazard type is allocated an uppercase letter representative of the active process involved. Processes which present lesser degrees of hazard ar represented by lowercase letter. Figure 3 shows the application of hazard zones to the slopes east of Waikawa township

## 4. LAND USE PLANNING

*Methodology*

Following the analysis of hazardous geological processes, land use constraint maps were produced. Again using the principal of the GASP program, these maps are essentially the equivalent of the Geotechnical Land Use maps (GLUM). The development constraint evaluation is derived from the hazard zonation maps, however the assessments are not equivalent. The constraint assessments represent the suitability of land for the purposes of residential development. Therefore, the maps represent the expected foundation conditions, and the necessity for additional, site specific geotechnical investigations. Figure 4. shows the land use planning map derived from the hazard map presented in Figure 3.

Development constraints are subdivided into four classes. Class IV areas have extreme limitations to development due to poor foundation conditions and require extensive geotechnical investigations and remedial measures prior to development. Commonly, such areas equate to zones of high hazard. Class III indicates moderate limitations and the requirement of site specific investigations before residential development is permitted. Class II land is generally suitable for development. Additional geotechnical assessment may be required. Finally, Class I development constraints are applied to the land best suited for urban development and that which is of low or negligible hazard.

While hazard zonation maps represent the natural state of the land at the present time, no attempt is made to interpret land conditions should additional urban development occur. The land use planning maps indicate the expected geotechnical problems and foundation conditions of future landscapes. For example areas of mass movement which are not considered to be active and are of moderate hazard, may be assigned an extreme classification due to the requirement of significant geotechnical investigations prior to development.

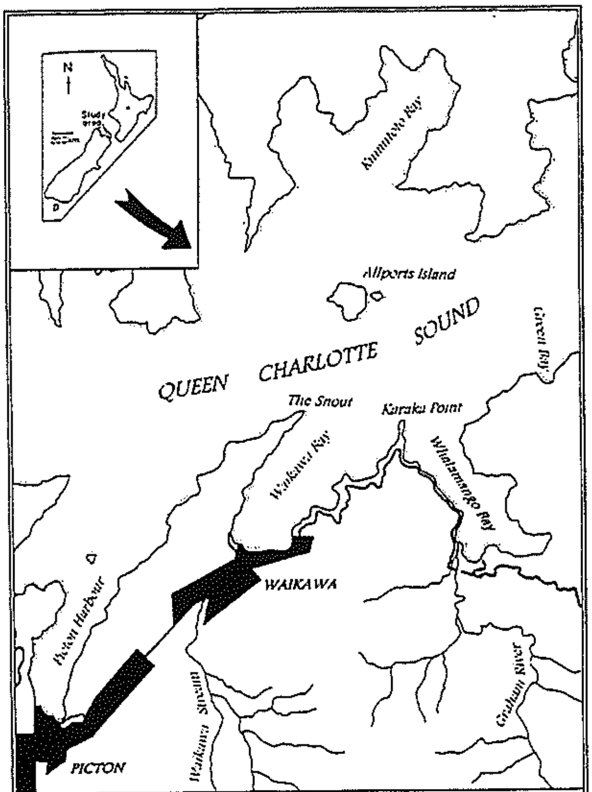



*FIGURE 1. Location diagram of the Marlborough Sounds.*

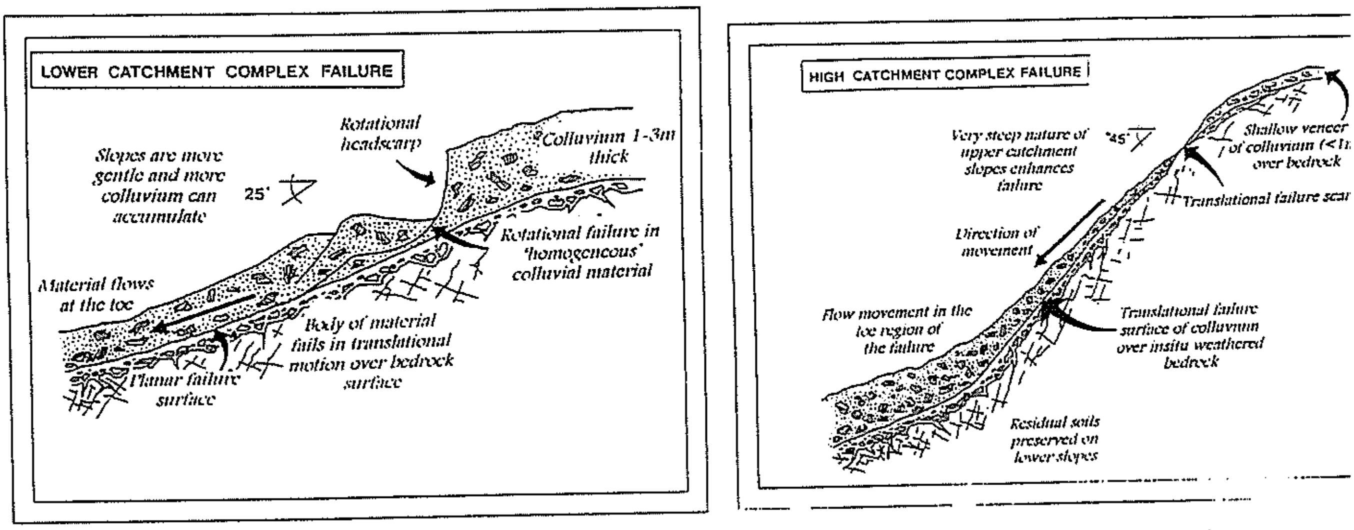
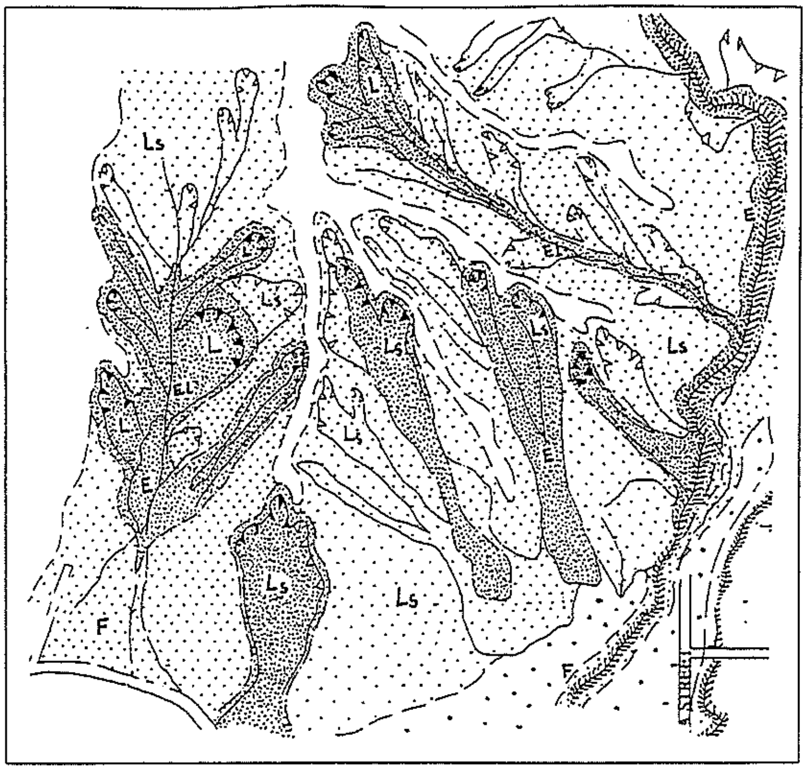



***FIGURE 2.** Schematic diagrams of complex flow types in the Marlborough Sounds*




GEOLOGICAL HAZARD ZONATIONS

| HAZARD | SLOPE MOVEMENT L | FLOODING F | STREAM BANK EROSION E | TUNNEL GULLY T | SEISMIC S |
|---|---|---|---|---|---|
| *HIGH* | Active within the last 36 years | Inundation or siltation within the last 36 years | Active erosion within the last 36 years | Observed collapse or rilling within the last 36 years | 20m either side of active fault traces due to rupture. |
| *MODERATE* | Active more than 36 years ago. Slopes over 35° | Inundation and siltation within a 100 year return period. | Active erosion more than 36 years ago. | Old inactive tunnels formed more than 36 years ago. | Areas of unconsolidated material on 35°+ slopes. Jointed rock faces. Liquefaction in reclaimed areas. |
| *LOW* | Unmodified slopes between 15° and 35° | Inundation and siltation only if flood control measures fail. | | Areas of unconsolidated material less than 1m thick. | Slopes less than 15° and low angle rock faces. |
| *NEGLIGIBLE* | Unmodified slops less than 15° | No Hazard | No Hazard | No Hazard | No Hazard |

*FIGURE 3. Hazard zonation classification and example map showing varying degrees of natural hazard.*

DEVELOPMENT CONSTRAINTS FOR URBAN LAND USE

| | SLOPE MOVEMENT L | FLOODING F | STREAM BANK EROSION E | TUNNEL GULLY T | SEISMIC S |
|---|---|---|---|---|---|
| CLASS IV | Extreme geotechnical limitations. Development generally unsuitable. | Extreme geotechnical limitations. Development generally unsuitable. | Extreme geotechnical limitations. Development generally unsuitable. | Extreme geotechnical limitations. Development generally unsuitable. | Not enough information to assign a constraint classification |
| CLASS III | Significant geotechnical limitations. Extensive site investigations essential. | Significant geotechnical limitations. Extensive site investigations essential. | Significant geotechnical limitations. Extensive site investigations essential. | Significant geotechnical limitations. Extensive site investigations essential. | Not enough information to assign a constraint classification |
| CLASS II | Generally favourable for development. Some site investigations required | Generally favourable for development. Some site investigations required | Generally favourable for development. Some site investigations required | Generally favourable for development. Some site investigations required | Not enough information to assign a constraint classification |
| CLASS I | No limitations to development Residential development recommended. | No limitations to development Residential development recommended. | No limitations to development Residential development recommended. | No limitations to development Residential development recommended. | Not enough information to assign a constraint classification |

*FIGURE 4. Classification for development constraint zonation.*

# USING BENDER ELEMENTS TO DETERMINE ELASTIC SOIL PARAMETERS

**Meyer, V. M. and Pender, M. J.**

Department of Civil and Resource Engineering
The University of Auckland, New Zealand

## SUMMARY

This paper presents results from several bender element tests performed on undisturbed samples of differing soil type. The bender elements enabled both P- and S-wave velocities to be measured. Using the theory of elastic wave propagation, the dynamic elastic parameters $G_{max}$, E, and ν for the soil samples were evaluated. These parameters showed general agreement with soil type.

## INTRODUCTION

The three basic constants of the theory of elasticity are E, G and ν, where E is the modulus of elasticity (Young's modulus), G is the shear modulus of elasticity and ν is Poisson's ratio. These constants are essential parameters in the design and low strain analysis of many geotechnical applications. However, the non-linear inelastic behaviour of soil does not readily lend itself toward experimental determination of these elastic properties.

The small strain shear modulus ($G_{max}$ or $G_o$) of a soil is associated with shear strains (typically < $10^{-3}$%) at which the soil can be considered to behave in an elastic manner. This condition is represented by the maximum shear modulus being invariant with shear strain, as shown by the *strain plateau* in Figure 1.

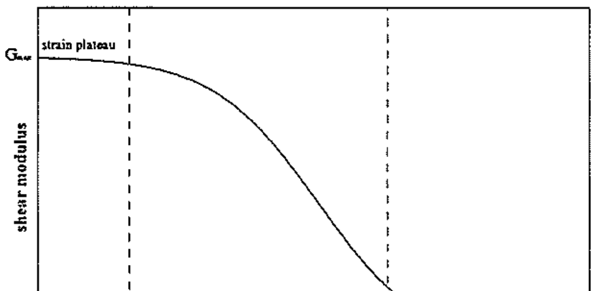



**Figure 1 - Shear modulus variation with shear strain**

$G_{max}$ is commonly evaluated through dynamic in situ or laboratory testing. In situ tests commonly include down-hole (or up-hole) and cross-hole seismic tests. Laboratory tests include dynamic strain or stress controlled triaxial tests, resonant column tests, free vibration torsion tests and bender element tests.

This paper deals with the bender element method of determining $G_{max}$, though the following theory is equally applicable to in situ seismic methods.

## THEORY OF ELASTIC WAVE PROPAGATION

Seismic methods of determining $G_{max}$ are based on the fact that the velocity at which seismic waves travel through a medium is dependent upon the elastic properties of the material. From the theory of plane wave propagation in a homogeneous isotropic elastic material, it can be shown that two types of body waves propagate. The first type is a compressional wave (P-wave), which propagates with a velocity $v_p$, given by:

$$v_p^2 = \frac{E(1-\nu)}{\rho(1+\nu)(1-2\nu)} \tag{1}$$

where ρ is mass density, E is the modulus of elasticity, G is the shear modulus of elasticity and ν is Poisson's ratio.

The second type of body wave is a shear wave (S-wave), which propagates with a velocity, $v_s$, given by:

$$v_s^2 = G/\rho \tag{2}$$

Comparison of equations (1) and (2) reveal that, for the same material parameters, P-waves propagate at a much faster velocity than S-waves, with the difference between the velocities dependent upon the value of Poisson's ratio.

Measurements of $v_p$ and $v_s$ can be made by generating a seismic disturbance at a specific point and recording the time required for the disturbance to reach one or more receivers positioned in the medium. If the distance travelled by the body waves is known, it is possible to calculate $v_p$ and $v_s$ respectively. It is common practice to reverse the polarity of the seismic disturbance, which correspondingly reverses the polarity of the received signal. This enables the first arrival of the shear wave to be more clearly defined (Abbiss [1]).

The dynamic elastic parameters $G_{max}$, E and $\nu$ can be evaluated if the mass density, $\rho$, $v_p$ and $v_s$ of the medium are known, since:

$$G = \frac{E}{2(1+\nu)} \tag{3}$$

Using equations (3), (2) and (1) it can be shown that:

$$\nu = \frac{\frac{1}{2}(v_p/v_s)^2 - 1}{(v_p/v_s)^2 - 1} \tag{4}$$

A $\nu$ value approaching 0.5, corresponds to a very large bulk modulus, or a condition of zero volume change (ie an undrained test). This criteria may well be satisfied for bender element tests on a saturated clay with a low value of $G_{max}$ and a high P-wave velocity.

## PIEZOCERAMIC BENDER ELEMENTS

A bender element is a small transducer comprised of two thin piezoceramic plates rigidly bound together in a sandwich type arrangement. The configuration of the ceramic material is such that it enables the bender element to convert electrical energy in to mechanical energy and vice-versa. Hence, an applied voltage causes the bender element to bend or deflect a small amount, and conversely, the bender element generates a small voltage as it bends. Figure 2 shows a bender element before and after an excitation voltage is applied.


**Figure 2 - Shape of piezoceramic bender element before and after application of excitation voltage (after Dyvik and Madshus [2])**

The use of bender elements to measure the shear wave velocity of soil in laboratory specimens has been described in detail by Dyvik and Madshus [2]. Test results presented in this paper were obtained by mounting bender elements in a triaxial cell using the techniques developed by Dyvik and Madshus. A brief summary of the techniques employed is given below.

The bender element must be protected from moisture to prevent electrical shorting of the transducer. This was achieved by encasing the bender elements in an epoxy resin. The encased bender elements were then mounted in both the top and bottom end platens, as shown in Figure 3.

A porous disc with a corresponding slot could be mounted over the bender element if required. The slot in the porous disc was large enough to ensure that it did not interfere with the operation of the bender element. The resultant length of bender element protruding in to the soil specimen was of the order 9.5mm with the porous disc and 11mm without.

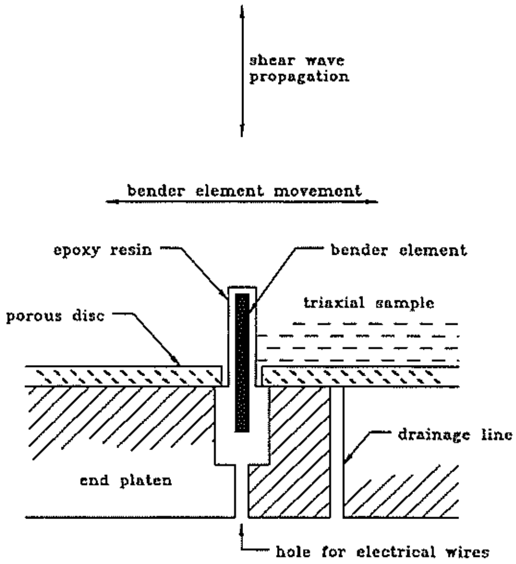



**Figure 3 - Bender element mounting in triaxial cell**

Electrical wires connected to the bender elements were sealed in the end platens and exited the triaxial cell using pressure resistant fittings.

The bender elements were configured such that the top element could act as a transmitter and the bottom element as a receiver. Using a function generator, an electrical signal was sent to the transmitter element and the time interval until arrival at the receiver element was measured using an oscilloscope, as shown in Figure 4.

## INTERPRETATION OF BENDER ELEMENT RESULTS

Initial interpretation of bender element results is relatively straight-forward. To determine the velocity of a particle in motion (or a wave) it is necessary to know the time required for the particle to travel through a known distance. Previous test results (Viggiani and Atkinson [9], Nishihara [4]) have shown that for bender element testing the distance through which the first arrival waves travel, is the length between the tips of the bender elements. This length is commonly referred to as the *tip length* or *effective length*. The time required for the waves to travel through the effective length can be assessed as the time interval between sending the input signal to the transmitter element and the point of first arrival of the attenuated waveform at the receiver element.

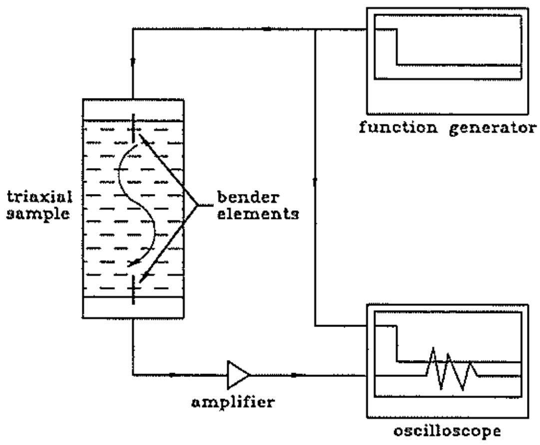



**Figure 4 - Schematic representation of equipment used for bender element tests**

Figure 5 shows an idealised trace record taken from an oscilloscope. From this trace it can be observed that an initial pulse is sent to the transmitter at time, $t_0$, and the first arrival wave is recorded at the receiver at time, $t_1$. The body wave velocities, $v_p$ and $v_s$, can then be defined as:

$$v_p \text{ or } v_s = {}^{l_e}\!/_{\Delta t} \qquad (5)$$

where $l_e$ is the effective or tip length and $\Delta t = (t_1 - t_0)$.

The above method of determining body wave velocities is based on visual estimation of the time of arrival at the receiver element. This is the most commonly used method to calculate seismic wave velocities and is termed the method of *direct times of arrival*. Body wave velocities presented in this paper were determined using this method. Other methods include interval times of arrival, methods based on the cross-correlation function and methods based on dispersion curves obtained from the cross spectrum or transfer function. Viggiani and Atkinson [9], Mancuso, Simonelli and Vinale [3], and Sánchez-Salinero, Roesset and Stokoe [5] present further information on these methods of analysis.

## DIFFICULTIES ASSOCIATED WITH BENDER ELEMENT TESTING

Figure 5 is a rather idealised presentation of actual bender element results. Whilst there is general agreement that the effective wave path length is the tip length of the specimen, determination of the point of first arrival is rather more subjective, as highlighted in Figures 6 and 7. Visual methods of estimating arrival times require a degree of judgement by the observer, which is unlikely to produce exact solutions for body wave velocities. The development of mathematical/analytical methods for determining arrival times can produce solutions which are more accurate than those determined by visual methods, though it should be noted that analytical methods were developed primarily for in situ seismic testing which usually incorporate more than one receiver.

### Near-Field Effects

Analytical studies by Sánchez-Salinero, Roesset and Stokoe [5] have shown that for two-dimensional in-plane

(SV-motion) and three-dimensional elasto-dynamic motion two types of body waves were present in the waveform solutions. One wave travelled at the P-wave velocity, the other at the S-wave velocity. It was observed that for S-wave excitation the amplitude of the P-wave attenuated much faster than the dominant S-wave. It was concluded that the P-wave was only significant at distances which were 'close' to the source and is commonly referred to as the *near-field wave*. The S-wave in this case is referred to as the far-field wave. Similarly, for P-wave excitation, a near-field S-wave and a far-field P-wave were observed. When the direction of the excitation impulse is reversed both the near- and far-field terms change polarity.

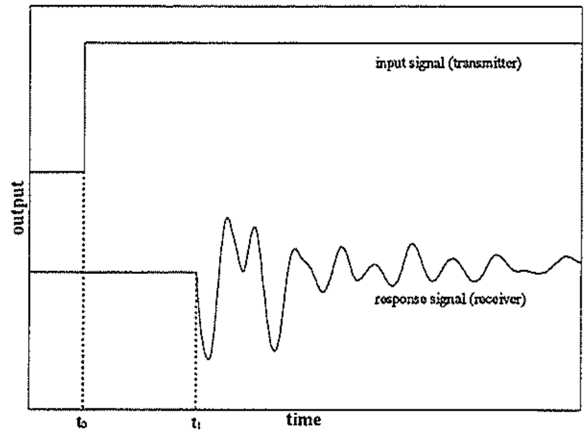


**Figure 5 - Idealised bender element trace record**

The orientation of the bender element shown in Figure 3, is such that the element is effectively a cantilever surrounded by soil. When the bender element is excited, this configuration results in shear waves emanating from the element and propagating through the soil in a direction parallel to the axis of the specimen, ie SH-waves. Considering a bender element test of a triaxial sample in three-dimensions, it is expected that a near-field P-wave will be present. The effect of this near-field wave varies, though in general, can mask the arrival of the shear wave.

## EXPERIMENTAL TEST PROCEDURES

The results presented in this paper were obtained from tests performed on samples contained in conventional triaxial cells. All samples were in their natural (undisturbed) state, typically 76mm in diameter and 152mm in length. Bender elements were installed in the top and bottom platens, as described previously. It was necessary to pre-cut a fine slot in each end of the sample to facilitate insertion of the bender elements without damage. The bender element tests were performed on the samples immediately following the consolidation stage.

The equipment used in each bender element test included a Yokogawa FG110 2MHz synthesised function generator and a Yokogawa DL1200A 4 channel 100MHz digital oscilloscope. It was necessary to incorporate an amplifier (gain=56) to improve readability of the output signal. From previous testing it was also observed that earthing the sample similarly improved the signal output. The polarity of the input signal was reversed to assist determination of wave arrival times.

The oscilloscope permitted direct reading of time intervals using on-screen measurement cursors. Additionally, many signal traces were down-loaded directly from the oscilloscope to a personal computer for later analysis. The maximum resolution of down-loaded signals was 10000 points per trace.

## TEST RESULTS

Bender element tests were performed on four undisturbed samples of different soil type. Sample descriptions and a summary of soil properties are detailed in Table 1. A series of tests were performed on each sample utilising a square pulse with an amplitude in the range of ±4V to ±20V as an input signal. The frequency of the square pulse was 10Hz. Figures 6 and 7 show bender element trace records for sample 1. All wave velocities were determined using the method of direct times of arrival and the tip length of each specimen

Figures 6 and 7 show very different first arrival times, 0.99ms and 0.18ms respectively. Considering the theory of near field effects, it is apparent that the these arrival times correspond to $T_s$ and $T_p$ respectively. Reversal of the polarity of the input signal reverses the output for both frequencies tested. Table 2 summarises arrival times and wave velocities for the square input wave.

Using equations (2), (4) and (3), the dynamic elastic parameters $G_{max}$, E and $\nu$ can calculated for the square impulse, as shown in Table 3. The undrained shear strength, $s_u$, for each sample, was measured using a consolidated undrained triaxial test. A loading rate of 10% axial strain per hour was adopted for all tests. Table 3 shows a summary of measured undrained shear strengths.

**Table 1 - Summary of sample properties**

| Sample No. | Soil Description | Sample Depth (m) | Water Content (%) | Bulk Density (kg/m$^3$) |
|---|---|---|---|---|
| 1 | Firm light brown silty CLAY (Waitemata Series) | 2.7 | 48.4 | 1680 |
| 2 | Soft dark grey CLAY with black organic material | 2.9 | 52.3 | 1640 |
| 3 | Firm dark brown clayey SILT with coarse sand intermixed | 2.0 | 53.9 | 1480 |
| 4 | Firm light brown silty fine SAND with occasional pumice sand intermixed | 5.1 | 50.4 | 1620 |

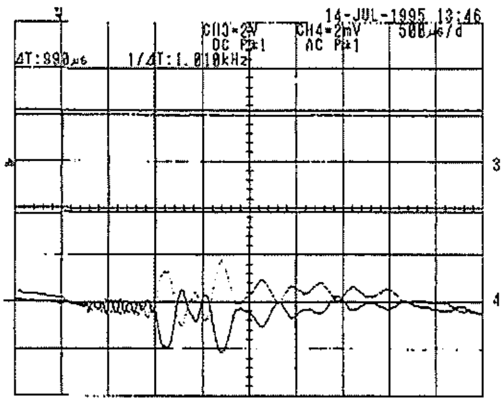


Figure 6 - Square impulse 10Hz (0.5ms/div)

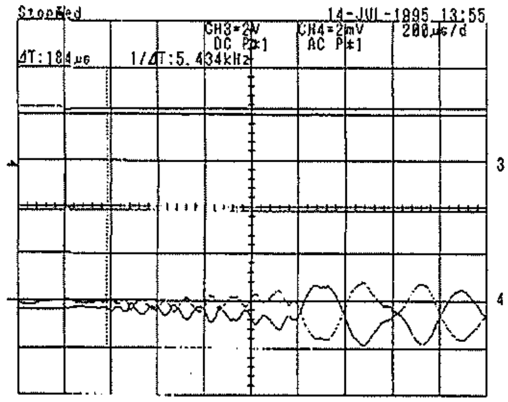


Figure 7 - Square impulse 10Hz (0.2ms/div)

**Table 2 - Body wave velocities (square impulse)**

| Sample No. | Confining Pressure (kPa) | $T_p$ (ms) | $T_s$ (ms) | $v_P$ (m/s) | $v_s$ (m/s) |
|---|---|---|---|---|---|
| 1 | 27 | 0.18 | 0.99 | 783 | 143 |
| 2 | 30 | 0.14 | 2.05 | 949 | 65 |
| 3 | 20 | 0.17 | 1.08 | 762 | 120 |
| 4 | 53 | 0.21 | 1.09 | 613 | 118 |

**Table 3 - Elastic soil parameters (square impulse)**

| Sample No. | $G_{max}$ (MPa) | E (MPa) | ν | $s_u$ (kPa) |
|---|---|---|---|---|
| 1 | 34.0 | 100.8 | 0.483 | 60 |
| 2 | 6.9 | 20.7 | 0.498 | 22 |
| 3 | 21.3 | 63.3 | 0.487 | 138 |
| 4 | 22.6 | 66.9 | 0.481 | 412 |

## DISCUSSION AND CONCLUSIONS

The results have shown that it is possible to measure both P- and S-wave velocities using bender elements mounted in a triaxial cell. Knowledge of both these velocities enable the dynamic elastic parameters $G_{max}$, E and ν to be calculated. For the tests presented in this paper, ν values greater than 0.48 were calculated for all samples, indicating that bender element tests can be modelled essentially in an undrained manner.

The P-wave velocity measured in these tests is likely to be the velocity of the near-field P-wave associated with primary SH-wave motion in three dimensional space, as described by Sánchez-Salinero, Roesset and Stokoe [5]. To measure the P-wave velocity of a soil, it is best to use a source rich in P-waves and not rely upon a near field P-wave. However, the results for the square impulse presented in this paper, indicate that $v_p$ can be consistently measured with reasonable accuracy, enabling elastic parameters for a soil specimen to be obtained quickly from a bender element test.

The use of a sine pulse as an input signal was investigated, however it was observed that the sine impulse demonstrated the frequency dependent response of the soil. This response is to be expected, as a input signal will impart energy to the specimen with a frequency range dominated by the frequency of the input signal. Consequently, it was easier to observe the higher frequency P-wave response of the soil by testing with a higher frequency sine impulse. The square input wave, on the other-hand, has a very wide range of input frequencies, enabling P- and S-waves to observed independently of the frequency of the input signal. The sharp fronted wave induced by the square impulse provides output similar to that observed from in situ seismic tests (Stokoe and Woods [7]), with better wave discrimination than that produced by the sine impulse. For these reasons, it can be concluded that, in general, the square input wave provides the clearest bender element response.

The elastic parameters calculated for each specimen show general agreement with soil type; $G_{max}$ tending to increase with $s_u$. It should be noted that $s_u$ for samples 3 and 4 (sandy samples) were associated with large negative pore water pressures. Considering samples 1 and 2 (clays) only, $G_{max}$ is of the order of 300 to 600 times $s_u$. In comparison, Seed and Idriss [6] present in situ test results where $G_{max}$ ranges from approximately 1000 to 5000 times $s_u$, This would indicate that the $G_{max}$ values determined from the bender element tests may be low. Several factors may contribute to this discrepancy and include sample disturbance, soil anisotropy and confining pressure.

Some sampling disturbance is inevitable with laboratory testing of 'undisturbed' soil samples. The implications of sample disturbance on bender element testing is not well documented, though any damage to a specimen would most likely result in a reduction of $G_{max}$. This reduction would be dependent on the degree of sample disturbance. The effects of soil anisotropy are discussed by Sully and Campanella [8], who highlight that in situ crosshole and downhole seismic tests may yield different shear wave velocities. It was concluded that this variation in velocity with test direction could provide an indicator to structural anisotropy of the soil. The variation of $G_{max}$ with confining pressure has been well documented (Viggiani and Atkinson [10]), and in general, $G_{max}$ approximately varies with confining pressure raised to the power of 1/3 to 1/2.

It should be noted that the in situ test results presented by Seed and Idriss [6] do not account for the effect of either soil anisotropy or confining pressure, thus direct comparisons of these results with those from bender element tests may not be accurate. In any case, it is apparent that further research into the use of bender elements to determine dynamic soil properties is required, particularly for soils in their natural state.

## ACKNOWLEDGMENTS

The authors are grateful Mr. G. C. Duske for setting up the equipment necessary to perform the bender element tests, and Dr. T. J. Larkin for providing assistance with interpretation of test results.

## REFERENCES

1. Abbiss, C. P., 1981, Shear wave velocity measurements of the elasticity of the ground, *Geotechnique* 31(1): 91-104.
2. Dyvik, R. and Madshus, C., 1985, *Lab measurements of $G_{max}$ using bender elements*, Advances in the Art of Testing Soils Under Cyclic Conditions, Proceedings of a session of the ASCE Convention, Detroit, pp. 186-196.
3. Mancuso, C., Simonelli, A. L. and Vinale, F., 1989, *Numerical analysis of in situ S-wave measurements*, Proceedings of the 12th International Conference on Soil Mechanics, Vol. 3, pp.277-280.
4. Nishihara, A., 1993, Unpublished laboratory work.
5. Sánchez-Salinero, I., Roesset, J. M. and Stokoe II, K. H., 1986, *Analytical studies of body wave propagation and attenuation*, Report GR86-15, The University of Texas at Austin.
6. Seed, H. B. and Idriss, I. M., 1970, *Soil Moduli and damping factors for dynamic response analysis*, Report No. EERC 70-10, Earthquake Engineering Research Centre, University of California, Berkeley, California.
7. Stokoe, K. H. and Woods, R. D., 1972, In situ shear wave velocity by cross-hole method, *ASCE Journal of the Soil Mechanics and Foundations Division* 98(SM 5): 443-460.
8. Sully, J. P. and Campanella, R. G., 1995, Evaluation of in situ anisotropy from crosshole and downhole shear wave velocity measurements, *Geotechnique* 45(2): 267-282.
9. Viggiani, G. and Atkinson, J. H., 1995, Interpretation of bender element tests, *Geotechnique* 45(1): 149-154.
10. Viggiani, G. and Atkinson, J. H., 1995, Stiffness of fine-grained soil at very small strains, *Geotechnique* 45(2): 249-265.

# PIEZOCONE PENETROMETER TESTING AND DIMENSIONLESS EXCESS PORE PRESSURE

Adam B. Milligan
Department of Civil Engineering
University of Canterbury

October, 1995

## Introduction

The use of the piezocone (CPTU) probe has become increasingly popular and is currently one of the most widely used *in situ* investigation devices. The addition of a pore pressure measuring system to the cone penetration test (CPT) provides much more information about the *in situ* soil and thus enables a much clearer picture of the subsurface soil conditions to be obtained.

The pore pressure measured during a CPTU test in a saturated soil consists of two principal components. Firstly, the hydrostatic pore pressure, $u_0$, and secondly a component, $\Delta u$, generally termed the excess pore pressure, that represents the change in water pressure caused by penetration of the instrument. This excess pore pressure is a function of the soil permeability and drainage, and therefore its grain size, as well as the mechanical characteristics of the soil, in particular, the density and dilatancy characteristics. Interpretation of excess pore water pressure is complicated by the fact that it is also affected by the drainage conditions, soil consolidation behaviour, and the rate of loading during the test.

Berrill et al. (1992) proposed a simple non-dimensional form for excess pore pressure in an attempt to minimise the effect of differing soil drainage conditions and rate of loading. With these factors eliminated the resulting dimensionless excess pore pressure, $\Delta U$, is intended to provide a better indication of the *in situ* state of the soil than does the raw excess pore pressure, $\Delta u$, and thus enable better interpretation of piezocone results. The dimensionless form of excess pore pressure is given by:

$$\Delta U = \frac{\Delta u}{\gamma_w v_f t_{50}}$$

In this expression, $\Delta u$ is the raw excess pore pressure, $\gamma_w$ is the unit weight of water, $v_f$ is the penetration rate, and $t_{50}$ is the time for half of the excess pore pressure to dissipate.

The initial intention of this project was to test this expression for the normalisation of excess pore pressure. However the investigation became sidetracked into a technical problem which arose from a combination of the modification of the *in situ* soil state and the clogging of the piezocone filter element.

## Observations

The results of a series of CPTU tests carried out using differing penetration rates, at the St Johns Street Water Pumping Station in Christchurch are shown in the plots of point resistance ($q_c$), friction ratio ($R_f$), and pore pressure (u) against depth of penetration in Figure 1 to 4. The subsurface material at the site is interpreted to consist of a large cohesive layer up to three metres thick overlying dense sand.

The Fugro piezocone appears to provide reliable data for $q_c$ and $R_f$; however the values for pore pressure are somewhat uncertain. Of particular interest is the zone of extremely high negative excess pore pressures from approximately 2.6 m through to about 3.6 m below the ground surface. These negative pore pressures are particularly strange since they follow a region of large positive pore pressures in apparently the same material (based on the similar $q_c$ and $R_f$ values). The change in behaviour follows the rod change and dissipation test at approximately 2.6 m for tests SJS003, SJS004, and SJS005.

Originally the high negative pore pressures were thought to be due to inadequate saturation of the hydraulic circuit of the piezocone. Test SJS006 was performed following the use of an extremely rigorous deairing procedure to ensure complete saturation. The results of this test (Figure 4) indicate that inadequate saturation was not a primary factor contributing to the unusual pore pressure records.

Inspection of the filter used for test SJS006, and subsequently that used for the previous tests, shows that clogging of the filter in the fine-grained layer is the principal cause of the spurious pore pressure records (Figure 5). A soil sample of the fine-grained layer from a depth of 2.2 m was tested in the laboratory. A summary of the test results and a grain size distribution are shown in Table 1 and Figure 6 respectively. The soil consists predominantly of silt material with a minor clay fraction (approximately 15%). The filter element used in these tests has a pore opening of approximately 150 μm. The average particle size of the soil (13 μm) is therefore considerably finer than the pore size of the filter element and clogging is therefore possible and likely to occur.

**Table 1: Results of the laboratory tests performed on a soil sample from a depth of 2.2 m at the St Johns Street Water Pumping Station site.**

| | |
|---|---|
| $d_{30}$ | 6 μm |
| $d_{50}$ | 13 μm |
| $d_{60}$ | 17 μm |
| $I_L$ | 33 |
| $I_P$ | 23 |
| Unified Classification | ML, clayey silt |

It is interesting to note that for tests SJS003, SJS004, and SJS005 the filter clears itself and the pore pressures measured in the clean sand layer seem to be accurate. However for SJS006 the filter is much more clogged than for the previous three tests (Figure 5) and does not clear itself at any point during the test. Furthermore, there does not appear to be any consistency between the three tests in the point at which the filter appears to clear itself.

## Interpretation

This apparently anomalous behaviour may be explained as follows. The large positive excess pore pressures measured at the beginning of penetration in the fine silt layer are as expected for this type of loose, estuarine material. The large negative pore pressures measured following the first rod change and dissipation test are not expected. When penetration is stopped at this first rod change and the large positive excess pore pressures allowed to dissipate, the soil surrounding the tip of the piezocone will consolidate as the local effective stresses increase. This consolidation will result in the soil being more dense than its original *in situ* state.

As penetration resumes, movement of the cone causes dilation of the now dense material and generation of negative excess pore pressures. Cavitation may also occur within the pore pressure transducer if the pore pressure decreases sufficiently. These negative excess pore pressures lead to a tendency for outflow from the (now imperfectly?) saturated pore water pressure system. As the cone passes through the consolidated region and into virgin material (in a matter of centimetres of movement) the excess pore pressure becomes positive again and therefore leads to an inflow into the pore water pressure system and a corresponding clogging of the filter by the fine silt particles.

## Conclusions

The problems of filter saturation and clogging cast doubt on the accuracy of the recorded values of $\Delta u$. Until this doubt is removed it is difficult to draw any conclusions about the correctness of the relationship for the normalisation of excess pore pressure.

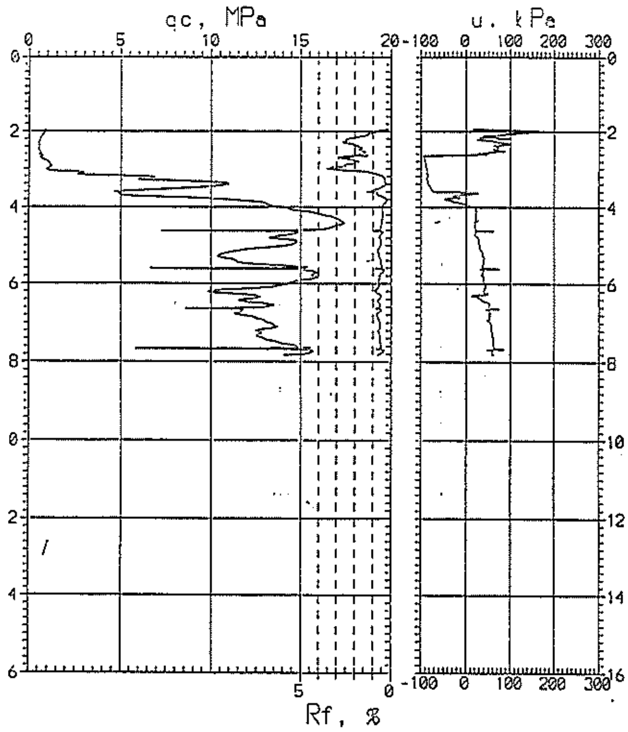
This investigation highlights the importance of complete saturation of the pore water pressure system of the piezocone in order to obtain accurate and significant results. It appears that the procedure currently in use at the University of Canterbury for deairing the Fugro piezocone is inadequate for testing in fine-grained soils and should therefore be modified.

Filter pore size has a significant effect on the reliability of pore pressure records. If the pores of the filter element are large relative to the soil particles at the site to be tested than clogging of the filter may occur and the pore pressure measurements will be adversely affected. On the other hand, too fine a filter may adversely affect response time in free-draining soils. Thus special consideration needs to be given to the pore size of the filter and the filter material used when performing CPTU tests.

The problem of filter clogging that occurred during piezocone testing at the St Johns Street Water Pumping Station site clearly adversely affected the pore pressure records. To further investigate the proposed normalisation of excess pore water pressure, either a sandy site free from overlying silt should be sought, or alternatively, more tests should be carried out at the present site using filters with finer pore sizes that will not clog during the test.

## Reference

Berrill, J.B., Canou, J., Foray, P. and Pautre, J.-L. (1992) "Piezocone testing of liquefaction sites: Normalisation of excess pore pressure", Proc. 10th World Conference on Earthquake Engineering, Madrid, pp 1421-1424.




igure 1: CPTU Test SJS003 with 10 cm$^2$ Fugro cone at St Johns St Water Pumping Station, hristchurch, New Zealand. $v_r = 20$ mm/s.

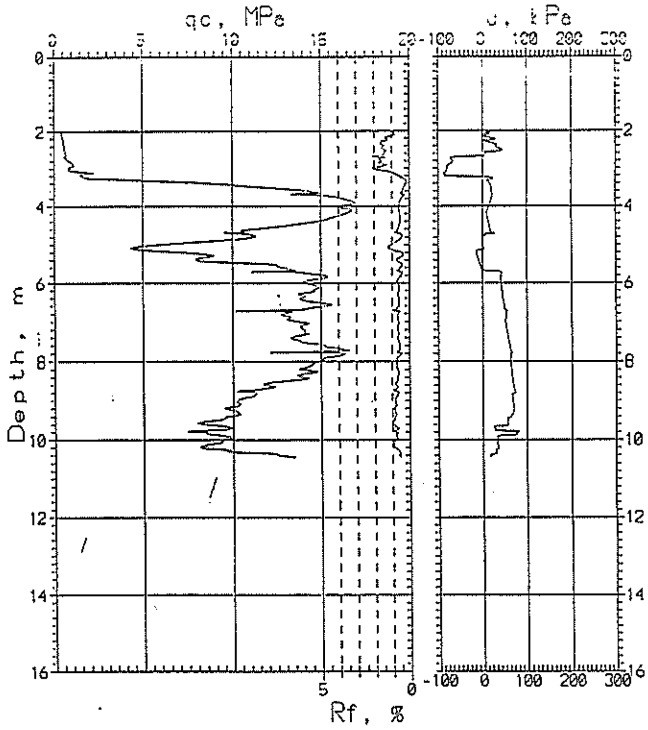
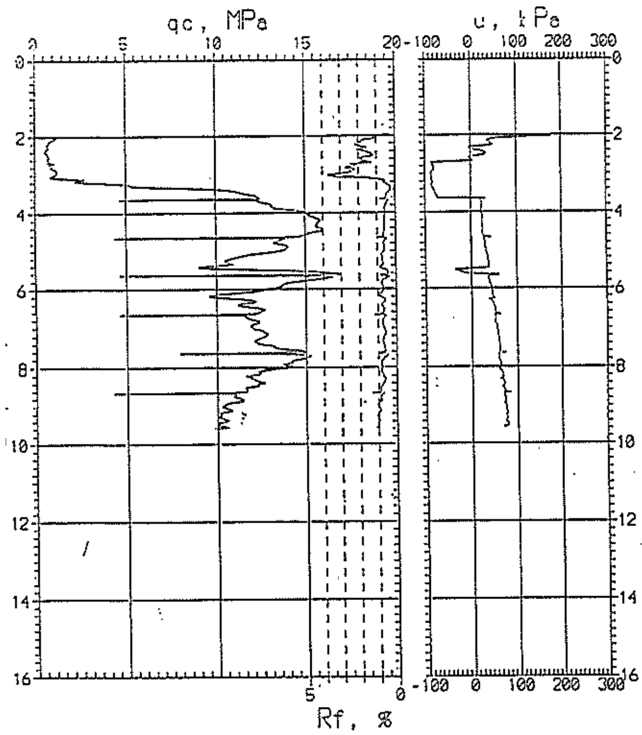



Figure 2: CPTU Test SJS004 with 10 cm$^2$ Fugro cone at St Johns St Water Pumping Station, Christchurch, New Zealand. $v_r = 40$ mm/s.




'igure 3: CPTU Test SJS005 with 10 cm$^2$ Fugro cone at St Johns St Water Pumping Station, Christchurch, New Zealand. $v_r = 10$ mm/s.

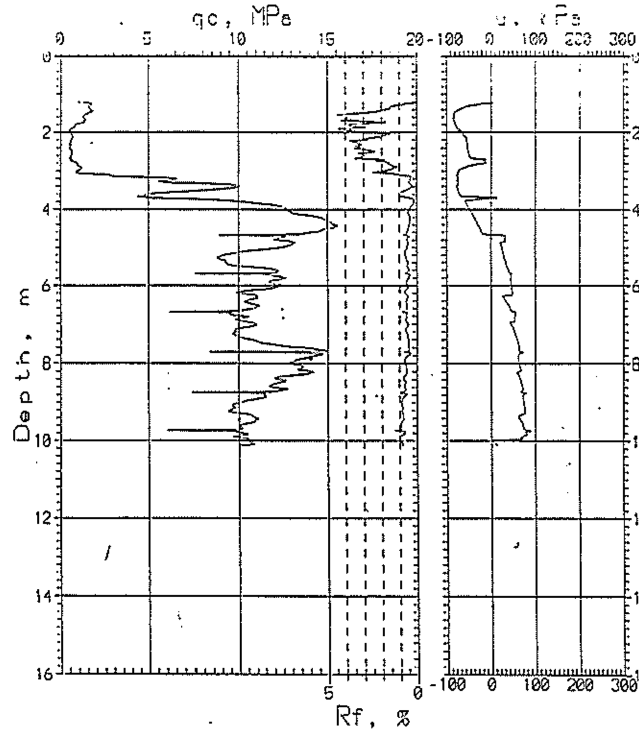



Figure 4: CPTU Test SJS006 with 10 cm$^2$ Fugro cone at St Johns St Water Pumping Station, Christchurch, New Zealand. $v_r = 20$ mm/s.

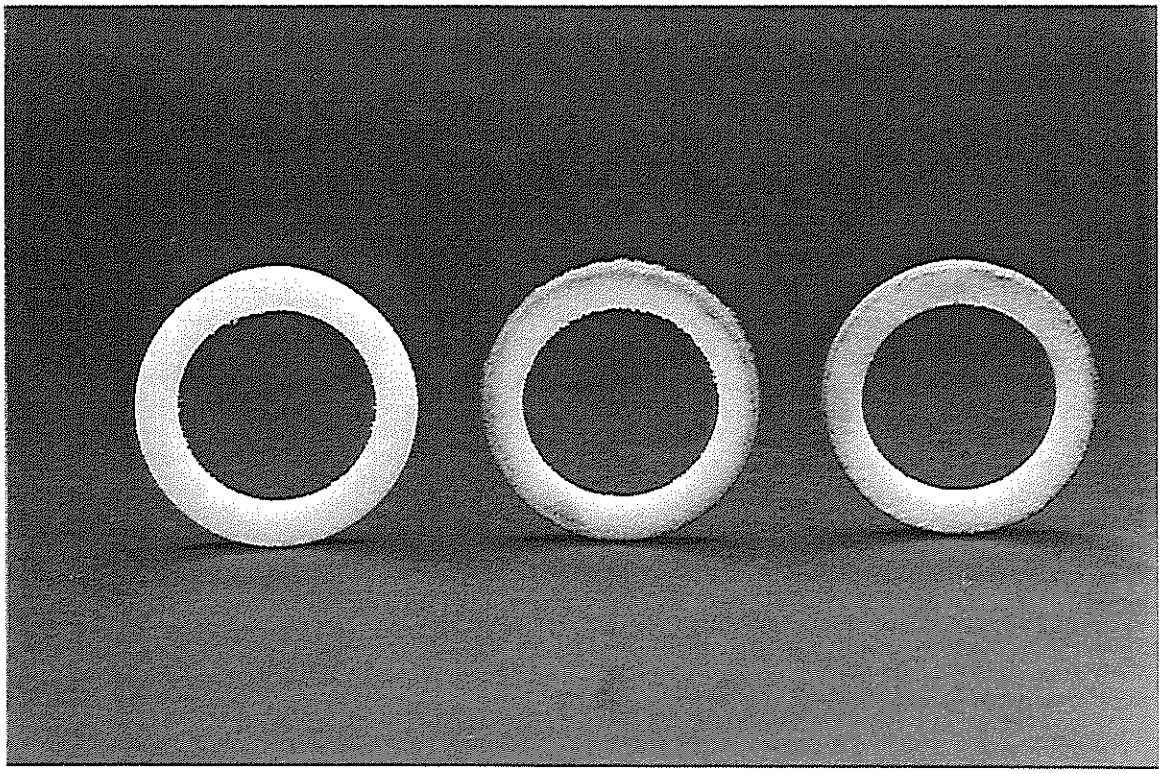


Figure 5: The filters used in the CPTU tests at the St Johns Street Water Pumping Station site. From left to right these are (1) an unused filter, (2) the filter used in for test SJS006, and (3) the filter used for tests SJS003, SJS004, SJS005.

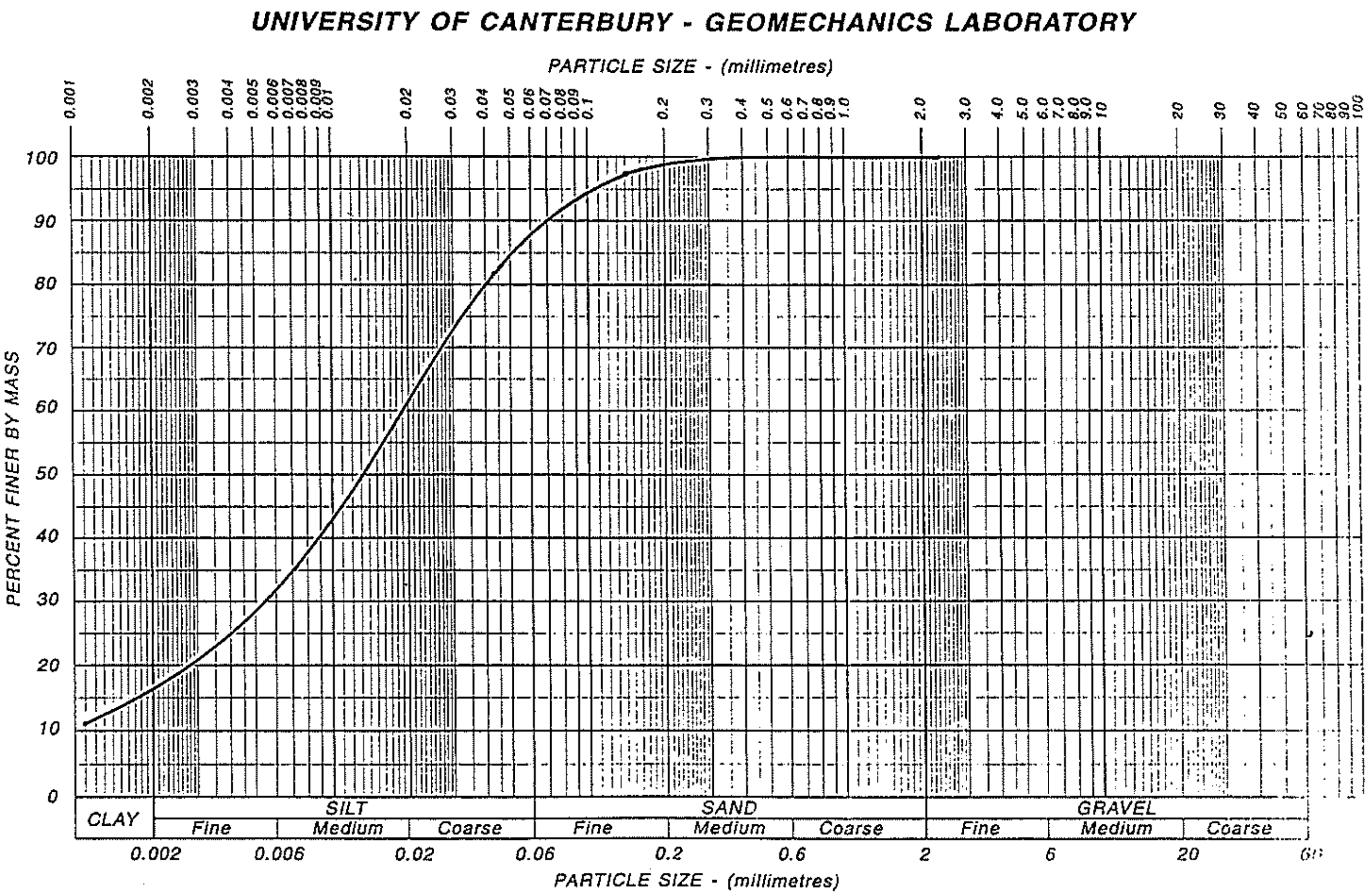



Figure 6: Grain size distribution curve for a soil sample from a depth of 2.2 m at the St Johns Street Water Pumping Station site.

# PRECONSOLIDATION EFFECT IN NORMALLY CONSOLIDATED AGED CLAYS

A K MURASHEV

Beca Carter Hollings & Ferner Ltd
Wellington

## SUMMARY

This paper provides evidence from research and practice that most of natural normally consolidated clays are in fact preconsolidated due to aging. Examples of normally consolidated aged clays, including a clay investigated by the author at the site for the proposed Seaview Sewage Treatment Plant (New Zealand) are presented. Test procedures specifically designed to detect preconsolidation due to aging are described.

## INTRODUCTION

A number of investigators have noticed that many clays and silts expected to be normally consolidated are shown by laboratory test results or field settlement monitoring data to be apparently preconsolidated. The laboratory tests on these clays and silts indicate a preconsolidation pressure ($p_c$) which is greater than present effective overburden pressure ($p_o$) in the ground at the level from which the samples are taken. Very often in addition to the laboratory consolidation test evidence, the field shear vane tests indicate undrained shear strength greater than would be expected from normally consolidated material.

It is very often assumed that only two mechanisms can cause preconsolidation (either erosion of the ground surface which have reduced overburden pressure, or variations of the groundwater level). In many cases the preconsolidation effect can not be explained by these traditional explanations. As utilisation of the preconsolidation effect often permits substantial cost savings in practical applications, the attention of many investigators around the world has been drawn to this phenomenon.

Until 1994 the author worked as an Associate Professor at the Chelyabinsk State Technical University in Russia and for 10 years was involved in large scale investigations of natural clays and silts. As a part of these investigations a new system for triaxial testing of soils was designed and constructed. Tests undertaken as part of an extensive investigation programme led the author to the conclusion that most of the natural normally consolidated clays are in fact preconsolidated due to aging. The author believes that the aging effect is of sufficient importance as to be taken into account in engineering design and practice.

## EXAMPLES OF NORMALLY CONSOLIDATED AGED CLAYS

Leonards and his students first described this phenomenon of preconsolidation in apparently normally consolidated clay more than 30 years ago [8, 9]. They showed that normally consolidated soils can develop an apparent overconsolidation ratio of 1.4, simply by letting the clay age in secondary compression. After artificially sedimenting a clay and then letting it age for 200 days it was discovered that the clay had an overconsolidation Ratio (OCR) of 1.4 because of the ageing. Moreover Leonards noted that in his experience with natural clay deposits he had never encountered a supposedly normally consolidated clay that did not show some aging - preconsolidation effect.

Schmertmann [13] reviewing data of other writers noted that the soft Mississippi River delta clays around New Orleans and Baton Rouge, typically normally consolidated from a geologic view point, provide many examples of the aging - preconsolidation effect. Schmertmann refers to L Capozzoli who reported that he had many times successfully taken advantage in practice of overconsolidation caused by secondary consolidation to support structures over these clays, with only small observed settlement. Schmertmann reported experimental results showing the development of an apparent OCR equal to 1.5-2 for a normally consolidated Italian clay. Further he reported results showing the development of a 10% ageing preconsolidation effect in a clean uniform air-dry sand from Florida demonstrating that the age-strengthening effect can result not only from increased cohesion, but also from increased soil friction.

The most recent example is a silt encountered during geotechnical site investigations for the proposed Seaview Sewage Treatment Plan (New Zealand). In 1994, Beca Carter Hollings and Ferner Ltd was engaged by the Hutt City Council to investigate a number of sites in the Seaview area which were being considered as possible sites for a treatment plant. It was noted during those site investigations that silts at two sites adjacent to the Wellington Harbour and expected to be normally consolidated were indicated by test results to be apparently preconsolidated. The laboratory tests on the silt materials indicated preconsolidation pressures of approximately 200 KPa greater than effective overburden pressure in the ground at the level from which the samples were taken. In addition to the laboratory consolidation test evidence, the field shear vane tests indicated strength greater than would be expected from a normally consolidated material. The preconsolidation of the silts in this case could not be explained by any traditional explanation because geological evidence is that the silts have never had more than 3m greater overburden thickness than at present and have never been above the water table.

## CAUSES OF OVERCONSOLIDATION IN CLAY

The most essential factors which bring a clay to overconsolidation state are as follows: (1) Secondary compression; (2) Geotechnical processes due to weathering; (3) Cold - welding of mineral contact points between particles; (4) Exchange of cations; (5) Precipitation of cementing agents; (6) Desiccation stress; (7) Mechanical aging resulting from increased basic soil friction, including dilatancy effects.

Secondary compression is a phenomenon where the void ratio decreases with time under a constant value of vertical stress. According to the concept proposed by Bjerrum [4], the additional structure developed by secondary compression increases linearly with increasing overburden pressure. Chemical bonding is a generalised term including cold welding of mineral contact points between particles; exchange of cations; cementation [3]. It is supposed that a bond structure is developed under constant value of the void ratio and the additional structure developed by chemical bonding is constant irrespective of the value of overburden pressure [7]. Desiccation is accompanied by an increase in internal effective stress and in turn a decrease in a void ratio.

The most interesting point of view has been presented by Schmertmann [13] who reported experimental results indicating that during aging of all soils (clays, silts and sands) the soil structure is densified by dispersive particle movements. Schmertmann gave many examples of aging improvement in soil engineering behaviour, both from the laboratory and the field, for short (from a few days to 100 years) and long aging times, and for clays, silts and even gravel. He tried to answer the question as to whether these improvements in soil behaviour result from an increase in the cohesive component of mobilised strength as a result of some form of cementation or internal bonding, as almost all references suggest, or from an increase in the frictional or mechanical component of strength. On the basis of experimental investigation, Schmertmann stated that age - strengthening effects result from increased basic soil friction, including dilatancy effects, and not from increased cohesion. The ageing mechanism includes grain slippage, soil - structure dispersion, increased interlocking, and postulated internal arching of stresses. According to Schmertmann the age - strengthening effect can develop relatively quickly because of the horizontal stress fluctuation.

## TEST PROCEDURES

A great number of methods have been suggested to measure the preconsolidation pressure of normally consolidated clays. The simplest method is a conventional consolidation test using Casagrande's procedure to obtain the value of preconsolidation pressure. Frequently however, the e versus log p curve obtained in conventional oedometer test does not clearly distinguish preconsolidation. Therefore, other factors are examined.

For example, J E Bowles [5] states that the natural moisture content of the soil is one indicator of preconsolidation and if it is closer to the plastic limit than to the liquid limit the clay is almost certainly preconsolidated. R B Peck [11] and many others suggest assessment of the degree of preconsolidation using the Skempton's relation between undrained shear strength, plasticity index and overburden pressure. G Sallfors [12] suggests measuring pore pressure during a consolidation test in an oedometer. According to Sallfors the pore pressure is fairly constant during the early stage of the test. However, as the preconsolidation pressure is exceeded the pore pressure increases rather rapidly indicating a breakdown in the structure.

T Berre and L Bjerrum [2] proposed measuring preconsolidation pressure in the triaxial test. It is suggested that the specimens are consolidated under the insitu stresses for several days (so-called reconsolidation) and then compression test carried out. The authors indicated that the data obtained from drained triaxial compression test and a consolidation test carried out with no lateral deformation were very similar with respect to measured value of preconsolidation pressure. It was also noticed that preconsolidation pressure depended on the loading trajectory and was connected with critical shear stress - the maximum shear stress which can be mobilised under undrained conditions. The connection between preconsolidation pressure and critical shear stress noticed by Bjerrum was used by G Aas and others [1] to obtain a relationship among normalised field vane strength, plasticity index and overconsolidation ratio from oedometer test. The results obtained indicate that preconsolidation effect can be assessed from shear vane test data.

Large scale investigations of naturally and artificially aged clays were undertaken in the Chelyabinsk State Technical University (CSTU), Russia [10, 14]. A new system for triaxial testing of soils was designed and constructed for the investigations. This system comprises six working cells and a computer. The required air pressure in the cells is maintained and if necessary changed (according to the chosen test programme) automatically by an electronic pressure stabiliser. A sketch of the working cell is shown in Figure 1. The use of air allow the electronic strain transducers to be installed on the surface of a soil sample inside the working cell. Vertical and horizontal strain measurements are carried out in the central undisturbed part of the soil sample. The proposed strain measurement system makes it possible to avoid errors due to the higher compressibility of the thin disturbed soil layers adjoining the loading cap and the bottom platen. Stress in the piston 8 is measured by transducers 17 which are located inside the cell so as to avoid the influence of bush-friction on the measurement accuracy. The system allows deformations and loads to be measured with greater accuracy than in conventional triaxial tests. The electronic strain transducers were also used to measure deformation of soil samples in the modified oedometer (Figure 4a).

It was shown later [10] that preconsolidation pressure for aged clays can be assessed on the basis of the plate loading test data. If this data is available, then triaxial tests do not need to be performed.

## TEST DATA

A great number of naturally and artificially aged normally consolidated clays were tested in the CSTU triaxial testing machine. It was noticed during the investigation that all remoulded specimens which were saturated with water and aged in odemeters under the vertical load of 40 kPa to 200 kPa for one to three years and all specimens of natural clays (without any exception) demonstrated preconsolidation effect. Preconsolidation pressures were found to be strongly affected by the duration of ageing (for artificially sedimented clays) and storage (for all clays and silts) when the soil samples were kept in the laboratory under an unstressed condition prior to testing. In addition to storage duration, for natural clays and silts preconsolidated pressure depended on the nature of soil and the geological history of the deposits.

It was shown [14] that a typical triaxial compression curve for an aged clay follow three stages (Figure 2a): a linear stage AB in which deformation is a result of the elastic deformation of particles without fracturing and breakdown of particle contact points; a stage BC in which deformation becomes plastic as a result of fracturing and breakdown of particle contact points, accompanied by sliding and locking of the degraded particles into a denser array; a stage CD of ultimate resistance and failure which can be described by the Mohr-Coulomb failure theory.

It was discovered that Poisson's ratio $\nu$ ( $\nu = \varepsilon_3/\varepsilon_1$ , where $\varepsilon_3$ - lateral deformation, $\varepsilon_1$ - axial deformation) increases as soon as stage BC starts (Figure 1b). The p and q (where $p = (\sigma_1+\sigma_3)/2$ $q = (\sigma_1 - \sigma_3)/2$ , $\sigma_1$ – axial stress, $\sigma_3$ – lateral stress) corresponding to points A, B and C were plotted for several samples tested along different loading trajectories as co-ordinates defining a point in a stress space (Figure 3). The test data showed that the preconsolidated pressure varies depending on the type of a loading trajectory and very often is different from preconsolidation pressure Pc on the compression trajectory. All points representing elastic soil behaviour (stage AB) were found to be enclosed in the initial flow surface 2 which in fact consists of preconsolidation pressures measured on various loading trajectories.

The initial flow surface for aged clays was obtained in the form of a drop-shaped closed envelope. For slightly preconsolidated aged clays this surface does not contain the point O of no stress in all directions. Therefore, samples of the slightly preconsolidated clays experience plastic deformation as soon as they have been taken from an exploratory drill hole. These clays do not demonstrate a preconsolidation effect in laboratory testing unless they are tested immediately after sampling and consolidated at the same pressures as they carried in the field for 2-3 days prior to loading.

For heavily preconsolidated aged clays, point O is located inside the initial flow surface but the disturbance of the thin layers of soil adjoining the loading cap and the bottom platen makes it difficult to observe the

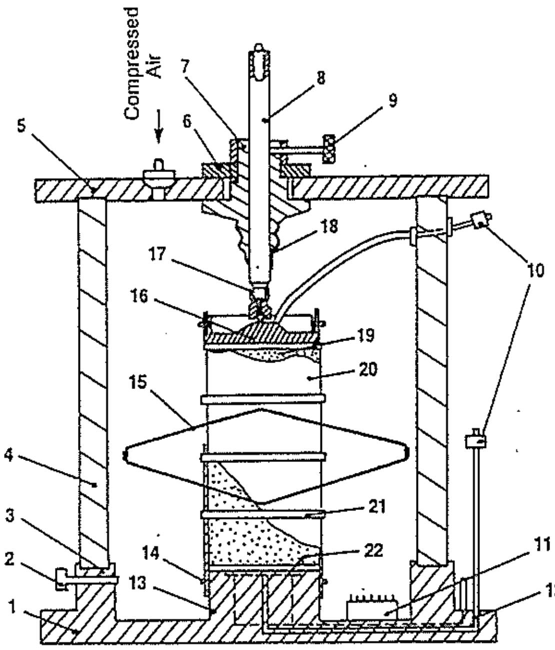



Fig.1 CSTU triaxial testing machine
1 - bottom platen; 2 - tensomanometer;
3 - gasket; 4 - cell; 5 - cover; 6 - nut;
7 - bush; 8 - piston; 9 - bolt; 10 - tap;
11 - connector; 12 - drainage; 13 - base;
14 - rubber ring; 15, 21 - strain transducers;
16 - loading cap; 18 - rubber insulator;
19 - sample; 20 - latex membrane;
22 - pore pressure transducer

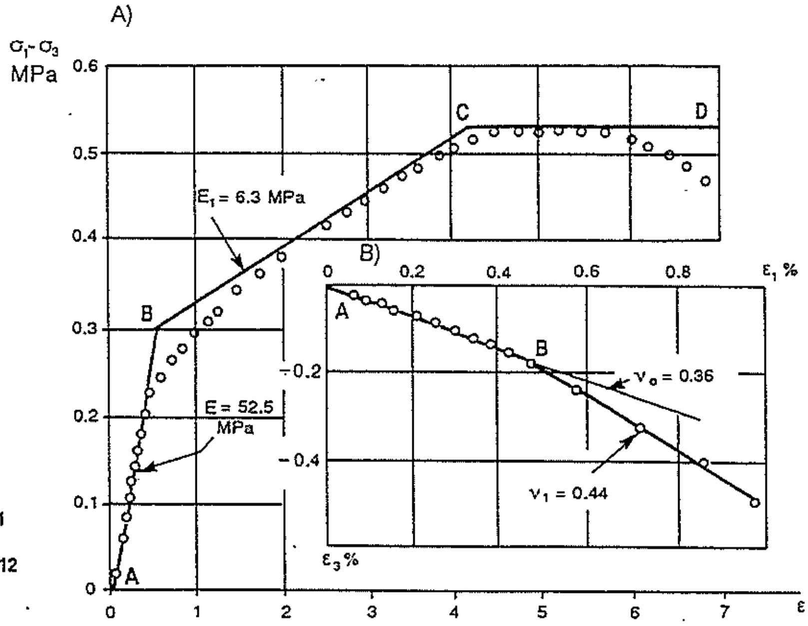



Fig.2 Results of a drained test in CSTU triaxial testing machine

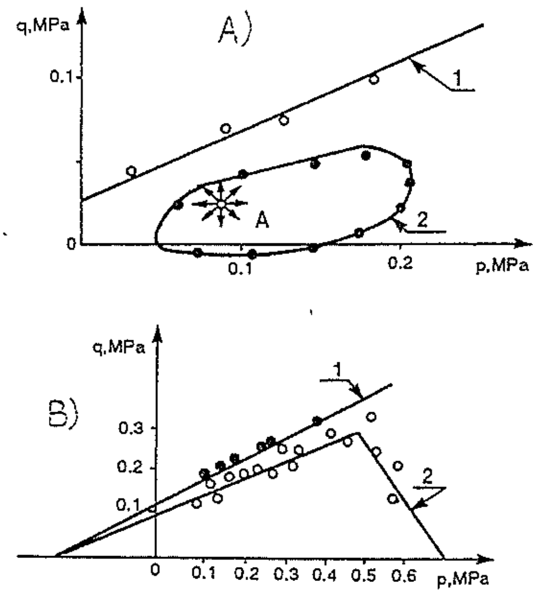



Fig.3 Failure envelopes (1) and initial flow surfaces (2) observed in drained triaxial tests on slightly (a) and heavily (b) preconsolidated aged clays

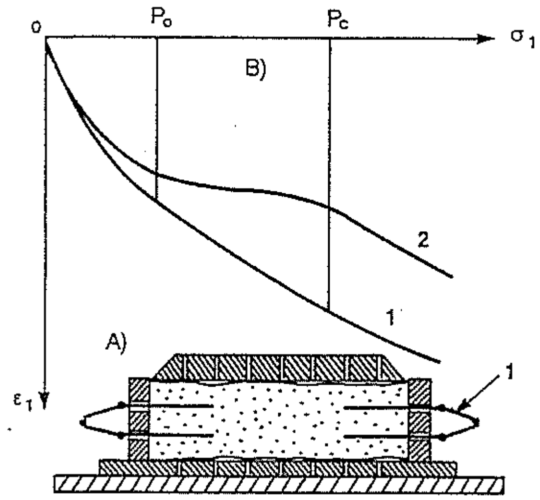



Fig.4 Modified oedometer (a) with electronic strain transducers (1) and compression curves (b) observed in conventional oedometer (1) and modified oedometer (2) tests

preconsolidation effect. The author compared conventional oedometer test data with that obtained in the modified oedometer shown on Figure 4a for 12 different aged clays. The preconsolidation effect was observed in all tests when vertical deformation was measured in the central undisturbed part of the samples. Two $\sigma_1 - \varepsilon_1$ curves shown on Figure 4b demonstrate typical conventional (curve 1) and modified (curve 2) oedometer test data.

## SETTLEMENT PREDICTIONS

Foundation settlements can be predicted with a high degree of reliability if the subsoil consists of a normally consolidated or slightly preconsolidated saturated young clay. However, if the clay is preconsolidated due to ageing, the calculated settlements are likely to be more than real ones. R B Peck [11] states that even if the settlement prediction is based on the e - log p curves for undisturbed samples, the calculated settlements are likely to exceed the real ones and the amount of the discrepancy cannot be predicted reliably.

Lipends [10] compared field settlements of several buildings in the Boston area with the settlements calculated on the basis of the oedometer test data and found that the predicted settlements were four to five times the field settlement. Crawford and Burn [6] compared data in Canada and found that predicted settlements were 4 to 13 times the field settlement values. Russian building code specifies a number of correlations between the foundation settlement calculated on the basis of the oedometer test data and the real settlement. According to this code the calculated settlement can be 2 to 8 times the field settlement.

The author believes that the discrepancies between the field and the predicted settlements can be explained in terms of the preconsolidation effect. If the conventional tests fail to detect preconsolidation pressure and the clay is assessed to be a normally preconsolidated material, then the predicted settlements are much higher than the real ones. Normally, consolidated clays often occur in river valleys and on the margins of rivers and harbours. These areas are also often the sites of heavy tank structures (e.g. oil storage installations, sewage treatment plant tanks), flood banks and heavy breakwater bunds. In many cases preloading of such sites are undertaken to improve ground strength and reduce settlements, on the basis of assumptions of no preconsolidation. Where preconsolidation due to aging is taken into account and established by appropriate testing, considerable savings can be made.

## CONCLUSIONS

1 Most of the normally consolidated natural clays and silts are preconsolidated due to ageing.

2 The nature of the aging is complex. The main factors affecting the aging effect are: secondary compression, desiccation stress, chemical bonding and mechanical aging. Mechanical aging is connected with fluctuation of horizontal stresses because of earthquakes and ground movement.

3. The preconsolidation pressure varies depending on the type of loading trajectory. In the stress space around the point corresponding to the insitu stresses there is to an initial flow surface containing a zone of elastic soils behaviour.

4 The parameters of preconsolidation can be assessed from modified triaxial and oedometer tests with high accuracy. The preconsolidation pressure can be approximately calculated from sheer vane test and plate loading test.

5 The preconsolidation effect influences the settlements of structures and must be taken into account.

## ACKNOWLEDGEMENTS

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## REFERENCES

1. Aas, G., Lacasse, S., Lunne, T. and Hoeg, K. 1986. *Use of in situ tests for foundation design on clay*, Publications of Norwegian Geotechnical Institute, Nº 6: 1-15.

2. Berre, T., Bjerrum, L. 1973. *Shear Strength of Normally Consolidated Clays.* Proc. 8th ICSMFE, Moscow, V.I.I.: 39-49.

3. Bjerrum, L. 1967. *Engineering geology of Norwegian normally consolidated marine clays as related to settlements of buildings.* 7th Rankine lecture, Geotechnique, Vol 17.

4. Bjerrum, L. 1973. *Problems of soil mechanics and construction on soft clays and structurally unstable soils,* General Report, 8th ICSMFE, Vol 3: 111-159.

5. Bowles, J.E. 1968. *Foundation Analysis and Design. McGraw-Hill, New York.*

6. Crawford, C.B., and K.N. Burn, 1962. *Settlement studies on the Mt. Sinai Hospital, Toronto,* Eng. J., Ottawa, 45(12).

7. Hanzawa, H. and Adachi, K. 1983. *Overconsolidation of all alluvial clays;* Soils and Foundations, Vol 23, Nº 4, pp 106-118.

8. Leonards, G.A., and Altschaffl, A.G. 1964. *Compressibility of clay,* J Soil Mech Found Div, ASCE, 90(5): 135-155.

9. Leonards, G.A., and Ramiah, B.K. 1960. *Time effects in the consolidation of clays,* STP 254, ASTM, Philadelphia: 116-130.

10. Murashev, A.K., Zelehin, A.V. 1993. *Determination of new non-linear model parameters taking into account structural strength of soils.* Proc of IV All-Russian Conference with foreign participation *"Non-Linear Soil Mechanics",* St Petersburg, Russia (in English).

11. Peck, R.B., Hanson, W.E., Thornburn, T.H. 1974. Foundation Engineering. John Wiley & Sons, New York.

12. Sallfors, G., 1975. *Preconsolidation pressure and secondary compression.* Discussion on normally consolidated and lightly over-consolidated materials, Proc of the conference *"Settlement of structures"* of the British Geotechnical Society: 528-530.

13. Schmertmann, J.H. 1991. *The mechanical aging of soils,* The 25th Karl Terzaghi Lecture, Journal of Geotechnical engineering, Vol 17, Nº 9 1288-1325.

14. Shirokov, V.N., Murashev A.K. 1988. *Computing bed settlements with allowance for the structural strength of soils,* Soil Mechanics and Foundation Engineering, Nº 5, September-October, Plenun Publishing Corporation, Great Britain.

15. Simons, J.H. 1975. *Normally consolidated and lightly over-consolidates cohesive materials.* Review Paper: Session II, Proc of the conference *Settlement of Structures* British Geotechnical Society, pp 500-525.

# SLOPE RISK ASSESSMENT OF NEAR VERTICAL CUTS IN HAWKESBURY SANDSTONE F3 FREEWAY, WAHROONGA TO HAWKESBURY RIVER, NSW

D.E. MURRAY

Douglas Partners Pty Ltd, Sydney

## SUMMARY

A slope risk assessment analysis was undertaken on a total of 83 high angle road cuts in Hawkesbury Sandstone along a 20 km length of the F3 Freeway, north of Sydney.

Details of individual cut features and methods used to prioritise cuts for remedial works are given, together with a discussion on risk assessment as part of the design process for future freeway works.

## INTRODUCTION

The F3 Freeway forms the major transport link between Sydney and the Central Coast and Newcastle regions located some 100 km to 150 km northwards, with traffic movements of the order of 15,000 vehicles per day (1989).

Within the study area, the route of the freeway lies within a geological setting of rugged terrain dominated by the near flat lying Hawkesbury Sandstone of Middle Triassic Age. Local relief is in the order of 100 m - 200 m.

Construction of the Freeway was largely undertaken in the late 1960s to replace the older, two lane Pacific Highway, with a further southern extension constructed during 1986-87. Excavation methods appear to have been largely by drilling and blasting.

The resulting roadway from both periods of construction is characterised by alternating cut and fill sections with the cuttings typically steeply excavated in medium to high strength sandstone with some mudstone interbeds and occasional volcanic intrusives.

Some cuttings have intermediate benches, although most are continuous with slope angles up to 80° and heights ranging up to 70 m.

During 1994, Douglas Partners were engaged by the Roads and Traffic Authority of NSW (RTA) to undertake a Slope Risk Priority investigation on cuttings along the southernmost 20 km of the F3 Freeway, between Wahroonga and the Hawkesbury River.

This paper presents a brief chronicle of methods used to prioritise cuts for remedial works. Discussion on features of the cuts posing a potential hazard to Freeway users and risk assessment during future freeway design are also presented.

## TYPICAL FEATURES WITHIN THE CUTTINGS REQUIRING REMEDIAL WORK

The Hawkesbury Sandstone formation dominates the landscape within a 100 km radius of Sydney. It is a quartz sandstone with a maximum thickness of 250 m. The formation contains numerous thin mudstone interbeds but sandstone exceeds mudstone interbeds by about 20:1. (Herbert, 1980). Sandstone facies of the formation are both of the sheet or massive types. The sandstone has high in situ horizontal stresses which are relieved on excavation.

The Freeway cuttings have resulted in large exposures of all facies of the formation. The exposed strata is generally of medium to high strength and is moderately to slightly weathered. However, numerous bedding horizons of lower strength, highly weathered strata also occur along the Freeway route. During excavation, horizontal stress relief movement occurred along these bedding planes.

The formation in the area of the Freeway is also often characterised by steeply dipping, semi-orthogonal joint sets which can, upon intersection with the cutting faces, create slabs, blocks and wedges of rock, some of which are unfavourably orientated.

Some of the principal features of the weathered rock faces that are considered hazardous include:

- individual loose boulders or dilated joint blocks,
- zones of loosened joint blocks caused by weathering-out of horizontal bedding planes and/or joint zones,
- weathering and slaking of low strength clayey sandstone beds, forming undercuts to the slope or rock above,
- dilation of joints running at an acute angle to the rock face; the 'feather edges' of such blocks are potentially subject to progressive tensile or shear failure.
- the presence of surface rubble or boulders on the natural slope above the crest of the cutting which could be undermined by strong overland rain-water flow.
- vegetation, particularly trees which can force joints open, due to 'root jacking'.
- stress relief between blast holes and blast damage, including areas where rock material has not been removed outside of the pre-split line, has also resulted in loose blocks or slabs,
- slot forming, weathered out of dolerite dykes or sheared zones.

It is considered possible that vibration from the traffic may also have contributed to slight general loosening of the cutting faces by encouraging further stress relief movement along bedding planes.

## SLOPE RISK RATING SYSTEM & INITIAL CLASSIFICATION OF CUTTINGS

Douglas Partners undertook an initial assessment of 83 cut faces along the 20 km section of the freeway in August, 1994, using the RTA Scientific Services "Slope Risk Rating System Guide" (version 1). This guide presents a recommended procedure for assigning a risk rating to slopes above roads based on quantitative levels for both the probability of occurrence and the severity of the consequences of slope failure.

The term, 'risk', is deemed to combine both potential for as well as consequences of slope failure.

The work was undertaken to supplement an earlier assessment of the cutting stability by the RTA Materials Services Branch in May. 1993.

Field observations of each cutting were recorded on Slope Inspection Reports (SIR), then used to establish an order of priority which reflected the need for further detailed geotechnical investigation; remedial or preventative action based on each observation being given a weighted score and recorded on a Slope Instability Score Sheet (SISS).

The instability score assigned to each slope allowed the slope to be placed in one of five classes which qualitatively represent the instability conditions. The five classes and corresponding ranges of instability score are given in Table 1.

Table 1. Classification of Slope Instability

| Instability Class | Instability Score Range |
|---|---|
| Very high (VH) | > 100 |
| High (H) | 75 - 100 |
| Moderate (M) | 50 - 75 |
| Low (L) | 25 - 50 |
| Very low (VL) | <25 |
| Source: RTA (1994) | |

A Consequence Assessment was also undertaken for each slope which qualitatively indicated the possible effects on road, traffic, human lives, services and property that could be caused by failure of that slope.

Road Class and traffic volume was a most important consideration in the Consequence Assessment. It is most likely that potential for disruption to traffic and services and loss of life would be highest on roads with the highest traffic flow.

A further consequence consideration involved assessment of potential disruption of services or loss of life resulting from the slope failure affecting occupied buildings above or below the slope.

A brief description of Road Class and a classification of Consequences Class are given in Tables 2 and 3 respectively.

Table 2 - Description of Road Classes

| Road Class | Road Type | Daily Traffic Volume |
|---|---|---|
| 1 | Freeways, Tollways, National or State Highways | >10,000 |
| 2 | National or State Highways<br>State Roads | 5000 - 10000 |
| 3 | Regional Roads (Main or Trunk) | 2000 - 4999 |
| 4 | Rural Roads , Local Roads | <2000 |
| Adapted from RTA (1994) | | |

Table 3. - Classification of Consequence.

| Consequence Class | Description of Consequence |
|---|---|
| Very High (VH) | High risk to life and/or total closure of a Class 1 road, resulting in an extended period of disruption to traffic and/or services and high cost of remedial works |
| High (H) | High risk to life and/or partial or total closure of one carriageway of a Class 1 road or total closure of a Class 2 road resulting in an extended period of disruption to traffic and/or services and high cost of remedial works |
| Moderate (M) | Moderate risk to life and/or partial or total closure of one carriageway of a Class 2 road, resulting in a temporary period of disruption to traffic and/or services and moderate cost of remedial works |
| Low (L) | Low risk to life and/or partial or total closure of a Class 3 road, resulting in a temporary period of disruption to traffic and/or services and low cost of remedial works |
| Very Low (VL) | Very low risk to life and/or partial or total closure of a Class 4 road, resulting in a temporary period of disruption to traffic and/or services and very low cost of remedial works |

Initially, a High consequence rating was proposed for all cuttings as a result of the Freeway being a Class 1 road. However, some of the cuttings are only very low (<3 m high) and in their present condition were considered to present only a moderate risk to life, regardless of traffic volume. A downgraded Consequence Class, where applicable, was given to those cuttings.

When Instability and Consequence classes are combined, a Slope Risk Rating can be given based on Table 4.

Table 4. - Slope Risk Rating

| INSTABILITY | CONSEQUENCE Very High (VH) | High (H) | Moderate (M) | Low (L) | Very Low (VL) |
|---|---|---|---|---|---|
| VH | 1 | 1 | 2 | 3 | 3 |
| H | 1 | 1 | 2 | 3 | 3 |
| M | 2 | 2 | 3 | 4 | 4 |
| L | 3 | 3 | 4 | 5 | 5 |
| VL | 3 | 3 | 4 | 5 | 5 |
| Source: | RTA, 1994 | | | | |

Numbers 1 to 5 in Table 4 indicate risk categories in order of priority for further investigations or implementation of remedial, preventative or maintenance works as follows:

Risk Rating 1 - Immediate action

Risk Rating 2 - Urgent action

Risk Rating 3 - Regular maintenance, action as soon as possible

Risk Rating 4 - Regular maintenance, frequent inspections

Risk Rating 5 - Periodical inspections and maintenance

On the basis of Consequence Class and Instability Class, a Slope Risk Rating was calculated for each of the 83 cuttings.

The results indicated that 19 of the cut slopes inspected were considered to warrant a Slope Risk Rating of 1. The bulk of these slopes were located within cuttings along the older northern section of the Freeway although five cut slopes from the newer, southern section were also included due to specific features.

In general, the findings were consistent with those presented in the RTA Material Services Branch Report, with similar features noted.

Several cuttings were allocated a high Slope Risk Rating due to the removal of undergrowth in bushfires that swept through much of the area in January, 1994. These fires had exposed large colluvial boulders on natural slopes, above the cut faces, which dipped towards the freeway. These problem areas may not have been easily identified during normal vegetation conditions unless a detailed walk-over inspection had been undertaken.

## COMMENTS ON THE SLOPE RISK RATING SYSTEM

Some general comments should be made regarding the workability of the draft RTA system as used for the investigation.

Whilst the Slope Inspection Reports allow for considerable detail of field observations to be recorded, the Slope Instability Score Sheets were often insensitive to specific details of defects.

For example, on each SISS a specific score was allocated to the slope, dependent on total height. However, no allowance was given for the height upon the slope where various defects occur.

Similarly, whilst much information was recorded on the SIR regarding size and location of loose debris, boulders or blocks, there was little variation in allowable scores on the SISS.

Furthermore, it is considered that scores may be allocated on the SISSs for components that may not be applicable for the slope type. This was evident during the investigation when scores were allocated for degree of seepage, which was largely weather dependent and not considered to be a major stability factor for these slopes.

## DETAILED SLOPE STABILISATION ASSESSMENTS

A second phase of the slope risk priority investigation involved recommendation of specific stabilisation measures on five selected cuttings that had been identified as having a Slope Risk Rating of 1 during the earlier phase.

Initially each cutting was photographed from the top of the opposite cutting, using a 35 mm SLR camera. This was carried out on an overcast day, to reduce shadow effects. Following development and printing of the films, a photomosaic was prepared for the full cutting comprising multiple plates with transparent plastic overlay.

The photomosaic was then taken into the field by a two man team of engineering geologists.

The cutting was taped longitudinally to identify horizontal chainages. It was also taped vertically at a number of points to allow heights of individual features to be estimated.

Following the taping, the cutting was carefully inspected from carriageway level, from the top of the cutting and from the opposite side of the cutting, noting various features which were considered hazardous and noting the necessary treatment. These features were directly highlighted on the photomosaic overlay and checked from various angles.

In the office, the recommended stabilising measures were transcribed onto the photomosaics and a table prepared describing the measures and giving a priority rating to each feature. These annotated photomosaics and the table were then independently reviewed by the Principal Engineering Geologist.

The recommended stabilisation treatment for the potentially unstable features noted included:

- scaling and cleaning of sections of the rock face and upper batter slopes.
- removal of vegetation from the rock face and from a 5 m wide zone at the top of the cutting,
- removal of individual dilated rock or joint blocks on the face.
- subhorizontal rock bolting across open feather-edge blocks.
- pinning by vertical dowels of undercut sandstone blocks at the top of the cutting.
- pattern bolting, meshing and shotcreting areas of rock being loosened by weathering-out of joints and bedding planes.
- scaling and then shotcreting of fretting low strength clayey sandstone and shaly beds,
- construction of a rock fence near the crest of the cutting to arrest any larger debris which may fall from the natural slope above.
- pattern bolting. meshing and shotcreting of dolerite dykes or sheared zones.

Based on experience, the priority of treatment was subdivided into six classes, namely 1, 1-2, 2, 2-3, 3 and 4.

Features with a priority of 1 or 1-2 were recommended to be treated in the near future. Features with a priority of 2 or 2-3 required treatment within two to three years, whilst features with a priority of 3 or 4 required treatment within five to ten years.

A brief report was prepared for each of the nominated five cuttings. Each report incorporated full colour photocopies of the photomosaics with the accompanying table listing all features to be treated on that cutting.

## GEOTECHNICAL SUPERVISION OF REMEDIATION WORKS

Geotechnical input has been provided by Douglas Partners during both the tendering process and construction phase of remediation works undertaken on the cuttings.

In some instances, nominated priorities for various features to be stabilised have been changed during the course of the work following closer inspection from a cherrypicker and from on site discussions between representatives from Douglas Partners, the RTA and the contractor. The RTA supervisor on site has usually been able to authorise variations to the work undertaken with minimum delay.

For example, where closer inspection indicated that rock slabs or blocks nominated for scaling were too large or awkwardly placed to be safely removed. bolting or dowelling the block or slab in place was given as an alternative.

## DISCUSSION AND CONCLUSIONS

A slope risk assessment of high angle cuttings along the F3 Freeway, undertaken by Douglas Partners was carried out as part of the RTA management and maintenance programme for the safe operation of the freeway.

This ongoing maintenance is expected to continue for the foreseeable future and will require the long term allocation of funding, given the magnitude of the area to be assessed and the potential recurrence of certain defects even after remedial work has been undertaken (for example, regrowth of the trees).

Given this high cost of controlling or managing the risk to road users associated with rock slope failures on steep cuttings, some consideration needs to be given to the proportion of capital funding allocated to protective measures on cuttings for future roadworks or for reducing slope angles during the construction phase.

Additional funding for benching cuttings or additional shotcreting or bolting may be more preferable than a "wait and see what happens" philosophy given the increasingly litigious nature of society.

Where potential risk of future slope instability is foreseen as a result of design of future roadworks, the degree of risk should be ascertained considering instability class and consequence of failure of the feature. Features identified as having an unacceptable level of risk should have remediation work implemented during the construction phase.

## ACKNOWLEDGMENTS

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## REFERENCES

DOUGLAS PARTNERS PTY LTD. - Report on Preliminary Slope Risk Priority Investigation for Cuttings on F3 Freeway, Wahroonga to Hawkesbury River. - Report to RTA. August, 1994.

DOUGLAS PARTNERS PTY LTD. - Report on Detailed Slope Stabilisation Methods for Cutting N 25 (Left) on F3 Freeway, Wahroonga to Hawkesbury River. - Report to RTA. January, 1995.

HERBERT, C. AND HELBY, R., 1980 - A Guide to the Sydney Basin. Geological Survey of NSW Bulletin, 26.

ROADS AND TRAFFIC AUTHORITY, NSW, - Guide to a Slope Risk Rating System (Version 1), Geotechnical Engineering Unit, Scientific Services Branch, Technical Services Directorate. 1994.

# RESIDENTIAL DEVELOPMENT ON COASTAL CLIFFLINES

C.K. OLDHAM

HARRISON GRIERSON CONSULTANTS LTD
AUCKLAND, NEW ZEALAND

## SUMMARY

The threshold criteria for determining when a detailed stability analysis is required is dependent on a number of factors but primarily the geology of the site. Where previous studies have been undertaken guidelines may be present to determine when detailed studies are required. A case study of a detailed stability analysis of a redevelopment located on a cliiffline is given.

## INTRODUCTION

The principle geotechnical hazard affecting residential development on coastal clifflines is stability of the cliffline. Properties located on coastlines are a finite resource within any locality and usually command premium real estate values. With increased demand new development and redevelopment of existing properties on clifflines becomes more profitable.

Increased demand for coastal properties has resulted in pressure to develop marginal sites and intensively redevelop existing residential properties, by demolition of existing structures and replacement with a number of townhouses. Such redevelopment can result in a demand to locate structures as close to the cliff edge as possible to maximise usage of a site. Over the last 20 years well publicised cliffline failures affecting existing residential development has resulted in an increased awareness of the risks associated with cliffline development. These factors have lead to increased demand for stability assessments of sites located on clifflines to satisfy the requirements of the Local Authority and the New Zealand Building Code.

In the past Local Authorities have accepted stability assessments from registered engineers. Recently some Local Authorities have introduced a policy of having a vetted list of professionals from whom stability assessments will be accepted. The issue of who should be undertaking such assessments is clearly, suitably qualified and experienced professionals. For the purposes of this paper it is assumed that the person undertaking a stability assessment has the requisite qualifications and experience.

## STABILITY ASSESSMENT THRESHOLD CRITERIA

The initial stability assessment is to determine if stability is an issue at the site and therefore the need for a detailed stability assessment. The criteria used to determine if stability is an issue are dependent on the site geology, land form, slope geometry, any existing slope failures and history of slope failures in the area. The assessor can from a desk study of geological maps and published data, existing site data, coupled with an examination of aerial photos and a walkover of the site determine whether further investigation of stability issues is required. In areas where previous studies have been undertaken threshold guidelines may be present for when a detailed stability assessment is required. Threshold guidelines can be in the form of projected regressed slopes or setbacks from cliff edges.

Safe building areas on sites located on clifflines comprised of soil or soil strength materials can be defined in terms of a regressed slope profile projected from the toe of the cliff with an allowance for toe erosion during the design life of the structure. A regressed profile that yields a factor of safety of 1.5 under fully saturated conditions will give a worst case regression profile for the assumed soil parameters.

## Tauranga Area

In the Tauranga area of the Bay of Plenty, a study of slope failures in the developed areas concluded that coastal slopes with a slope of greater than 1V:2H were potentially at risk to slope failure. The projected regression line of 1V:2H is therefore used as the guideline as to when it is considered necessary to undertake a detailed stability assessment of the site. Failure modes can be either deep seated semi circular or planar failure surfaces.

# CASE STUDY

## INTRODUCTION

In July 1995 a geotechnical investigation was undertaken for a proposed redevelopment of an existing residential site located on a coastal cliffline. The proposal was to remove the existing building and construct four new residential buildings on the site. The geotechnical investigation was undertaken to determine the subsurface conditions at the site to detail any identifiable geotechnical hazards affecting the site. It was determined that the coastal cliffline was influencing the site, therefore a detailed stability assessment was undertaken.

Based on a visual inspection of the site and the data provided by the developer it was apparent that a significant part of the area to be redeveloped was located seaward of the nominal 2H:1V profile projected from the toe of the slope. It was therefore determined that the stability of the coastal slope required assessment to determine its influence on the proposed development at the site. The coastal slope geometry and a stratigraphic profile, through the cliff is shown on Figure 1.

## SITE DESCRIPTION

The site was located in a developed residential area located on the cliffline on the western side of the Maungatapu Peninsula within Tauranga Harbour. The site was generally flat to gently sloping up to the crest of the steep coastal cliff which was about 27m high. The cliff has a slope angle of about 54° with a 3m to 5m subvertical face at the toe of the slope, which is coincident with the harbour foreshore. Immediately north of the site is a large gully. The gully side slope falls from the sites northern boundary via moderate to steep slopes ranging from 26° to 50°.

## GEOLOGY

The regional geological map[1], indicated that the area was underlain by fluviatile sands and silts of Pleistocene age. These alluvial sediments are referred to as "Tauranga Beds" in this paper. Late Quaternary volcanic ash deposits mantle the region and are indicated to overlie the Maungatapu Peninsula [1]. The inferred stratigraphy at the site is of Tauranga Beds, older rhyolitic ashes, younger ashes, and recent ashes [2]. The results of the investigations confirmed that volcanic ashes overlie alluvial sands and silts at the site.

Generally within the Tauranga area landslip failures have occurred in the older ashes or within clay rich layers within the Tauranga Beds [2]. The map accompanying Houghton and Hegans report[2] indicates that the gully feature immediately north of the site is a possible deep seated landslide based on aerial photo interpretation. Deep seated failures are confined to areas adjacent to the sea cliff where the weathered, clay rich older volcanic ashes are very thick or the Tauranga Beds are unusually clay rich [2].

## FIELD INVESTIGATIONS

Two machine boreholes MB1 and MB2 were drilled during the investigation to determine the subsoil conditions at the site. Due to restricted access to the seaward side of the existing house a trailer mounted Gemco HPC7 drill rig was lifted over the house using a mobile crane. Boreholes MB1 and MB2 were drilled to a depth of approximately 24m and 14m respectively. Plasticity Index test and particle size grading curves were determined on samples obtained from Borehole MB1.

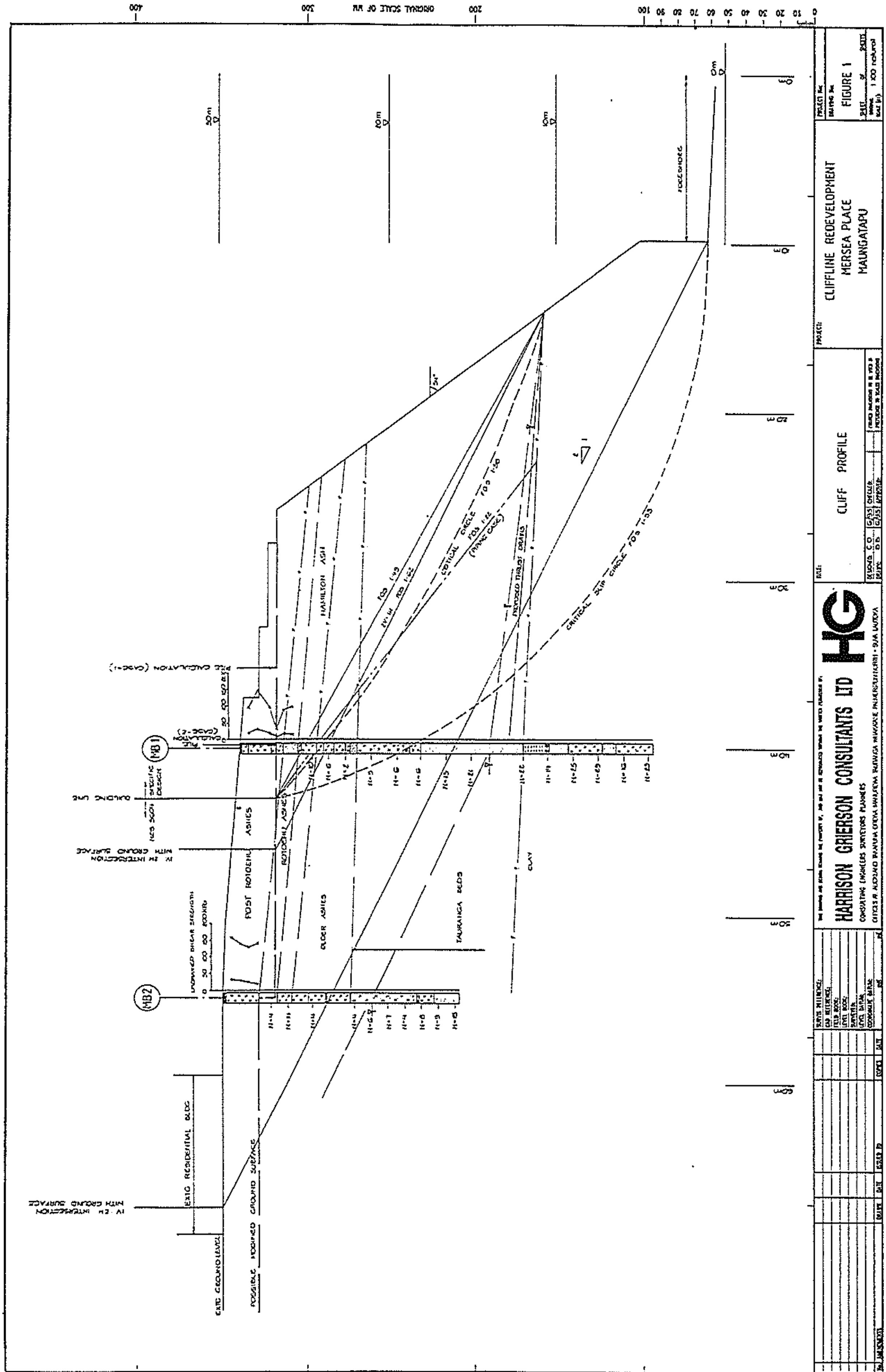


Figure 1: Cliff profile

## SUBSURFACE CONDITIONS

The soils encountered in the boreholes indicate that the site is underlain by a stratification of silts with some sand layers, overlying interbedded silts and sands. Good correlation of strata between boreholes was obtained although some variation in colour was noted which may be due to weathering effects.

### Volcanic Ashes

The surface soils at the site as encountered in the boreholes consisted of pumiceous, silts and sands about 7m thick inferred to be volcanic ashes. The silts ranged in shear strength from 57 kPa to 157 kPa and were sensitive to disturbance. SPT N values ranged from 2 to 14 indicating very loose to medium dense material. The inferred sequence of volcanic ashes was between 6.8m and 7.5m thick in boreholes MB1 and MB2 respectively. The volcanic ashes are underlain by sands and silts.

### Alluvial Sediments

The materials encountered consisted of sands, interbedded silty sands and sandy silts, with some silt layers and fine gravel. The silts and sands were thinly interbedded, with banding and faint bedding observed in the sand horizons. In Borehole MB1 a clay layer 1.4m thick was encountered at a depth of 16.8m. SPT N values from approximately 7m to 13m ranged from 4 to 9 indicating loose material. SPT N values increased from a depth of about 13m to the end of the boreholes with N values ranging from 13 to 29 indicating medium dense material.

## SLOPE STABILITY

### Effective Strength Parameters

The inferred profile of the surficial ashes was gently dipping towards the coastal slope. A shallow planar failure mode within the surficial ashes at the crest of the coastal slope was considered to be unlikely due to the low groundwater levels measured at the site in combination with the height and geometry of the slope profile. Based on the slope profile and geometry it was determined that the critical mode of slope failure was likely to be a deep seated circular failure. The laboratory data indicated that some soils may be close to their liquid limit, and it was inferred that an increase in pore water pressures due to a increase in ground water levels may result in a deep seated failure occurring.

In order to assess the stability of the coastal slope effective stress parameters for each inferred soil horizon were assumed. Based on the assumed parameters and the measured groundwater levels a stability assessment for circular failure using Bishops simplified method was undertaken.

Using the existing ground profile, as measured groundwater levels and the assumed soil parameters the minimum theoretical factor of safety for the cliff slope was less than 1.0. As the slope had not failed it was therefore apparent that the initially assumed parameters were too low. From inspection of the slope profile it was apparent that the parameters assigned to the Tauranga Beds were the major factor affecting stability analysis.

A friction angle of 36° was considered realistic for the materials encountered, however a zero cohesion value was overly conservative for existing conditions. By iteration it was determined that a minimum cohesion value of 22 kPa for the Tauranga Beds was required to yield a minimum theoretical factor of safety of approximately 1.0 for the critical circle for the existing ground water profile. It was concluded that the revised effective cohesion value of 22 kPa and friction angle of 36° were realistic parameters for the Tauranga Beds, which we believed were conservative parameters as a raised ground water condition was not used in the iterative analysis.

In order to check that the revised parameters were reasonable, the adjacent gully profile was analysed assuming that the gully was the result of a deep seated failure of the cliffline. It was assumed that originally the cliffline extended northwards with a similar geometry and soil profile to that encountered at the site and that the existing slope profile, approximated a deep seated failure surface, with the slumped material assumed to have been eroded away by tidal action.

The slope was analysed for a circular slip surface using the revised parameters which yielded theoretical factor of safety values of 2.03 for as measured groundwater conditions and 0.74 for fully saturated conditions. It was therefore assumed that at the time of failure the groundwater condition was raised above that measured within the site. It was therefore concluded that the revised parameters listed in Table 1 are reasonable conservative parameters based on the available data.

Table 1: Revised Effective Strength Parameters

| Soil Horizon | Bulk Density ($kN/m^3$) | Effective Cohesion (kPa) | Internal Friction Angle (°) |
|---|---|---|---|
| Younger Ashes | 14 | 2 | 32 |
| Rotoehu Ashes | 14 | 4 | 32 |
| Hamilton Ash | 16 | 4 | 32 |
| Older Ashes | 15 | 4 | 32 |
| Tauranga Beds | 16 | 22 | 36 |

Cliff Stability

A possible development option was to cut down the ground surface by 1.5m to 3m below existing as shown on Figure 1. Based on the revised parameters, existing groundwater conditions and the proposed modified ground surface it was determined that the existing cliff slope had a theoretical factor of safety of less than the minimum value of 1.5 required by the Local Authority. Various slip circles were analysed to determine the slip circle with a theoretical factor of safety of 1.5, which is plotted on Figure 1. The slip circle was further analysed for the case where cohesion equals zero under existing groundwater conditions which yielded a theoretical factor of safety(FOS) of 1.28.

Based on the results of the investigations at the site and knowledge and previous experience with soils of the type encountered at the site, it was our opinion that the area of the site landward of the point where the FOS = 1.5 slip circle intersected the existing ground surface would have an acceptable stability state in terms of conventionally accepted criteria, for the subsurface conditions assumed to exist or likely to occur at the site.

The intersection of the FOS = 1.5 slip circle with the proposed modified ground surface determined the location of the "Recommended Building Line Limitation" (RBLL) which defined the extent of the area suitable for conventional residential foundations. The area seaward of the RBLL defined the area within which building foundations required specific design to provide a stable building area with a minimum factor of safety of 1.5. It was recommended that the location of the RBLL should be reassessed when the development design was finalised to take into account any modifications to the site.

In the foregoing analysis no allowance for earthquake effects had been made. It was our opinion that the proposed development had the same risk of being affected by an earthquake as most other existing residential developments located on the coastline and elsewhere in the area.

Toe Erosion

The coastline on the eastern boundary of the site varied from steep slopes, up to sub vertical banks. The harbour at the site location is tidal and exposed to limited wave action. It was considered that active erosion of the coastline along the eastern edge of the site was occurring due to the combined erosive action of waves and tidal currents. The erosion rate of the cliffs at the site locality was not known, but in it was considered unlikely to have any significant influence on the area defined as suitable for conventional building construction, located approximately 43m from the toe of the coastal slope.

## REVIEW

Shortly after the investigation of the above site on the western edge of Maungatapu Peninsula, a landslip occurred on the cliffline at the northern end of the peninsula about 150m north of the site.

Detailed investigation of the slip was undertaken by another consultant on behalf of the Local Authority. The failure surface of the slip at the north end of the peninsula was understood to have exited the slope above the toe at the upper level of a clay horizon. A clay horizon was encountered in Borehole MB1 at our site, which was assumed to be similar to the clay layer encountered at the north end of the peninsula. We therefore reassessed the stability of the cliff at the site taking into account the analysis of the recent slips at the north end of the peninsula.

## STABILITY APPRAISAL

Taking into account the recent slips and the results of our investigations, it was concluded that the likely slip failure surface would not be deep seated exiting through the slope toe. A more probable failure mechanism was for a slip surface to develop which exits the slope immediately above the upper surface of the clay horizon within the overlying silts and sands. The existing vegetation cover and steepness of the slopes prevented an inspection of the cliff face being made.

It was assumed that the clay layer was laterally continuous at the site and acted as an aquiclude to groundwater movement. The clay layer was inferred to be exposed in the cliff face. The existing cliff slope was back analysed for a circular mode of failure assuming a groundwater level and failure surface controlled by the clay layer. It was determined that for the Tauranga Beds layer effective stress parameters of 35° and 10 kPa cohesion were required to approximate unity for a shallow circular failure at the cliff face. The soil parameters were revised and the cliff analysed for a circular failure exiting the cliff face at the top of the clay layer.

It was determined that the critical slip circle with a back scarp located at the previous Recommended Building Line Limitation (RBLL) had a factor of safety of approximately 1.58. It was further determined that a 2H:1V line extending from the top of the clay layer at the cliff face intersected the proposed ground surface approximately 3m behind the RBLL, which had a factor of safety value of 1.62 for a planar faliure and the existing ground water conditions. A planar failure surface from the clay layer at the cliff face extending to the RBLL location yielded a theoretical factor of safety value of 1.49 for the existing groundwater conditions.

The cliff was further analysed for a non circular failure mode assuming that a failure plane develops immediately above the upper clay surface. It was assumed that groundwater seepage at the cliff face along the clay surface resulted in piping erosion of the overlying sands and silts. The piping was modelled by assigning residual parameters to the slip surface along the level of the layer. The residual parameters used were 18° internal friction angle and zero cohesion. The residual internal friction angle was considered to be a conservative value for volcanic soils based on test results reported by Wesley [3].

By iteration the minimum theoretical factor of safety for a non circular failure was 1.22 for existing groundwater conditions assuming that a piping layer developed along the clay interface for a horizontal distance of 9m into the slope. The back scarp of the failure was located to be coincident with the RBLL determined in the original analysis. This value was considered to be satisfactory provided that groundwater levels are maintained at existing levels. It was therefore recommended that thrust drainage be installed at the site immediately above the clay layer to intercept any ground water seepages and to maintain the existing groundwater conditions. The various failure planes and the proposed thrust drains are shown on Figure 1.

## CONCLUSIONS

Based on the above results it was considered that the original RBLL on the site had a satisfactory stability state provided that the groundwater level does not rise above the inferred existing levels and that piping erosion does not occur. The installation of appropriate drainage measures should maintain the ground water levels at the levels inferred to exist at the site in the long term. It was our opinion that horizontal drains thrust from the cliff face coincident with the top of the clay layer as shown on Figure 1, should provide sufficient drainage to maintain the existing ground water conditions. The critical slip failure mechanism assumes that piping develops along the clay surface. The installation of thrust drainage with suitable filter cloth should minimise the potential for piping erosion to occur.

## References:


1 Healy, J, Schofield, J C and Thompson, B N 1964. *Sheet 5 - Rotorua.* Geological map of New Zealand 1:250 000. Map(1 sheet). Wellington, New Zealand. DSIR.

2 Houghton, B F and Hegan, B D 1980. *A preliminary assessment of geological factors influencing slope stability and landslipping in and around Tauranga City.* NZ Geological Survey Engineering Geology Report EG 348.

3 Wesley, L D 1992. Some Residual Strength Measurements on New Zealand Soils. *Proceedings 6th Australia New Zealand Conference on Geomechanics*: 157 -162.

# LIME STABILISATION OF ROAD SUBGRADES BENEFITS AND RECEPTIVE SOIL TYPES


## R.M. OSBORNE

**WOOD & PARTNERS CONSULTANTS LTD**
**CONSULTING ENGINEERS**
**AND REGISTERED SURVEYORS**
**AUCKLAND, NEW ZEALAND**



## SUMMARY

Soil stabilisation is not new in New Zealand having been practised some thirty years or more. Lime stabilisation, however, is only now being considered as a primary option in the process of road construction.

Lime reacts with clay to varying degrees depending on the mineral composition of the clay. Plastic soils whether they are fine grained clay or gravel-clay in nature are responsive to lime, whereas organic soils and soils with a low Plasticity Index (P.I.) are generally not.

The advantages of lime treatment is that it leads to a reduction in the construction time and cost if carried out correctly. It thus makes it imperative to recognise when it is a viable option. Simple tests can identify lime responsive soils, but further laboratory tests are required to enable a cost efficient result to be achieved.


## INTRODUCTION

Lime stabilisation is a technique being utilised on more and more occasions as an alternative to conventional pavement construction. The New Zealand Institute of Highway Technology have described it as "a means of permanently consolidating subgrade soils and basecourse materials by substantially increasing their strength and bearing capacity whilst, at the same time decreasing their sensitivity to the ingress of water and volume change during wet/dry cycles". Today it involves spreading burnt lime over the surface, slaking it, hoeing the resultant material into the underlying surface, compacting this mixture and then trimming it to the desired shape.

With advances in equipment in carrying out these processes and the increasing cost of metal of the required quality, it is of no surprise that lime stabilisation in some instances is the most efficient option.

Consequently it is imperative to understand what type of soils are responsive to lime. However, before we are able to achieve this we must familiarise ourselves with the chemical reactions resulting from such interactions.

## GENERAL LIME REACTION

The addition of burnt lime and water to clay soils results in two main chemical reactions:

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| $CaO$ (Burnt Lime) | + | $H_2O$ (Water) | = | $Ca(OH)_2$ (Calcium Hydroxide) | + | Energy (Heat) | | | (1) |
| $Ca(OH)_2$ (Calcium Hydroxide) | + | $H_2O$ (Water) | = | $Ca^{2+}$ (Metal Ions) | + | $H^+$ (Hydrogen ions) | + | 3OH (Hydroxide Strong Base) | (2) |

Calcium hydroxide forms through the reaction with calcium oxide with water (1). The hydroxide is a strong base and dissolves in water to liberate metal, hydrogen and hydroxide ions (2).

The first main reaction is a base ionic exchange which occurs with the clay particles. The clay minerals react with ionised water molecules and bond into permanent ionic configurations. The fine plastic clay particles collect together in a mass of coarse friable particles and the ionic exchange is completed, taking only minutes if the proportion of reagents are correct and mixed thoroughly.

The second reaction is a slow chemical response in which lime reacts with silica and alumina minerals in the original clay to form calcium silicates and alluminates. This is the hardening action and is the result of the formation of these very hard dense minerals which have great stability. This process continues for a year or more if sufficient moisture is available to drive the reaction.

## SUITABILITY OF SOIL TYPE

The prime consideration as to whether a soil type will respond to lime treatment is thus the content of clay minerals contained within.

Cohesive soils react both physically and chemically with lime to achieve an improved pavement material. The exceptions are the organic soils that contain 20% or more of organic material and some volcanic based soils. Soils with a low P.I. and non plastic soils can have the lime silica reaction induced through the addition of a secondary additive (i.e. fly ash).

Granular soils with a P.I. greater than 10 have been known to react with lime, but cement stabilisation is a more efficient alternative for these types of soils.

## ON SITE ASSESSMENT

Lime reactive soils can be assessed within the field, however, this only comes through experience and gives only a qualitative account of the soils nature. Laboratory tests are required to enable us to achieve the optimum application of lime for the desired result.

Field testing is by no means definite, with many Engineers utilising their own techniques.

However, the general "Palm Squeeze Test" is used on most occasions to determine the plastic nature of the soil and thus it's ability to react with lime.

The New Zealand Institute of Highway Technology have suggested a technique requiring only plastic bags and a small amount of slaked lime to determine the potential reactivity of a soil sample.

The test involves mixing two samples with water and kneading them until they become plastic in nature and are thoroughly mixed. Both samples are then placed in plastic bags. One sample has 10% by volume of slaked lime added and this combination is then mixed by kneading the outside of the sealed plastic bag. The other bag is sealed and used as the control. After two days of storage both samples are removed from the plastic bags and placed in two litre packs of water to determine the degree of disintegration after 24 to 36 hours of submersion.

If the lime stabilised sample shows no significant disintegration compared to the control then the soil warrants further laboratory testing.

## CONCLUSION

Lime stabilisation has significant advantages over conventional pavement construction. These include;

1. It's ability to increase the workability of the subgrade.

2. It's ability to reduce the compactive effort required by reducing the moisture within the soil.

3. It's ability to reduce the shrinkage swell characteristic of clay soils.

4. The increase in unconfined compression strength of the subgrade that can be obtained.

5. The formation of a working platform resistant to water.

6. The reduction in depths of high quality metal required in pavement construction.

7. The reduction in time and cost indicative of all of the above.

Lime reacts with clay minerals, plastic clay soils with P.I. greater than 10 tend to be the most responsive to the lime stabilisation process.

Simple field test can determine whether a soil is receptive to lime; laboratory tests however are required to identify an efficient application rate.

## ACKNOWLEDGEMENTS

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## REFERENCES

1. New Zealand Institute of Highway Technology 1995. Second National Pavement Stabilisation Symposium.

2. Mahan & Myers; University Chemistry (Fourth Edition 1987) (Pg 727-731)

# WELLINGTON REGIONAL STADIUM
# SITE ASSESSMENT

**S J PALMER**

**Beca Carter Hollings & Ferner Ltd**
**Wellington, New Zealand**

## SUMMARY

The Wellington Regional Stadium is one of a number of projects planned or under construction along Wellington's reclaimed waterfront. Each of these projects has had to consider ground stability with respect of liquefaction and lateral spreading, taking into account Wellington's relatively high seismic activity and the weak and variable nature of the reclaimed land.

The proposed stadium site straddles two distinct areas of reclamation; one of hydraulic fill and the other of end tipped gravel fill. The potential for liquefaction and lateral spreading of the site has been assessed. A relatively high potential for liquefaction of the hydraulic fill sands has been concluded. On the basis of an approach proposed by Newmark [4] a potential for upto 1 m of lateral movement of the site has been predicted under the design earthquake (600 to 800 year return period). Ground improvement options have been considered to mitigate these predicted movements. The formation of gravel columns by vibro-replacement has been concluded to be an appropriate form of ground improvement.

## INTRODUCTION

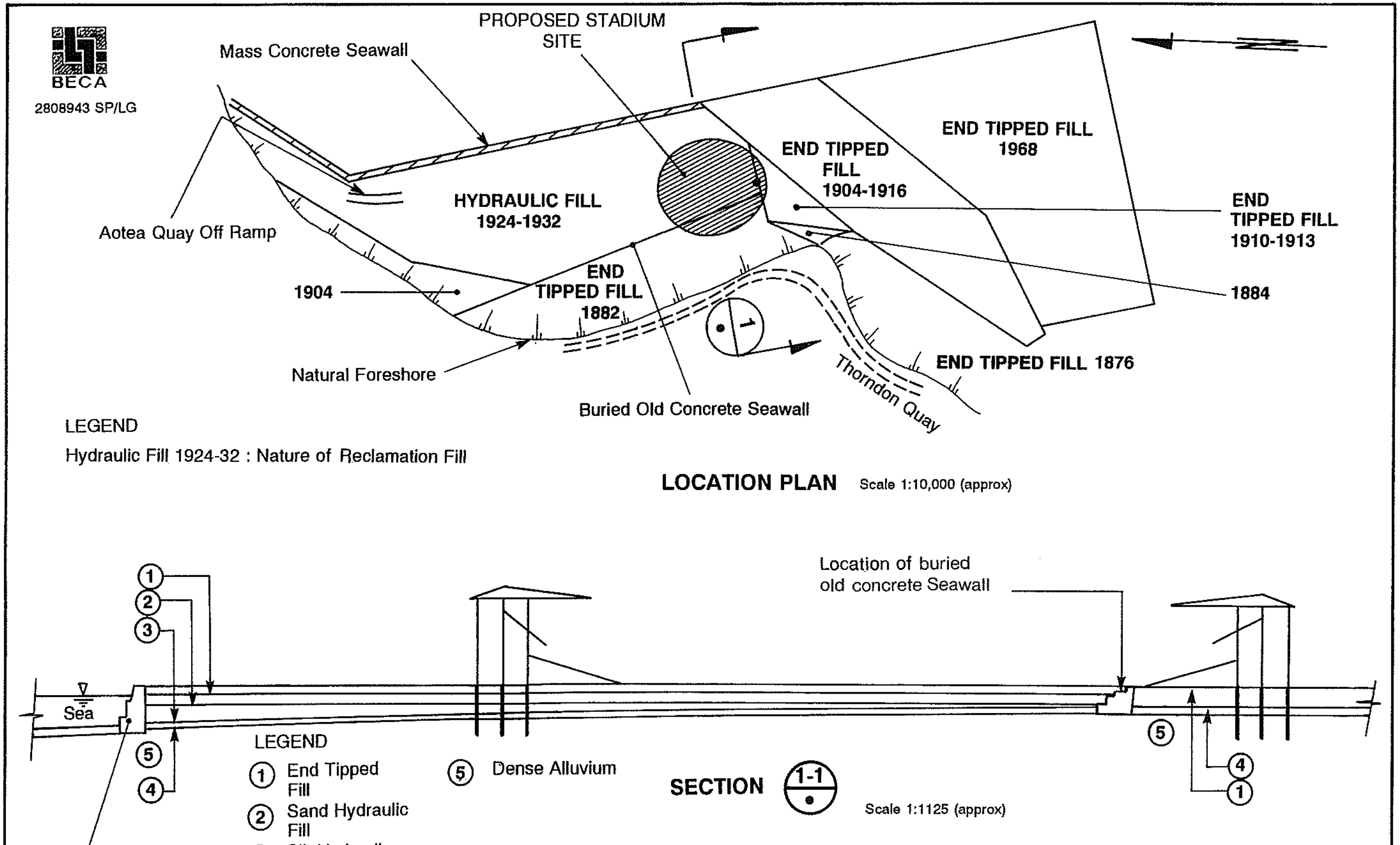
The boarder between Wellington's central business centre and the foreshore has been formerly occupied by railway and port facilities and carparks. Now with this land being surplus to railway and port requirements a unique opportunity is offered for its redevelopment, linking the city to the foreshore with public facilities and space. This waterfront land comprises reclamations varying from end tipped quarried rock to dredged and hydraulically placed marine silts. The reclamations were constructed in stages over a 100 year period. With these ground conditions and Wellington's relatively high seismic activity the redevelopment has presented geotechnical engineers with some special challenges. Each development has had to consider ground stability with respect to liquefaction and lateral spreading and develop appropriate ground improvement and/or foundation systems considering the specific site ground conditions and the importance and public risk associated with the development.

This paper discusses the site assessment and conceptual design of ground improvements for one of these Wellington waterfront developments; the Wellington Regional Stadium. The proposed stadium is to provide an outdoor sports and cultural venue with two levels of covered grandstand seating for 35,000 people.

## SITE DESCRIPTION

The proposed stadium site is located on reclaimed land lying immediately to the west of the junction of Aotea Quay and Waterloo Quay, Wellington. The site is approximately level with a ground surface level 2.0 m to 2.5 m above mean sea level. Available information relating to the construction of the reclamation has been researched [2] and is summarised on Figure 1.

As can be seen from Figure 1, the majority of the site was reclaimed by a hydraulic filling operation during the period 1924 to 1932. The western and southern sections of the site were reclaimed by end tipping of rock fill during the periods 1882 and 1910 to 1913 respectively. The old concrete sea wall forming the boundary between the 1882 and 1924-32 reclamations remains buried beneath the site. The location of this old seawall has been determined from historical legal survey data and confirmed by excavation.




LOCATION PLAN AND CROSS SECTION
Figure 1

The reclamation fill beneath the site varies between 4 m and 7 m in thickness and compact beach sands and dense alluvial deposits underlie the fill. The old seabed surface dips beneath the site at a gradient of typically 5% to the east. A mass concrete wall of approximately 10 m in height forms the edge of the reclamation approximately 80 m to the east of the stadium site.

As indicated on Figure 1 there are two distinct areas of reclamation within the site. Namely the 1924 to 1932 hydraulic fill area and the earlier end tipped fill areas. Borehole investigations revealed fill materials within the hydraulic fill area to typically comprise 1 m thickness of compact gravel over 3 m thickness of loose sand over 0.5 m to 3.5 m thickness of very soft silt. The fill materials in the end tipped fill areas typically comprise 1 m thickness of compact gravel over 3 m to 4 m thickness of loose gravel.

The sites location is 500 m south-east of the Wellington fault.

## LIQUEFACTION

The liquefaction potential of the site was assessed using empirical correlations developed by Seed [6] which relate normalised SPT blowcounts to ground accelerations causing liquefaction in the field.

The influence of particle size grading on liquefaction potential was also assessed for the various materials encountered at the site. In particular the silt content of the sands and the susceptibility of the loose gravels to liquefaction were considered. Work by Evans [3] has demonstrated by the way of triaxial tests that gravelly soils can be susceptible to liquefaction.

From these assessments it was concluded that the loose sand hydraulic fill could be expected to liquefy as a consequence of an earthquake with a 200 year return period (magnitude M=7.4, peak ground acceleration a max = 0.2g).

The end tipped gravel fill could be expected to temporarily lose strength as a result of pore water pressure build up initiated by the shaking of a major earthquake. Build up of pore water pressures in this material to the extent that it liquefies cannot be discounted but is considered unlikely. Such shaking could also be expected to cause up to 200 mm of settlement in the end tipped fill.

The cohesive nature of the silt hydraulic fill is such that liquefaction potential of this material is assessed to be low.

Due to the density of the beach sand, indicated by the SPT results, this soil is expected to have a relatively low liquefaction potential.

## LATERAL SPREADING

### Introduction

Lateral spreading is the permanent displacement of ground along a weak or liquefied soil layer toward a free edge, such as a reclamation edge or river, as a result of earthquake ground accelerations.

The phenomena of lateral spreading has been observed as a consequence of a number of recent major earthquakes including the 1987 Edgecombe, 1989 Loma Prieta and 1995 Kobe earthquakes.

The very soft and potentially liquefiable soil layers in the sites hydraulically filled areas provide weak layers along which lateral spreading could be expected to occur. It is postulated that severe earthquake shaking could cause the collapse of the seawall which forms the edge of the reclamation 80 m to the east of the stadium site. Lateral spreading type shear movements could then occur along a series of scallops extending inland, resulting in permanent lateral surface displacements reducing with distance from the collapsed seawall. The magnitudes of these potential displacements were predicted using the approach proposed by Newmark [4].

## Method of Analysis

In the approach proposed by Newmark [4] the horizontal ground acceleration that reduces the factor of safety against sliding along critical shear plains to 1.0 is first assessed. This acceleration level is the threshold "yield" acceleration for that shear plain, beyond which accelerations will result in permanent displacements. Displacements are computed by integrating over the portion of the acceleration time history of the design earthquake where the applied ground acceleration exceeds the assessed threshold yield acceleration. Newmark [4] and later Ambraseys and Menu [1] analysed the acceleration time history of a number of earthquakes in the magnitude range of 6.6 to 7.3. For each of these acceleration time histories they computed maximum displacements for various ratios of yield acceleration to peak ground acceleration. From this analysis they prepared charts relating the acceleration ratio to maximum permanent displacement. The two cases of symmetrical resistance, or a level shear plane, and unsymmetrical resistance, or a sloping shear plane, were considered by these researchers. In the case of the unsymmetrical resistance it was assumed that the yield acceleration in the upslope direction was high such that no upslope movements occur. Charts were produced for both of these cases.

These charts were applied in assessing the potential displacements by lateral spreading at the proposed stadium site.

## The Analysis and Conclusions

The undrained shear strength of the hydraulic fill silt was assessed on the basis of 75 mm diameter insitu shear vane tests to be 15 kPa. The residual undrained shear strength of the liquefied hydraulic fill sand was assessed by correlations with SPT results proposed by Seed and Hander [5] and Stark and Mesri [6]. These correlations suggest a residual undrained shear strength of 15 kPa.

Simple block stability analyses assuming these shear strengths gave yield accelerations of 0.1 g.

To be consistent with the design of the structure peak ground accelerations as specified by the Loadings Code NZS4203 [7] for both the ultimate limit state and the serviceability limit state were considered. The ultimate limit state design earthquake has a peak ground acceleration of 0.6 g (estimated to be a 600 to 800 year return period event for this site) while the serviceability limit state design earthquake has a peak ground acceleration of 0.1 g.

Calculating the yield to peak ground acceleration ratio for the ultimate limit state case and applying this to the Newmark [4] and Anbraseys and Menu [1] charts, the potential maximum permanent displacement was predicted for the hydraulic fill area of the site to be of the order of 1 m. Under the serviceability limit state earthquake liquefaction of the hydraulic fill sand and yielding along this plain would not be expected. The estimated yield acceleration for shear of the soft silt equalled the serviceability limit state case peak ground acceleration, thus any yielding of the soft silts in this case could be expected to be small (<25 mm).

In the end tipped fill reclamation areas the magnitude of lateral displacement would depend on the degree of pore water pressure build-up and weakening in the loose gravel layer. This layer would not be expected to become as weak as the silt and sand layers in the adjoining hydraulic fill area thus any lateral displacement of the end tipped fill could be expected to be less than that of the hydraulic fill. Differential lateral displacements along the boundary between hydraulic fill and end tipped fill (the buried seawall) could therefore be expected. These differentials could be up to the full displacement of the hydraulic fill (in the case of no yield of the end tipped fill).

Potential total and differential lateral ground movements of the order of 1 m were thus predicted as a consequence of the design earthquake. Ground movements, of this order could be expected to leave the stadium in an irreparable condition and (depending on structural form) the risk of a collapse type failure of the stadium, could be high. Ground improvement options were investigated to mitigate these ground movements and other consequences of liquefaction.

## GROUND IMPROVEMENTS

The ground improvement options of dynamic compaction and vibro-replacement were considered in detail while other methods including soil grouting, excavation and replacement, and driven gravel columns were quickly discounted on technical and cost grounds. Isolating the structure on piles from the potential ground movements was found to be impractical because of the high lateral loads imposed on the piles by the potential ground movements.

### Dynamic Compaction

Ground improvement by dynamic compaction has been successfully applied at three Wellington reclaimed sites in recent years. The process involves repeatedly dropping a large weight (say 15 tonnes) from height (say 15 m) on a specified grid pattern to compact the reclamation fill. The hydraulic fill area of the stadium site differed from the Wellington sites recently compacted in that a substantial proportion of the fill comprised soft silt. It is unlikely that the silt hydraulic fill will compact or consolidate to the required degree under dynamic compaction. It is expected that the energy of the dynamic compaction would be absorbed in displacing the silt and thus the overlying sand hydraulic fill is also unlikely to be improved to the required degree.

The end tipped fill can be expected to be adequately improved by dynamic compaction. Similar materials have been successfully compacted at other Wellington sites. The Department of Survey and Land Information (DOSLI) house vibration sensitive equipment in their building 180 m south-west of the site boundary. The vibrational effects of dynamic compaction were assessed at this distance on the basis of monitoring data from other sites improved by dynamic compaction. It was concluded that it was probable that dynamic compaction would adversely affect the operation of DOSLI's equipment.

Because of dynamic compactions predicted poor performance on the hydraulic fill and its probable adverse affect on DOSLI's equipment this method of ground improvement was not considered further.

### Vibro-Replacement

The process of vibro-replacement involves probing the full depth of the ground to be improved with a vibrating probe on a specified grid basis. On extraction of the probe gravel is introduced and compacted to expand into the void left by the probe. The final product is a series of dense gravel columns of 0.8 m to 1.2 m in diameter on a grid at spacings of 2 m to 3.5 m.

Vibro-replacement was last used in New Zealand in Queenstown in 1986 but the equipment is readily available from Australia where it is commonly used to improve Australia's west coast sands.

The hydraulic fill sand and end tipped gravel fill can be expected to be readily compacted by vibro-replacement, mitigating their potential to liquefy and laterally spread. The improved drainage provided by the free draining gravel columns will also reduce liquefaction potential.

Little or no improvement in the strength of the hydraulic fill silt is expected by vibro-replacement, however the replacement of the silt with the stronger gravel columns will improve the average shear strength along this potential shear plane reducing the potential for lateral spreading.

The conceptual design set the size and spacing of the gravel columns to be produced by vibro-replacement such that the average shear strength along the critical failure plain through the silt would be increased to 35 kPa. With this shearing resistance the Newmark [4] approach suggests lateral spreading under the ultimate limit state design earthquake would be limited to 200 mm. It is considered feasible to design the stadium's foundations and structure to tolerate these movements with any damage being relatively minor and repairable.

The conceptual design has selected ground improvement by vibro-replacement for the stadium site.

## CONCLUSION

There is no standard solution of ground improvements and foundations for developments on Wellington's reclaimed waterfront. The reclamations were formed in stages over a 100 year period employing a variety of techniques and a variety of materials. Appropriate ground improvement and foundation systems must be developed for each project considering the specific site conditions and the importance and public risk associated with the specific development.

With the variable and weak ground conditions at the proposed stadium site, vibro-replacement has been concluded to be a necessary and appropriate form of ground improvement for this site.

The Museum of New Zealand is another Wellington waterfront development. This building is currently under construction and since it is to house the nations treasures its design attracts a high importance factor. The ground conditions at the site contrast the stadium site in that they predominantly comprise end tipped granular fills allowing effective ground improvement by dynamic compaction.

## ACKNOWLEDGEMENTS

The author acknowledges the permission of the Wellington Regional Stadium Steering Group to report on the site assessment.

## REFERENCES

1 Abraseys and Menu (1988), "Earthquake Induced Ground Displacements", Earthquake Engineering and Structural Dynamics, Vol. 16.

2 Dellow & Perrin (1991), "Wellington Railway Yard Assessment of Liquefaction Potential During Earthquake Shaking" DSIR report Nº 1991/4.

3 Evans and Seed (1992), "Membrane Compliance and Liquefaction of Sluiced Gravel Specimens", Journal of Geotechnical Engineering, ASCE, Vol. 118, Nº 6.

4 Newmark (1965), "Effects of Earthquakes on Dams and Embankments", Geotechnique, 15.

5 Seed and Harder (1990), "SPT - Based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Strength", Proceedings of H Bolton Seed Memorial Symposium, University of California, Berkeley.

6 Seed, Tokimatsu, Harder and Chung (1985), "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations", Journal of Geotechnical Engineering, ASCE, Vol. 111, Nº 12.

7 Stark and Mesri (1992), "Undrained Shear Strength of Liquefied Sands for Stability Analysis", Journal of Geotechnical Engineering, ASCE, Vol. 118, Nº 11.

8 Standards Association of New Zealand, 1992, "Code of Practice for General Structural Design and Design Loadings for Buildings", NZS4203, Wellington.

# SOME ENGINEERING PROPERTIES OF A VOLCANIC SAND

S. PRANJOTO[1], T.J. LARKIN[2]
[1] FoRST Research Fellow, Dept. Of Civil and Resources Engineering, University of Auckland
[2] Senior Lecturer, Dept. Of Civil and Resources Engineering, University of Auckland

## SUMMARY

Pumice sand is widespread over the central part of the North Island due to frequent past volcanic activity. The pumice has distinct properties different from that of quartz sands. The index and engineering properties are reviewed. The most significant difference from typical quartizitic sands is the grain softness of pumice. The particles are easily crushed by a finger nail. The other major difference is the void ratio, which is about twice that of typical quartz sands. This two factors are suggested as being substantailly responsible for its different behaviour, as found in a series of consolidated drained triaxial tests on specimens with free end platens.

## INTRODUCTION

Extensive deposits of volcanic sand are found in the central part of the North Island. Problems have been encountered when this sand has been involved in some geotechnical projects in the recent years. The very high erodibility of the pumice sand in a hydro development has led to a catastrophic failure in the region of canals and intake structure. These occurences highlighted the unique behaviour and properties of this pumice sand, and stimulated an interest to study to the basic engineering properties and behaviour.

The distinctive property of pumice sand is the softness of the grains, which are easily crushable under only modest finger nails pressure on a glass plate. This is in contrast with quartz sand which has very high level of crushing pressure; perhaps reached at the base of large earth dams (in the order of several MPa). This difference is expected to lead to index and engineering properties of pumice sand that are different from those of quartzitic sand. As many empirical formula are derived from sand of quartzitic origin, it may be misleading to apply them to interpret test results on pumice sand.

In situ cone penetrometre tests have been been widely used to obtain data for sand deposits. Various methods for interpreting cone test result are summarised by Lunne and Christoffersen [6] and Robertson and Campanella [8]. Considering the very high pressures developed around the advancing cone the possibility of grain crushing in volcanic sands is very high.

Some testing has been done to obtain the index properties, strength properties, and the stress-strain-volume change behaviour. With the newly developed $K_0$ testing apparatus, the constrained modulus D, the coefficient of lateral stress at rest, $K_o$, and the slope of the normally consolidation line have also been obtained. The first two parameters will be useful to develop a mathematical model of expansion of a cavity in an infinite granular medium. Using this mathematical model, one can predict the difference of the cone penetration resistance between pumice sand and quartz sand, so that the commonly available formula to interpret cone test results for quartz sand may be calibrated to be used for pumice sand. The third parameter could be used to compare with the slope of the critical state line e-log p.

## BASIC PROPERTIES

### Grain Size Distribution

The volcanic sand used in these tests comes from the Puni river, hence it has become known as Puni sand. It containes a small amount of impurities. Mercer No 1 sand a quarzitic material, was also tested as a direct comparison. Mercer No 1 has a very small content of pumice. They have a similar range of particle sizes in the grading curve, although the distributions are not the same. A new composition of particle sizes for test specimens (of both sands) was made by excluding particle sizes bigger than 1.18 mm and smaller than 300 μm. The proportion of the particle sizes between 1.18 mm and 300 μm was based on the natural proportion of pumice sand in this range. The ratio of diameter of specimens (70 mm) to the maximum particle size was considered to be of a reasonable order. Figure 1 shows the grading curves for natural composition of both sands and that for the test specimens.

## Grain Features

The main feature of the pumice grain is the softness. A small amount of very fine particles was collected at the end of each of the traixial tests. Figure 2 shows the change in the grading curve from the inital specimen grading for tests under 150 to 500 kPa cell pressures. Increase in cell pressure results in more particle crushing and further shift in the grading curve.

Scanning electron microscope photographs were taken of both a pumice grain from Puni sand, and a quartz grain from Mercer No. 1 sand. Figure 3a below shows the vesicular nature of the pumice grain. This is typical for all range of sizes. The vesicles are formed in glassy igneous rocks by the expansion of a bubble of gas or steam during the solidification of the material. It is also apparent that there are internal voids trapped within each grain. The flaky grains have subangular to angular shapes. Some have a fin-like structure on the surface. All these features make it susceptible to grain breakage and crushing. Figure 3b shows the very different nature of a quartz sand grain, which is solid, subrounded shape, and hard.

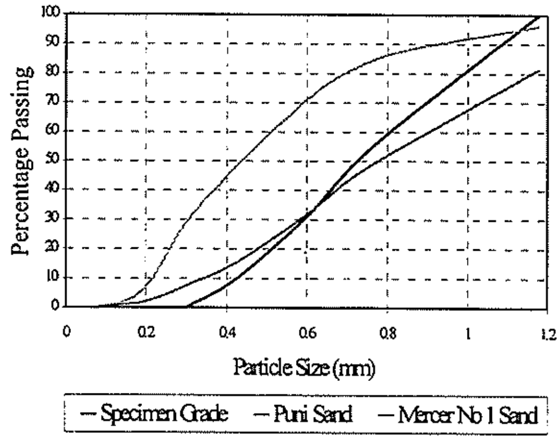



Figure 1 The natural and specimen grading curves

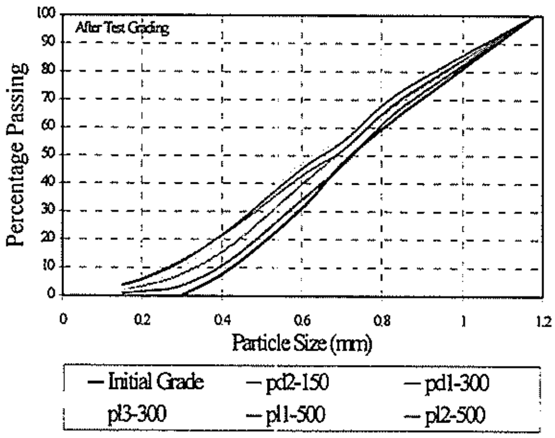



Figure 2 Grading curves initial and after test


Figure 3a Pumice grain on 710 μm sieve

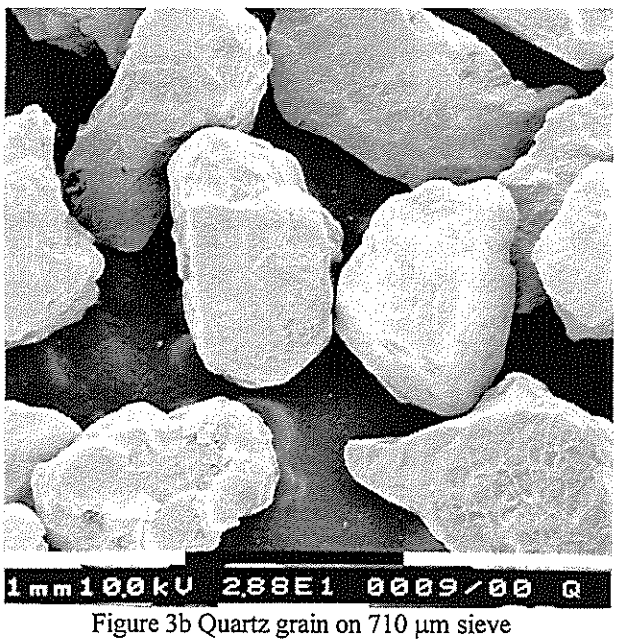



Figure 3b Quartz grain on 710 μm sieve

## Specific Gravity Tests

Three series of specific gravity tests have been done on the Puni sand for each particle size range retained on the series of sieves used in determining the grading curves. The first series, which contains a small percentage of non pumiceous material, shows a clear tendency of increasing solid density with decreasing particle size, from 2.18 t/m$^3$ for grains on 1.18 mm sieve to 2.76 t/m$^3$ for grains on 75 μm sieve. As pumice is basically silica, it was expected to have an upper bound of solid density of 2.7 t/m3. However, for the fine particles of 75 to 150 μm size, which is beyond the range of sizes used in the specimen, Gs is found to be 2.76 t/m$^3$, greater than the supposed the upper bound. This is due to the non pumice content which has solid density greater than 2.7 t/m$^3$.

In the second series the impurities content was removed by a simple centrifugal technique for grain size from 300 μm up to 2.36 mm. This technique was developed as it was observed that pumice grains were light in water compared with the non pumice, some pumice even floated on the water surface. The results confirmed that larger pumice particles have a lower solid density, which implies more air trapped inside the solid. As it is very difficult to obtain pure pumice grains smaller than 300 μm, the pumice grain of 1.18 - 2.36 mm size were ground to break them into smaller particles. The very fine grains smalller than 150 μm were then collected and tested. The third test series of ground pumice shows a solid density of 2.48 t/m$^3$ for particles smaller than 63 μm, which is still below the solid density of silica. Figure 4 shows the consistent pattern of the solid density vs particle size.

The reported solid density of each particle size is actually an average of the results of 3 tests done simultaneously under the same conditions. It is worth noting that the variations in results were up to 0.04 t/m3, which is considerably more than that of quartz based sand. This variation was also found by Galloway [3]. The nominal solid density of the samples was calculated based on the proportion and solid density of each size range in the specimen material. For the Puni sand specimen, these procedures yields a value of 2.27 t/m3, while for Mercer No 1 sand a $G_S$ value of 2.58 t/m3 was found.

## Bulk Density and Void Ratio

Maximum and minimum densities of the specimens were obtained in accordance with NZS 4402, 1986 [12] and the corresponding minimum and maximum void ratio accordingly. The basic properties of both sands are summarised in Table 1. The void ratios for Puni sand are distinctly different, being about twice that of a typical quartz sand.

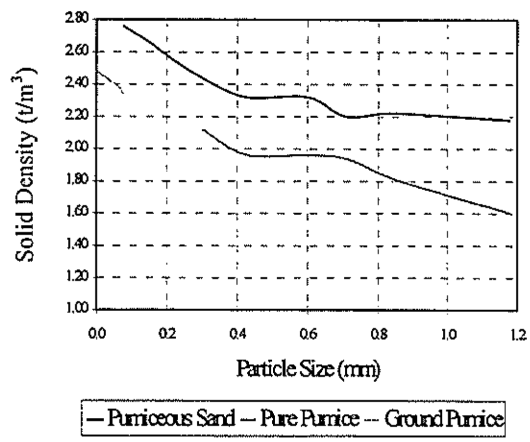



Figure 4 Solid density vs particle size

Table 1 The basic properties of the specimens

| | Puni Sand | Mercer No 1 Sand |
|---|---|---|
| $\gamma_{max}$ (t/m$^3$) | 0.926 | 1.471 |
| $\gamma_{min}$ (t/m$^3$) | 0.782 | 1.266 |
| $e_{min}$ | 1.452 | 0.754 |
| $e_{max}$ | 1.906 | 1.039 |
| $G_S$ (t/m$^3$) | 2.27 | 2.58 |
| $d_{50}$ (mm) | 0.73 | 0.73 |
| $C_U$ | 0.741 | 0.741 |
| Grain shape | flaky subangular to angular | solid subrounded |

## TRIAXIAL TESTING

A series of isotropically consolidated, dry, drained triaxial testing with free end platens has been carried out on both Puni and Mercer no 1 sands with 25 - 500 kPa cell pressures. These tests were done on dense and loose specimens, which were made up at the maximum and minimum void ratios. The test programme was designed to approach the critical state line both from above and below, i.e. prior to shearing some samples have void ratios larger than those at the critical state, and some others have lower void ratios.

## Critical State Line

The aim of compression testing with free ends was to achieve a deformed sample in a condition known as the critical state. In this state the original structure of particles have been completely altered (Ishihara [4]), and it is achieved when the whole sample deforms homogeneously with zero volume change, zero deviatoric stress change under a constant strain rate of loading (Chu and Lo [1] ; Schofield and Wroth [11]). Developed from the idea of yield surfaces and critical voids ratio line (Roscoe, Schofield, Wroth [9]), the concept of critical state was proposed by Schofield and Wroth in 1968 [11]. This state can be achieved from any original structure and density and needs mobilisation of large strains. The critical state concept presents a unified explanation of stress-strain-volume change behaviour, although experimentally it is not easy to achieve. Hence, much discussion has occurred concerning the existence for such a state for any soil (Konrad[5],Parry [7]).

## The Free Ends

Free ends were used to provide almost friction free radial expansion on the boundary of the sample. The free end was made up of an enlarged polished platten, and two layers of a rubber membrane with High Vacuum Silicon grease smeared between the membranes, and between the platen and the first membrane (Rowe, [10]; Chu and Lo [1]). The membrane was cut in a radial and circumferential fashion to lessen the constraint when it expands radially. The enlarged platen is 20 % larger than the initial sample diameter to accomodate the expanding diameter of sample as the test proceeds.

## Sample Preparation

Techniques to make loose and dense specimens have been developed with reliable consistency. A dry deposition method was applied by using a small opening glass funnel to place carefully and slowly the sand at the centre of a 3 way split mould. As the funnel is raised slowly the sand will pour from the centre downward radially toward the mould at the angle of repose, and build a sand cone with the top at the tip of the funnel. When the elevation of sand in the mould has reached the required height, the surface is levelled off by a vacuum process to remove the excessive sand. The same technique is applied for a dense specimen, except that the sample is placed on a vibrating table prior to removal of the excess sand to achieve the minimum volume.

## Results and Discussion

Figure 5a and 6a show that the deviator stress vs axial strain curves of Puni sand and Mercer No 1 sand have a different pattern. The peak and residual stress for Mercer sand were typically achieved at 5 - 15 % axial strain, while for Puni sand the strain was from 10 % to 50 % (even more for loose specimen under 500 kPa cell pressure, test pl6-500). A rapid and marked decrease from peak stress to residual stress is also demonstrated for Mercer sand, as opposed to a lower gradient and smaller decrease in stress for Puni sand specimens.

For the Puni specimens, some portion of the voids are actually recesses on the grain surface, while some occur between the fin-like structures on the surface. This portion is not accesible by the moving grains during the shearing process. However, it seems of no major relevance as indicated in Figure 5b and 6b that a much larger percentage of void ratio in contractive Puni sand specimens was reduced than that of Mercer No 1 sand specimens. In other word Puni sand is highly compressible, and it requires a large strain mobilisation before the peak and/or residual stress are achieved. The high compressibility is very likely facilitated by the flaky angular shaped grains, which is responsible for the high void ratio, and the softness of the grains which results in fracture during compression tests and give leads to a large reduction in the void ratio. The amount of particle crushing is not completely reflected in the shift of the grading curve as some of the fractured grains derived from angular particles will still have the same effective particle diameter.

The critical state lines are shown for both Puni and Mercer No 1 sands on the e vs log p' curves in Figure 5b and 6b. The critical state parameters for the e vs ln p' plane are derived as follows:

| | | |
|---|---|---|
| Puni sand: | $\lambda = 0.347,\ \Gamma_1 = 4.400$ | where: $\lambda$ = the slope of critical state line |
| Mercer No 1 sand: | $\lambda = 0.119,\ \Gamma_1 = 2.603$ | $\Gamma_1$ = the specific volume at p'= 1 kPa |
| | | p' = the mean effective pressure = $(\sigma_1'+\sigma_2'+\sigma_3')/3$. |

The pumiceous sand results give a clearer definition of critical state line than those of the quartzitic sand. This may be due to the difficulty in achieving stabil dilatative samples which expand uniformly along the height.

Compared with other sands' critical state lines (Collins et al [2] ), the location of Puni sands is very remote from the other reported results. For example, Ottawa sand and Reid Bedford sand were reported as having $\lambda$= 0.028, $\Gamma_1$= 1.754; and $\lambda$= 0.065, $\Gamma_1$= 2.014, respectively. The slope of critical state line for Puni sand is also found to be parallel with the normally consolidated line obtained from the $K_0$ tests ($\lambda$=0.375), an agreement which is expected within the framework of critical state theory (Figure 7).

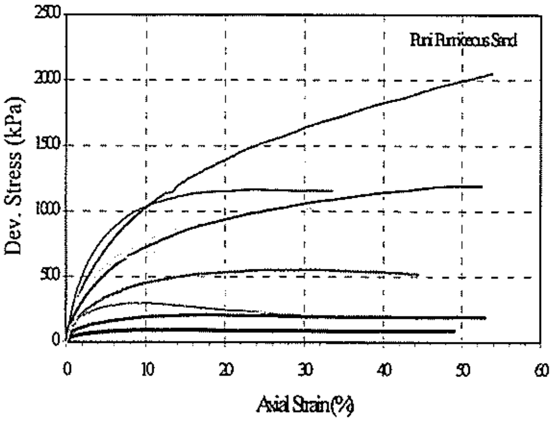



Figure 5a Deviator stess vs axial strain curves
(Puni Sand Tests)

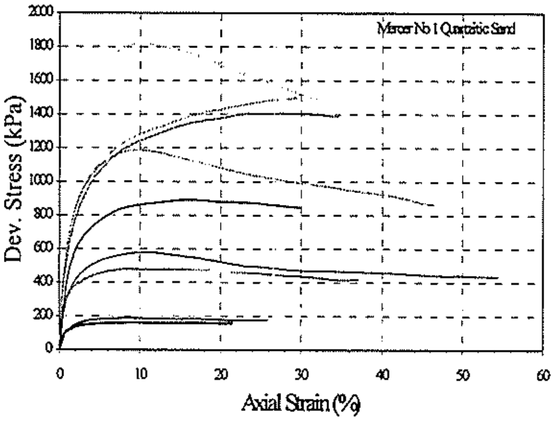



Figure 6a Deviator stress vs axial strain curves
(Mercer No 1 Sand Tests)

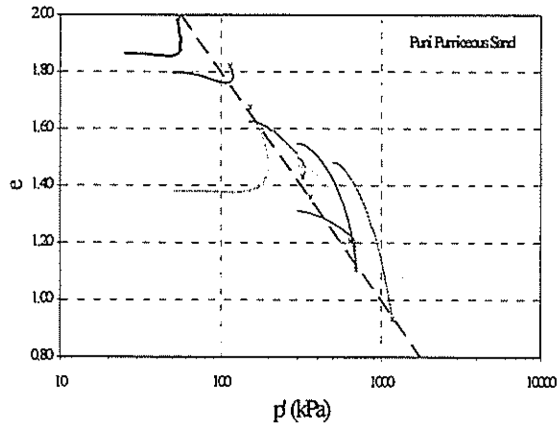



Figure 5b Void ratio e vs log p' curves
(Puni Sand Tests)

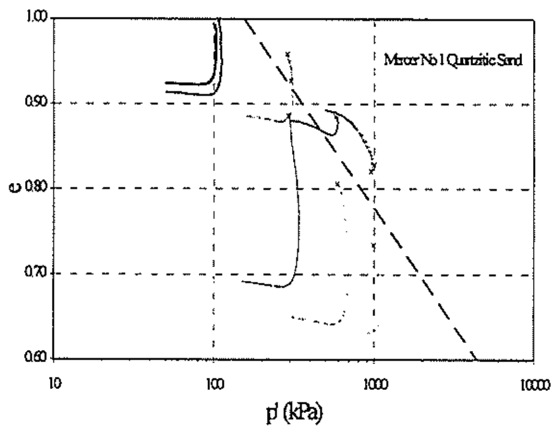



Figure 6b Void ratio e vs log p' curves
(Mercer No 1 Sand Test)

Legend for Figures 5a and 5b:

| | | |
|---|---|---|
| p 18 - 25 | p 15 - 50 | p 14 - 150 |
| p 13 - 300 | p 17 - 300 | p 16 - 500 |
| pd 5 - 50 | pd 2 - 150 | pd 3 - 300 |
| cs - line | | |

Legend for Figures 6a and 6b:

| | | |
|---|---|---|
| q 15 - 50 | q 17 - 50 | q 12 - 150 |
| q 16 - 150 | q 11 - 300 | q 13 - 500 |
| q 18 - 500 | qd 3 - 150 | qd 1 - 300 |
| qd 2 - 500 | cs - line | |

Table 2 indicates that the internal friction angle $\phi'$ is not solely dependent on the hardness of the grain nor the compressibility. Nevertheless the high maximum strength of Puni sand is unlikely to be utilised since it requires mobilisation of deformation beyond the range of normal engineering practice.

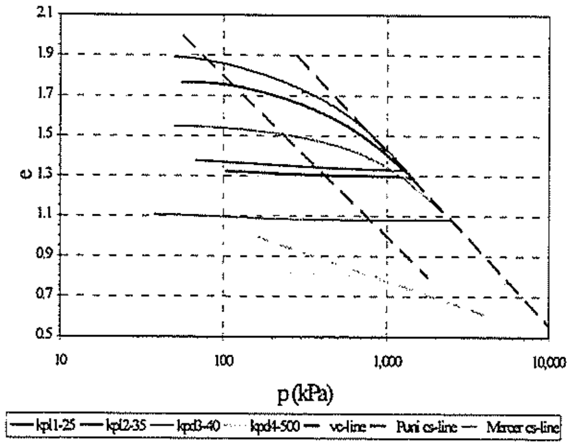



Figure 7 e vs p' from $K_o$ test for Puni Sand

Table 2 Peak and residual internal friction angle $\phi$

| | Dense | Loose | Residual |
|---|---|---|---|
| Puni Sand | 44 | 41 | 41 |
| Mercer No 1 Sand | 41 | 38 | 38 |

## CONCLUSION

The basic engineering properties and behaviour of Puni sand has been identified. The trapped air in the "solid" grain of Puni sand results in a decreasing solid density with increasing particle size, and a high variation of the solid density results between the simultaneously conducted tests for a particular particle size range. The high compressibility of Puni sand specimens is responsible for the large strain mobilisation required to achieve the peak and residual stress. The softness and flaky angular shape of the grains are suggested as the key factors of the high compressibility. Although the Puni grain is easily crushed by finger, it has peak and residual internal friction angles greater than 40 degrees. The critical state line was also successfully determined for both sands. The one dimensional consolidation line, from $K_o$ tests, was found to be parallel with the critical state line.

## REFERENCES


1 Chu, J and Lo, SCR 1992. On the measurement of critical state parameters of dense granular soil, *Geotechnical Testing Journal*, the American Society of Testing and Materials 16(1): 27-35.

2 Collins, IF; Pender, MJ; Yan, W 1992. Cavity expansion in sands under drained loading conditions, *International Journal for Numerical Method in and Analytical Methods in Geomechanics* 16: 3-23.

3 Galloway, JHH 1963. The specific gravity of a volcanic soil, *The Proceedings of the Fourth Australia - New Zealand Conference on Soil Mechanics and Foundation Engineering*, Adelaide.

4 Ishihara, K 1991. Liquifaction and flow failure during earthquake, *Geotechnique* 43(3): 351-415.

5 Konrad, JM 1990. Minimum undrained strength of two sands, *Journal of Geotechnical Engineering*, ASCE, 116(6): 932-947.

6 Lunne, T and Christoffersen, HP 1983. Interpretation of cone penetration data for offshore sands, *Proceedings of the 15th Offshore Conference Houston*: 181-188.

7 Parry, RHG 1958. Correspondence on "On the yielding of soils", *Geotechnique* 8(4): 183-185.

8 Robertson, PK: Campanella, RG 1983. Interpretation of cone penetration tests: part1, sand, *Canadian Geotechnical Journal* 20(4): 718-733.

9 Roscoe, KH; Schofield, AN; Wroth, CP 1958. On the yielding of soils, *Geotechnique* 8(1):22-53.

10 Rowe, PW and Barden, L 1964. Importance of free ends in triaxial testing, *Journal of the Soil Mechanics and Foundations Division*, ASCE 90(SM1): 1-27

11 Schofield, A and Wroth, P 1968. *Critical State Soil Mechanics*, McGraw-Hill, London.

12 Standard Association of New Zealand. 1986. *Method of testing soils for civil engineering purposes.* NZS 4402, Wellington.

# DESIGN AND CONSTRUCTION OF CASTLEMAINE LANDFILL

**DON RICHARDSON**
Golder Associates Pty Ltd
Melbourne, Australia

## SUMMARY

In 1994 a new landfill was designed and constructed in accordance with strict environmental and regulatory requirements on a former alluvial gold mining site in Castlemaine, central Victoria. The landfill includes staged filling in specially constructed cells, leachate management and progressive rehabilitation. A major geotechnical aspect of the project was lining the base of the landfill cells with a low permeability liner constructed by ripping, breaking down, moisture conditioning and compacting the on site weathered sedimentary rock. The liner construction works were generally performed in accordance with specified relative compaction, moisture and particle size requirements to achieve a median permeability of between 4.0 x $10^{-9}$ and 9.4 x $10^{-9}$ m/s.

## INTRODUCTION

Castlemaine is a rural Victorian centre with a regional population of about 15000. This paper outlines the design and construction of a new landfill for the centre. Included is a description of the regulatory framework at the time as well as the site conditions such as hydrogeology and geology which were main factors in design. A prime objective of the landfill design was to protect the local environment.

Of particular interest was the utilisation of the on-site weathered sedimentary rock to construct a low permeability liner in the base of the landfill cells. It is understood that this has limited precedent in Australia. The paper describes the construction of the liner including construction methods, compliance testing during construction, permeability testing using single-ring infiltrometers as well as some observations of the broader 'nuts and bolts' issues involved with building a major earthworks project.

## SITE DESCRIPTION AND HYDROGEOLOGY

The general layout of the landfill is shown in Figure 1. The site has an area of about 9 hectares. It slopes from south to north with a change in elevation of about 25 metres.

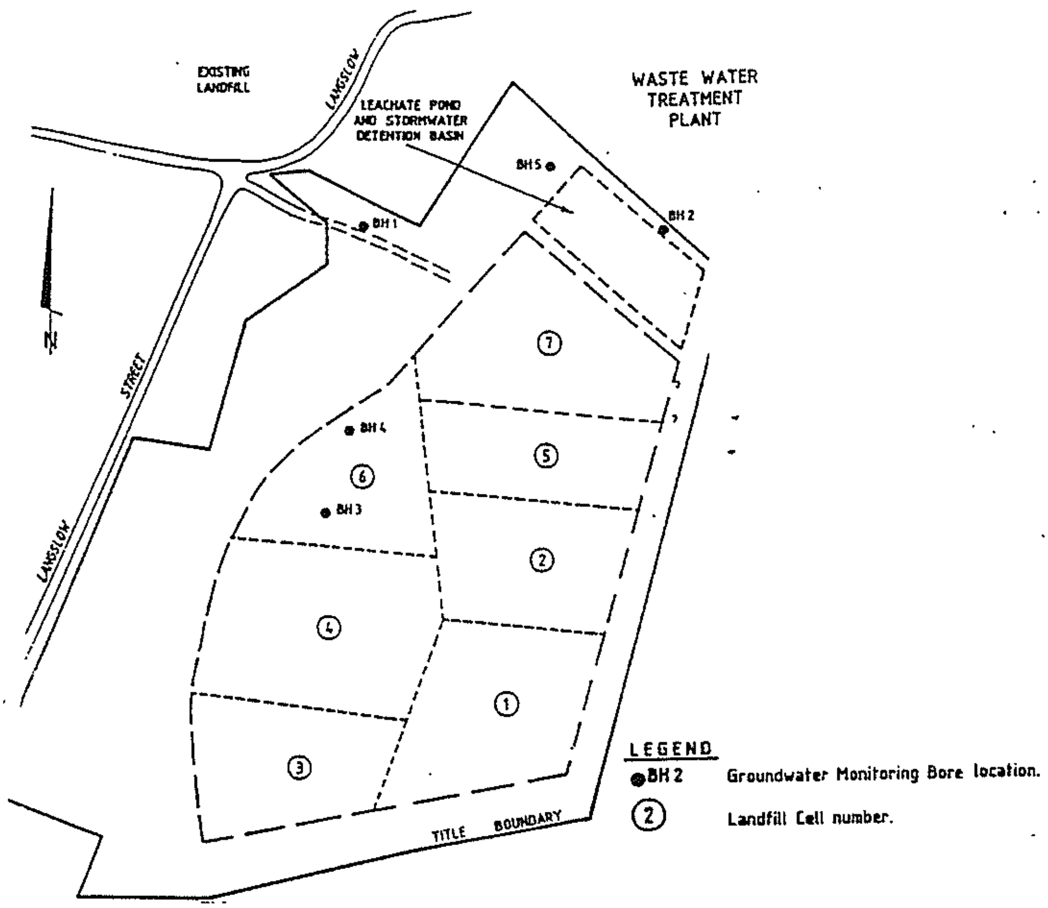
The site is underlain by weathered Paleozoic age sandstone and siltstone with well developed near vertical bedding. Soil cover is thin to absent on parts of the hillside due to gold mining activities using sluicing and alluvial mining techniques. However the remaining soils are of two origins:

- Tertiary age sand and gravels which cover the rock on the upper part of the site.

- Quaternary age alluvial and colluvial soils which occur in drainage lines on the lower part of the site.

Permeability tests indicated permeabilties in the rock ranging from 2 x $10^{-4}$ to 4 x $10^{-6}$ m/s. The depth to the groundwater table under the lower parts of the site was a minimum of about 2 m and increased southwards under the hillside. The groundwater flow direction was to the north east and the groundwater underflow was calculated to be about 22 $m^3$/day. [6] The groundwater quality at the down gradient site boundary was characterised by a mean total dissolved solids (TDS) concentration of 3740 mg/L. This indicates that the groundwater has a beneficial use as stockwater but is unsuitable for potable or irrigation water [4].

Figure 1. Site Plan (Scale 1:5000)




## REGULATORY FRAMEWORK

The Environment Protection Authority (EPA) in Victoria stipulate that new landfill be permitted through a Works Approval system. This requires a technical evaluation of the impact on the environment by the landfill. Of particular relevance in the regulatory guidelines for municipal waste landfills is the development of necessary control measures to protect the beneficial uses of the local groundwater and surface water. [5,1].

## LANDFILL DESIGN

Taking into account the site specific conditions such as hydrogeology, geology, climate and topography the landfill was designed to minimise environmental impact in accordance with the regulatory framework.

Leachate is produced within a landfill when moisture enters refuse, extracts contaminants into a liquid phase and produces sufficient moisture to initiate liquid flow. [3] The estimation of leachate volumes and quality during the design is beyond the scope of this paper.

Water quality at the landfill has been protected by implementing the following control measures to minimise leachate generation and minimise the transfer of leachate into surface water and groundwater.

i) Diversion of all stormwater around the site.

ii) Staged operation of the landfill in a series of cells which are capped and rehabilitated as they are completed.

iii) Minimising seepage of leachate through the base of the landfill cells by construction of a low permeability liner using the on-site weathered sedimentary rock. The liner can also attenuate contaminants such as metals in the leachate which could pollute the groundwater.

iv) A leachate collection system to remove leachate from the cells to maintain low hydraulic head on the liner

v) Progressive capping of the landfill cells to promote runoff and minimise leachate generation. The capping consists of about 500 mm of low permeability soil overlain by topsoil and with a minimum surface gradient of 5%. The cap will be revegetated as part of the site rehabilitation.

The following sections of this paper focus on the above item iii) as one of the major geotechnical aspects in the overall landfill design.

### Requirement for a Low Permeability Liner for the Landfill Cells

The siltstone/sandstone at the site has well developed bedding that dips near vertically. It was considered that this could potentially allow locally rapid infiltration of leachate and allow concentrations of leachate along the preferred flow paths. Therefore it was considered necessary to disrupt any preferred flow paths, to limit permeability and to provide an attenuating layer. Based on the quality of groundwater leaving the site and the potential beneficial uses of the water it was considered that an appropriate level of protection would be provided by the construction of a 600 mm thick liner in the base of the landfill cells with a minimum median surface permeability of $1 \times 10^{-8}$ m/sec. This was a requirement of the EPA Works Approval for the landfill.[2]

### Material Selection

It was considered that in the absence of sufficient quantities of suitable on-site clays that the on- site weathered siltstones and sandstones were the most cost effective materials to construct the liner. Preliminary laboratory testing indicated that the target hydraulic conductivity requirements could be met provided that the rock is ripped and compacted to achieve a Dry Density Ratio (AS 1289.5.4.1) of at least 95% Standard (AS 1289.5.1.1) and at a moisture content of not less than optimum (OMC).

### Liner Construction Trials

A construction trial was performed to assess construction methods that could satisfy the permeability requirement for the liner using the on site weathered rock and produce a specification for the liner construction. A 600 mm thick trial pad measuring about 21 m x 12 m overall was constructed in three layers. The following construction procedure was found to be satisfactory to meet the permeability requirement.

- rip the weathered siltstone and sandstone to a depth of about 600 mm. Track roll with a bulldozer to assist in breaking down the rock.
- stockpile the ripped material.
- moisture condition to just wet of OMC by spraying water from water truck and continually mixing with the dozer until evenly mixed.
- spread out moisture conditioned material and compact with eight passes of Dynapac 10 tonne vibrating padfoot roller.

Field density, laboratory classification and grading tests were performed on the compacted materials. The compacted material can be characterised as a Sandy Gravelly Clay of low plasticity. The fines had a Plasticity Index of about 6 %, Liquid Limit of about 22 %, linear shrinkage of about 2 to 3 % and moisture content of between about 11 % and 15 %.

The median vertical permeability of the trial pad was found to be $4.0 \times 10^{-9}$ m/s. This is less than the minimum of $1 \times 10^{-8}$ m/s required by the EPA Works Approval. The method of permeability testing is described in the following section.

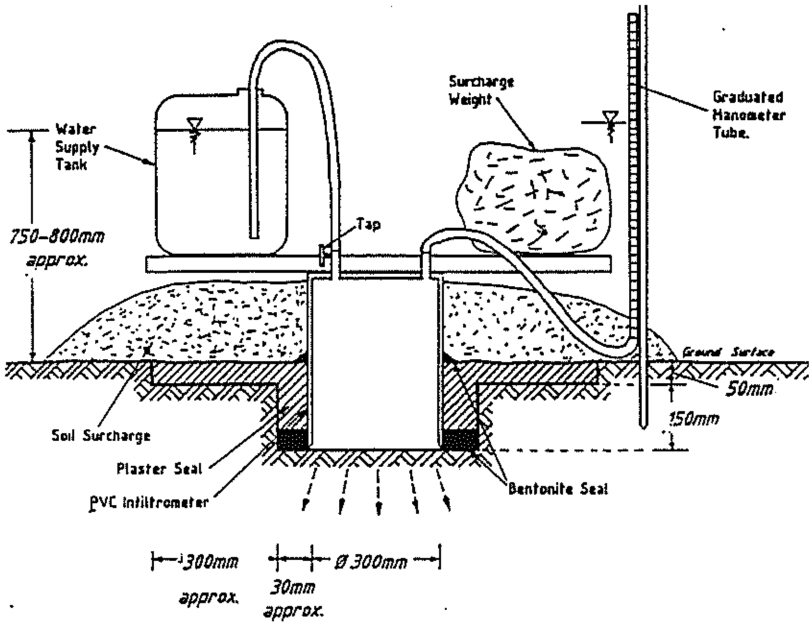
It was concluded from the Liner Construction Trial that the requirements of the EPA Works Approval could be met using the on site siltstone and sandstone materials as a base liner for the proposed landfill cells subject to the liner being constructed to meet specified requirements for relative density, moisture variation and particle size distribution as outlined below.

### Permeability Testing

The single ring infiltrometer permeability test was considered to be the most appropriate to measure permeability to meet the Works Approval requirement. Laboratory permeability testing of undisturbed samples was initially considered but because of the nature of the material and the range of particle sizes it would not have been practical to recover and test representative samples.

The infiltrometer set up is shown in Figure 2. It consists of a 300 mm diameter, 400 mm long P.V.C. tube sealed at one end and with a bevelled cutting edge at the other. The tube is seated into the bottom of a shallow flat based hole excavated to a depth of about 200 mm below the surface. The annulus around the PVC tube is then backfilled with bentonite and plaster to provide a seal. Soil is then mounded over the seal and surrounding ground to provide a surcharge.

Figure 2. Cross section of single ring infiltrometer for permeability testing




The infiltrometer is filled with water and connected to a water container to maintain a constant head. Also connected to the top of the infiltrometer is a graduated manometer tube. The test area inside the infiltrometer is saturated for about 24 hours and then a flow measurement is taken by turning off the water supply and measuring the fall in water level in the manometer tube over time (generally 30 minutes). The test procedure is repeated several times at intervals greater than 1.5 hours to monitor any change in the rate of infiltration.

**Specification for Construction of the Landfill Cell Base Liner**

Construction of the landfill cell base liners was specified to be as follows

i) Maximum layer thickness of 200 mm after compaction.

ii) The liner material shall be compacted to a Hilf Density Ratio (Rapid Method) AS1289.5.7.1-1993 of not less than 95% STANDARD.

iii) The moisture content of the compacted material shall be between 0% to 3% Wet of the Optimum Moisture Content. Testing was in accordance with the Hilf Moisture Variation (Rapid Method) AS1289.5.7.1-1993

iv) The particle size distribution of the compacted material shall have greater than 70% of material passing the 2.36 mm sieve and greater than 35% of material passing the 0.075 mm sieve. Testing is to be in accordance with AS1289.C6.1.

Compliance testing to be undertaken at the completion of each 200 mm layer at a rate of about 10 tests per hectare to assess compliance with items i), ii) and iii) above. Five field permeability tests were carried out per cell on the completed cell base liner to confirm that the required permeability had been achieved.

**Construction Method**

The 600 mm thick base liner for the landfill cells No. 1, 2 and 3 was constructed by Leech Earthmoving Pty. Ltd., Castlemaine. Each cell had a base area of between about 0.6 and 0.9 hectares and was constructed in two stages (Stage A and B). The cell locations are shown on the Site Plan, Figure 1. The other four landfill cells are proposed to be constructed at a later date. Site equipment generally included a bulldozer, scraper, water truck and vibrating padfoot roller. A grader was used at times to trim embankment slopes and areas subject to overfilling. The preparation of a 200 mm layer over an area of about 0.7 hectares generally took about 1 to 1.5 days.

After initial surface preparation the following construction procedure was considered to be the most effective and efficient by the contractor.

- Rip the weathered siltstone with a bulldozer and break up the ripped rock by track rolling with the bulldozer and with passes with the vibrating pad foot roller.
- Transfer to base of landfill cell using a scraper. Spread out the material in thin layers of about 50 mm thickness.
- Add moisture by spraying with one pass of water truck at the top and bottom of each 50 mm layer. Tyning was sometimes required to assist in achieving an even mix.
- Compact with about four passes of vibrating padfoot roller.
- At the end of each day or 200 mm layer seal the surface with the wheels of a fully laden scraper. Before placing further material the surface was prickled with the pad foot roller to prevent laminations and assist the bonding between layers.

### Results of Compliance and Permeability Testing

The results of permeability and the compliance testing are summarised in Table 2 below.

Table 2 . Summary of compliance and permeability testing
for the landfill cell base liner, Castlemaine landfill

| CELL | SPECIFICATION | 1A and 2A | 1B and 2B | 3A | 3B | overall |
|---|---|---|---|---|---|---|
| Number of tests | | 21 | 21 | 15 | 12 | 69 |
| HILF DENSITY RATIO (% Standard) - range of results - mean | >95% | 95 to 98.5<br>96.9 | 93 to 98<br>96.3 | 95.5 to 101.5<br>99.3 | 97 to 101.5<br>98.5 | 93 to 101.5<br>97.8 |
| VARIATION FROM OPTIMUM MOISTURE CONTENT (OMC) - range of results (% Wet) - mean (% Wet) | 0 to 3% Wet | 0.5 to 3.0<br>1.7 | 0 to 3.5<br>1.0 | 0 to 3.5<br>2.1 | 0 to 2.5<br>1.58 | 0 to 3.5<br>1.6 |
| PARTICLE SIZE DISTRIBUTION | | | | | | |
| % passing 19.0 mm - range of results - mean | not specified | 90.0 to 98.4<br>95.2 | 87.4 to 99.1<br>96.1 | 97.9 to 98.8<br>93.8 | 81.9 to 96.3<br>90.4 | 81.9 to 99.1<br>93.9 |
| % passing 2.36 mm - range of results - mean | >70% | 73 to 90<br>80.5 | 72.4 to 91.3<br>83.9 | 60.5 to 84.5<br>70.6 | 56.3 to 74.1<br>65.3 | 56.3 to 91.3<br>75.1 |
| % passing 0.075 mm - range of results - mean | >35% | 54.6 to 71.6<br>61.2 | 38.8 to 71.1<br>58.7 | 38.6 to 64.3<br>53.6 | 41.4 to 58.3<br>48.9 | 38.6 to 71.6<br>55.6 |
| MEDIAN PERMEABILITY (m/s) | $< 1.0 \times 10^{-8}$ | $8.8 \times 10^{-9}$ | $4.0 \times 10^{-9}$ | $4.4 \times 10^{-9}$ | $7.3 \times 10^{-9}$ | $4.2 \times 10^{-9}$ |

## DISCUSSION

Construction to meet the specified requirements for relative density, moisture conditioning and particle size distribution resulted in a liner with median permeability of not more than $1 \times 10^{-8}$ m/sec. As a point of comparison the permeability of the leachate pond liner constructed from the on-site clays was measured to be between $1.4 \times 10^{-11}$ m/sec and $4.8 \times 10^{-11}$ m/sec The decision to construct a trial pad prior to bulk earthworks at the site had proven to be very effective and useful. Based on the permeability results being close to the required minimum, it was evident that it was critical to enforce the specification requirements noted above. A general trend was noted that the greater the breakdown of the weathered rock to material passing the 0.075 mm sieve, the lesser was the permeability.

Difficulties were experienced with variations in the degree of strength and weathering in the rock across the site. Rock from several areas proved to be more difficult to break down than from other areas. It was suggested by the contractor that the materials used in construction of the test pad may not have been representative of some others at the site. Could it be argued that the variability was not adequately accounted for at the planning and design phase of the project? Or was it more an issue of how the contractor planned and managed sourcing of the rock? Ultimately this issue was resolved by allowing some flexibility with the location of sourcing the more suitable lower strength rock across the site. Similarly, there was also some flexibility with the design subgrade levels for the cell bases to allow sourcing a greater quantity of suitable rock from these areas if appropriate. This prevented effective burial of suitable source rock beneath the cells.

The project enhanced my understanding of the nuts and bolts of construction. This includes the practicalities of what can and cannot be achieved in the field. The two issues raised above are examples of situations when it is paramount to enforce specified requirements and others when they can be interpreted in a more flexible manner. A key to the whole process is to have a superintendent who can make clear, impartial and resolute decisions. This requires a sound understanding of the engineering and contractual issues as well as excellent communication and people skills. Mr. Malcolm Styles of the City of Castlemaine fulfilled this role admirably.

## CONCLUSION

The new Castlemaine landfill was designed in accordance with strict environmental and regulatory requirements. The site hydrogeology indicated that the design needed to incorporate various control measures to protect the beneficial use of the regional groundwater as stockwater. One of the main geotechnical control measures involved lining the base of the landfill cells with a low permeability liner by ripping, breaking down, moisture conditioning and compacting the on site weathered sedimentary rock. Construction was performed in accordance with specified requirements for relative compaction, moisture conditioning and particle size. The target permeability requirement stipulated in the Works Approval of not more than $1 \times 10^{-8}$ m/sec was achieved. In general, construction proceeded smoothly and was completed within the estimated timeframe and budget. The landfill is now operating successfully. On a personal level the project was an excellent opportunity for the author to be involved with current practice in landfill design. It also provided valuable hands on experience with laboratory and field geotechnical testing techniques. My site role gave me exposure to what I like to refer to as the 'nuts and bolts' or the day to day practical issues related to construction. These are not necessarily technically based but of no less importance in the construction of a large earthworks project.

## ACKNOWLEDGEMENTS

The author would like to thank Mr Malcolm Styles from the Wangaratta Rural City Council (formerly City of Castlemaine) and Mr Peter Thornton of Golder Associates for their assistance during the project and for permission to base this paper on the project.

## REFERENCES

1. Protection Authority Victoria 1991. *State Environmental Protection Policy (Siting and Management of Landfills Receiving Municipal Wastes),* Victorian Government Gazette No.S40

2. Environment Protection Authority Victoria 1992. *EPA Works Approval WA 1584*, Environment Protection Authority Victoria, Melbourne

3. Farquar G.J.1987, Leachate: Production and Generation, *Proceedings of the Canadian Society for Civil Engineering Centennial Conference, Montreal*

4. National Health and Medical Research Council 1990, *National Water Quality Guidelines Draft*

5. Parker, R J and Thornton, P N 1994. *Landfill Design - Controlling Leachate*, Papers for ACADS/AGS Geotechnical Landfill Design Seminar, Melbourne

6. Thornton, P N, Kerby, N E and Styles, M D G 1994. Evolution of a Landfill, Castlemaine Victoria, *Proceedings of the Second National Hazardous and Solid Wastes Convention, Melbourne*

# BIO REMEDIATION OF HYDROCARBON CONTAMINATED SOILS: EXPERIENCES FROM AUSTRALIA

W.M. TROTTER

Environmental Scientist for Groundsearch EES, Auckland, New Zealand

## SUMMARY

Bioremediation is an effective way of remediating hydrocarbon contaminated soils and is a widely used technique in Europe, the US and Australia. The landfarming technique involves the application of nitrogen and phosphate to the soil, monitoring the soil pile to ensure optimum moisture and pH conditions are met along with ongoing aeration to ensure an adequate supply of oxygen is available to ensure maximum micro-organism population growth. A landfarming bioremediation facility needs to be adequately managed to ensure the community and surrounding environment are not affected by the remediation process. On site management practices include: dust suppression, noise and odour control, stormwater runoff control, sediment and erosion control, groundwater monitoring and regular sampling to monitor the progress of the remediating soil.

## INTRODUCTION

Bioremediation is becoming increasingly popular as a method of cleaning up a wide range of contaminated sites around the world. In New Zealand it's application is relatively new but in other parts of the world such as Europe, the US and Australia the method is common place.

To date the most effective application of the method has been the clean up of hydrocarbon contaminated soils and water, but leading bioremediation consultancies are now developing methods for remediating soils with toxic metal contamination. This involves the introduction of genetically engineered micro-organisms which can accelerate and enhance their capacity to breakdown of these chemicals (Saville, 1991).

This paper is presented as an introduction to the landfarming technique and its application to hydrocarbon-based contaminated soils. The information presented in this paper is a combination of my experiences in the remediation of a decommissioned petrol depot in Eastern Australia (September 1995) and background information. For client security no details of the location of the site will be given.

## BIOTECHNOLOGY: AN INTRODUCTION

Bioremediation is based on harnessing natural bacterial activity of micro-organisms to decontaminate hydrocarbon polluted soils. Bacteria remove contamination by converting the chemical pollutant into cell biomass, carbon dioxide and water. It is essential during the process to optimise site conditions to ensure their activity is enhanced. This is achieved by a variety of management practices which will be discussed later. The energy needed to drive these biochemical conversions is generated by the bacteria transforming a portion of the absorbed organic carbon into carbon dioxide and water, as illustrated by Eq. 1 (Lapinskas, 1989).

$$C_nH_{2n+2} + O_2 \rightarrow C_5H_7N2O^* + CO_2 + H_2O \qquad (1)$$

Actual biochemical pathways of biochemical degradation of hydrocarbon contamination depends on the particular substrate and the type of organism involved. Heterotopic aerobic microorganisms are almost exclusively involved in hydrocarbon decontamination because oxygen is a vital component needed for the process to occur (Lapinskas, 1989).

Bioremediation provides an alternative to more expensive and environmentally harmful remediation techniques such as incineration and removal to landfills.

### *Landfarming*

This technique adopted by our company at the site in eastern Australia to clean up the hydrocarbon contaminated soil is commonly termed "enhanced landfarming". The technique involves the addition of nutrients, moisture and oxygen to the soil to produce the optimum conditions for micro-organism growth and hence degradation of the offending pollutant. Typically optimum degradation rates occur over the temperature range of 30-40° C (Lapinskas, 1989) at moisture contents that will vary for the different substrates encountered.

Vegetation or available mulch is often added to the remediating soil. Vegetation mulch has a composting affect on the soil thereby raising its temperature and increasing the microbial activity, while also breaking up the physical structure of dispersive soils and stiff clays, allowing fertilisers and oxygen to better penetrate the soil to greater depths as well as acting as a blanket to trap any remanent odours.

The same remediation technique is used for most types of hydrocarbon contaminated soil, whether it be from the light fraction, petrol end, or from the heavier oil and diesel end of the range. The soil volume affects the time taken for remediation of the soil, a smaller volume resulting in a more concentrated remediation effort. Warm weather conditions will also increase remediation rates.

## SETTING UP LANDFARMING FACILITIES

In eastern Australia I was involved in the removal and transportation of 2000 $m^3$ of hydrocarbon contaminated soil as well as the construction of the bioremediation facility. In many cases, with respect to urban service station remediation, bioremediation is carried out on site, however in this case a patch of land at the local landfill facility was made available.

Typically a bioremediation pad needs to contain a low permeability clay or synthetic base, which has the vegetation and topsoil removed. This base is constructed to gently slope to one end for stormwater management and compacted to prevent the infiltration of leachate and rainwater runoff.

The pad is bunded to redirect any upslope surface runoff from around the outside of the pad and to capture and retain any runoff from within the remediation pad area. These are typically designed to withstand a 1 in 5 year storm event.

A drainage sump is located within the bund in the downslope corner. In the case of the pad I was involved in a 4 metre wide and 1 metre deep sump was designed. This was rather an overkill given the rainfall characteristics of the region, and perhaps reflected the NZ influence in its design. The water collected in the sump is recycled and irrigated over the treatment area to keep the material at an optimum moisture content.

At the site existing gum trees surrounding the site were used as a screen which helped to minimise visibility and dust movement.

### *Problems encountered*

The site kindly made available by the local council was an area used for the disposal of night soil and grease trap waste from the local community for the past 30 years. This waste was pumped into 1.5 metre deep trenches which ran down the slope to the direction of the creek and into a sump 2 metres wide and at least 80 metres long. Five years prior to our involvement at the site the landfill manager filled in these liquid trenches with on site clay fill and let the area revegetate. We were completely unaware of the problem prior to construction of our facility. We were aware of the on the opposite side of the track in which grease trap waste from local restaurants and fast food outlets was dumped daily, but the past use of the other side of the track was not known.

The problem surface during rolling when the 20 tonne roller got bogged in the large sump on the lower part of the site. It was immediately evident from the smell! and the colour that this was grease trap and nightsoil waste. Samples were immediately dispatched to the laboratory to define its constituents so as to determine whether this would further contaminate our soil.

The waste contained several metals at toxic levels (ie. Zinc, Arsenic and Lead) along with the expected heavy oils and PAHs (poly aromatic hydrocarbons). While the oils would be likely to respond to bioremediation techniques, the metals would contaminate the soils we were bringing in. The heavy oils and even the PAHs can be successfully bioremediated but over a much longer time frame than that for petrol contaminated soils. Typically gas works soils contaminated with coal tars and phenols have been successfully cleaned up in within 15 months with a cost saving of 15-20% over landfilling (Saville, 1991). It was decided to remove as much of this grease trap and night soil material as possible and back fill the trenches with additional clay fill. The contaminated soil was stock piled on a corner of the pad and a proposal is underway to bioremediate this for the local council.

## REMEDIATION MANAGEMENT

Soil was transported to the site in tarpaulin covered trucks, with all trucks accounted for using signed tracking documents. The soil was spread out on the biopad in a 300- 400 mm thick layer. At this site the soil is kept at the optimum moisture content by;

1. recycling sump water and mobilising by use of a sprinkler system, and
2. use of recycled landfill leachate which is dispersed by onsite sprinkler trucks.

A state of optimum moisture content is maintained to enhance the growth of microbial bacteria within the waste and tensiometers will be placed at various locations in the soil to monitor the moisture levels.

During treatment the soil is tilled fortnightly, using a tractor to ensure ample oxygen is available for aerobic conditions required to enhance microbial population growth.

Samples are regularly collected from the remediating pile and analysed to evaluate how well the treatment is working and to determine if different nutrients or treatment methods are required. Results are required to be checked by the local council and the regulating authority the EPA (Environmental Protection Agency) before the soil can be returned to the site or used as topsoil or night cover within the landfill. In this case the site was backfilled with overburden clay from a local quarry and the remediated soil was given to the local council. Given adequate time the soil could be a high grade topsoil suitable for use in park and garden development around the town. Generally speaking this soil is expected to b designated clean for refilling after 4 months.

## ENVIRONMENTAL PROTECTION PRACTICES

### *Runoff Control*

As previously mentioned surface water runoff from within the pad is collected in a large sump and use to irrigate the remediating soil. Surface water from outside the pad is not an environmental concern and is directed around the bioremediation pad by 2 metre wide drains.

In periods of unusually heavy rainfall, excess water from the bioremediation pad by overflow the sump, in which case it is directed through a series of vegetated overflow drains with a total length of 20 metres arranged in a zigzag fashion to allow any sediment in the water to drop out and also to control and slow down the flow rate at which excess water reaches the creek.

### *Odour control*

Soil contamination by total petroleum hydrocarbons or benzene, toluene, ethyl benzene or xylene (BTEX), may produce odours for a short time when initially disturbed, but they tend to disperse quickly, disappearing within a few hours.

The case study site was located well away from residential properties and odour was not considered a problem. If odour is a problem mulch or polythene can be spread over the remediating pile to reduce the odour. The use of mulch is advantages as it provides green waste which increases the humic and nutrient content in the soil.

### *Dust suppression*

Factors which contribute to dust production include;

1. a cleared site,
2. stored backfill or topsoil material,
3. constant movement of machinery over a working site that is free of surface cover and,
4. wind blowing over a cleared site.

Generally dust can be controlled during traffic movement by water truck sprinkling. At the case study site with the access track was regularly used by small trucks dumping grease trap waste so the responsibility of the track was still managed by the landfill.

*Sediment control and erosion*

The control of sediment are important to consider when undertaking any earthworks. To recognise this the bunds were revegetated to further stabilise them and the overflow drain system previously described, was established.

*Noise control*

Noise producing machinery is needed when aerating the pile, and its use is logically kept to normal working week day hours.

## MONITORING SYSTEMS

The surrounding environment needs to be monitored to ensure that the remediation process is in no way affecting it. At this site we installed four piezometers. One piezometer was located upstream of the pad to act as a background reference point, while a second was situated between the grease trap ponds and the biopad to monitor any movement from the ponds to the pad. Two further piezometers were located in the creek lowland area to monitor any downstream effects. Water was collected monthly from the monitoring points and analysed for total petroleum hydrocarbons.

## ALTERNATIVE BIOREMEDIATION TECHNIQUES

The major limitation with traditional landfarming practices is the availability of space. For the technique to be effective soil at a maximum depth of 400 - 500 mm depth can be treated. This can pose logistical problems and a large surface area is required to treat fairly limited volumes of soil. Another restriction is the tilling or rotovation depth, usually 600 mm maximum. Further the maintenance of moisture content is a challenge given such a thin soil layer which is subject to the effects of wind and sun, and there is still a possibility of the downward migration of contaminated water or nitrogen into the groundwater.

Soil banking systems are an alternative to landfarming, however to go into their engineering, construction and management would warrant a paper in itself. Generally speaking the soil is thoroughly mixed then combined with a specialised inoculum of nutrients, substrate and trace elements followed by the set up of an active aeration, drainage and irrigation system. The soil is piled into a dimension that will depend on available space at the site with the systems mentioned above installed prior to covering the pile in polythene or some similar impervious material. Once again runoff of leachate from the soil pile or bank is used to irrigate the remediating soil (Lapinskas, 1989).

Insitu bioremediation is also typically carried out especially on urban service station sites. In many cases the site is excavated in stages and a part of the site is remediated then reinstated and remediation then moves onto the next stage. The method is typically slower than landfarming techniques but often necessary in cities where a bioremediation pad can not be established due to the lack of available or suitable land.

## CONCLUSION

Biotechnology has proven effective for the remediation of hydrocarbon contaminated soils around the world. I have been involved in the enhanced landfarming aspect of biological treatment systems in Eastern Australia where they are used with significant success in the clean up of hydrocarbon polluted soils. This technique provides a cost effective alternative to incineration or landfilling (Lapinskas, 1989).

Disadvantages of bioremediation include its chemical specific nature where all organic pollutants may not be treated at one site, the procedure may produce toxic compounds through for example the oxidation of PAH's, and nutrients used in the process if not carefully managed may become polluting to groundwater aquifer systems (ie. nitrate and phosphate).

## REFERENCES


Environmental and Earth Sciences, 1995. *Detailing the preparation, operation and monitoring of the bioremediation pads at the..., Eastern Australia.* Environmental and Earth Sciences Report.

Lapinskas, J 1989. Bacterial degradation of hydrocarbon contamination in soil and groundwater, *Chemistry and Industry* (4 Dec 1989): 784-789.

Saville, J 1991. Bioremediation of sites contaminated by organic chemicals - the natural answer, *Synopsium of the Victoria Branch of the Institute of Engineering, Australia.*

# SLOPE MOVEMENTS AROUND THE TUTAMOE PLATEAU (CENTRAL NORTHLAND NEW ZEALAND)

S. J. TYSON
Department of Geology, University of Auckland,
Private bag 92019, Auckland, New Zealand.

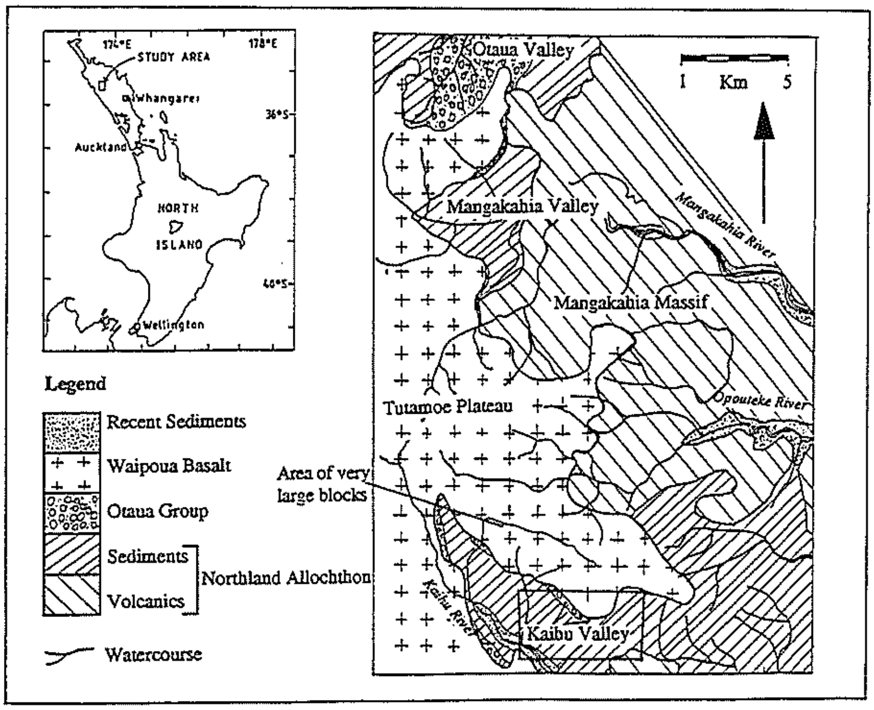
## SUMMARY

The Tutamoe Plateau consists of a prominent upstanding tableland of resistant well jointed basaltic rock and regolith. Underlying sediments are highly to extremely weathered, eroded, and very weak, with subdued topographic expression. Stream superposition and erosion of these sediments by slope movement results in oversteepened slopes which maintain the edge of the Plateau in a state of disequilibrium. Continued and successive undercutting of the Plateau creates large and varied slope movements. Degradation and retreat of the Plateau edge is continuous and ongoing.

## INTRODUCTION

The Tutamoe Plateau, (Fig.1.) consisting of some 500 square kilometres of basaltic rock and regolith extends inland from Northlands west coast for 15-20 Km forming a gently south westerly dipping table land which in places reaches up to 700m asl. Very weak sedimentary rock and sheared tectonic melange underlie the basalt which are practically everywhere subject to slope movement. This undermines the fringes of the basalt cap which is sustained in a barely stable state showing evidence of prolonged slope failure, with aprons and streams of slope debris extending for up to 3 Km from the plateau edge.

Figure 1. Location of the study area showing dominant geological and geomorphological features. See figure 3 for enlargement of inserted boxed area.



Hydrological conditions on the Plateau suggest that groundwater flow patterns are controlled by the basalt acting as an aquifer (with high secondary porosity) and the underlying sediments acting as an aquitard. The resulting groundwater flow is directed along the interface between these two units resulting in seepage at the base of the Plateau cap. This promotes instability with degradation of the underlying sediments and undercutting of the basalt cap.

Debris flows and earthflows along with block slides, rockfalls and small scale translational sliding are common and widespread in the sedimentary and volcanic material commonly found on slope below the Plateau cap. These cause gradual degradation of the undercut basalt Plateau cap which retreats by method of rock fall and rock avalanche.

## STRATIGRAPHY

The regional stratigraphy of the study area consists of one major tectonic and two stratigraphic units. The Northland Allochthon is a tectonically emplaced ophiolite consisting of a sequence of nappes and olistoliths of Cretaceous - early Tertiary sediments and volcanics over 3 Km thick thrust across Northland at the Oligocene-Miocene boundary [1,2]. Within the study area the allochthon consists of two complexes: (1)Mangakahia complex sediments consisting of weak and weathered olistostromes which are represented in the study area by the Punakitere sandstone and Whangai Formation. (2)Tangihua complex lithologies contain Early Cretaceous-Palaeocene sediments and volcanics.

Highly weathered and weak Otaua Group sediments consist of a regressive sequence of bathyal-terrestrial sediments which unconformably overlie the upper surface of the Northland Allochthon and interfinger with the lower surface of the other major stratigraphic unit, the Waipoua subgroup [4].

The Waipoua subgroup is the eroded remnant of a large basalt shield volcano and consists of early Miocene basalt flows and volcanogenic sediments [2].

## ENGINEERING GEOLOGY

Table 1. provides an engineering geological summary of the lithological units together with their topographic expression and propensity for slope movement. Notable features reflected in the table are the relationship between weathering values, strength values and topographic expression. The volcanic lithologies (Waipoua basalt and Tangihua complex) show that a wide range of weathering is reflected in a wide range of strength values. The sedimentary lithologies (Otaua group and Mangakahia complex) are highly weathered and as a result show a narrow range of low strength values. The presence of slightly weathered material in the volcanic lithologies, reflected in the high strength values, forms the resistant and prominent bluffs seen in these units. The low strength due to a high degree of weathering in sedimentary units is reflected in gentle, highly eroded hillsides.

Grain size varies with lithology, which all show a decrease in grain size with increasing weathering. Scanning Electron Microscopy (SEM) and X-Ray Diffraction (XRD) analysis show that associated with this is an increase in clay content and interlayered clay abundance.

Atterberg limit values are related to clay percentage and clay species within soil units. All units with the exception of the Waipoua basalt (residual soil) plotted above the A line within the Inorganic clays of high plasticity zone of the plasticity chart, indicating high deformation potential of remoulded (disturbed) material (Fig. 2)[5]. The Waipoua basalt unusually plotted as an inorganic silt of high compressibility, indicating that the unit has a large compressibility component in its deformation potential, possibly due to a large silt component (as indicated in hand specimen and preliminary SEM work). Atterberg limit values within the Mangakahia complex sediment would seem to indicate greater instability within Whangai Formation material than the Punakitere sandstone, however this was not reflected in the field, each unit proving to be equally prone to slope movements. An explanation of this may prove to be related to a variation in microstructure, of which an analysis is currently ongoing.

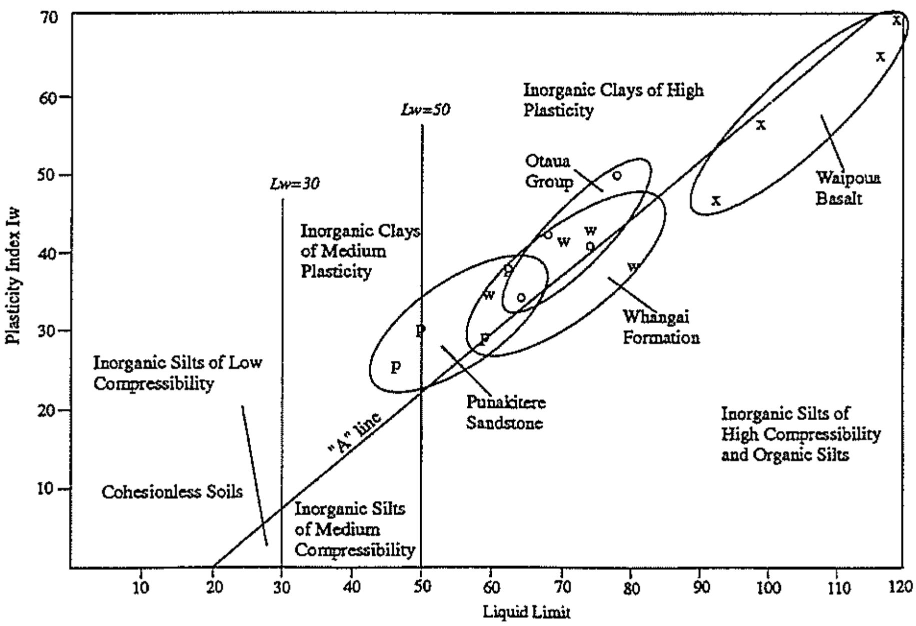
Table 1. Engineering geological summery of the lithological units and their topographic expression.

| Lithology | Weathering range<br>f mw hw ew | Strength range<br>es s ms w ew | Grain size<br>G Sa Si C | Atterberg limits (%)<br>(Residual soil) | Topographic expression and slope stability. |
|---|---|---|---|---|---|
| Waipoua basalt | | | | PI= 58<br>LL= 101<br>PL= 43<br>NMC= 75 | Forms steep 70-80° oversteepened slopes. |
| Otaua Group | | | | PI= 40<br>LL= 72<br>PL= 28<br>NMC= 40 | Forms gentle to moderate sloping hillsides. Unstable on slopes greater than 20-30°. |
| Mangakahia Complex<br>Punakitere Sst<br>Whangai Fm | | | | Pun \| Wha<br>PI= 20 \| 42<br>LL= 50 \| 75<br>PL= 30 \| 33<br>NMC=35 \| 38 | Forms gentle to moderate sloping hillsides. Unstable on even gentle slopes (>15°) |
| Tangihua Complex | | | | | Forms steep to very steep slopes. Rockfalls and soil creep are common. |

**Key to abbreviations:**

| | | | |
|---|---|---|---|
| Weathering terms: f | fresh | Strength terms: es | Extremely strong |
| mw | Moderately weathered | s | Strong |
| hw | Highly weathered | ms | Moderately strong |
| ew | Extremely weathered | w | Weak |
| | | ew | Extremely weak |
| Grain size terms: G | Gravel | Atterberg terms: PI | Plasticity Index |
| Sa | Sand | LL | Liquid limit |
| Si | Silt | PL | Plastic limit |
| C | Clay | NMC | Natural moisture content |

Figure 2. Plasticity chart showing classification of soil units.



## GEOMORPHOLOGY AND SLOPE MOVEMENT PROCESSES

Superficial features such as soil slips, flows, rockfalls, and talus development are all found in the study area. Block slides and rock avalanches form where lateral support of the basalt cap has been removed by erosion resulting in the down slope movement of large coherent masses and smaller displaced blocks of basalt. Figure 3 is an enlargement of a portion of the study area showing geology and geomorphology and an associated cross section to indicate the topographic expression of individual units.

Mass movement by debris flow is the dominant landform process within the study area. this creates a landscape dominated by large lobate, hummocky areas with poorly defined stream courses, scattered swampy patches, and poorly graded soil profiles. Debris flows develop where there is an abundance of material which can be mobilised by the addition of water. This usually occurs after a prolonged period of high intensity rainfall or a series of high intensity storms causing saturated ground conditions. Within the study area hydrological flow conditions suggest that prolonged storm events would result in high seepage rates at the base of the basalt cap resulting in super saturated ground conditions. Catastrophic flow follows upon reductions in viscosity with increased shear rates, down to values associated with hyper-concentrated fluids (ie. at or above the liquid limit). Catastrophic flow is most likely where a trigger is provided by a sudden event such as an earthquake or landslide creating an initial movement [6]. Within the study area two mechanisms of debris flow initiation exist: Away from the Plateau edge rotational or translational sliding will produce sufficient energy (providing the slip is large enough) to remould debris incorporating water beyond the liquid limit, resulting in a highly viscous and mobile flow. On the edge of the Plateau, collapse of the soil/rock mass off the Plateau edge onto the underlying sediments may raise pore water pressures in the lower mass to such an extent that overburden weight is transferred to the fluid leading to liquefaction. This method of "undrained loading" as proposed by Hutchinson and Bhandari [3] will again lead to extremely viscous and mobile flows, some extending for up to 3 Km from the Plateau edge. The Otaua and Mangakahia valleys are dominated by composite debris flow features (multi storied debris flow) which represent the continued and successive down slope movement of material over a period of time.

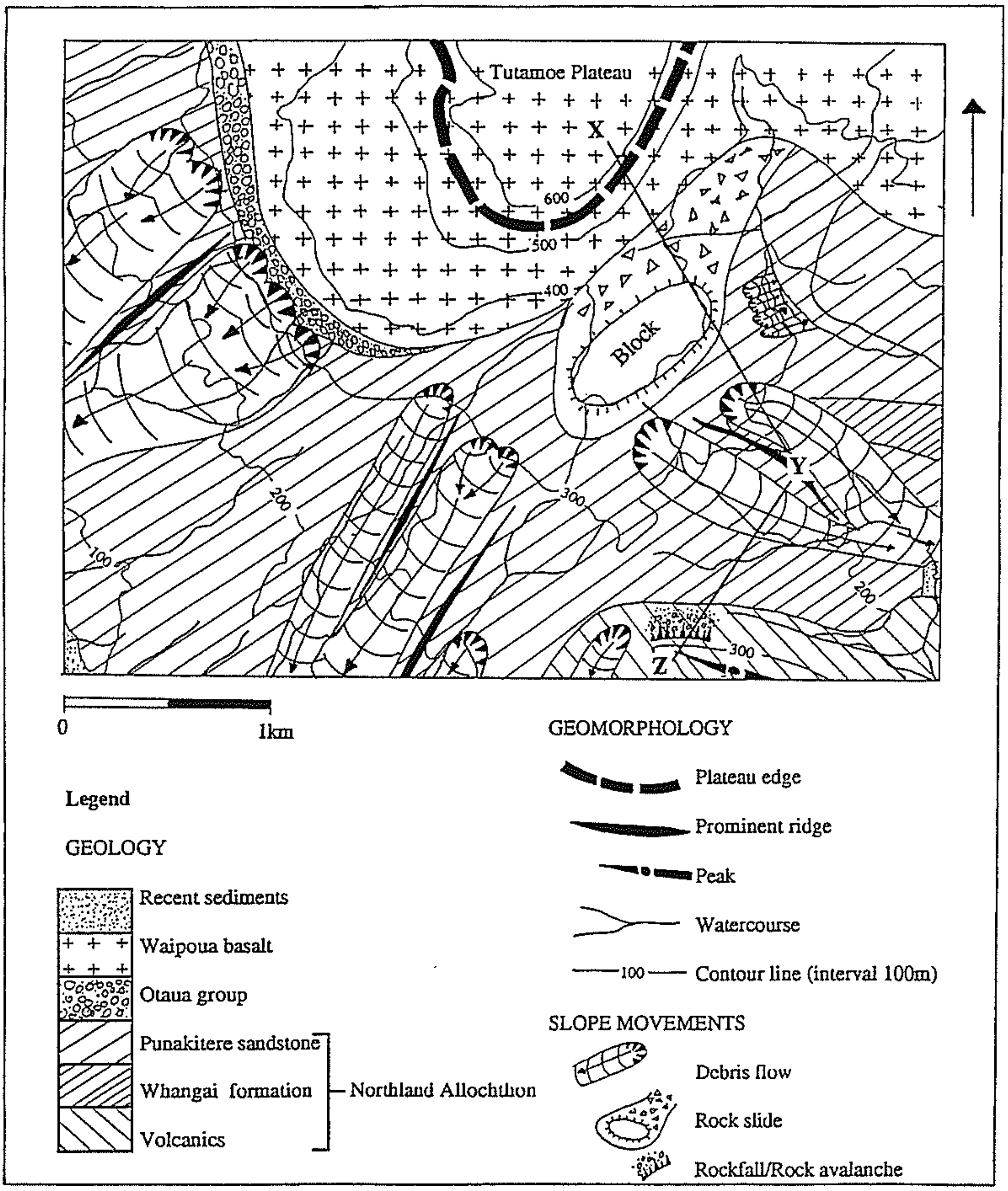
Rockfalls and rock avalanches are found at the base of basalt cliffs. These deposits provide debris for boulder fans which are well developed in major valleys. Rock falls are confined to the failure of individual blocks from the cliffs by joints sliding or toppling in the rock. Rockfalls are common in both the Waipoua subgroup and Tangihua complex where prominent bluffs out crop. Rock avalanches are confined to the Waipoua basalt. This unit exhibits high, very steep bluffs with sub vertical, continuous, and widely to moderately widely spaced columnar joints. Removal of support by erosion of the underlying sediments results in failure of the defects in shear or by outward rotation on a fixed base.

Block slides although rare in the study area are notable due to the large size of individual coherent blocks. Removal of lateral support has allowed individual blocks to break off from the Plateau edge and slide down slope on a basal rupture surface of highly weathered and extremely weak remoulded sediments. The resulting failure is a planar glide as the hard basalt mass is rafted on the remoulded sediments (Fig.3.). Near the head of the Kaihu river, on the western head of the Tutamoe Plateau (Fig.1.) there are several benches at different levels , each up to 1.5 Km long and backed by a steep scarp. These benches are interpreted as very large block slides of basaltic plateau cap on a basal rupture surface of underlying soft sediment.

Smaller rotational and translational movements are common in the study area, developing in highly weathered (HW) to extremely weathered (EW) volcanic and sedimentary units. They often lead to other slope movements such as debris flows or rock avalanches. Smaller rotational and translational movements are not shown in Fig.3 due to their relatively small size.

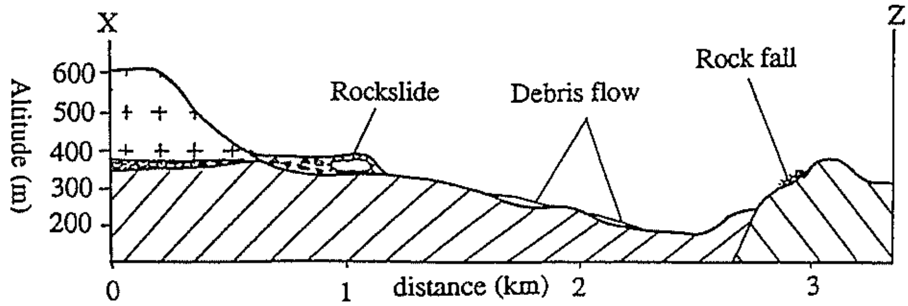
The highly weathered (HW) to extremely weathered (EW) (rock-soil) transition boundary correlates with the border between regolith related slope movements and failures involving fresh rock.

Figure 3. Geology and geomorphology of the Kaihu valley area with an associted cross section of line X-Y-Z. (see Fig.1. for location).




**Cross section along line X-Y-Z (Kaihu valley)**

VE=1.7



## CONCLUSIONS

The Tutamoe Plateau consists of a gently southwesterly dipping table land composed of well jointed basalt rock and regolith. Very weak sedimentary rock and sheared tectonic melange underlie this basalt cap. Continued and successive slope movements in these materials undermine the basalt cap resulting in failure along joints by removal of lateral and underlying support.

Weathering has a significant influence on strength and slope movement. Physical strength, density, and grain size decrease with weathering increasing propensity for slope movement related phenomena. Clay content and the presence and abundance of reactive species (Smectite and interlayer clays) increase with weathering further increasing a slopes propensity for movement.

Debris flows and soil slips (translational and rotational) are basically regolith slides (occurring below the HW-EW rock-soil transition boundary) caused by catastrophic collapse generally as a result of hydraulic overloading.

Rock avalanches, rock falls, and block slides, involving slightly weathered to moderately weathered rock (material above the HW-EW rock-soil boundary) are related to defect orientation and spacing. Defects directly influence the geometry of slope movements by providing failure surfaces. Failure occurs as a result of erosion undermining the rock face, removing support.

## REFERENCES


1 Ballance, P F and Spörli, K B 1979. Northland Allochthon. *Journal of the royal society of New Zealand. 9: 259-275.*

2 Evans, R B 1994. Profile of a piggy back basin; early Miocene Otaua Group and Waipoua Subgroup, western Northland, New Zealand. *New Zealand Journal of Geology and Geophysics. 37: 87-99.*

3 Hutchinson, J N and Bhandari, R K 1971. Undrained loading, a fundamental mechanism of mudflows and other mass movements: *Geotechnique, 21:353-358.*

4 Isaac, M J, Herzer, R H, Brooke, F J, and Hayward, B W 1994. *Cretaceous and Cenozoic sedimentary basins of Northland, New Zealand.* I.G.N.S. Monograph 8. Institute of Geological and Nuclear Sciences. Lower Hutt, New Zealand.

5 Mitchell, J K 1993. *Fundamentals of soil behaviour 2nd Ed.* John Wiley and Sons, Inc. New York.

6 Selby, M J 1993. *Hillslopes materials and processes. 2nd Ed.* Oxford University Press, Oxford.

# DRAINAGE OF THE CAIRNMUIR LANDSLIDE, NEW ZEALAND

C. R. WATTS

Electricity Corporation of New Zealand, Clyde Hydro Group

SUMMARY

The 20 million $m^3$ Cairnmuir Landslide is located on the right bank of Lake Dunstan about 15km upstream of the Clyde Dam. Investigations and remedial works concentrated on the 8 million $m^3$ active segment, a 40-70m thick translational slide composed of schist debris. The groundwater model is a relatively simple stepped watertable below the slide but a highly compartmentalised system of aquifers within the debris. Groundwater below the slide was easily drained by drainage drives and drainholes. However, the thin compartmentalised aquifer system within the debris was difficult to drain. Prior to remedial works the active segment was moving at rates of up to 100 mm/yr. Since the completion of the underground drainage and surface infiltration protection works the slide velocity has reduced to approximately 5 mm/yr.

## 1. INTRODUCTION

Seventeen landslides are located around the 35km long reservoir of the Clyde Dam in the Central Otago region of New Zealand. Remedial works were completed on 9 landslides including the Cairnmuir Slide as part of the Clyde Power Project development [1]. The Cairnmuir Slide is not directly affected by the lake as the slide base is approximately 60m above the reservoir. However, an Active Segment of the slide has sufficient volume (8 million $m^3$) to block the reservoir and rapid movement could cause a wave that would overtop the dam. This hazard necessitated stabilisation measures which would isolate the segment from the effects of lake filling and limit its demonstrated sensitivity to rainfall.

The Cairnmuir Slide is north facing and receives a low average annual rainfall of 400mm. Generally the climate on the slide is extreme, warm in summer and cold in winter. Consequently before remedial works the vegetation was sparse with up to 50% of the surface made up of bare ground or rock. A high rabbit population ensured vegetation was kept to a minimum.

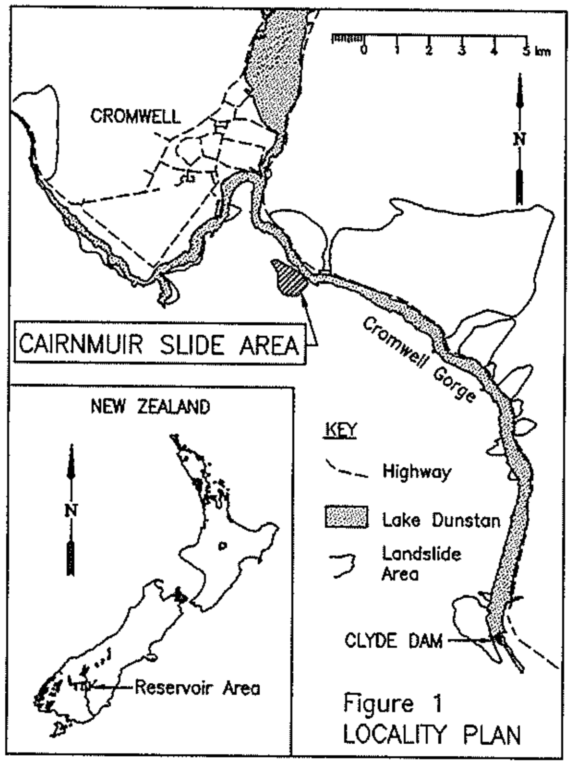



Figure 1
LOCALITY PLAN

## 2. GEOLOGY AND GROUNDWATER

The Active Segment, a 40-70m thick translational slide composed of schist debris, is 600m in length with an average width of 450m. The segment has an average slope angle of 20° which increases to more than 35° at the toe and head of the slide (Figure 2). The Active Segment occupies a generally deflated basin and fresh scarps (up to 1.5m in height) extend around its perimeter. Frontal Lobes at the toe of the slide are surrounded by a set of arcuate fresh scarps (up to 3m in height). Approximately 400 sinkholes were present over the surface of the Active Segment, generally associated with tension zones and formed in lines along active scarps.

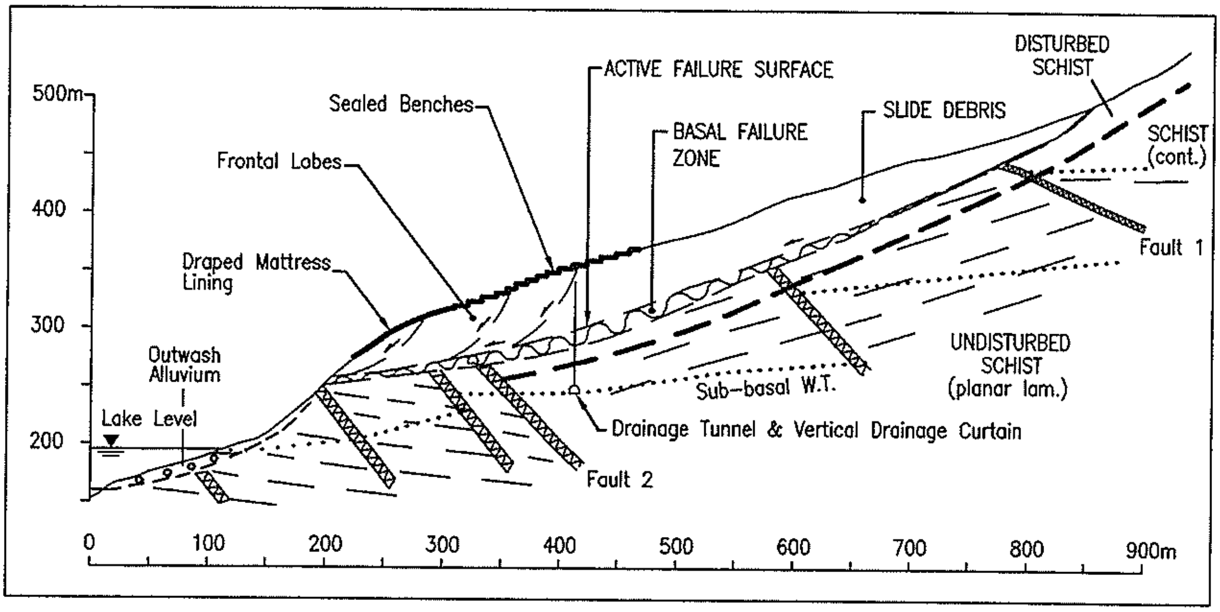



Figure 2. Typical cross section

### 2.1 Undisturbed Schist

Undisturbed schist beneath the slide comprises unweathered, strong, foliated, quartzo-feldspathic mica schist (greyschist). The predominant tectonic defects within bedrock are a series of faults which dip into the slope at 20-40°. These features generally consist of crushed, sheared and shattered schist with occasional gouge seams. The faults are up to 26m thick and traceable across the full width of the slide area.

Foliation below the majority of the slide dips out of the slope. However, downslope of Fault 2 foliation dips into the slope. At the head of the slide Fault 1 separates two lithological distinctive schist types; "planar laminated" greyschist below Fault 1 and "contorted" greyschist above.

The faults which dip into the slope act as low permeability groundwater barriers and divide the sub-basal watertable into large groundwater compartments. Head differences of up to 50m occur across the faults. Before drainage the sub-basal watertable was confined beneath the basal failure zone (BFZ) over a limited area upslope of Fault 2 and it was inferred that these uplift pressures could rise with or following lake fill.

### 2.2 Disturbed Schist

A layer of disturbed schist just below the BFZ extends from the head of the slide down to Fault 2. The material is similar to undisturbed schist but is slightly weathered and the rock mass is generally more open. It is inferred that this schist has been disturbed by stress relief processes [4].

The disturbed schist has a higher permeability than the underlying undisturbed schist. It is also likely that the faults dipping into the slope which compartmentalise the sub-basal aquifers may be disrupted in the disturbed schist zone resulting in increased leakage between the sub-basal compartments. Therefore, the disturbed schist provides a flow path for infiltration from above the headscarp to migrate beneath the slide. Monitoring has detected transient responses to significant rainfall events within the disturbed schist down to approximately the mid-slope area of the slide.

During significant rain events it was possible for groundwater pressures within the disturbed schist aquifer to pressurise parts of the BFZ and AFS for short periods. Therefore, drains drilled from the sub-basal drives were installed within this unit.

## 2.2 Basal Failure Zone

The BFZ above the disturbed and undisturbed schist is a 1-17m thick crushed and sheared zone with occasional gouge seams, shattered zones and blocks of relatively intact schist. Groundwater within the BFZ is trapped in shattered schist clasts which form local, discontinuous aquifers. The degree of interconnection between these blocks is limited or non existent. Consequently this zone was difficult to drain but did not recharge after drainage. Typically when a shattered clast was hit in the drainage drives flows of approx. 5 *l*/min were evident but would dry up within a week.

## 2.3 Active Failure Surface

The active failure surface (AFS) is located at the top of the BFZ and is generally a 100-300mm thick zone of moderately to highly plastic gouge. The gouge is slightly weathered, sandy silty clay with minor gravel (CL/CH) and contains thin discrete slickenslided surfaces (c. 0.5mm thick). Slickenslides were generally orientated downslope, correlating with overall slide movement.

Contorted schist occurs above the gouge zone and planar schist is present below it. This marker horizon proved useful for identifying the AFS and the positioning of instrumentation. During tunnelling an attempt was made to excavate the drive along the failure surface and the lithological change was used to steer the drive. The AFS and the BFZ are effectively impermeable and act as a perching horizon for infiltrating groundwater.

## 2.4 Slide Debris

The bulk of the slide debris in the Active Segment is made up of "contorted" schist, derived from above Fault 1. This implies that the original slide volume has been completely evacuated downslope, a displacement of at least 600m [2]. Slide debris generally consists of slightly weathered, schist blocks in a sand/gravel matrix with some silt. Schist blocks vary from cobble size up to 20m in length and also vary from completely shattered to competent. Joints and fractures are common in these blocks and are usually open 1-100mm. In some locations these fractures are infilled with debris or loess. Commonly the loess was cross bedded indicating water deposition or reworking.

Gouge and crushed zones define Frontal Lobe failure surfaces. Randomly orientated gouge and crushed zones were also logged within the debris. These defects, combined with the AFS, compartmentalise the debris and make the permeability of this material highly variable. Scarps and sinkholes allow the ingress of water into the debris. Shattered boulders and tension zones provide flow paths for infiltration from the surface to the AFS. Before drainage the perched watertable was generally 2-4m thick, but up to 19m in some localised areas.

During the early stages of drainage drilling the perched watertable was modelled by contouring the head of water above the AFS both before and after drainage. The differences between these surfaces was then used to estimate the effective drainage of the perched watertable. This methodology was discontinued when the extent of compartmentalisation within the debris was realised. A piezometer within the debris reflects a groundwater level of a compartment that is typically less than 15m in width and can not be reliably used to infer the depth of perched water between piezometers.

# 3. MOVEMENT HISTORY

Outwash terrace remnants deposited during the Hawea-Mt Iron Advances (16 000-23 000 years BP) outcrop below the toe of the slide. By calculating the amount of debris on this outwash surface it is inferred that the toe of the slide has moved some 16-40m in the last 16 000 years (1-2.5 mm/year). Boulder displacement plots from comparison of aerial photos from 1949 and 1990 indicate that the main body of the slide has moved downslope 1-2m (50 mm/year) and the Frontal Lobes 4m (100 mm/year) in the last 41 years. This indicates a significant increase in the slide velocity in at least the last 40-50 years. Survey pillars located on the upper slide mass confirm the higher velocity with deformation rates of 30-35 mm/year between 1984 and 1989 (pre remedial works).

It is postulated that the increase in the velocity of the Cairnmuir Slide is related to the removal of vegetation. Approximately 150 years ago the vegetation on the slide was adversely affected by the introduction of grazing from sheep and rabbits. During the 1940's a further increase in the rabbit population occurred in Central Otago resulting in reduced vegetation cover and increased soil erosion. This general denuding of the slide is believed to have increased the infiltration into the Active Segment elevating the perched watertable and resulting in greater deformation over the last 40-50 years.

Since detailed monitoring began in 1989 six movement events triggered by rainfall have been detected. Generally these rainfall events have been relatively small with an average recurrence interval (ARI) of 1-2 years. Total observed movements during individual episodes have been up to 100mm in the toe and up to 10mm in the head of the slide, with rates of up to 3 and 0.5 mm/day respectively.

## 4. REMEDIAL MEASURES

The sensitivity to rainfall indicates that the general stability of the Active Segment is controlled by the perched watertable. Movement appears to be a retrogressive type of failure with the toe of the slide moving before and to a greater extent than the upper slide mass. This mechanism is consistent with the boulder displacement plots which show the Frontal Lobes moving twice the distance of the head of the slide over the period 1949-1990. Therefore the majority of the remedial works concentrated on the Frontal Lobes of the Active Segment. Sub-surface drainage was installed to drain the perched and sub-basal aquifers and isolate the slide from the effects of lake filling. Surface protection works were also implemented to control infiltration. Buttressing of the slide was not considered practical because of the height of the slide base above the valley floor (90m).

### 4.1 Drainage Drives.

A total of 1.2km of drainage drive has been installed within and below the Active Segment. The tunnels are nominally 3.5m wide and horseshoe shaped. Approximately 800m of the tunnels were excavated within the undisturbed schist to provide both sub-basal drainage and access for perched drainage drilling. The sub-basal drives were excavated using a conventional drill, blast and muck cycle with hand held air-leg drills and ST6 Wagner muckers. Excavation rates of up to 50m per week were achieved in this competent material and no significant stability problems were encountered. Rockbolts and crown shotcrete were used to line the majority of the sub-basal tunnels. Limited use of the NATM was made when excavating through the sub-basal faults.

Approximately 400m of tunnel was excavated within the BFZ and slide debris. These drives, constructed for both investigation and drainage purposes, were excavated along the AFS to form a drainage cut off and intercept all the water perched upon the failure surface. The AFS had more relief than had been previously inferred and was only able to be followed over approximately 60% of the length of the tunnels. However, generally this method of drainage was successful, with seepage and minor flows (<5*l*/min) from above the AFS and other crushed zones within the debris intercepted over the majority of the tunnel length.

Conventional drill, blast and muck cycle was intended to be used for excavating the BFZ and slide debris material. However, when excavating through the BFZ a possible correlation between blasting and slide acceleration was detected. Blasting was limited to 0.5kg per delay for the popping of isolated boulders. The remainder of the tunnelling within the slide debris was completed using the wagners and air picks to excavate the face. Despite this restriction excavation rates of approximately 25m/week were achieved.

Steel sets at 1m spacing and heavy to medium timber lagging with the walls prelined 1.5-2.0m high were used to support the drive during excavation. Full reinforced concrete lining was used as permanent support. A standard pattern of weep holes which included a line of holes parallel to the failure surface, was drilled through the lining to intercept any seepage. A gravel side drain was also installed outside the lining adjacent to the country to maximise tunnel drainage.

Tunnel stability problems were encountered within the debris where a water bearing shattered/sheared unit was intercepted in the crown. Flows of 10-20 *l*/min produced some running ground and the crown became unstable developing a 5m chimney. A bulkhead was constructed and the drive terminated 2.5m short of its target.

### 4.2 Drainage Drilling

A total of 320 drains with a combined length of 22km have been installed within and below the Active Segment. Drainage of the sub-basal compartments was easily achieved by 12 drainholes which drained the confined groundwater and accomplished widespread lowering of piezometric pressures. However, the debris was difficult to drain due to the low head conditions and its highly compartmentalised nature. Drainage responses indicate that the compartments are less than 25m in width and typically 5-10m. Therefore, many drainholes at close spacing were required to obtain effective drainage.

Two types of drilling have been used at Cairnmuir to drain the perched aquifers; "target" drilling implemented to drain specific targets and a "drainage curtain" installed from the sub-basal drives across the full width of the Active Segment just behind the Frontal Lobes (Figure 3). The curtain, consisting of 200 drains drilled at 5m centers and extending 15-30m through the AFS, was installed to intercept groundwater migrating downslope from the upper slide mass before it could influence the more active Frontal Lobes. Most of the drainholes produced only small flows of less than 1 $l$/min, but a limited number flowed initially at 50-150 $l$/min. All high initial flows reduced rapidly (2-14 days) also indicating that the compartments are relatively small.

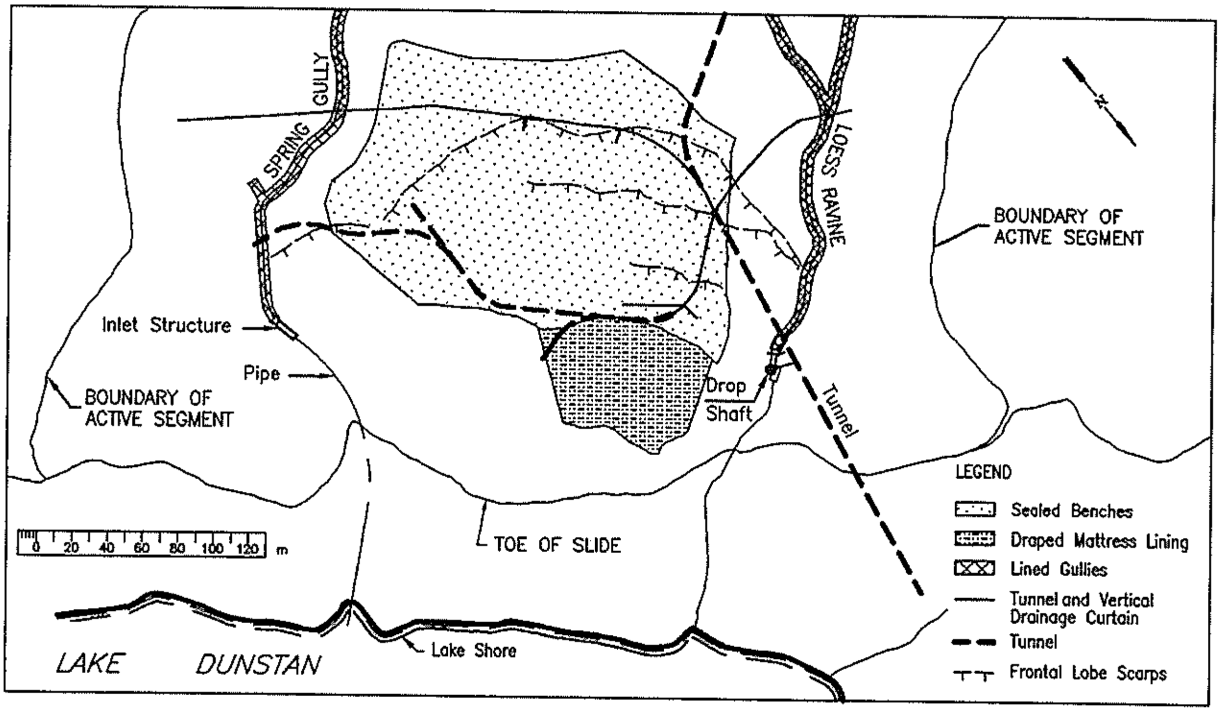



Figure 3. Plan of stabilisation measures

Drains are up to 170m long and were drilled using down the hole hammer and limited Odex type drilling. Generally the holes were drilled to total depth, the hammer and drill string extracted and a slotted 100mm diameter steel casing installed with a 50mm diameter slotted plastic liner pipe. This "open hole" method of drilling proved to be successful even in relatively poor drilling conditions within the debris. A 24 hour drilling operation enabled drilling and screen installation before the hole deteriorated and became unstable. Where collapsing or open ground were encountered the hole was typically advanced using a tricone.

During the drainage drilling cutting samples were collected and logged to determine the change from "planar laminated" to "contorted" schist and hence the location of the AFS. Logging of the cuttings proved to be a quick method of establishing if significant drillstring deviation had occurred and could help determine if the drain had hit or missed its target.

### 4.3. Surface Works

The surface works are not considered in any detail as a full description of these is presented by Gillon and Saul [3]. Surface infiltration protection works include:

- Lining of two gullies within the Active Segment (1250m) to ensure that all water collected is transported off the slide.
- 15 reinforced earth retaining walls with sealed benches between (2.8ha) which collect water from the Frontal Lobe area and divert it to the lined gullies.
- A lower sealed slope (0.6ha) which collects water from the very steep toe of the slide and diverts it to the lined gullies.
- Vee drains located in the upper slide area which also collect run off and divert it to the lined gullies.

• Revegetation, oversowing and fertilising of the slide area. A rabbit-proof fence has been constructed around the perimeter of the slide and a rabbit control programme implemented.

## 5. SLIDE PERFORMANCE

Monitoring of the Active Segment before the remedial works indicated the upper slide area was moving at about 30 mm/year. In the toe several movement events were monitored with velocities up to 3 mm/day recorded for a short duration. Current monitoring (September 1995) suggests the upper slide is stable or creeping slowly (1-2 mm/year) and the toe of the slide is moving at approximately 5 mm/year.

From December 1993 to February 1994 the slide experienced a long period of extremely wet weather (by local standards) with 226mm of rain in 60 days. This event with an ARI of approximately 150 years occurred when the drainage works were nearly complete and during the early stages of the surface infiltration protection works. Deformation responses to the rainfall were detected, but were less than had been observed in previous movement events and were relatively limited considering the size of the rainfall event.
Only one significant rainfall (ARI of 2 yr) has occurred since the completion of the remedial works. No increase in the velocity of the slide was identified and only limited piezometric response was recorded, entirely from outside the area of the Sealed Benches. The lack of deformation and minor piezometric response to this rain event, and the relatively limited movement during the December 1993 event combined with the continued slowing trend of the slide, suggest that the stabilisation measures have been successful.

## 6. CONCLUSIONS

The Cairnmuir Slide is perched 60m above Lake Dunstan and is not directly affected by the lake. However, an Active Segment of the slide has sufficient volume to block the reservoir and rapid movement could form a wave that would overtop the dam. This hazard, combined with the sensitivity of the Active Segment to rainfall, necessitated intensive stabilisation involving drainage and surface infiltration protection works.

The Active Segment is believed to have moved more than 600m. The extent of movement is the largest of any of the Cromwell Gorge Landslides and has also produced crushed and gouge zones throughout the slide debris resulting in a highly compartmentalised internal aquifer system. Geological evidence indicates that the Active Segment has been creeping at an average rate of 1-2.5 mm/year since the last glacial advance. However, it is inferred from boulder displacement plots that the slide velocity has increased over the last 40-50 years to rates of up to 100 mm/year. This increase in movement is believed to be due to de-vegetation and increased infiltration.

Drainage of the sub-basal aquifers was easily achieved with the use of drainage drives and a limited number of drainholes. However, the highly compartmentalised nature of the slide debris made drainage of this unit difficult and many drains at close spacing were required. A total of 1.2km of tunnel and 320 drainholes totalling 22km of drilling were installed.

Limited deformation response to a large rainfall event (ARI 150 years) occurred during the later stages of the drainage works and during the early stages of the surface works. A small rain event (ARI 2 years) after the completion of the works caused no increase in the slide velocity. These results, combined with the continuing slowing trend of the slide, suggest that the remedial measures have been successful.

## REFERENCES


1. Brown R., Gillon M.D. & Deere D.U. 1993. Landslide Stabilisation at Clyde Power Project, New Zealand, *ASCE Conference on Geotechnical Practice in DAM REHABILITATION, Raleigh, North Carolina, April* 1993, PP 299-319.
2. Gillon M.D., Riley P.B., Halliday G.S., and Lilley P.B. 1992. Movement history and infiltration, Cairnmuir landslide NZ : Proc. *Sixth Int. Symposium on Landslides, Christchurch New Zealand* (Vol. I) pp 103-110.
3. Gillon M.D., Saul G.J. 1995. "Cairnmuir Landslide Infiltration Protection Stabilisation Works", *Proc. IPENZ Anuual Conference, Palmerston North, New Zealand: Vol 1 pp 235-239.*
4. Macfarlane D. F., Riddolls B. W., Crampton N. A., and Foley M. R. 1992. Engineering geology of schist landslides, Cromwell, NZ: *Sixth International Symposium on Landslides, Christchurch, NZ.*