NZ Geomechanics News

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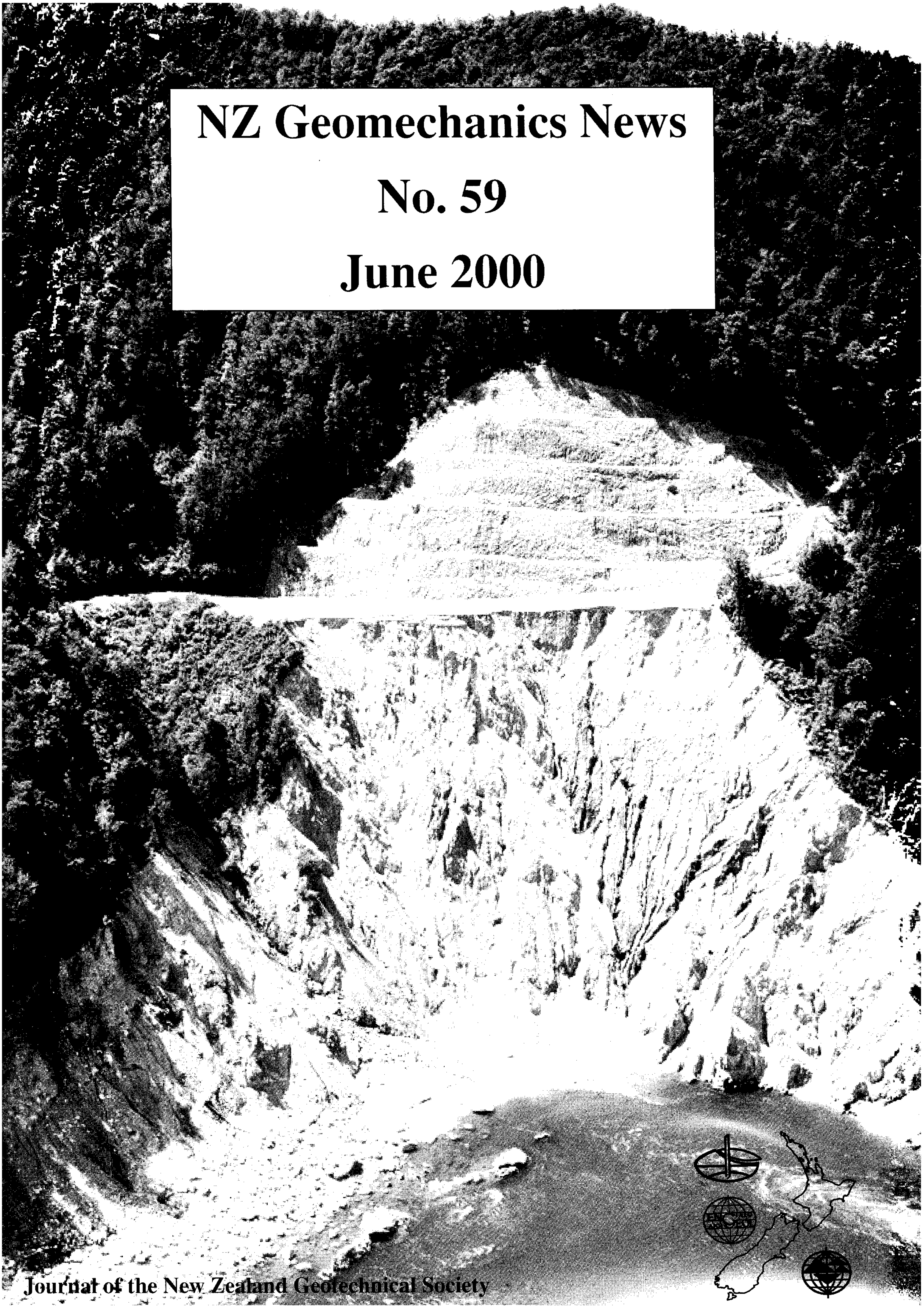

Journal of the New Zealand Geotechnical Society

New Zealand Geomechanics News

Number 59 June 2000

Contents

Chairman's Corner

This is the first edition of Geomechanics News under its new format. Grant Murray as Editor has put in considerable effort to bring about some significant changes to improve the quality of the magazine, and is to be congratulated for this. We would appreciate any feed back on the new style and its contents.

MANAGEMENT COMMITTEE

I would like to record my thanks to Geoff Farquhar, our immediate past Chairman, and the members of the out going committee. Geoff has been largely responsible for the progress made to date with incorporation of the Society. There is still a little way to go with resolving a few outstanding issues, and Geoff is still actively assisting the new management committee with these aspects. Under Geoff and the previous management committee, the Geotechnical Society has been left in very good heart, both financially and from the viewpoint of services to members.

The new management committee commenced work in February this year. In addition to myself as Chairman, Debbie Fellows continues as Secretary, Grant Murray is Editor of Geomechanics News, Ian McPherson has the role of treasurer, and John Berrill is assisting with local branch activities.

SERVICES TO MEMBERS

A questionnaire has recently been sent out with details of the services the Geotechnical Society provides to its members. I would like to urge all members to spend a few minutes to review the current list of services, and to forward any comments on ways in which these can be improved or new services provided commensurate with the current subscription level.

LOCAL BRANCH MEETINGS

Local branch meetings are one of the core activities of the Society, and I am very keen to ensure that local branches are active and provide our members with a good range of topical speakers. Local branches are established in Auckland (local co-ordinator Chris Bauld), Waikato/Bay of Plenty (Paul Burton in Tauranga, Mark Mitchell in Hamilton), Wellington (Ian McPherson), Christchurch (John Berrill), and Dunedin (John Henderson). If any member of the Society is keen to give a talk, or is aware of any eminent person who would be appropriate as a speaker, this information can be conveyed to any of the local branch co-ordinators or to our secretary Debbie Fellows.

CONFERENCES

The highlight of this year's conference calendar will be GeoEng 2000 to be held in Melbourne between 19 – 24 November. A number of our members will be active either as Issues Lecturers (Mick Pender; John Berrill & Trevor Matuschka), Chairmen of paper presentations (Bruce Riddolls) and as presenters of papers.

The date and title of the NZ Geotechnical Society's next symposium was recently announced. The theme of the symposium is "Engineering in hazardous terrain", to be held in August 2001 in Christchurch. Further details can be obtained via the Secretary or the Chairman of the Symposium organising committee at k.mcmanus@civil.canterbury.ac.nz (Tel 03 351 6808).

Guy Grocott

Editorial

It was my first time. Was it good for you?

I am writing this column very early on in the production process, whilst I am still very much in the honeymoon period. It is therefore easy for me to say that I am more than pleased to have taken over the role of the NZ Geomechanics News Editor. I am hoping that you will have immediately noticed some differences in the style of presentation and formatting of the GN. Any appreciative comments will be well received and may even be published. Any detrimental letters of castigation will end up in the editor's rubbish bin.

There are of course many people involved in the production of GN and my special thanks go to Debbie Fellows, Doug Johnson, Jaime Bevin, Jon Sickling and Allanah Anderson. Their efforts are to be applauded.

Apart from the formatting you will notice there have also been some subtle changes to the content. Regular features including the Chairman's Comment, Letters to the Editor, Branch Activities and Meeting Reports will obviously continue and new features such as Laurie's Brain Teaser, the Book Review's and Photo' Competition will be become regular items. We are a professional society and therefore technical papers will always be the backbone of GN but we shall also be showcasing more general interest items in the Project News section. Contributions for this section over the coming months will be welcomed. They need not necessarily be glamorous flagship projects, if they are interesting, they will be considered.

GN is pleased to introduce as regular contributors some expert columnists in specialist areas of the geo-professions including, mining & quarrying, geo-environmental, landfill engineering, earthquake engineering, analytical modelling and groundwater. It is hoped that these features will generate some interest and comment. Bob Wallace in particular promises to be very provocative.

I believe that the journal of any professional society is always a reflection of the membership it represents. Our membership is growing and I have detected a groundswell of opinion that believes we can only enhance our professional status by continuing to provide a better, more rigorous service to our clients. If this means improving what we do and how we do it, then it is my objective for the GN to set the pace.

Grant Murray

Editorial Policy

NZ Geomechanics News is a journal issued to members of the NZ Geotechnical Society. It is designed to keep members in touch with recent developments within the Geo-Professions both locally and internationally.

Persons interested in applying for **Membership of the Society** are invited to complete the application form at the back of the journal. Members of the Society are required to affiliate to at least one International Society and the rates are included with the membership information details at the back.

The editor's team is happy to receive submissions of any sort for future editions of *NZ Geomechanics News*. The following comments are offered to assist potential contributors:

Technical contributions can include any of the following:

- technical papers which may, but need not necessarily be, of a standard which would be required by the international journals and conferences
- technical notes
- comments on papers published in *NZ Geomechanics News*
- descriptions of geotechnical projects of special interest.

General articles for publication may include:

- letters to the NZ Geotechnical Society
- letters to the Editor
- articles and news of personalities
- news of current projects.

Submission of text material in camera-ready format is not necessary. However, typed copy is encouraged particularly via e-mail to the editor or on floppy disk. Diagrams and tables should be of a size and quality appropriate for direct reproduction. Photographs should be good contrast black and white gloss prints and of a suitable size for mounting to magazine format. *NZ Geomechanics News* is a journal for Society members and papers are not necessarily refereed. Authors and other contributors must be responsible for the integrity of their material and for permission to publish.

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ASSISTANT EDITORS

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Advertising

NZ Geomechanics News is published at least twice a year and distributed to the Society's 400 members throughout New Zealand.

This magazine is issued to society members who comprise professional geotechnical and civil engineers and engineering geologists from a wide range of consulting, contracting and university organisations as well as those involved in laboratory and instrumentation services.

Advertisement Location	Single Issue	Advert. Size (mm)
Black & White Back Cover	$275	210 wide x 297 high
Inside Cover (Front or Back)	$225	210 wide x 297 high
Full Page Internal	$200	185 wide x 265 high
Half Page Internal	$150	90 wide x 265 high
Quarter Page Internal	$125	90 wide by 130 high
Colour Full Page Internal	$350	210 wide x 297 high
Inserts Insert to be posted with magazine $100/flyer Max size single A4 page by 1 Nov 2000 Special price given on request for other types and sizes.		
***Note**: 1. All rates are excluding GST 2. Space is subject to availability 3. 3mm bleed 4. Advertiser provides all flyers		

The deadline for advertising copy for the next issue is 1 November 2000. Arranging artwork for your advertisement can be carried out at a reasonable additional cost if requested. However, advance notice is required for these additional services.

If you are interested in advertising in the next issue of ***NZ Geomechanics News*** please contact:

Management Secretary
Debbie Fellows
6 Sylvan Valley Ave
Titirangi
AUCKLAND

Tel: 0-9-817 7759
Fax: 0-9-817 7035
Email: dfellows@xtra.co.nz

Letters to the Editor

Discussion on Expansive Clays, Translating AS2870 into NZS3604

By Bruce Grayson

Dear Sir

This is an interesting paper and is no doubt a practical approach to a difficult problem. There are however a few technical over-simplifications on the geological side, as follows:

- The expansion of clay minerals, especially smectites, are affected by salinity and pH as well as overall moisture content, which is why lime stabilization works. The pH is not usually too variable during construction, but ionic substitution can be important.
- Soils in New Zealand, especially those developed on rhyolitic pyroclastics, can contain a large amount of halloysite and allophane, which are prone to large volume changes, more so than typical basaltic soils. The extent to which these are relevant depends on what we define as the "Auckland Area", since they are more important closer to the Central Volcanic Region and in localised occurrences in Northland. However, they do occur in the Auckland urban area as well. These are materials that would not be commonly found in Australia.
- Diatomite is another material that does occur in Auckland in infilled craters that were the site of temporary lakes. This has a number of amusing engineering properties due to its fine grain size and high surface area. These include swelling, shrinking and in some cases being thixotropy. I understand that these difficulties were encountered a few years ago on the Onehunga motor way approaches!
- There are ways of determining clay mineralogy other than XRD and SEM (though engineers should not be scared of XRD, it is something we routinely carry out in-house for a very modest cost). The most usual is the methylene blue test for smectites, which are often the culprit in swelling soils. The test is semi-quantitative so we have used it in aggregate specifications in Fiji.

Regards

Jim Lawless
Earth Science Manager, Sinclair Knight Merz

Dear Sir

Based on my initial read of this paper I suggest the following:

1) The author needs to spend at least one year here and maybe re-consider the content of his paper. Why? Because the water table variation (by which I mean saturation line) varies in most areas by over four metres and often up to six metres. This variation completely dominates the shrink-swell behaviour of local materials.

2) If one is to look at the influence properly, then one needs to consider also the geologic formation and the strength of the soil. I had a case where, due to an unfortunate set of circumstances, one corner of the subject building ratcheted down (settled) due to shrink-swell behaviour.

3) One is often more concerned about differential settlement, rather than total settlement. This point seems to be missed. I do not know the meteorological conditions in Victoria, but I suspect that they are drier. In which case a waffle slab would cause significant differential movement due to moisture content variation. This does not happen here to the same extent due to the moisture variation described in item 1. Most cases of shrink-swell related damage in Auckland that I am aware of are due to a local feature, for example a tree that is causing a relative change in moisture conditions.

Regards

Sergei Terzaghi
Senior Geotechnical Engineer, Sinclair Knight Merz

Dear Sir

The introduction of proprietary raft slab systems constructed either directly on the ground surface or at shallow depth has led increased interest in shrinkage swell movements and how they may be quantified or at least allowed for in a rational foundation design. At a recent Geotechnical Society Meeting in Auckland AS 2870 (The Standard) was proffered as a possible way forward. While use of this document could be used by designers I do not favour nor would I recommend the means of determining the surface movement set out within the standard.

AS 2870 is based heavily on Australian experience and uses very broad categories to allow a range of design parameters to be selected and used by structural engineers designing foundations. The bulk of the standard sets out design rules for a variety of footing sizes and depths in table form for a wide range of design conditions. Of some benefit is that the standard requires site specific investigation of each building platform developed in accordance with the standard, albeit with a minimum of 1 borehole to 1.5m or a predetermined depth ranging from 1.5 to 4m. The standard includes a note but not a requirement that soil types and site conditions should be inspected at footing excavation stage by the classifier to confirm the soil profile. The classifier may however be a builder with local knowledge although the standard tends to promote the use of Geotechnical Engineers and Engineering Geologists in this role. There are also some useful suggestions regarding advice to home owners on the actions they can take to minimise the adverse effects of high shrinkage swell soils.

The standard notes however that for a site with controlled fill of a material other than sand and deeper than 0.4m the site shall be classified as Class P. This classification covers:

"Sites which include soft soils, such as soft clay or silt or loose sands; landslip; mine subsidence; collapsing soils; soils subject to erosion; reactive sites subject to abnormal moisture conditions or sites which cannot be classified otherwise".

The designer must then justify by reference to "engineering principles" that a less severe classification is appropriate although AS 2870 Supplement 1 1996 states

"Generally clay fill should be avoided unless great care is taken".

This runs contrary to New Zealand practice. Typically, in Auckland at least, on completion of a controlled fill operation, a Foundation Completion Report with a clear statement by the Geotechnical Engineer, responsible for the overview of the quality assurance of the fill, is issued to his client and the Local Authority. This generally includes recommendations as to the constraints on developing over such fill. Often this statement will refer to development of residential structures in accordance with NZS 3604 with additional comments as to depths of foundations where shrinkage swell is of concern. Such statements may include broad recommendations on foundation depth but no direct reference to the parameters necessary for the specific design required for raft slab designs.

AS 2870 sets out to ensure foundation systems designed in accordance with the standard have only a 5% chance that loads will be exceeded. For the affected property owners AS 2870 includes within its Appendices the following statement:

" under adverse conditions Category 2 [Damage Category – ie cracks less than 5mm] should be expected although such damage should be rare".

Adverse conditions are not defined however I presume this is intended to cover ground movement greater than the range assumed in the design and not the effects of poor workmanship. The CSIRO Guide to Home Owners on Foundation Maintenance and Footing Performance states with reference to designs in accordance with AS2870:

"Larger cracks, up to 5mm, may occur in some houses with properly designed and constructed footings, if reactive clay sites have been subject to large changes of moisture. Cracks larger than 5mm are regarded as significant damage".

While not terribly reassuring for the lay person sensible advice on "Care of Clay Foundations", and "Performance of Footing Systems" is included in the CSIRO document as well as in the standard and Supplement 1. The effect of trees and the low cost remedy of watering the ground adjacent to foundations are discussed. The inevitability of shrinkage cracks in concrete slabs is also highlighted. The need for change in moisture content for ground surface movement to occur is clearly recognised in these documents however in my opinion the methods used in AS 2870 are essentially empirical.

The approach within AS 2870 to the assessment of site specific shrinkage swell movements appears to be predicated by the assumption that measurement of soil suction profiles is expensive and unreliable. Measurement of soil suction profiles should however allow site specific suction envelopes to be developed and differential shrinkage and swell to be predicted. Of the various laboratory methods for measuring soil suction, such as the Thermocouple Psychrometer, the Porous Block, Thermal Conductivity or Heat Dissipation Sensor, Suction Plate, Pressure Plate and the Calibrated Filter Paper Method the latter is the simplest, least expensive and applicable to a wide range of suction measurements (Wray 1997, Swarbrick 199?). This method also has the ability to measure both total and matric suction and hence allows the solute suction to be calculated.

In my opinion solute suction in New Zealand soils will be found to be significantly different from that of Australian soils. Such a difference would potentially explain the significant difference in the shrinkage swell movements observed in the two countries. The climate as well as the geological origin and mineralogy of the soils will influence the salts within the soil giving rise to solute suction. The arid conditions within Australia can reasonably be expected to give rise to a greater concentration of salts within the soil than typically present in New Zealand soils. For example, large areas of Australia are affected by high salinity, a condition largely unknown in New Zealand.

Undertaken as a straightforward test in accordance with ASTM D5298, Filter Paper Suction (FPS) tests should allow appropriately equipped laboratories to determine site specific soil suction profiles (Wray 1997, Swarbrick 1995). Routinely moisture content profiles are determined in site investigations, the calibrated Filter Paper Suction test uses similar equipment and can be undertaken using samples recovered from auger holes as part of the same investigation. Where specific concerns exist regarding the response of a structure to changes in moisture content, a suction profile could be determined using FPS testing to allow surface changes to be calculated. This approach should automatically take into account the presence of fill and its clay content and not require the use of figures drawn from AS 2870. What will however also need to be undertaken is the determination or assessment of the suction compressibility index either using the clay percentage and clay mineralogy (McKeen 1977, Bratton 1991) or using the PI, % of soil passing the # 200 sieve and the Cation Exchange Capacity. The latter could be estimated using the PI (Mojekwu 1979).

In my opinion there is currently no basis for the direct use of values presented in Table 2.4 "Recommended Soil Suction Change Profiles" in New Zealand. It is acknowledged however that seasonal fluctuations in moisture content and suction at any given site cannot be determined on the basis of a single set of samples. Wray (1997) reports however that use of soil suction theory to determine soil suction envelopes has over a nine year research study period allowed monthly estimates to be made accurate to within ± 6mm of the surface movement actually measured.

AS 2870 requires that one of various methods set out in AS 1289 be used to determine a Shrinkage Swell index. This property is determined either by measurement of swell within a consolidometer or in a loaded shrinkage cell or by measuring shrinkage of a core of soil. Both of the last two methods also require the measurement of soil suction of part of the sample. The Building Services Authority case study commissioned in 1996 (in response to growing concerns regarding damage to dwellings designed and constructed using AS 2870) strongly recommended against the use of the Core Shrinkage Test and that Atterberg Limit Testing not be accepted as a means of assessing site classification. Of interest is that the AS 1289 test methods are all individually more expensive than the determination of a suction profile and will provide only a single data point and not a means of assessing the volume of soil that can be potentially affected by moisture content change and the resulting surface movement.

While the FPS test and suction theory offer a rational way for designers to assess the potential movement that structures on reactive soils may be subject to alternative design techniques are also available. Traditionally New Zealand has tended to use timber structures set above ground level. Smarter designs and systems using timber or other flexible materials and systems may be a better approach than trying to make structures stiffer and stronger. Unless some basic research is undertaken however both options may result in overly conservative and expensive solutions being promoted. The current trend promoted by NZS 3604 for shallower house foundations has however the potential to lead to the same increase in claims against Engineers, Local Authorities and contractors that led to the commissioning of the 1997 BSA report.

Malcolm Stapleton
Geotechnical Engineer
Principal
Babbage Consultants Ltd

References

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2. Swarbrick GE, Measurement of soil suction using the filter paper method. Proc. 1st Int Conf on Unsaturated Soils, Paris 1995.

3. Wray WK, The principle of soil suction and its geotechnical engineering applications. Proc. 5th Int Conf on Expansive Soils, Adelaide 1984.

4. CSIRO Sheet No 10-91 Rev November 1988, Guide to home owners on foundation maintenance and footing performance. (Updated for AS2870 - 1988).

5. AS 2870 - 1996, Residential slabs and footings - Construction.

6. AS 2870 Supplement 1 - 1996, Residential slabs and footings - Construction - Commentary.

7. Wray WK, Using soil suction to estimate differential soil shrink or heave. ASCE Special Geotech. Pub No 68, pp66-87.

8. Geo-Eng Australia Pty Ltd, Subsidence of residential buildings, a case study. Building Services Authority - Queensland 1997

Report from the Secretary

Society membership is currently flourishing with a total of 434 members.

New Members

It is a pleasure to welcome the following new members into the Society since the last issue of Geomechanics News:-

Michael Knocker	Nigel Edger	T Crow
Trevor Banks	N Watkins	John Higginbottom
Brent Hawthorn	Rodney Hutchison	Greg Cook
Jeffrey Swanney	Trevor Hill	Jaime Anderson
Bruce Grayson		

Resignations

R Ni, S D Scott, C Wills, and P Arnold have tendered their resignations from the Society.

Subscriptions

A number of members are still outstanding subscriptions for this financial year. Prompt payment of your outstanding subscription would be appreciated.

Remember the new society rule if you owe 1-year's subscription you will be removed from the membership.

PLEASE PAY UP!!

Society Incorporation

To let you know the status of the Societies Incorporation. The third draft of the society rules were sent to a lawyer for legal advice to ensure that the content of the rules was acceptable and meet the requirements of the Incorporated Society's Act. The feedback we received was favourable with some suggestions for improvement. Geoff Farquhar will be including these comments in the rules and we are expecting to send the rules out to a membership vote by mid June.

Debbie Fellows
Management Secretary

International Society Reports

ISRM

INTRODUCTION

This report covers ISRM business for the period November 1999 to April 2000. The majority of the Board's activity has been by email. The next Board meeting will be held in Melbourne at GeoEng2000 on 18 November, 2000. A combined Board meeting with ISSMGE and IAEG will also be held on this day. The next Council meeting will be held in Melbourne on 19th November.

NEW BOARD

President :	Prof. Marc Panet
Secretary-General :	Jose' Delgado Rodrigues
Vice - Presidents :	
Africa	Dr Riza Guner Gurtunca
Asia	Prof Chung In-Lee
Australasia	Prof Chris Haberfield
Europe:	Prof Pekka Sarkka
North America	Prof Alfredo Sanchez Gomez
South America	Prof Euripeded do Amaral Vargas Junior

COMMISSIONS

The following ISRM Commissions have been retained (Chairman in brackets) :

Commission on Swelling Rocks (F. Madsen)
Commission on Preservation of Natural Stone Monuments (C. Tanimoto)
Commission on Application of Geophysics to Rock Engineering (K. Sassa)
Commission on Education (M. Kwasniewski)

At this time I have not received any information regarding possible new Commissions.

INTEREST GROUPS

Invitations have been made to develop the proposal for the formation of the following Interest Groups :

Interest Group on Mining (N. van der Merwe)
Interest Group on Fragmentation by Blasting (W. Fourney)

Please contact me if there is any interest in forming an Interest Group.

ROCHA MEDAL

No nominations were received. There will therefore be no Australasian nomination forwarded to the Board.

ISRM NEWS JOURNAL

I would greatly appreciate receiving articles for the ISRM News Journal.

Associate Professor Chris Haberfield
Vice-President for Australasia
International Society for Rock Mechanics

9th International Congress on Rock Mechanics
25-28 August, 1999 Paris, France

The 9th International Congress on Rock Mechanics had the theme of '20th Century Lessons and 21st Challenges'. The congress was attended by about 550, and the technical programme was divided into four topics:

1) Rock engineering – environmental safety and control
2) Coupling between mechanical, thermal, hydraulic and chemical phenomena
3) Rock dynamics and tectonophysics
4) 'In situ' tests, measurements, monitoring.

This programme was interspersed with an array of keynote lectures, case history presentations, the Muller Lecture, plus several workshops. The timetable was basically run as a single plenary session, though the workshops were generally in parallel sessions.

Topics were handled in the traditional congress manner – an invited general reporter giving a state-of-the art overview, followed by a summary of the submitted papers by the co-reporter and short presentations of selected papers by authors. As in the past the success of sessions related closely to the breadth of effort put in by the general reporters, and the selection of author presentations. The keynote lectures covered a diverse array of topics from field measurements in tunnelling (Sakurai, Japan), to rock mechanics in petroleum engineering (Roegiers, USA), and a history of mining rock mechanics (Hood, Australia). The Muller lecture 'Puzzles in Rock' by Prof H Einstein (USA, and former ISRM President) dealt with modelling jointed rock masses. Workshop themes covered a diverse range of topics (squeezing rocks, usefulness of rock mechanics tests, movement of rock masses, mechanical effects of underground nuclear tests).

Posters are extensively used at scientific conferences, and for the first time, I think, were offered at the congress. Unfortunately less than half the authors took the opportunity, and as the latter sessions were much more attentively attended than the first sessions, I hope the approach continues. Several prominent companies (MTS, Coyne & Bellier) and publishing companies (Elsevier) occupied technical exhibition booths. The two post-congress field trips to western or southern France with their cultural and technical mix were not heavily subscribed.

Theme 1 (113 papers) - General Reporters N Barton (Norway) & N Van Der Merwe (South Africa)

While the debate between rock mass classification systems (e.g. RMR, Q, GSI) continues the general reporter agreed that input data should not be presented to decimal point accuracy, and that such classifications are a valid approach when applied by experienced practitioners who can visualise the consequences of their choices. Another problem is how to approach the problem of cohesion in Mohr Coulomb and which computational method to use in analyses (e.g. jointed as a continuum against discontinuous).

The use of empirical methods was well shown in several of the selected papers with simple slope v height charts with RMR classifications in Australia (Duran & Douglas) and application of Barton indices (J_A and J_R) as a best or worst case in Ghana (Goel & Wezenberg). Many other papers were vastly more analytical with display of some intense 3-D computing such as used for modelling the enhancement oil recovery in the USA (Dershowitz *et al*).

Theme 2 (92 papers) – General Reporters E de Tourney (USA) & J Van Sint Jan (Chile)

Poro-elasticity modelling and the important contribution of Biot was introduced by the general reporter before delving into the realm of hydraulic modelling in discontinuous media. A challenge is to perform laboratory testing that is representative of field behaviour, particularly with respect to dilation, anisotropy, and time effects. Many of the selected papers reflected either a pre-occupation with either simple tests such as point load strength (Romana, Spain) or intact tensile strength (Risnes *et al*, Norway), larger scale direct shear laboratory testing (Pearce & Haberfield, Australia) or acoustic emission (Carlsson *et al*, Sweden).

Theme 3 (44 papers) – General Reporters A McGarr (USA) & M Dubinski (Poland)

Because of the importance of underground safety and economic extraction of ore the general reporter concentrated on mining induced seismicity, in situ stress and rock excavation by either cutting or blasting. This aspect of tectonics and stress field impact plays a lesser role in New Zealand, and no further comment is made.

Theme 4 (43 papers) – General Reporters O Stephansson (Sweden) & M Steiner (Switzerland)

The theme reflected trends for the strong use of technology and the application of computer technology to the characterisation of rock masses and vision in tunnelling. GPS, seismic energy tomography, fractals, digital image analysis, 3-D lasers, artificial neural networks, impact echoes, and borehole slotters abounded. These were countered by some simpler approaches, such as a pin test for determining weak rock strength, together with some interesting case history monitoring of large excavated slopes (Yoshinaka *et al*, Japan), comprehensive assessments of grouting performance (Kikuchi *et al*, Japan).

Some of the workshops had very lively discussions. John Hudson successfully advocated contrary views on the value of laboratory testing – consensus agreed on relevance being the most critical factor whether for simple index tests for calibration against case histories or sophisticated expensive tests. In Europe movement mechanisms for large scale slopes and landslides is gaining a lot of attention – including mapping, monitoring and modelling inputs.

The keynote lectures gave timely historical industry overviews with particular points for the future – e.g Hood examining cuttability as excavation rates continue to accelerate. The case histories generally showed pragmatic solutions – e.g. Kovari for squeezing ground in deep alpine tunnels. The 'Puzzles in rock' lecture, while recognising persistence as a key rock mass feature, showed that modern computer modelling can always show an impressive looking solution. Overall rock mechanics continues to provide a mix of empirical approaches backed by experienced judgement and sophisticated numerically-backed assessments as computing power continues to grow. The balance will always be somewhere between, as long as linkages are developed between practical observation, sophisticated modelling, and real behaviour.

The four yearly congress by its very nature covers a broad range of activities and research, and therefore provides a good opportunity to catch up on a variety of subjects. The 9th congress was no exception for this generalist and sole kiwi attendee, one who cannot stop being astounded by the much larger amounts of money available in research facilities overseas compared with New Zealand.

Stuart Read
Institute of Geological & Nuclear Sciences Ltd (GNS), Lower Hutt

IAEG Activities

ESTABLISHMENT OF REGIONAL GROUP

The Australasia IAEG Group is gradually becoming a reality, within the umbrellas of the NZGS and AGS. There is now a Steering Committee of 9 representing both countries (in NZ, Bruce Riddolls, Guy Grocott, Don Macfarlane, & Warwick Prebble) and through email communication, key items identified to date for action are:

- establishment of database of members
- education
- technical activities
- professional registration.

Anyone wishing to join the Steering Committee will be most welcome.

Arrangements have been made with IPENZ to make use of its existing NZ Geotechnical Society database, and so the NZ arm of the Australasia IAEG Group is essentially now up and running. Any member wishing to air an issue, contact: Australasian IAEG Group, c/o bwr@rgl.co.nz.

The first meeting of the group will take place in Melbourne at 5.30 pm, Tuesday 21st November, during GeoEng 2000 conference.

EXECUTIVE AND COUNCIL

The 2000 Executive and Council Meetings of IAEG are scheduled to coincide with the International Geological Congress in August (www.31igc.com), and the Executive will also meet at GeoEng 2000 in Melbourne in November. Please let me know if you have any issues you would like raised at these meetings.

NEWSLETTER & BULLETIN CONTRIBUTIONS

New Zealand engineering geology has been under-represented in both of these publications, which provide a good opportunity for members to make a difference to their profession world-wide. Please think about planning some time to contribute.

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ISSMGE Activities

A Board meeting of ISSMGE was held in London in March 2000. The following main issues were addressed:

- The accounts for 1999 indicated a smaller deficit than anticipated in the budget, partly due to delays in some items of expenditure. The plan is still to run at an operating loss over the next 3 years, gradually reducing the cash reserves of the ISSMGE. A 12.5 % increase in membership fees is anticipated for the year 2002, the last increase having been in 1991. There are also moves to revitalise corporate sponsorship of the ISSMGE, to help fund initiatives.
- Initial plans were discussed to introduce an International Geotechnical Services Directory linked to the ISSMGE web-site, with companies paying a fee to advertise on the Directory. This scheme would work in conjunction with the corporate sponsorship of ISSMGE.
- There was a move to increase the frequency and detail of reports from Technical Committees (TCs). A motion was carried at the Council meeting for the Chair of each TC to submit a report, in a form to be suggested by the Secretary-General, for inclusion on the ISSMGE web-site. The report would be updated at least annually. A current list of Technical Committees is attached.
- The new web-site http://www.issmge.org is being actively maintained and is receiving an exponentially growing number of visitors. Although the web-site will eventually replace the printed newsletter, the latter is being maintained throughout 2000.
- Progress on key forthcoming international conferences was presented, including GeoEng 2000 (November 2000, Melbourne), XVth ICSMGE (August 2001, Istanbul) and 4th ICEG (2002, Rio de Janeiro). The next (XVIth) ICMSGE will be hosted by Japan, and be held in Osaka in 2005.
- National delegates are invited for the first International Young Geotechnical Engineers Conference to be held in Southampton in September, 2000. Note that delegates from Australia and New Zealand have been chosen from the very successful YGPC held in Perth in February, 2000.
- The Swedish Geotechnical Institute (SGI) has decided to offer free subscription to its database, merely charging for any reprints that are ordered. The web-address to register as a user is http://www.swedgeo.se

The next Board meeting will be in Melbourne in November 2000, followed by an informal Council meeting.

Mark Randolph
Vice-President (Australasia), ISSMGE
Centre for Offshore Foundation Systems
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Fourth ANZ Young Geotechnical Professionals Conference
Perth, Western Australia, 16-19 February 2000

The Fourth Young Australia-New Zealand Geotechnical Professionals Conference was held at the picturesque St George's College on the grounds of the University of Western Australia in February earlier this year. It was attended by 40 delegates from prominent consulting and contracting companies as well as research institutions from throughout the Australasian Region, and included 7 delegates from New Zealand.

The conference began with registration and welcome drinks on the Wednesday night and was followed by members of the Organising Committee leading a party of delegates to a local restaurant and then on to "Steves", the local student Pub. The atmosphere from the start was very friendly and it was interesting to hear other people's backgrounds and the work they were involved in. For those of us who were at the previous conference in Melbourne it was also a great chance to catch up.

The conference itself kicked off on the Thursday morning and continued through until Saturday afternoon with each delegate allocated 15 minutes for their presentation (including 5 minutes for questions and discussion). In addition, three keynote lectures were given by prominent senior Australian Geotechnical Professionals. The presentations throughout were of a very high standard and covered a wide and diverse range of topics with projects stretching as far afield as Papua New Guinea and the United Kingdom.

On the Friday afternoon a site visit was held in and around the Fremantle Harbour area, taking in the historical landmarks of "Freo" and some of the interesting and challenging geotechnical issues that have been faced in the development of the harbour. We were also introduced to what can only be described as a very "fresh" Fremantle Doctor. The conference organisers were also kind enough to treat some of the delegates to a tour of the legendary Centrifuge Facility at the Centre for Offshore Research at the University of Western Australia.

A full calendar of social functions was also organised and included a BBQ next to the lovely shores of a badly algal affected Swan River, conference dinner and farewell drinks. The conference dinner was held at a restaurant adjacent to Cottesloe Beach, enabling the diners to view a spectacular sunset over the Indian Ocean as well as entertainment from windsurfers making full use of the waves and the Fremantle Doctor. Following the dinner a tour of the cities nite spots was undertaken, with several of the senior participants seen on various niteclub dance floors in the early hours of Saturday morning!

On the Saturday afternoon before the conference closed, a workshop was held on "Professional Development in Geotechnical Engineering". Despite most of the participants still suffering from the previous night's conference dinner, some lively and intense debate was sparked by comments regarding the value of a PhD to a successful career in the geotechnical world. The main outcomes of the workshop were:

- Top executives tend to have postgraduate qualifications of a masters degree or PhD, however at a lower level, postgraduate qualifications may inhibit a job applicant.
- Postgraduate studies are sometimes too specialised and may not satisfy all of the problems that a consultant may encounter. However, postgraduate research does instill a more general understanding that covers a scope far beyond the esoteric research topic.
- Employers however do value postgraduate qualifications due to a lack of geotechnical exposure in current undergraduate coursework.
- Short courses are not an effective substitute for a formal postgraduate qualification.
- Research is more beneficial than a Masters/PhD by coursework.
- Company sponsored professional development has a very important role to play.
- A proportion of the delegates in attendance considered a higher degree in geomechanics such as a PhD, as unnecessary for a successful consulting career. However, senior participants argued the worth of a higher qualification in the longer term quality of engineering judgement.

Notably, approximately half of those in attendance at the workshop session did not envisage practicing in the field of geomechanics within the next 15 years (surely after a career in rape and pillage of the landscape they weren't considering repenting their sins for alternative green/vegan/hippy lifestyles?!?!).

The conference also saw the introduction of awards to send delegates to the International Young Geotechnical Engineers Conference to be held in Southampton (United Kingdom) in September later this year. The NZGS/EQC award for attendance to this conference was won by Paul Horrey of

Riddolls and Grocott Limited in Christchurch.

Overall the conference was very enjoyable and worthwhile both in terms of professional development and for the future of the geotechnical profession. Although the traditional NZ/ Aussie rivalry was constant throughout (not helped by hourly updates of the Australian cricket team thrashing NZ, sheep jokes and constant references to Phar Lap being an Australian horse) the conference aims of professional development and fostering, expanding and strengthening the ties between young geotechnical professionals were easily achieved. The organising committee should also be congratulated for what was a superbly organised and very successful conference.

On behalf of the New Zealand delegates, I would like to thank the New Zealand Geotechnical Society and the Earthquake Commission for their financial assistance and support. I would also like to take this opportunity to encourage all companies to fully support future YGP Conferences.

Jaime Bevin
FOUNDATION ENGINEERING

Opening of the Ralph B. Peck Library at NGI

By John Dunnicliff, Geotechnical Instrumentation Consultant, Bovey Tracey, Devon, England

The Ralph B. Peck Library was opened at Norwegian Geotechnical Institute (NGI) in Oslo on 8 May, 2000. The festive occasion was attended by many of Ralph's colleagues and friends, and also by his daughter Nancy and her husband Allen, his son Jim, and Karl Terzaghi's grandson Sergei, who is now practicing as a geotechnical engineer in New Zealand.

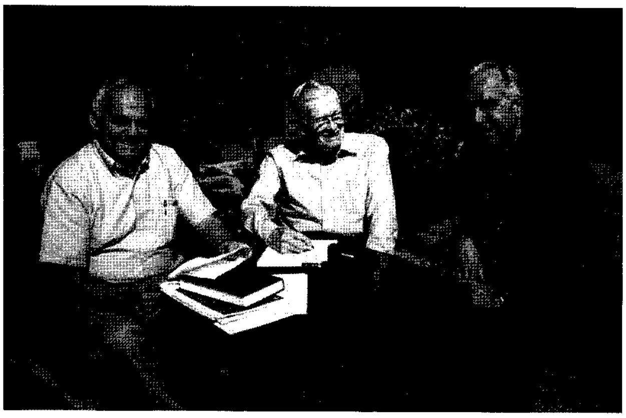
The library contains copies of all Ralph Peck's publications, together with his job files, diaries, his medals and awards, and a fascinating scrapbook compiled by his parents. It is housed in a room adjacent to the Terzaghi library, which was established at NGI in 1967. In her welcome, Suzanne Lacasse, Managing Director of NGI, explained "The Norwegian Connection" - *There is really an interweaving of many reasons [why the library is to be found in Norway], but the origin of Ralph Peck's Norwegian connection is no doubt the magnetism, close friendship and mutual professional respect that existed between him and NGI's first director Laurits Bjerrum.*

The program for the opening included a welcome by Suzanne Lacasse, a lecture by Ralph Peck during which he described his experiences with Karl Terzaghi during construction of the tunnels for the Chicago subway (an exciting presentation during which the importance of making decisions during construction that are based on observations rather than on analysis was emphasized), and a presentation by Elmo DiBiagio. In the 1960s Elmo and Kaare Flaate were both doctoral students of Ralph Peck, and in March this year they spent a week with Ralph at his home in Albuquerque, New Mexico to "interview" him. The result was the presentation, and a publication that was given to each attendee entitled "NGI Publication Number 207, Ralph B. Peck, Engineer, Educator, A Man of Judgment". The publication includes:

- "The Norwegian Connection" - the welcome
- "A Profile of his career - presented in the format of a borehole log
- "In His Own Words" - the result of the interview
- A list of awards and citations
- Reprints of three publications with powerful behavioural messages
- "Words of Wisdom" - a series of pungent quotations
- A list of publications.

NGI Publication Number 207 is available from NGI. Norwegian Geotechnical Institute, P.O. Box 3930, Ullevaal Stadion, N-0806 Oslo, Norway, e-mail ngi@ngi.no. Requests should be marked "for the attention of the library".

All interested are welcome to visit NGI and browse in the library. NGI's website is http://www.ngi.no

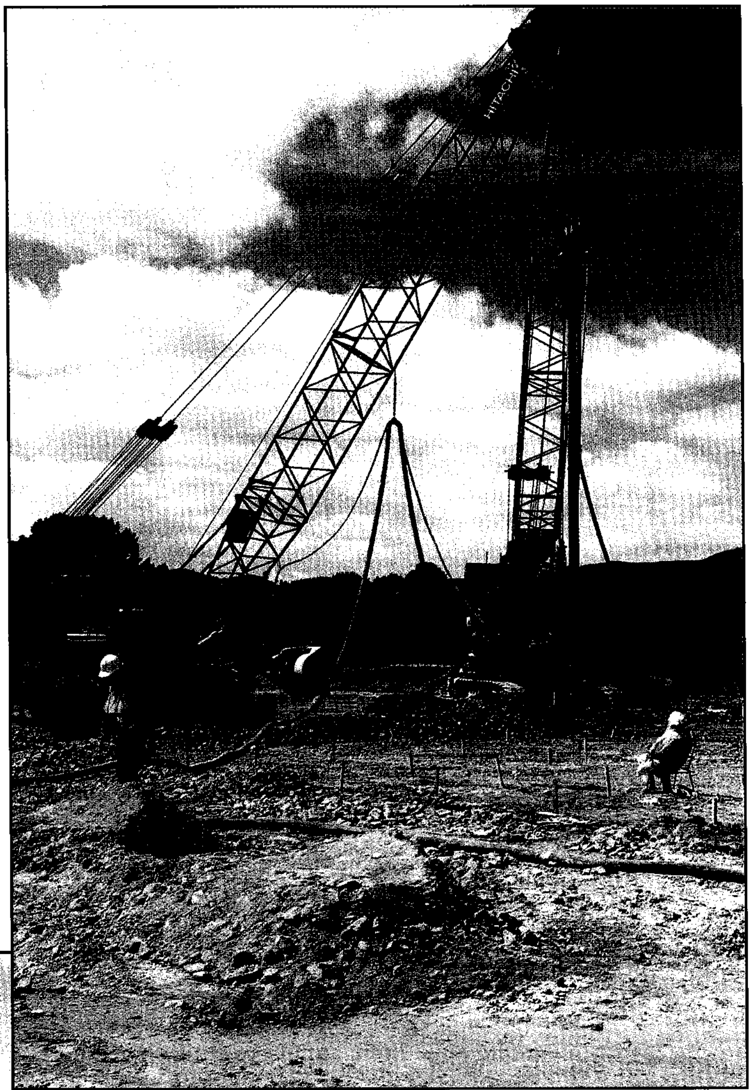


Elmo DiBiagio and Kaare Flaate interviewing Ralph Peck

2000 Photo Competition

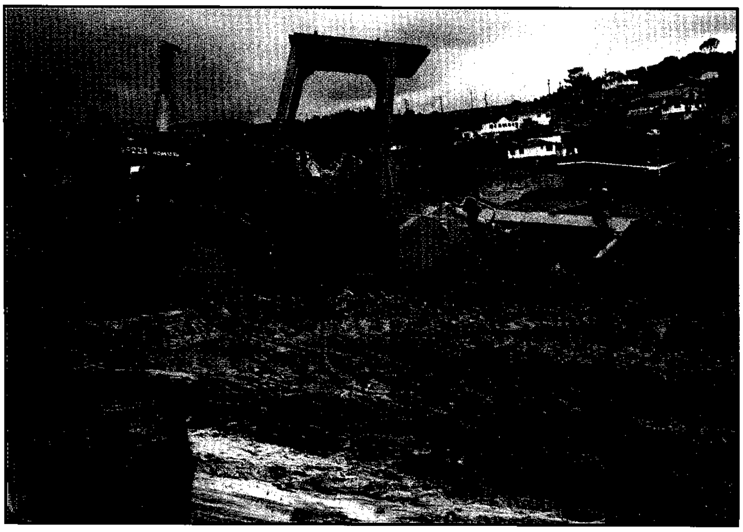
Awarded First Place

Evan Giles comes a close second in the First International Musical Chairs Competition.
(Stone column installation, Mokai Geothermal Power Station, Central North Island)
Entered by Tony Fairclough

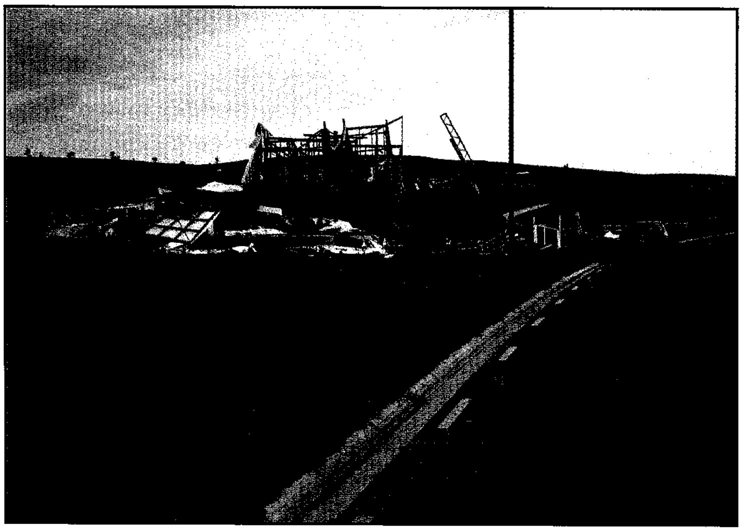


Suddenly Peter Riley wondered if the soles of his shoes had a high enough friction angle to prevent a very nasty accident. (Cosseys Dam overflow outfall inspection)
Entered by Tony Fairclough

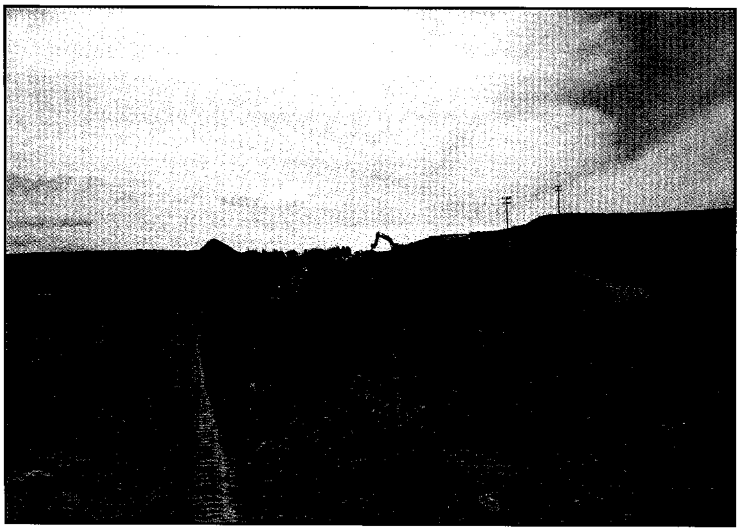
'Client Liaison' Evan Giles at Gillies Ave Rock Cutting
Entered by Debbie Fellows



"Fill slightly wet of optimum?'
Entered by Jamie Bevin, Foundation Engineering



'Geotech Engineer did his job, pity about the Structural Engineer.....'
Entered by Jamie Bevin, Foundation Engineering



'Contractor Humour - ALPURT B1'
Entered by Jamie Bevin, Foundation Engineering

NZGS Branch Activities

Auckland Branch Activity Report

The year has started with two talks to the Society in Auckland. In March we were presented with talks from the Auckland delegates to the Fourth Australia-New Zealand Young Geotechnical Professionals Conference held earlier this year in Perth. This was followed in May by Bruce Grayson discussing Expansive Clays and the potential of translating the expansive clay sections of AS 2870 into NZS 3604.

Young Geotechnical Professionals

The speakers and topics were:

Jaime Bevin	Seismic microzoning of the ground shaking hazard from soil geotechnical properties, Gisborne, New Zealand
Tony Davies	Tirohia Quarry landfill geotechnical investigation
Tony Fairclough	Predictions of de-watering related settlement in Waihi township, New Zealand
Nicola Ridgely	Geotechnical investigations associated with the Axis Fergusson expansion

All four of these presenters gave polished performances for the society. Obviously honed by the previous practice at the conference, we were given four slick graphics intensive presentations. All parties managed to answer a number of searching questions on their topics from the more senior members of the society watching the presentations.

The quality of the presentations and the level of responsibility required for the works described, show the future of the industry is in good hands.

Expansive Clays

Bruce Grayson provided a very enlightening talk about the procedures used in AS 2870 to assess and deal with the problems of expansive clays. He discussed the theory and practice of these methods and provided some case studies of use of these in real projects. He finished with his assessment of the applicability of these methods to NZ conditions. Most importantly was the need for additional research work to calibrate the Australian methods. The details of his talk are given in a paper to be published in the next edition of Geomechanics News.

This talk was extremely well attended by a large number of society members along with a strong representation from Structural Engineers. The turnout reflected the impact of this issue on the industry, especially in recent seasons. An excellent and robust debate followed that highlighted the depth of thought that has recently been required on this matter.

Future Talks

A summary of proposed future talks are included below. This may be considered as preliminary and we welcome further additional talks or presentations of interest to the society.

May: *Planning Strategies*

Ever wondered how regional and district planning is carried out in Auckland and where geotechnical engineers are likely to be carrying out site investigations in the next 10 to 20 years? Two planners have agreed to take us through the process and enlighten us on the factors that are taken into account in the planning process.

Speakers Noel Reardon from Auckland Regional Council
Bruce Harland from Manukau City Council

June: *Major Project*

A talk highlighting a technically interesting project (to be decided)

July: *Debate On Section 36(2)*

This meeting will be an open floor discussion prompted by several speakers. The intention is to provide a forum for the industry view of the recent court rulings and the current usage of this portion of the act by local territorial authorities.

August: *Natural Hazards*

The implication of natural hazards on geotechnical engineering and a discussion of the risks associated with these hazards.

September: *Major Project*

A further talk highlighting a major overseas project (to be decided)

October: *Economist/Lawyer*

A talk by a prominent professional, from outside the industry, to provide an alternative view of potential challenges facing tomorrow's society.

November: *Student Prize Evening*

Presentations from students vying for the annual Northern Region Student Prize.

Others: *Professor G V Rao*

We are currently looking at the possibility of a talk from this leading Indian Geosynthetics expert in June.

If you have a potential topic or wish to present something to the Society in Auckland please do not hesitate to contact me.

Chris Bauld
AUCKLAND BRANCH CO-ORDINATOR
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Wellington Branch Activity Report

The Wellington Branch has had a slow start to the year trying to track down potential speakers. One presentation has been given this year by Kelvin Berryman on "Seismicity Along the Alpine Fault" which received a good turnout of 12 to 15 people.

At this stage five other presenters have been confirmed as follows:

- Russ van Dissen on the Turkish Earthquake (May)
- Bruce Symmans presenting his Young Geotechnical Professionals Conference paper (June)
- Alexi Murashev on Dynamic Compaction at Seaview (July)
- Graham Hancox on Earthquake Induced Landslides (August)
- Greg Saul on the SH73 (Otira Gorge) Candy's Bend roadworks (November)

Also proposed is a presentation on the Taiwan Earthquake.

The date of each speaker will be confirmed by a flyer one to two weeks before the presentation.

Ian McPherson
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mcphersoni@conwag.com (note this is a new address)

Canterbury Branch Activity Report

The Southern Zone Student Prize Competition Presentations will be the first formal activity of the year. They will be held in Room E6 f the School of Engineering, University of Canterbury, at 4:00 p.m. this Friday, May 5th. Two presentations will be made:

Baishen Peng, (candidate for M.Sc. in Engineering Geology):
Laboratory and Field Tests for the Coefficient of Restitution
The coefficient of restitution is an important parameter input in computer simulation of rockfalls. It is usually determined by a field test, which is expensive and risky for most practical situations. Aimed at finding an easy solution to this problem, laboratory tests have been carried out to obtain the coefficient of restitution of different rocks, based on an analysis which has been done of the relationship between the coefficient of restitution and rock properties. Field tests have been carried out to verify the results from laboratory.

Christopher Lyons (candidate for M.E. in Civil Engineering):
Critical Depth in Pile Design.
The theory of a critical depth for deep piled foundations in cohesionless soils is based on the concept that below some 'critical depth' the effect of overburden pressure on shaft and end bearing resistance of a pile reaches a limiting value, failing to increase further with depth. This method of pile design is widely used in many foundation engineering texts, but is it actually a valid approach for pile design? Does this critical depth actually exist and if so how does it affect theoretical pile design?

Other confirmed meetings this year (5:30 p.m., Room E6, School of Engineering):

July 19th
Mr Graham Hancox (IGNS) will present a talk on the West Coast landslide dam that formed in late 1999 south of Hokitika. The resulting debris dam presented a significant flooding hazard.

August 9th
Dr Jim McKean, Lecturer in Engineering Geology at the University of Canterbury, will talk on Digital terrain modeling, particularly with regard to forestry development.

October 4th
Kevin McManus, Rob Davis and John Berrill will present some recent results from the Geomechanics Group of the Department of Civil Engineering, University of Canterbury. Topics should include, the response of piles and shallow foundations to earthquake shaking; recent liquefaction research; experiments into the effect of layer thickness on CPT cone resistance.

Related seminar:
At 4 pm. Tues June 6th, in Room E5 of the School of Engineering, Prof. Rick Sibson of Otago University will give a seminar entitled: "Stress and power dissipation during shallow earthquakes". All are welcome.

Proposed meetings:
We have invited Prof. G. V. Rao of I.I.T. New Delhi, a visitor to Auckland, to give a seminar in Christchurch on geotextiles, about mid-June.

Finally, it is proposed to hold a half-day or (long) evening discussion late in the year on local geotechnical problems such as design practice in Port Hills Loess, and rockfall hazard.

John Berrill
CHRISTCHURCH BRANCH CO-ORDINATOR
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email: berrill@canterbury.ac.nz

Otago Branch Activity Report

Nothing Received

Waikato/BOP Branch Activity Report

The Waikato and Bay of Plenty Branches have held no recent meetings and are looking for people out there who have worked on any interesting projects or are undertaking any fascinating research to volunteer some their time to give a small talk or presentation.

We would also like to hear from any NZGS/Geotechnical/Geological people from other parts of the country who may be passing through Hamilton or Tauranga and would be able to give a short presentation or share some of their experiences. This can easily be organised with a little advance warning.

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Limit State Design Seminar
By Prof M Pender

The New Zealand Geotechnical Society shall be running a seminar on Limit State Design in August. The seminar shall be run at two locations, the Avon Centra Hotel in Christchurch on the 11th and 12th and the University of Auckland Conference Centre on the 18th and 19th.

A preliminary note from the organiser indicates that the seminar will cover the background to ultimate limit state design - namely that slopes, walls and bearing capacity of foundations are really the same problem.

The seminar will cover:-

- Background
- Ultimate state ideas
- Application to shallow foundations
- Retaining walls - gravity
- Deep foundations - vertical and lateral capacity
- Pole retaining walls - cantilever, tie-back & RE?
- Choice of soil parameter

Members will be notified of the costs and will have the opportunity of signing up for the seminar at the venue of their choice in due course.

Project News

ALPURT Sector B1

In 1998 Beca Carter Hollings & Ferner Ltd (Beca) were awarded the contract to undertake detailed design and construction supervision for sector B1 of the Albany to Puhoi Realignment (ALPURT). This section of State Highway 1 extends some 4.5km from the Silverdale interchange to the western edge of Orewa. The investigation and design was undertaken in early to mid 1998 with the majority of the construction taking place in the 1998/1999 earthworks season.

Geotechnical investigations had been performed along the alignment over the previous 15 years. This data was collated together and areas where further information was required were identified and investigations undertaken. These were primarily in the locations of structures, areas of large cuts and difficult "geotechnical" conditions. This investigation comprised machine boreholes, hand augers and test pits along with laboratory consolidation, triaxial, and compaction testing.

The geology of the alignment was variable with Onerahi Chaos Breccia and deep soft alluvial deposits south of the Orewa South River and Waitemata Group materials outcropping north of the river and underlying other materials elsewhere. The Onerahi Chaos comprised both siliceous mudstones and muddy limestone, the former well known for its stability problems even on shallow slopes. The alluvium was soft and wet with organic rich lenses. It was up to 8m deep in the area adjacent to the southern abutment of the Orewa South River Bridge. The Waitemata Group had a variable weathering profile (>20m up near the Link Road exit) and contained sensitive volcanogenic soils.

The project involved moving over one million cubic metres of soil and rock from cuts up to 30m deep and placing it into fills up to 20m high. The designation was tight, based on 3:1 (Horiz:Vert) for the Onerahi Chaos and 2:1 for the other materials. Batter slopes were generally designed for a Factor of Safety of 1.5 and significant subsoil drainage was included.

Generally the batter slopes behaved very well. The exceptions occurred in areas where the soil profile changed suddenly due to faulting (and batters were set at those for the stronger material) and on a 10° to 15° sliding plane within the highly weathered Waitemata Group material. These areas could be satisfactorily remediated by cutting back the slopes to flatter angles.

Following discussions with the client, a large cut within the Onerahi Chaos was designed with a lower factor of safety than the other slopes. This was done to avoid the client having to purchase the extra land beyond the designation. The slopes were designed at 2:1 batters with 6m wide benches to catch the debris from the expected slips. Subsequently, during construction, additional land beyond the designation was able purchased and the slopes were cut back to a more stable slope of between 4:1 and 5:1.

The earthworks were largely completed in a single season, with an average of 4,000 m^3/day and a single day maximum of 12,000m^3. Scrapers were used to move all the materials unless the soils were too wet, in which case it was cut to waste. Blasting was also used to loosen the rock and allow the scrapers to excavate it.

In 1999 Beca were successful in winning the contract to design sector B2 of ALPURT. Investigations have been largely completed to date and the design of this project is well underway, with earthworks proposed to commence this season (2000 / 2001). This final section of the realignment is expected to be open to traffic in 2003. Updates on progress will be provided in future issues.

For further information on this project or any other areas in which Beca are involved please contact Dr D V Toan on (09) 300 9185.

Looking South from the Northern End

Looking North over Orewa South River Bridge

Standards, Law & Industry News

Standards

NZS 4404

NZS 4404:1981 Code of practice for urban land subdivision is currently under review by Standards New Zealand (SNZ). The revised document will be issued in 2001, providing opportunities for public and technical comment over the next year.

With the advent of the Resource Management Act in 1991, subdivision provisions of the Local Government Act 1974 have been repealed. The aim of the review is to include provisions for rural subdivision along with updated procedures for guidance, approval, design and construction. The review will reflect advances in civil engineering, including use of modern materials for water reticulation and drainage, while obsolescent materials (eg. Asbestos cement) will be removed. References to old National Roads Board specifications are updated, along with changes to Acts and Regulations associated with privitisation of services.

Any comments or enquiries about this review should be directed to Shafiq Islam, Standards Development Consultant (04) 498 3954 or shafiq.islam@standards.co.nz

(The above is based on the article by Graeme Drake *Land development and environmental sustainability* in the February/March issue of "Standards" Magazine.)

NZS 3604

REMINDER: NZS 3604:1990 *Code of practice for light timber frame buildings not requiring specific design* loses its status as an acceptable solution on 31 May 2000.

Can anyone explain why NZS 3604:1999 *Timber Framed Buildings* advocates only the Scala Penetrometer as a reliable foundation investigation tool?

Assuming that a large proportion of light timber framed structures constructed each year in New Zealand are located in the North Island on residual clays or soils with a significant clay component, it seems odd that the shear vane coupled with a hand auger is not suggested as an optional investigation technique within the standard. Admittedly the shear vane does not present a "continuous" profile as the Scala does, but surely this is outweighed by the fact that the shear vane readings at least offer undrained strengths that may be designed upon without the use of dubious correlations. Further to the letter by "N J Near" in issue No. 58 of the Geomechanics News, what is the justification of blindly using the Scala?

Is NZS 3604 taken to mean that a territorial authority could reject a site investigation for light timber buildings on the basis that a vane was used instead of the Scala?

Law & Industry News

Further to the discussion raised by Steve Crawford (Tonkin & Taylor) at the March NZGS Auckland Branch meeting on Section 36(2), Geomechanics News presents below an article reproduced from BIA News No. 101 April 2000. The article reflects on the Court of Appeal decision on Section 36 notices with reference to the case Logan v Auckland CC.

Also appended are some summary comments prepared by Helen Rice, a solicitor with Heaney & Co, Auckland, and published in Local Government News. We would encourage feedback and comment on the issues as they might be applicable to slope stability and the geotechnical profession.

Court of Appeal Judgment

The Court of Appeal has ruled in effect that:

- *A section 36(2) condition is to be attached to a building consent for building work on land subject to a listed hazard unless "adequate protection" against the hazard is provided for the land itself, as well as the building and other property.*
- *A structure that is part of a network utility operator's system is not a "building" for the purposes of the Building Act.*

[1] *Logan v Auckland CC* 9 March 2000, Richardson P, Thomas and Tipping JJ, CA 243/99. Copies of the judgment can be purchased from the Registrar of the Court of Appeal, P O Box 1606, Wellington.
[2] See *BIA News* No. 97, November 1999.
[3] See *BIA News* No. 95, September 1999.

Background

The judgment[1] was on an appeal against the High Court decision quashing Determination 99/004[2]. The judgment is complex, and this article does not attempt to do more than highlight some of the salient points. In particular, this article should not be seen as explaining how the law as laid down in the judgment is to be applied in practice. Territorial authorities and others affected should consult their own legal advisers in any particular case.

Section 36

Substantive requirements
Section 36 applies when the land on which the building work is to take place is subject to, or is likely to be subject to, erosion, avulsion, alluvion, falling debris, subsidence, inundation, or slippage.

In this case (Logan v Auckland), the parties agreed that the land was subject to inundation. They also agreed that the building complied completely with the building code. The central question was whether the building consent for the building was to be subject to a condition that an entry shall be made on the certificate of title under section 36(2). Such an entry is generally regarded as a "blot" on the title, and has insurance implications[3]. The entry also protects the territorial authority against legal liability if the building is subsequently damaged by, in this case, inundation.

In Determination 99/004 (and also in Determinations 98/003 and 99/010) the Authority had in effect taken the view that an entry on the title was required only if the territorial authority had granted a waiver or modification of the building code's requirements in respect of inundation. That was incorrect, and the proper interpretation is as set out in the Court of Appeal judgment.
In particular:

- A section 36(2) entry is required as a condition of building consent unless "adequate protection" against the hazard is provided for the land itself, as well as for the building and other property.
- The Court of Appeal did not say what is meant by "the land", so that the proper interpretation appears to be that given by the High Court, which said in effect that it meant "the land intimately connected with the building".

As to what constitutes "adequate provision" to protect the land, the Court of Appeal said:

. . . in determining whether the statutory risk threshold . . . has been reached, and what will be adequate provision to protect the land . . . given, too, that adequate provision for protection does not require the elimination of any possibility in all conceivable circumstances of inundation or other relevant hazards, a territorial authority can be expected to take a common-sense approach. Whether the risk is at the level and frequency to justify the expense and other implications of making adequate provision to protect the land and, if not, to require a warning notice, which is a blot on the title and may have significant insurance implications, will always require a sensible assessment involving considerations of fact and degree.

Procedural requirements

Two procedural requirements are that:

- A building consent can be issued under section 36(2) only if the applicant is the owner of the land in fee simple (other provisions apply in respect of Crown land and Maori land).
- The territorial authority must notify the District Land Registrar to make an entry on the title "forthwith on the issue of" the building consent.

In the *Logan* case, the territorial authority did not notify the Registrar until a significant time after the building consent was issued, by which time the applicant for building consent was no longer the owner of the land. The Court ordered the territorial authority to ensure that the entry was removed.

When a building is not a "building"

Determination 99/004 also decided that the building code applied to a proposed bund (a wall or stopbank) to be constructed alongside a drainage channel so as to increase its capacity. The Court of Appeal was asked:

> *Are pipes, drains, bunds and structures not intended for occupation by people, machinery or chattels, which are owned or operated by a network utility operator and form part of a system for reticulating other property, excluded from the definition of "building" under section 3(1)(a) of the Act?*

The Court ruled that:

> *. . . while the proposed bund may otherwise constitute a building within the opening words of section 3, it is excluded under para (a), being a system operated by a network utility operator.*

It follows that such structures are not required to comply with the building code.

Unfortunately, the Court did not discuss the meaning of "system". Apparently, an office building or the like owned by a network utility operator is not exempt. Such a structure is clearly not part of a system. However, the status of other structures owned or operated by network utility operators is less clear.

The Authority has heard of cases where the owner has not realised that a building consent was subject to a section 36(2) condition, or has not realised what the condition meant. The Authority advises owners who have been granted building consent to check whether it is subject to section 36(2) condition. The check should be made before the consent is issued, which usually means before the owner uplifts it. If the building consent is subject to a section 36(2) condition, the owner should consider whether or not to uplift it and continue with the building project.

General Comments on Section 36
By Helen Rice

The following item has been uplifted from Helen Rice's article in Local Government News and relates specifically to the Court of Appeal decision on Logan v Auckland CC.

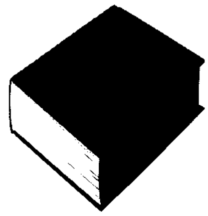
"The Court of Appeal went on to record in some detail its conclusions on the interpretation of s36 and the inter-relationship between subsections (1) and (2). In summary:

> *Where a building is to be constructed or major alterations to a building are to be made, it is not reasonable to issue a building consent unless adequate provision is made to protect the land concerned as well as the building work itself from the listed hazards. Section 36(2) provides flexibility to allow for the issue of a building consent if the set requirements of paragraphs (a), (b) and (c) of sub-section (2) are met with notice to the world then being given through the entry on the title and with consequential exemption from civil liability of the Council under section 36(4). The exemption protects the Council against being charged with issuing a building consent in the knowledge that either the building or the land was or was likely to be subject to damage (or inundation) arising from the listed hazards.*
> *Adequate provision for protection does not require the elimination of any possibility in all conceivable circumstances of inundation or other relevant hazards. A Council can be expected to take a common-sense approach. Whether the risk is at the level and frequency to justify the expense and other implications of making adequate provision to protect the land and, if not, to require a warning notice, which is a blot on the title and may have significant insurance implications, will always require a sensible assessment involving considerations of fact and degree."*

Jon Sickling
Sinclair Knight Merz

Book Reviews

Wanted - Book Reviewers



NZ Geotechnical Society has a number of recently published books available for review. These books have been supplied free to the Society, by the Publishers, for review purposes. We are looking for eager volunteers to review the following books:-

Advances in Aggregates and Armourstone Evaluation. (J P Latham (ed)) 1998 Geological Society of London Special Publication No 13.

Slope Stability Engineering (N Yori, T Yamagami, J Jiang (eds)).A 2 volume proceedings of the International Symposium on Slope Stability Engineering, November 1999.

Stone: Building stone, rock fill and armourstone in construction. (M R Smith (eds)) 1999 Geological Society Engineering Geology Special Publication No 16

The reviews are to be succinct and critical appraisals of the books in the order of 1 or 2 A4 pages in length. Reviews will be forwarded to the publishers. Upon completion of the review the book reviewers can keep the book. (Now there is a good incentive for you!!).

If you are interested please contact:

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Dynamic Soil-Structure Interaction

This book is the output from a Swiss-Chinese Workshop in 1997 on the topic of dynamic soil-structure interaction, and consists of some 18 papers arising from this workshop covering a range of topics from simple analytic models through to advanced numerical modelling. The book is one of the more recent titles in this series from Elsevier and follows much of the same format as other titles in the series.

The first paper is by John Wolf who has been active in this area for some time. He has prepared a number of books on the topic of dynamic soil-structure interaction, with the more recent books providing alternative analytic solutions to classic problems such as a vibrating foundation on a 2-layer half-space. These solutions are based more on a 'cone' of wave propagation (ie 3-d approach), rather than a 'wedge' of wave propagation (ie 2-d approach). The results are compared with answers from a number of other analytic methods. This paper provides a summary of these techniques, and is accompanied by lengthy appendices providing specific solutions.

Many of the other papers present discussions on variations/combinations of various numerical modelling techniques including finite element, boundary element, infinite element, infinite boundary element, finite difference and distinct element applied to specific problems. These papers include derivations of the particular method/element for the problem, discussion of applicability and why the approach was taken and often comparison of results with more standard programs. Many of the studies seem focussed on the integration of infinite elements into either conventional numerical analysis or a hybrid numerical analysis (combination of say finite element and boundary element techniques). This integration appears to be successfully completed for programs that operate in the frequency domain, though the work presented indicates that further research is required for programs that operate in the time domain.

A number of the papers focus on the non-linear analysis of an arch dam in a canyon. This is certainly an interesting problem and a hot topic in China due to the proposed construction of a number of high arch dams in seismically active areas. However, much of the work appears to be esoteric in nature rather than useful, even though the studies are directed towards particular dam sites.

A few of the papers present interesting discussion on problems that crop up from time to time but are always difficult to find information on such as vibrations due to air-blast or the particular problems associated with soil-structure interaction in tall buildings.

The last remaining group of papers are really parametric studies for a variety of problems. Amongst others, this includes a study of the impact of structural vibration control on soil-structure interaction and a theoretical study on wave motion.

A major difficulty in the book is the layout of the papers. It appears that the papers are perhaps ordered in the sequence of presentation rather than topic. It would have been very useful to have the papers ordered by topic for example, Dams, Theoretical Considerations, New Numerical Models, Application of Infinite Element, Application of SSI. It is recognised that some of the papers actually could be listed under several topics. In looking for one paper, I have often got lost largely because of the scattering of the papers. The other major criticism would be that some of the authors could be perhaps more widely read, as some of the information was found many years ago and the papers seem to be going over old ground.

The book represents a useful reference for the advanced/ frequent practitioner or researcher in the dynamic soil-structure interaction field. It is not for a person starting in the field or even the casual practitioner, as much of the information will not be relevant and is certainly not easy to digest, as could be expected from a book that is essentially the proceedings of a speciality workshop. Though, having said that, the first paper is certainly very useful, and at times provocative.

REVIEWED BY :- SERGEI TERZAGHI, SINCLAIR KNIGHT MERZ

Dynamic Soil-Structure Interaction
Developments in Geotechnical Engineering, 83
Edited by:- C. Zhang and J.P. Wolf
Published by:- Elsevier
Date: 1999
Web shopping on: http://www.elsevier.nl/inca/publications/store
Price : $US 169.50

The Geotechnics of Hard Soils - Soft Rocks

In NZ in the 1970's and 80's, uncertainty over the geotechnical characteristics of soft rocks (usually taken to mean the widespread Tertiary sediments) led to research in response to various engineering objectives. Most was in relation to roading, but hydropower and coal resources also figured. Having been involved in that work, I put my hand up to review this title to see what has been done more recently, albeit from an international perspective. (Upon receiving it, it wasn't immediately obvious from the cover that the title is in fact conference proceedings, rather than a text).

The Foreword notes how such materials "are characterized by peculiar features, as bonding, discontinuities, or inhomogeneities that strongly govern their mechanical behaviour (make) them different from both soils and rocks", and how "the geotechnical engineer that deals with such materials has a hard life since there are not widely accepted investigation techniques, modeling criteria, design and construction procedures."

This and the first symposium (Athens 1993) result from initiatives of workers on a Technical Committee of ISSMGE devoted to the same theme, which probably accounts for the content being oriented towards material characteristics rather than in situ geological variability or optimizing of site investigation. The papers in volumes 1 and 2 are grouped as follows:

1. "Investigations, classification and testing for soil characterization " (45 papers).

2. "Selection of soil parameters: modeling the soil behaviour" (60 papers).

(One could nit-pick about whether "soil" should have been omitted from both of these headings.)

3. "Tunneling and underground openings" (12 papers).

4. "Cuttings and natural slopes" (19 papers).

Volume 3, containing state-of-the-art papers, panel reports, and keynote lectures, is not due for publication until the end of the year.

The papers, from many parts of the world, are based on a wide range of lithologies – a good proportion of sediments like New Zealand's Tertiary, along with various pyroclastics, products of residual weathering, and clays (some fissured). Most testing is laboratory-based, with some in situ assessment. A number of papers go into microstructure, often a useful step for full understanding of behaviour of many of these materials. One paper of general interest is by Czerewko & Cripps, "Simple index tests for assessing the durability of mudrocks".

Deformation monitoring, groundwater, and support issues feature in the underground papers, while failure modeling and analysis feature in those dealing with slopes.

These proceedings and volume 3 containing the overview papers (still to come) are definitely worth a look if you have an interest in "hard soils – soft rocks." These materials might not cause such a "hard life" for geotechnical practitioners if we weren't still burdened by the tradition of having to split the geological continuum according to "is it a rock?" or "is it a soil?", but I suspect we are some way off from recognizing the need to change that.

REVIEWED BY: BRUCE RIDDOLLS, DIRECTOR, RIDDOLLS AND GROCOTT LTD

The Geotechnics of Hard Soils – Soft Rocks
Proceedings of the Second International Symposium, Naples, Italy, 12-14 October 1998. Vols. 1+2.

Edited by:- EVANGELISTA, A, AND PICARELLI, L, (EDS)
Published by:- A.A.Balkema, Rotterdam
Date: 1999
Web shopping on: http://balkema.jcn.nl/ima/balkema/index
Price : $EUR 159.50

Volume 3 not yet published

Landslide Risk Assessment

The book presents the proceedings of a workshop on "Landslide Risk Assessment" which was convened as a Working Group of the International Union of Geological Sciences (IUGS). The objectives of the workshop included review and standardisation of landslide risk terminology, review and suggest methods for applying national standards of acceptable and tolerable risk level to landslides, and to review methods of predicting vulnerability of property and life to landslides.

The text provides an excellent introduction to the developing field of Quantitative Risk Assessment (QRA) and the state of the art of landslide risk assessment in general. It provides further reading to the recent short courses on Quantitative Risk Assessment for landslides presented by Professor Robin Fell at the University of New South Wales and in Auckland during 1999.

A brief summary paper outlines the state of the art and level of consensus achieved at the workshop and includes agreed definitions for risk terminology.

Four theme papers provide a broad introduction to landslide risk assessment:

- The paper by Fell and Hartford titled "Landslide Risk Management" presents a general framework for risk management and examples of different frameworks. Guidance is provided on acceptable and unacceptable societal risk. The definition of societal risk is explained in some detail. Risk criteria adopted by other engineering disciplines is reviewed including that adopted by the Health and Safety Executive (UK), BC Hydro (Canada) and ANCOLD. The ALARP (As Low As Reasonably Practicable) concept for tolerable risk is explained. Advice is provided on zoning for landslide risk.
- The consequences associated with landslides in Hong Kong are outlined in a paper titled "Assessment of consequence of landslides" by Wong, Ho and Chan. This paper provides useful information on the vulnerability of facilities and human life in landslides and evaluation of slide travel distance. A number of different approaches are presented for the quantification of consequences for buildings and roads.
- Two papers by Morgenstern and Einstein outline the status of landslide risk in geotechnical practice, methods and systems of assessment and management.

Twenty submitted papers cover a relatively broad range of case studies and potential approaches to landslide risk assessment. The submitted papers can be broadly categorised as follows and the papers, which may be of most interest to Geotechnical Society members, are highlighted:

- Historical Evidence of Risk (3 papers)
  Of note is a paper by Evans, which covers the analysis of frequency of fatal landslides in Canada from a database over a 156 year period.
  A paper by Cruden addresses the problems of estimating risks from historical evidence of rockfall using the Argillite Cut in British Columbia as a case study.
- Hazard and Risk Mapping (4 papers)
  Landslide intensity mapping on a nation wide basis is outlined in two papers by Perkins and Highland of the USGS.
- Approaches to Risk Assessment and Management (8 papers)
  Approaches to risk assessment for reservoir slopes are addressed in papers by Imrie and Moore of BCHydro, and Moon provides a case study for prediction of probability for slow moving landslides in the Roxburgh Gorge in New Zealand.
  A paper on quantitative and semi-quantitative estimation of landslide probability by Mostyn and Fell provides further assessment frameworks and includes a large reference section for background research.
  Risk assessment of debris flows is covered by Sassa et al.
  Tang and Stark indicate the reliability of cut slope design from back analysis of a large number of landslides in Berkeley, California.
  A methodology for risk assessment and management for development below potential natural slope instability in Hong Kong is covered by Roberds et al.
- Regulation and Guidelines for Landslide Risk (3papers)
  A regulatory perspective to slope hazards in Canada is presented by Morgan including a review tolerable societal risks.
  An interesting paper by Oboni and Oldendorff considers the integration of risk and crisis management using an example of rockfall risk mitigation with netting.
- Problems and Pitfalls (2 papers)
  Potential pitfalls with geological characterisation for Quantitative Risk Assessment are outlined in a paper by Baynes. Four case studies are presented including examples of rockfall and slow moving landslides threatening communities.

Overall this book provides an excellent introduction to landslide risk assessment and the various papers cover all aspects of the developing field of Quantitative Risk Assessment. The numerous case studies and various frame works presented provide valuable assistance for assessing the risks associated with landslides. The information presented on acceptable levels of risk provides guidance on appropriate recommendations for the management of landslide risks and should permit more defensible recommendations to be made.

**REVIEWED BY: GREG SAUL, SENIOR GEOTECHNICAL ENGINEER,
OPUS INTERNATIONAL CONSULTANTS LTD**

Landslide Risk Assessment
Proceedings of the International Workshop on Landslide Risk Assessment. Honolulu 19-21 Feb 1997.

Edited by:- David Cruden & Robin Fell

Published by:- A.A.Balkema

Date: 1997

Web shopping on: http://balkema.jcn.nl/ima/balkema/index
Price : $US 104

Special Interests

The Management of Old Gasworks Sites

Keith G Delamore
Director, Pattle Delamore Partners Limited,
(Auckland, Wellington & Christchurch)

The management of old gasworks sites for redevelopment needs to be based on a good scientific understanding of the site conditions and appropriate strategies for limiting the environmental and public health risk associated with the new development. By selecting a low sensitivity use and applying a robust site management plan for works at the site, gasworks sites can be redeveloped with limited remedial costs.

From late in the 19th century for approximately 100 years, many towns in New Zealand obtained light and heat from the local town gas works. Initially, gas was produced from coal, later (about the 1960s) from petroleum. In the 1980s, natural gas replaced gas manufactured at gas works and the gas works were abandoned. There are approximately 55 gas works sites through out New Zealand.

At most sites, following the closure of the gas works, the brick buildings, steel gas-holders and associated pipework were demolished. The sites were levelled and left vacant for future development. Because of their location, in town and close to the water body that often gave the town its focus, gas works sites are frequently valuable sites for redevelopment.

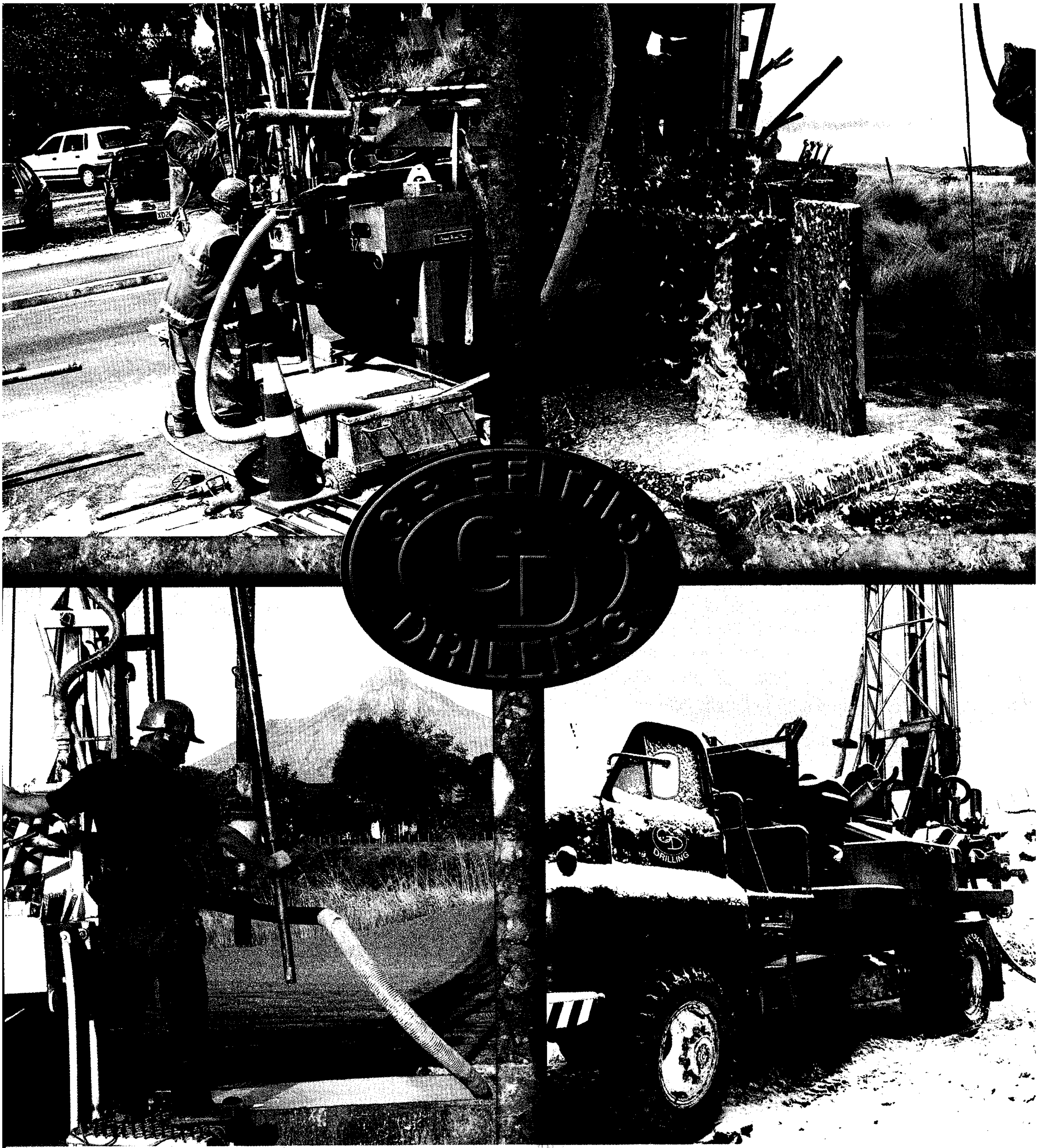
Gas manufactured from coal was a dirty process involving storage of materials, and disposal of wastes. Stored materials (coal, gas and coal tar) resulted in chemicals leaching or leaking into the ground. Wastes disposed into the ground (demolition rubble, furnace wastes, and spent iron oxide) resulted in chemicals being added to the natural ground. Often wastes were used to fill the adjacent water body, so that chemicals from the wastes could enter the water directly.

The main chemicals of concern at disused gas works sites are sulphur, cyanide, polycylic aromatic hydrocarbons (PAHs) and heavy metals. Benzene, toluene, ethyl benzene and xylene (BTEX) are often associated with coal tar but because they are volatile compounds, they are often absent from wastes that have been exposed to the atmosphere. Cyanide is usually present as an iron complex and is in a relatively non-toxic form compared with "free" cyanide.

Typically a gas works site can be investigated using standard contaminated site investigation techniques (historical review of site activities, initial site reconnaissance, test pits, boreholes, groundwater monitoring wells, and soil and water samples for laboratory analysis). Based on this information, the site can be zoned according to the wastes that have been found. The zoning can be used to decide the extent of any cleanup that might be required. In most cases, it is impractical to remove all of the gasworks wastes so that returning the land to single-family residential use is not appropriate. Instead, the preferred approach is to recognise that the site is contaminated, maintain an industrial or similar low sensitivity use, and ensure that an appropriate management plan is in place to control the redevelopment of the site. This information can be kept on the property's Land Information Memorandum (LIM) to advise future users of the site.

A special problem with gas works sites is the contamination of groundwater aquifers by the chemicals associated with the wastes. Various techniques can be applied to confine the contamination in the water but a major problem is the coal tar which is a dense non-aqueous phase liquid (DNAPL). Since the coal tar is heavier than water it can sink to the base of the aquifer resulting in contamination over a large area that cannot be easily confined.

In summary, gas works sites can be studied to understand the extent and nature of the contamination. Providing that a low sensitivity site use is planned, no major remediation may be necessary and the study information can be used to manage redevelopment of the site. Where groundwater has been contaminated, special studies are necessary to ensure that groundwater supplies are not affected by gas works chemicals.



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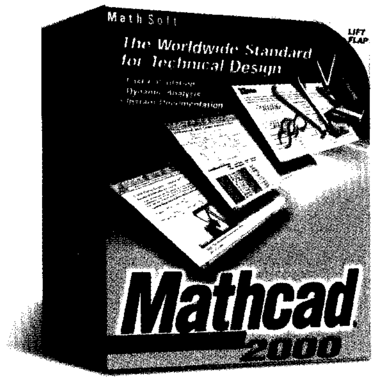

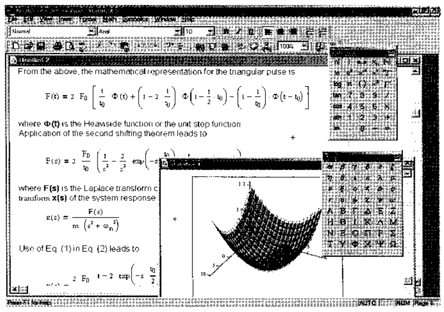

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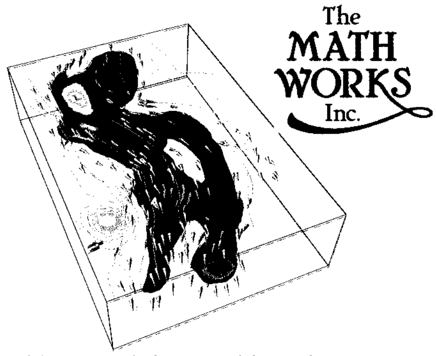

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Numerical Analysis in Soil Mechanics, Part 1

Sergei Terzaghi
Senior Geotechnical Engineer, Sinclair Knight Merz

We hear a lot of discussion about the applicability of numerical analysis for practical soil mechanics problems. The fact is that numerical analysis has a lot to offer to practicing engineers provided that the engineer keeps an eye on some very fundamental issues, and of course, they have access to a code that is robust and straight forward to use.

Soil has a number of peculiarities virtually unknown or not relevant in the realm of more conventional mechanics including:

- non-linearity of strength and stiffness with both shear and volumetric strain (also confining pressure)
- plasticity with associated and non-associated flow
- two or three phase material (air, water, solid) with the concurrent issues of dry, saturated, drained, or undrained behaviour, often all in the one problem.
- threshold strain/stress levels
- time dependent effects including strain rate effects, consolidation, and creep
- element locking
- anisotropy of permeability, strength and stiffness

Through this and successive articles, I hope to examine some of the consequences of these peculiarities, and the impact that they have on the analysis that one is performing.

To clear the air a little, it makes little difference whether one is using a finite element, finite difference, boundary element or some other technique. All of these techniques represent an alternative discretization and/or solution method of the same problem and face the same issues. Each method has its strengths and weaknesses with regard to a particular problem, and certainly none of the techniques are as universal as some proponents might claim. Hydridization of the different techniques are occurring more and more frequently. At the end of the day, it is the implementation of the techniques that will be more important than which particular technique is used, and of course, how carefully they have addressed the above and other issues.

The issue of non-linearity of strength in combination with plasticity is an issue that fairly frequently gets overlooked, and yet the importance of both of these issues is paramount, and indeed intertwined. The non-linearity of strength manifests itself on several levels, many of which are still not satisfactorily resolved.

The first order non-linearity is of course the assumption that the soil is elastic up to the strength envelope (mohr-coulomb or alternative), and then fully plastic thereafter. A number of second (or higher order) non-linearities then start becoming apparent. Some of the obvious ones include the curvature of the strength envelope, the anisotropy of the material, whether or not we are considering the drained or undrained behaviour of the material, and then the consequent impact of volumetric/shear/pore pressure changes. Also to be considered is the shear hardening/softening of the material. Right now we will focus on the first order non-linearity and consideration of that with the onset of plasticity.

Consider an embankment on soft normally consolidated clay and assume that it will be constructed in such a fashion that it will have a short-term factor of safety (slope stability) using conventional analysis of 1.25. We can also analyse the structure using a number of different models being linear elastic, Figure 1, elasto-plastic assuming undrained conditions (in other words effective stress parameters with pore pressure generation) Figure 2 and elasto-plastic assuming total stress conditions (in other words total stress parameters $\phi = 0$ & c_u) Figure 3.

Each of these models has been set-up as closely as possible to represent the same strength, stiffness, and behaviour. Provided that the strength parameters chosen in Cases 2 and 3 are compatible they should have identical results in terms of stability. The figures show the distribution of shear strain resulting from the construction of the embankment in a qualitative sense.

These diagrams are hugely informative as to the impact of the different assumptions. Figure 1 is purely elastic and represents the classical stress bulb obtained from standard charts (Lambe and Whitman, Soil Mechanics, 1969) modified by gravitational stresses. Figures 2 & 3 exhibit varying degrees of plasticity and associated re-distribution of stress/strain. The re-distribution is dependent on the assumed material behaviour and the consequent flow laws assumed. Figure 2 is perhaps the most realistic of the three cases whereas Figure 3 represents the commonly assumed case.

Figure 2 shows some impact of the classical stress bulb and also shows the concentration of shear stress/strain immediately underneath the toes of the embankment. Figure 3 shows a large zone of uniform shear strain reflecting the classic stress bulb but the deeper, more circular shape suggests that this zone has gone plastic. Smaller zones of high strain exist underneath the embankment toes, as can be seen by the density of contours.

These three simple cases indicate the magnitude and distribution of stress/strain through the foundation soils is very dependent on the model used. This will directly impact the predictions made for both settlement and ultimately the 'true' factor of safety. It will also impact on the way one designs and interprets the ground investigation program.

In my next column I will look at some of the reasons for the different stress distributions and the reason these differences will impact on settlement, FOS and investigations.

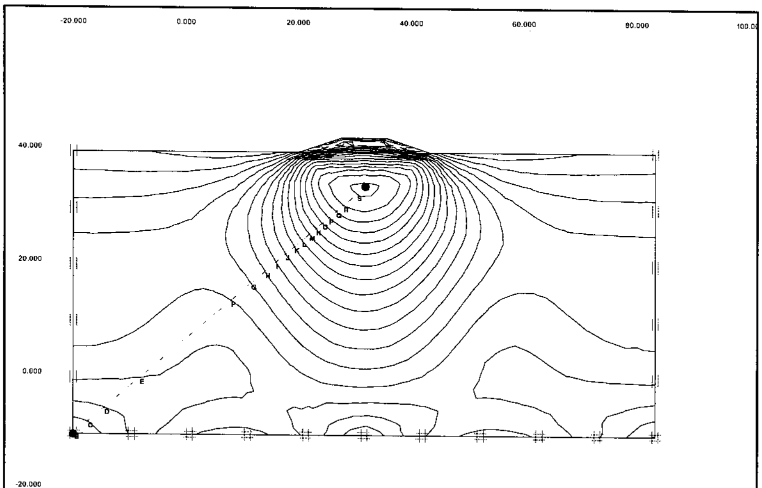


Figure 1- Elastic Behaviour

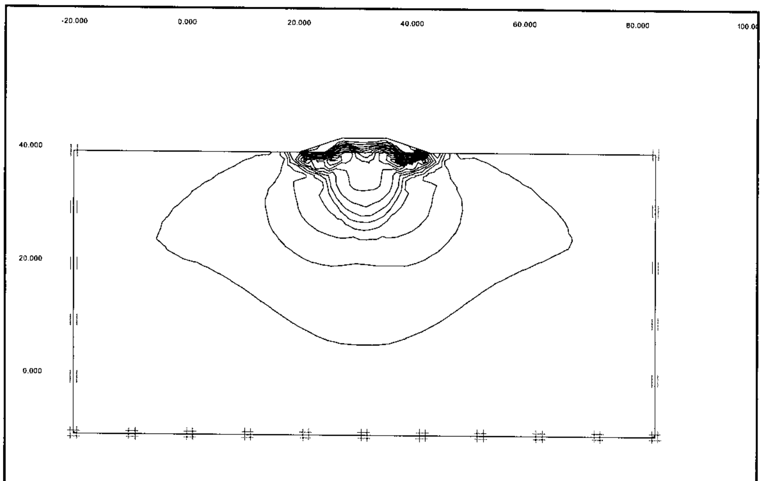


Figure 2 - Undrained Elasto-Plastic

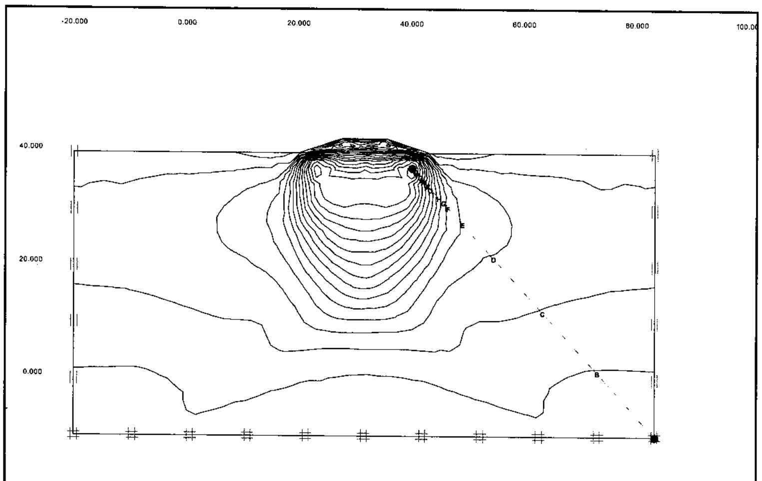


Figure 3 - Total Stress

The Bob Wallace Column

Ethic *n.* a moral principle or set of values held by an individual or group:

Ethical *adj.* In accordance with the principles of conduct that are considered correct, esp. those of a given profession or group.

The Geotechnical Society is a technical group of IPENZ and as such we are expected to understand and implement their Code of Ethics. IPENZ publishes a useful document titled Guidelines for the Code of Ethics that includes an interesting discussion on the general principles that should be followed. It is not a big document, and it is worth reading at least once every year or two to remind us of the high ideals we should be striving for in our professional life.

So where are we in relation to these high ideals? Are we kidding ourselves about our Profession? Compare us to Doctors – covering up their own mistakes to the detriment of public health and welfare; Dentists – advertising on the television like car salesman; Lawyers – like flies around ***t *(Ed's note, blanked out since we cannot use this term in a "professional" journal)* never slow to make money out of misfortune or mishap. By comparison we must be doing fairly well.

Unfortunately, Engineers (and particularly Geotechnical Engineers) operate at the lowest common denominator. Why do a hand auger if we can get away with a Scala? Why do a trial pit if we can get away with a hand auger? Why do a drill hole if we can get away with a trial pit. Why recover core if we can get away with an air hammer or washbore?

We bid competitively as a matter of course and the lowest tenderer is commonly selected. We draft proposals and lay claim to experience that is often misleading. We tailor our reports to suit the whim of our clients (often excused as "accentuating the positive"). We understand that protecting our client's interests means firstly and foremost saving them money. We let commercialism take precedence over technical rigour or efficacy. In the changing professional world there has been a steady erosion of those ideals, to an extent that almost on a daily basis we face conflict with our ethical archangel.

However, there is hope. Ernest Greenwood pointed out as far back as 1957 that ethics demand behaviour that is cooperative, equalitarian and supportive to our colleagues. Despite this being in direct conflict with our daily competition for clients and almost impossible to follow in an adversarial contractual environment, it remains one of the most well defended principles.

Clients have been encouraged to recognise that the lowest lump sum bid is not necessarily the correct or best way to procure professional services. In an attempt to protect their interests, and ensure they do get what they want, the concept of Peer Review has been commonly adopted. Unfortunately, in our defence of the last bastion of ethical endeavour this approach has become an unqualified disaster. Clients are now paying one consultant to do a job inadequately and another expert to confirm that it will probably be OK. We do not confront the incompetent and their mistakes are not exposed.

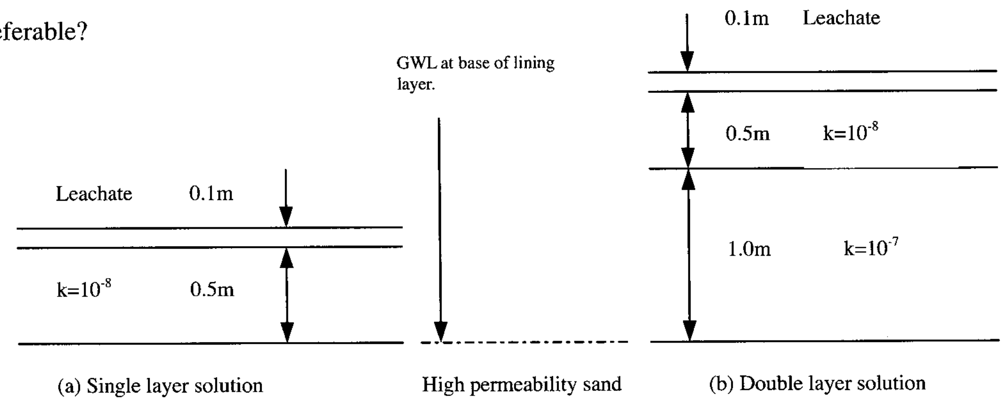
Perhaps we can after all stand shoulder to shoulder with the medical profession, proud in the knowledge that some of our ethical standards are still intact.

Laurie's Brain Teaser (No. 2)

A landfill is to be created at a disused quarry where excavation has been carried out down to the water table depth; this is fixed by the level of a nearby river. The base is in high permeability sand. The drawing below shows two possible lining proposals for the base of the landfill. There is only enough clay with permeability of 10^{-8}m/s to form a layer 0.5m thick, but there is an abundant supply of clay with a permeability of 10^{-7}m/s.

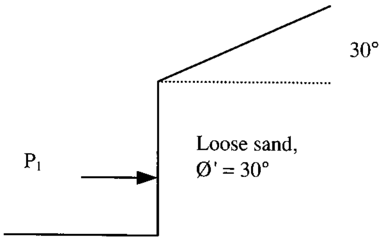
Engineer Smith proposes solution (a), but Engineer Jones thinks a more conservative solution is needed and proposes solution (b).

Which is preferable?

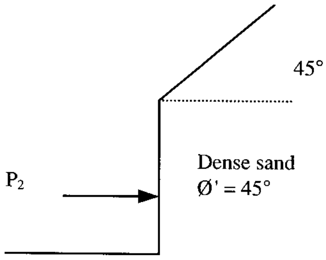


Solution to Brain Teaser No 1

The answer is (a): the wall supporting the loose sand. In this case the value of K_a is 0.75, while for the dense sand the value of K_a is 0.5, so that the wall supporting the loose sand needs to be 50% stronger than the wall supporting the dense sand.



(a) Wall retaining loose sand



(b) Wall retaining dense sand

Explanation

Assuming the walls yield sufficiently to give a failure condition, then an analytical solution can be found using a series of Coulomb wedges. For the general case of a smooth vertical wall with a slope at angle ω, and a cohesionless material with a friction angle of Ø', the value K_a is given by:

$$K_a = \cos^2Ø'/(1+\sqrt{\sin Ø'.\sin(Ø'-\omega)/\cos\omega})^2$$

and

$$P = 0.5K_a\gamma H^2$$

If the slope angle ω is equal to Ø' this becomes:

$$K_a = \cos^2Ø'$$

This is an interesting finding, as it means that the value of K_a decreases as ω (=Ø') increases, and hence the answer to the question is the wall retaining the material with a back slope of 30°. This wall has to be 50% stronger than the wall with slope of 45°.

An analytical solution is also available for the angle of the critical failure wedge α. This is given by:

$$\tan(\alpha-\omega) = \tan(Ø'-\omega) + \sqrt{\tan(Ø'-\omega)\{\tan(Ø'-\omega) + \cot Ø'\}}$$

and when ω = Ø' this gives α = Ø' = ω. Hence the wedge becomes a slice extending an infinite distance up the slope.

I became aware of the above solution when looking into the estimation of retaining forces for cuts made into very steep slopes which appeared to be of fairly marginal stability without cuts being made. The slopes I encountered were often of almost unlimited extent above the cut site and were in residual soils of rather mixed properties, being made up of zones of soil and partially weathered rock. The reliable estimation of strength parameters was a very uncertain undertaking.

The alternative to attempting measurement of c' and Ø' in such a situation is to make the assumption that the slope is at limiting equilibrium and obtain c' and Ø' by back analysis. This is a tedious procedure and involves assumptions about the relative magnitude of c' and Ø', as well as about the seepage conditions in the slope. The "back-analysed" parameters can then be fed into a Coulomb wedge analysis to obtain the force levels. An analytical solution is not possible (at least I have not been able to find one) but by trial wedges and a graphical procedure a solution is possible.

This solution turns out to give K_a values similar to those for the purely frictional (and dry) case discussed above. As the slope becomes steeper the value of K_a invariably decreases, regardless of the assumptions made about relative magnitudes of c' and Ø', or the seepage conditions.

The above finding is immediately clear if the limiting cases of a horizontal slope and a vertical slope are considered. If a horizontal slope is in a state of limiting equilibrium, it means the material has no shear strength and is in fact a liquid; the K_a value would then be 1.0 (ie the hydrostatic case for a fluid). If the slope is vertical then no force would be required to support it and K_a would zero.

Laurie Wesley.

Technical Articles

Prediction of Dewatering Related Settlement in Waihi Township, New Zealand

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SUMMARY:

Between 1995 and 1998, Woodward-Clyde (NZ) Limited (WCNZ) conducted a settlement and rebound study of the Martha Gold Mine and Waihi township. The objective of this study was to support a resource consent application to extend the Martha pit.

Dewatering of the Martha pit has caused the groundwater level in most of the soil and rock layers immediately adjacent to the Martha pit to fall. Lowering of the general groundwater level has resulted in an increase in the level of effective stress within these soil and rock deposits. This increase in effective stress, and four other unrelated factors, has caused some of the soil and rock layers around the pit to consolidate resulting in settlement of the ground surface.

In order to estimate the magnitude of settlement and rebound that is likely to occur in Waihi due to operating and decommissioning the Martha pit, WCNZ completed an extensive program of research, investigation, laboratory testing and modelling. This paper summarises the methodology and results of this work.

While there has been a change in effective stress leading to some consolidation of the ground surface, dewatering of the Martha Pit has not caused any structural distress in the township, and engineering predictions indicate that no distress due to this cause is likely in the future. The magnitude of dewatering related settlement measured in Waihi Township is low compared to that measured in New Zealand and the United States at sites which have been affected by fluid extraction.

1.0 NOTATION

C	Cohesion.
D	Constrained modulus.
D_C	Constrained modulus of a soil or rock layer during initial consolidation.
D_{UR}	Constrained modulus of a soil or rock layer during unload and reload cycles.
e	Void Ratio.
E	Young's Modulus.
m_v	Coefficient of Compressibility.
s	Estimated settlement
t	Layer thickness
$\Delta\sigma'_v$	Change in effective stress at the centre of a soil or rock layer.
μ	Possions ratio
ρ_B	Bulk Density
Ø	Angle of friction.
Note:	$D = 1/m_v$ *
	$D = E(1-\mu)/\{(1+\mu)(1-2\mu)\}$ *
	* After Lambe and Whitman (1979).

2.0 INTRODUCTION

Settlement of the ground surface due to the extraction of fluids such as groundwater, oil, gas, and geothermal water has been documented at numerous sites throughout the world.

The extraction of water from the ground, for either water supply or dewatering purposes, has resulted in ground settlement of more than 6 metres in Mexico City (Rivera *et al* 1991) and around 9 metres in the San Joaquin Valley, California (Johnson 1991).

Mine dewatering related settlement has been documented in New Zealand at the Huntly East Coal Mine. During 1983, an area of approximately 7 hectares experienced settlement of up to 800mm (Kelsey, 1985).

Historically, underground mining occurred at the Martha Mine from the late 1880's through to the early 1950's. Old mine records indicate that the underground workings extend to 457m below sea level (BSL). Dewatering of the old mine occurred to the maximum depth of the workings (McAra, 1988).

The Waihi Gold Mining Company Limited (WGC) has operated an open pit mine at the Martha Mine site in Waihi since 1989. Originally Martha Hill outcropped at 160m above sea level (ASL). Resource consents have recently been granted allowing the Martha pit to extend to a depth of 95m below sea level (BSL). The location of the Martha pit is shown on Figure 1.

The Martha pit has been progressively excavated and dewatered over the last ten years and, as at June 1999, the pit excavation extended to a level 10m ASL. Since 1989, the groundwater level has been held at a level 5 to 20 metres below the base of the pit excavation by pumping. This has caused the groundwater level in most of the soil and rock layers adjacent to the pit excavation to fall.

Lowering of the general groundwater level around the pit has increased the magnitude of effective stress within the dewatered zones. This has caused some of the soil and rock layers adjacent to the Martha Pit to consolidate. Monitoring has shown that dewatering of the Martha Pit has not adversely affected buildings or services within Waihi Township. Engineering predictions also indicate that buildings or services are unlikely to be damaged as a result of mine dewatering.

In addition to mine related settlement, four other causes of settlement were identified in Waihi Township:

i) Natural consolidation settlement of fill and the underlying natural soils;

ii) Cyclical shrinkage and swelling, of the near surface soils, due to rainfall and changing soil moisture levels;

iii) Movement or collapse of historic mine workings; and;

iv) Consolidation of upper soil layers due to reduced water infiltration.

Considerable work was undertaken to evaluate these, and separate their effects from the mine related settlement.

A percentage of the settlement due to mine dewatering is recoverable if the groundwater level is allowed to recover after the pit is decommissioned. As the groundwater level rises, the effective stress levels will decrease, and a proportion of the settlement will be recovered through rebound.

3.0 STUDY APPROACH

The settlement and rebound study was conducted in eight stages.

The first stage of the study was a review and analysis of all the available data. This included a review of published and unpublished maps, reports, aerial photographs, historical records and survey data.

The second stage of the study was the development of plans summarising the settlement measured in Waihi Township, and preliminary two dimensional geological, geotechnical and hydrogeological models. During this stage of the study, areas of insufficient data and areas of high or unusual settlement were identified.

The third stage of the study comprised field investigations and laboratory tests targeted to address information gaps.

The fourth stage of the study was the generation of detailed two dimensional geological, geotechnical and hydro-geologic models, and a detailed analysis of these models in conjunction with the measured settlement and rainfall data. During this stage, a preliminary estimate of the potential settlement and an interim report was completed.

The fifth stage comprised additional field investigations targeted to confirm the extent of the potentially compressible layers, confirm the geotechnical characteristics of key units, and clarify issues identified during stage four of the study.

The sixth stage of the study was the development and refinement of simple one-dimensional geotechnical models. These models were constructed using records of drillholes, readings from groundwater piezometers installed within these drillholes, and laboratory data. The stiffness properties of the different geologic units were refined using the one-dimensional models by comparing predictions of surface settlement made by the models to measured survey data.

The seventh stage of the study was the development of a finite element model along a section through the Martha Mine and Waihi Township. This was achieved using the computer program "Plaxis" (Version 6), the geological, geotechnical and hydrogeologic cross-sections, and geotechnical data from the refined one-dimensional models. Where possible, measured groundwater profiles were used in the finite element models, however, the maximum number of layers able to be modelled by Plaxis version 6 is ten. Because of this, adjacent geologic units with similar geotechnical properties were modelled as a single layer. The groundwater profiles in these combined layers were initially estimated using field measurements, and then refined by comparing predictions of surface settlement with survey data.

Finally, the refined one dimensional and finite element models were used to predict settlement and rebound due to operating and decommissioning the Martha Mine. By subtracting the predicted rebound from the predicted settlement, an estimate of the permanent ground surface settlement was made.

4.0 GEOLOGIC SUMMARY

The near vertical ore-bearing vein systems that originally outcropped on Martha Hill are located within andesite. The andesite is overlain to the east, south and west of the Martha pit by younger ignimbrite, ash, tuff, and alluvial deposits which, in places, are overlain by fill.

For the purposes of the settlement and rebound study, the geological units present at Waihi were grouped into three main deposits according to their geological and geotechnical characteristics. These units were :

(i) Older deposits comprising andesite and altered andesite,

(ii) Younger Deposits comprising ignimbrite, ash, tuff and alluvial deposits, and,

(iii) Man-made Deposits comprising fill.

Figure 2 shows a typical subsurface distribution of the geological units under Waihi Township. Several cross-sections were developed using published maps, field observations and machine drillhole information.

The overall strength of the older deposits is relatively high, and this, together with the fact that they have previously been dewatered, means that they are relatively incompressible.

The overall strength of the younger deposits is highly variable and the unconfined compressive strength of these deposits range from >100 MPa (welded ignimbrite) to <0.1 MPa (alluvium). Most of the younger deposits comprise materials which will consolidate if dewatered or depressurised.

In many locations within Waihi Township, particularly adjacent to historic mine workings and railways, there are man-made deposits of unconsolidated to poorly consolidated fill. In addition, many old swamps have been drained or filled, and the land used for residential development. Most of the man-made deposits are still undergoing post depositional consolidation, or are causing the consolidation of underlying natural deposits independently of the current mine workings.

5.0 HYDROGEOLOGIC SUMMARY

The permeability of the materials encountered at Waihi appear to be largely dependant on defect and fracture frequency in the case of welded materials or rock, and on porosity in the case of a soil. For the purposes of the settlement models, the geological units were broken into three groups of comparable permeability:

i) Highly to moderately permeable. Unaltered andesite and welded ignimbrite fall in to this group. The fractures in these units were generally open and moderately widely to widely spaced in the andesite, and closely spaced in the welded ignimbrite. The boulder alluvium is also highly permeable. This unit comprises cobbles and boulders in a silt, sand and gravel matrix.

ii) Moderate to low permeability. The geological units that fall into this group are the unwelded to moderately welded ignimbrite, ash and tuff deposits. The fractures in these units are very widely spaced, tight, and poorly interconnected. Some of the silty and sandy alluvial layers had a low to medium permeability and are also included in this group.

iii) Low permeability. The units in this group include the altered andesite and clay alluvium layers.

Under Waihi township, the presence of an almost continuous layer of low permeability altered andesite, on the surface of the older deposits, separates the older and younger deposits into separate groundwater regimes.

Historic underground mining dewatered the andesite rocks under Martha Hill and Waihi Town via a system of stopes, shafts and drives. This dewatering only drained those overlying younger deposits able to drain through the altered andesite cap into the andesite. Drainage of the younger deposits was, therefore, mainly limited to areas immediately around the shafts and drives that penetrated the younger deposits.

When historic mining ceased in 1952, pumping stopped and the groundwater levels returned to an elevation close to the ground surface. Re-establishment of the groundwater level within the dewatered younger deposits almost certainly resulted in a partial rebound of the ground surface.

Data from piezometers, installed prior to mine dewatering recommenced in 1989, shows that the groundwater level in 1988 was close to the ground surface in all layers.

Piezometric and survey data indicates that the present day limit of groundwater draw-down (and mine related settlement) is approximately 700 metres north, 2000 metres east, 1200 metres south and 400 metres west of the Martha pit. Within the andesite, these limits appear to correspond with the location of major structural faults.

Piezometric data indicates that the water level in the andesite rock immediately adjacent to the Martha pit has been drawn down to a level similar to the dewatering pump intake in the old Waihi No.7 shaft. The groundwater level measured in piezometers within andesite layers generally rises as the distance from the pit increases. Piezometers installed within altered andesite indicate that the piezometric pressure within this unit is variable and the unit is only locally drained via the historic underground workings.

Piezometers within the ignimbrite, ash, tuff and alluvial layers indicate that the groundwater level within the younger and man-made deposits follow a separate regime to the underlying older deposits.

The welded ignimbrite layer appears to have been dewatered to a level which is controlled by the units exposure in the Martha pit wall. Some recharge of this unit appears to occur via infiltration from overlying layers, and through outcrops of welded ignimbrite in the beds of streams.

Piezometers in the pumiceous ignimbrites, ash and tuff layers, that underlie the welded ignimbrite, indicate that the groundwater levels within these units are variable and are only drained locally.

Piezometers in the fill, alluvium and boulder alluvium layers, analysed in conjunction with rainfall data and six-monthly survey data, indicate that the groundwater level in these layers is controlled by :

i) the lowest level of the unit concerned exposed in the Martha pit wall,

ii) rainfall recharge, and,

iii) under-draining of the surface boulder alluvium by the welded ignimbrite layer.

6.0 GEOTECHNICAL SUMMARY

Review and analysis of historic, survey, rainfall, geological and geotechnical data identified five physical processes that produce settlement of the ground surface within Waihi Township. In most areas at least three of these causes of settlement occur simultaneously. The five settlement processes which were identified are :

i) Natural consolidation settlement of fill and the underlying natural soils;

ii) Cyclical shrinkage and swelling, of the near surface soils, due to rainfall and changing soil moisture levels. At some locations, seasonal shrink and swell of ±20mm has been measured;

iii) Movement or collapse of historic mine workings such as the Milking Cow block cave;

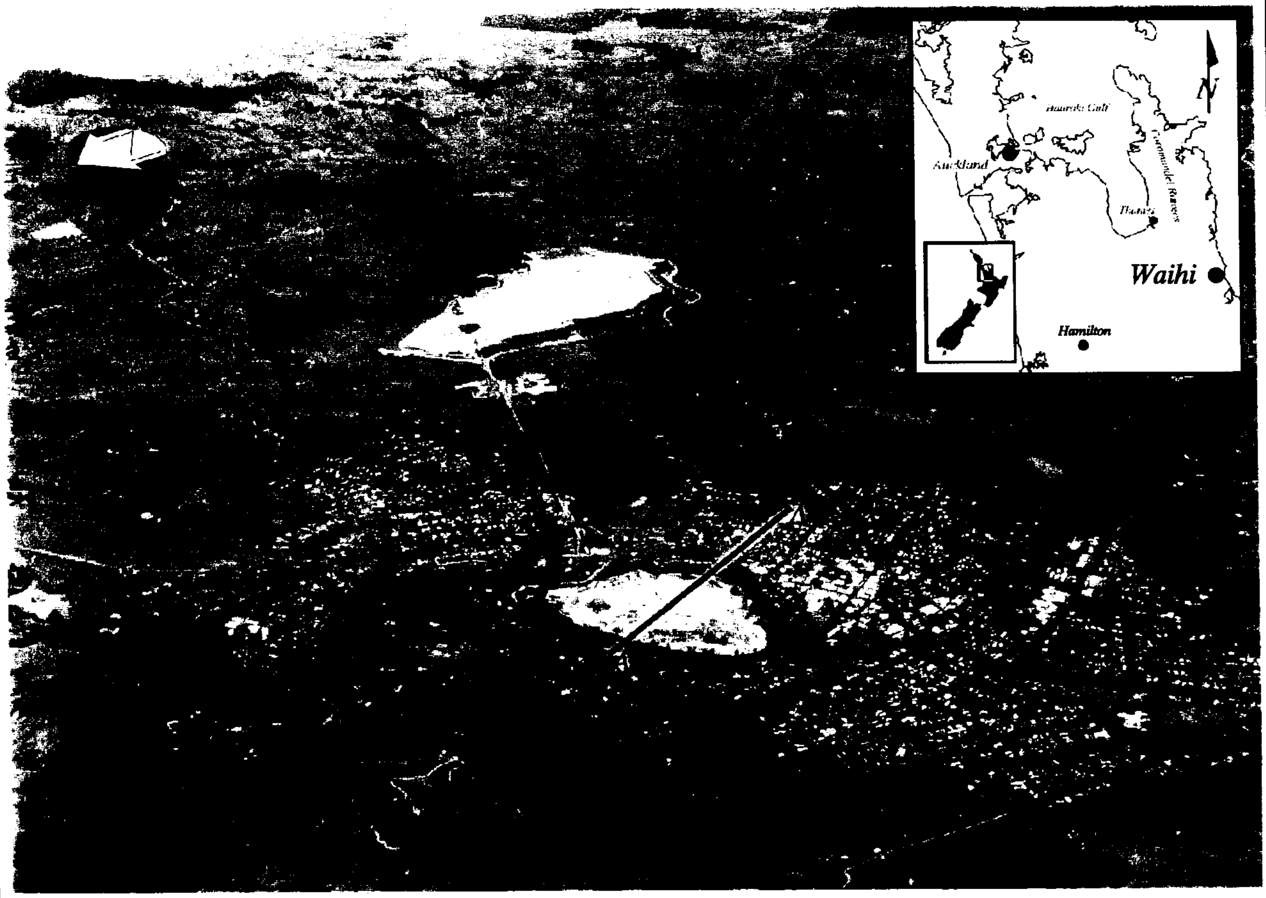


FIGURE 1
Location Plan

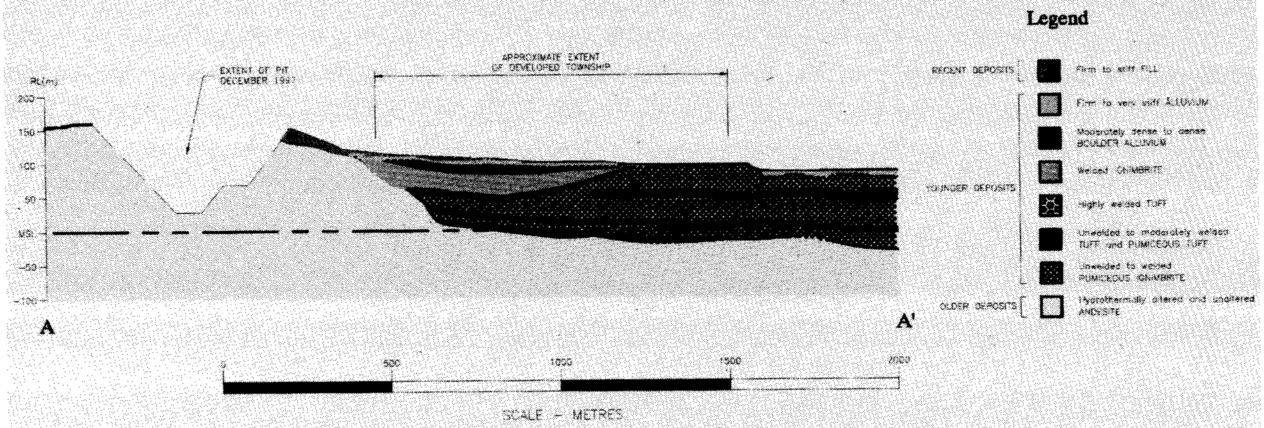


FIGURE 2
Typical Geological Cross Section A-A'
(Refer to Figure 1 for location)

iv) Consolidation of upper soil layers due to reduced water infiltration. This reduced infiltration is due to a combination of :

- El Nino weather patterns and a lower than average rainfall over the five or six years prior to the study;
- the installation of sewage and storm water reticulation systems in Waihi township since the mid 1980's;
- the clearing or modification of streams; and;
- the local removal of vegetation.

v) Dewatering of the present day Martha Mine.

Settlement caused by i, ii and iii frequently results in differential settlement and damage to settlement intolerant structures. The consequences of settlement causes iv and v are usually regional, more subtle in nature, and therefore less destructive.

The results of the geological and geotechnical investigations, observation of laboratory tests, analysis of survey monitoring results, and the analysis of laboratory tests results are summarised below :

i) The calculated pre-consolidation pressures of the ignimbrite, tuff, ash, and alluvial layers (younger deposits) underlying Waihi Township suggest that these layers had not undergone significant consolidation during historic dewatering.

ii) Many of the unwelded to poorly welded ignimbrite samples tested in the laboratory exhibited low to moderate consolidation characteristics up to a critical pressure. Once this critical pressure is exceeded the samples exhibit moderate to high consolidation characteristics. Observations of laboratory tests indicate that this critical pressure appears to reflect the collapse strength of pumice within the material, and welding of the material, rather than preconsolidation of the material;

iii) The pumiceous ignimbrite and tuff layers are brittle and compressible once critical pressures are exceeded;

iv) Some of the welded ash layers are brittle and moderately compressible once critical pressures are exceeded;

v) In places, the alluvium and fill layers are highly compressible;

vi) Most of the near surface cohesive alluvial soils are susceptible to cyclical shrink and swell;

vii) The consolidation coefficient of compressibility (m_v) is between two and ten times greater than the rebound m_v.

Following the eventual pit closure, and recovery of the groundwater level, rebound of the ground surface is expected to occur. Rebound of the ground surface will occur as the effective stress levels in the dewatered soil and rock layers decrease as a result of a rise in the level of the groundwater table.

Settlement developed during operation of the Martha pit is unlikely to be fully recovered for two reasons :

a) Most of the younger alluvial and man-made deposits are normally consolidated. Soil stresses, as a result of dewatering the Martha pit, are anticipated to exceed the existing pre-consolidation pressure in some locations.

b) The final lake level is anticipated to be around RL 104 m ASL. This represents a permanent lowering of the groundwater table around the pit and Martha Hill by between 0 and 30 metres. As a result, the original effective stresses in the ground will not fully recover to pre-existing conditions.

7.0 SETTLEMENT MONITORING DATA

WGC has measured a network of 180 survey pins, on a six-monthly basis, since dewatering of the Martha pit commenced in 1989. This network comprises surface settlement marks which were installed throughout the township before major excavation or dewatering of the pit commenced.

In addition, several close order survey networks have been established around buildings which exhibit cracks, or properties whose occupants or owners claim have been damaged by mine activities.

The close order survey networks comprised a combination of piezometers and closely spaced survey pins. Survey pins were installed at a spacing less than 50 metres across properties and around buildings. Deep seated and near surface survey pins were installed to separate mine related settlement from the other causes of settlement which were known to be present in Waihi.

Six-monthly pit contour plans, and the corresponding survey monitoring data, were reviewed in conjunction with piezometer data to identify links between pit operations, groundwater movement and ground settlement in the township area. The results from this review indicated that:

i) The Township of Waihi could be divided into seven zones of similar settlement behaviour demonstrated by the survey monitoring. Figure 3 shows the extent of the seven settlement zones

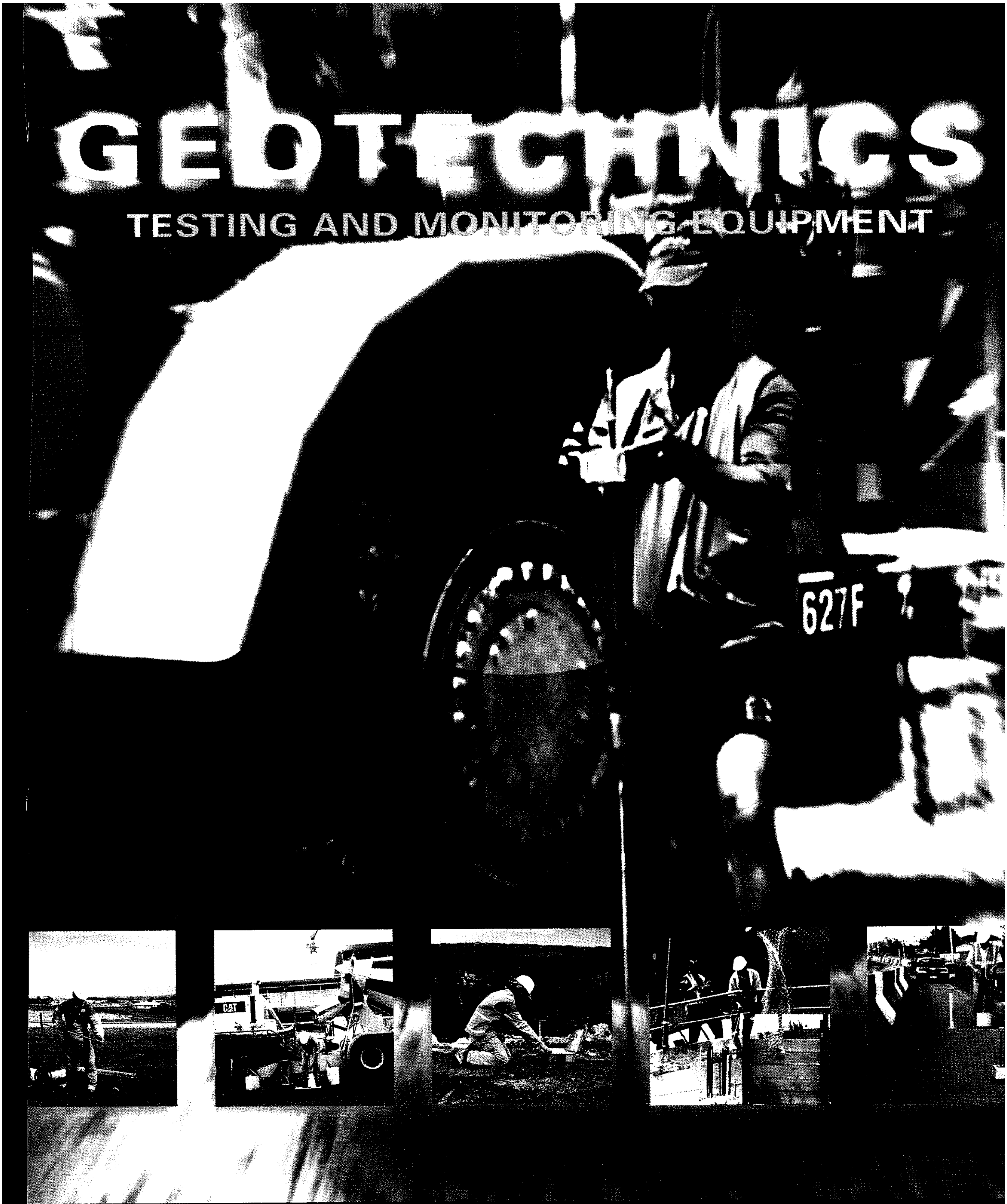
ii) There is a moisture dependent shrink-swell cycle, which is generally seasonal, that is unrelated to the mining operations;

iii) It was observed that ground movements occur rapidly, and many of the younger deposits exhibit a near-elastic response to changes in the groundwater level.

iv) The periods of greatest settlement usually correspond with pit wall cutbacks that result in a lowering of the exposure of younger deposits in the pit wall.

v) To date, the measured settlement has not been consistent, or symmetrical, around the Martha pit. The complex geology surrounding the pit is considered to be the main reason for this.

Survey results indicated that settlement Zones 1 and 2 were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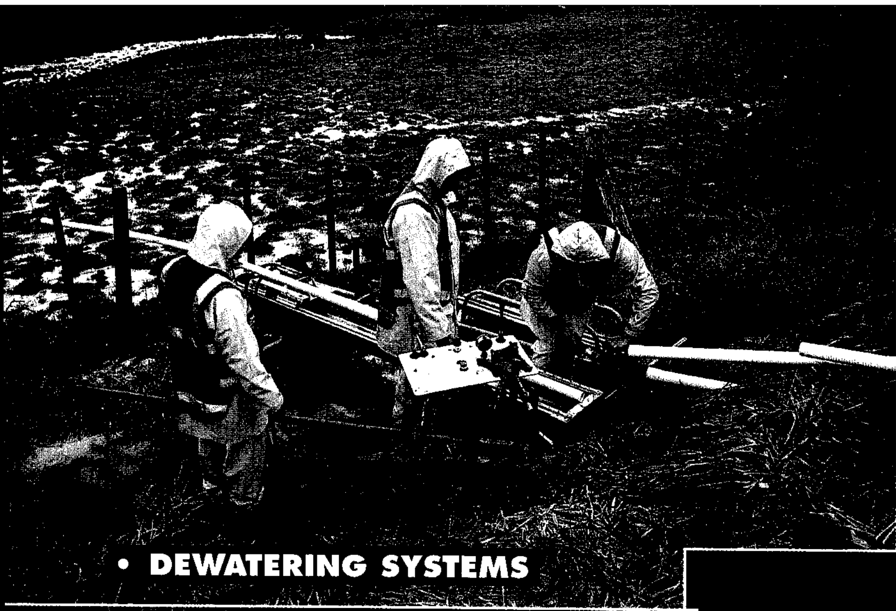

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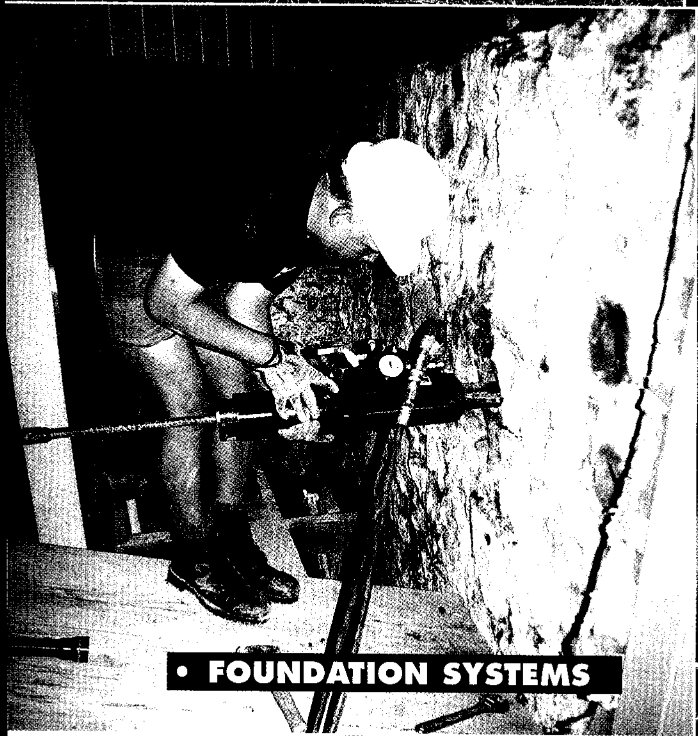

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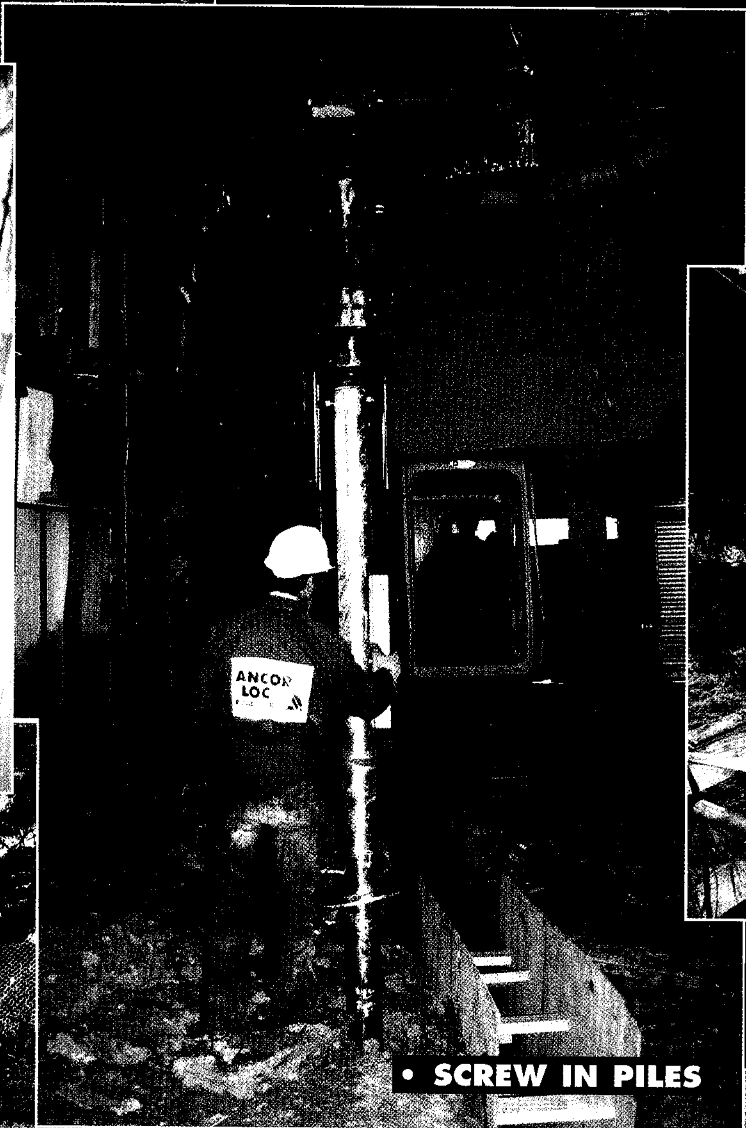

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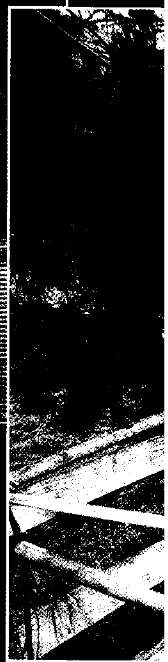

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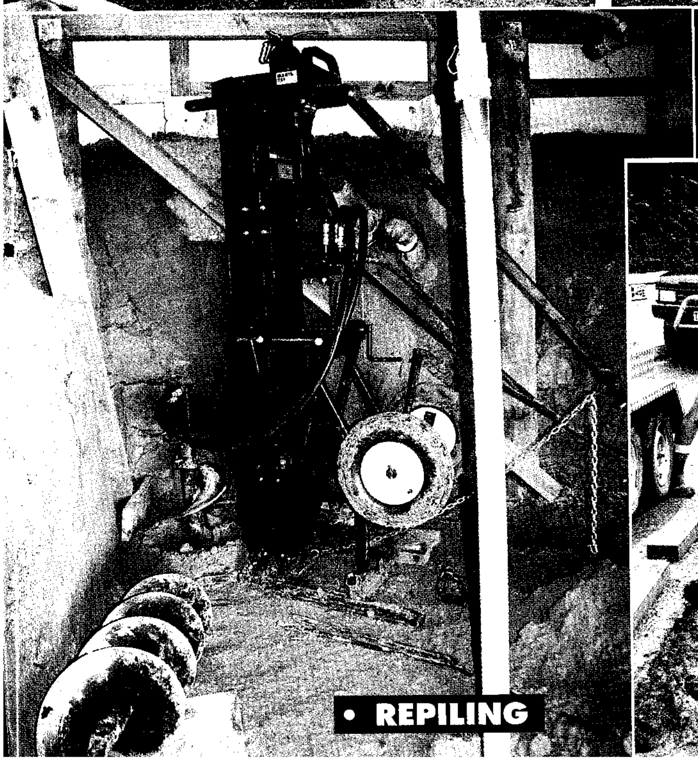

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probably unaffected by mine dewatering related settlement. In Zones 1, 2 and 3, settlements appear to be predominantly controlled by cyclic variation of the soil water content, with some settlement due to mine dewatering in Zone 3.

Settlement within Zones 3 to 7 is due to a combination of any 5 of the causes of settlement previously discussed in the geotechnical summary. Of Zones 3 to 7, Zone 3 is the zone least affected by mine dewatering and Zone 7 is the zone most affected by mine dewatering.

Settlement in Zones 4 to 7 appears to be primarily due to changes in the groundwater level caused by present day mining activities. Survey data indicates that considerable seasonal fluctuations are also present in these zones. The settlement in Zone 7 is primarily due to the combined effects of mine dewatering and stress redistribution within the Milking Cow block cave.

The Milking Cow is an historic block cave feature. Old mine records indicate that this underground excavation is over 300 metres deep, and was extensively mined and backfilled with waste soil and rock between 1910 and 1945. Prior to present day mining, the Milking Cave was visible on the ground surface as a 20 metre deep water filled depression. The approximate location and extent of the Milking Cow is shown on Figure 3

To date, the six-monthly survey results indicate that settlements in Zones 4 to 7, due to mine dewatering, have been rapid, and many of the younger deposits exhibit a near-elastic response to changes in the groundwater level. These movements are expected to be partially recovered when the pit is decommissioned and the pit-lake reaches its final level.

In all settlement zones, differential settlement, unrelated to mine dewatering, can be expected due to the local occurrence of unconsolidated alluvium, fill or rock outcrops, which respond differently to changes in load, rainfall recharge and surface water conditions.

8.0 ONE DIMENSIONAL GEOTECHNICAL MODELS

Once the soil property data had been collated and analysed, and the soil and rock characteristics identified, work focused on the development of geotechnical models which could be used to estimate settlement and rebound of the ground surface due to operating and decommissioning the Martha pit.

Preliminary one dimensional geotechnical (POD) models were initially constructed for several drillhole locations using :

a) borehole logs;

b) geotechnical laboratory data; and

c) measured piezometric levels.

For all one-dimensional models, the following equation, originally developed by Terzaghi during the 1930's, was used to estimate the settlement of each layer :

$s = m_v \times t \times \Delta\sigma'_v$ (Lambe & Whitman, 1979)

The estimated settlements, for all the layers encountered in a drillhole, were summated to provide an estimate of the ground surface settlement.

The values of m_v used in the POD models were selected based on the layer material types, and the mid-layer effective stress, both before and after dewatering. Estimates of the material properties were obtained from the results of laboratory tests.

Once the POD models had been constructed, an estimate of settlement was made using piezometric readings taken in December 1995. These predictions were then compared to the average settlement measured around the drillhole locations by the six monthly surveys. The average difference between the actual and estimated settlement due to mine dewatering was found to be approximately 30%.

Next, the values of material stiffness (m_v) used in the POD models were refined so that the difference between the measured and estimated settlement was less than 10%.

Refinement of material stiffness, in general, was limited to an adjustment of ± 20% and concentrated on those layers that most significantly influenced the total settlement. The exception to this was when laboratory data from adjacent drillholes indicated that a sample anomaly had occurred. The refined models were known as the refined one-dimensional (ROD) models.

A second calibration was undertaken using the ROD models and additional survey and piezometric data measured in May 1996. The May 1996 piezometric data was keyed into the ROD models to obtain an estimate of settlement. These predictions were then compared to the measured settlement. Differences between the estimated and measured settlements were

Material Type	m_v, (m^2/ MN) Calculated from Consolidation Test Results.	m_v, (m^2/ MN) Calculated from Triaxial test Results.
Fill	0.02 to 0.27	---
Alluvium	0.02 to 7.65	0.01 to 0.23
Boulder Alluvium Matrix	0.01 to 0.08	0.02 to 0.22
Weathered Welded Ignimbrite	0.01 to 0.05	---
Welded Ignimbrite	---	0.00005 to 0.00017
Moderately Welded Pumiceous Ignimbrite	0.005 to 0.10	0.0002 to 0.0085
Unwelded Pumiceous Ignimbrite	0.006 to 0.035	0.002 to 0.013
Moderately Welded Tuff	---	0.0006 to 0.0054
Slightly Welded Tuff / Ash	0.012 to 0.24	0.0014 to 0.019
Unwelded Tuff / Ash	0.01 to 0.25	0.0006 to 0.065
Altered Andesite	---	0.0003 to 0.057

Table 1. Summary of Material Compressibility

Material	Compressibility Ratio (Virgin m_v / Unload:Reload m_v)
Fill	3 to 9
Alluvium	3 to 7
Boulder Alluvium	3 to 6
Welded Ignimbrite	1
Ash and Tuff	3 to 5
Pumiceous Ignimbrite	3 to 5
Altered Andesite and Andesite	1

Table 2. Summary of Virgin / Unload : Reload Compressibility Ratios

Layer Name	C (kPa)	Ø (^{0})	ρ_B (t/m^3)	D_C (MPa)
Andesite	80,000	50	24.0	38,000
Altered Andesite	3,000	42	22.0	300
Moderately Welded Pumiceous Ignimbrite, Ash & Tuff	300	42	16.0	2500
Unwelded to moderately welded Ash & Tuff	200	34	17.0	150
Highly Welded Ignimbrite	120,000	46	22.5	10,000
Boulder Alluvium	10	34	20.0	800
Alluvium	10	28	16.5	20
Fill	10	28	15.0	4.5

Table 3. Material Properties Used in the Refined Finite Element Models

less than 10%, indicating that reliable models had been constructed. A summary of the material stiffnesses used in the one dimensional settlement models is presented in Table 1.

The ROD models were used to predict settlement and rebound of the ground surface at the drillhole locations due to operating and decommissioning the Martha pit. These predictions were made by feeding an estimate of the appropriate groundwater levels and rebound stiffness into the ROD models. The reload stiffnesses were estimated from the laboratory test results and are summarised in Table 2 in the form of Reload Compressibility Ratios.

9.0 FINITE ELEMENT MODELS

A preliminary finite element model (PFE model) of a section through Martha pit and Waihi township was initially constructed using Plaxis and :

i) Geological cross-section A-A (refer to Figure 2);

ii) Material data from the refined one dimensional geotechnical models; and

iii) The groundwater draw-down cones for May 1996.

The selection of the material stiffness, to be used in the finite element models, was very difficult due to the variation in geotechnical properties between adjacent drillholes.

Initially, the average layer stiffness was calculated using all the one-dimensional settlement models. Estimating material stiffness using this method resulted in finite element models which gave an estimate of settlement which was within 40% of the survey results.

A second value of the material stiffness was based on the one-dimensional settlement models of drillholes WC202 and WC206 only. This method gave an estimate of settlement which was generally within 20% of that measured by survey methods.

Table 3 summarises the finite element model material input parameters. For the purposes of the PFE model, a Mohr-Coulomb type model was selected to represent the behaviour of all material types.

The complex sequence of inter-bedded tuff, ash and pumiceous ignimbrite units; that underlie the welded ignimbrite, is modelled by two layers in the finite element model. This simplification was possible because these units have similar geotechnical and hydrogeologic properties, and appear to have a compatible level of dewatering.

A simplification of the geology was unavoidable, as the maximum number of different layers able to be analysed by Plaxis version 6.31, is ten.

Predictions of settlement made by the PFE model were compared to the settlements measured during May 1996. In general, the difference between the predicted and measured settlement was less than 20%.

Using the PFE model as a base, the two-dimensional groundwater level within the altered andesite, pumiceous ignimbrite and ash layers (initially estimated / interpreted and considered the most variable set of input data) was adjusted to reduce the difference between predicted and measured settlement. This model is referred to as the Refined Mohr-Coulomb Finite Element Model (RM-CFE model).

The RM-CFE model predicted settlements that were generally within 8mm (approximately 10%) of that actually measured.

Finally, more sophisticated material models, that have the ability to model inelastic consolidation and rebound of soil and rock, were constructed. Using the RM-CFE model as the base, a Preliminary Advanced Finite Element model (PAFE model) was developed using advanced mathematical models to model soil and rock behaviour.

Both the RM-CFE model, and the RAFE model, were used to predict maximum settlement due to mine dewatering. This prediction was made by feeding the predicted maximum groundwater draw-down profiles into the finite element models. In general, the magnitude and pattern of settlement predicted by the two models was very similar.

The main difference between the two finite element models was the prediction of settlement within 100 metres of the pit crest. The RM-CFE model predicted approximately 300mm of movement at the pit crest while the RAFE model predicted approximately 150mm. Within Waihi Township, both models predicted approximately 100mm of settlement at a distance 150 metres from the pit crest reducing to 10mm at a distance 1100 metres from the crest of the pit. The results of the finite element modelling are summarised in Table 4.

Node Number	Distance Between the Node & Pit Crest (m)	Estimate of Maximum Settlement using the RM-CFE Model (mm)	Estimate of Maximum Settlement using the RAFE Model (mm)	Estimate of Rebound using the RAFE Model (mm)	Estimated Long Term Settlement of the Ground Surface (mm)
629	0	301	153	34	267
663	50	196	122	8	188
697	100	182	114	12	170
731	150	105	95	18	87
765	200	88	81	25	63
799	220	82	79	27	55
833	260	85	86	38	47
867	320	86	81	47	39
901	330	92	84	51	41
935	450	80	72	37	43
969	480	75	68	32	43
1003	520	67	62	27	40
1037	700	37	40	20	17
1071	800	28	31	14	14
1105	825	27	30	13	14
1139	975	17	23	4	13
1137	1075	10	11	4	6
1207	1125	2	2	0	2
1241	1313	>1	>1	0	>1
1275	1500	>1	>1	0	>1
1309	1600	0	0	0	0
1343	1700	0	0	0	0
1377	1800	0	0	0	0
1411	1900	0	0	0	0
1445	2000	0	0	0	0

Table 4. Predicted Settlement and Rebound

A prediction of ground surface rebound due to future recovery of the groundwater levels was made using the RAFE model. The RAFE model was used to predict rebound as the RM-CFE model does not have the ability to directly model inelastic rebound.

An estimate of the nett long-term ground surface settlement was made by subtracting tne predicted rebound from the maximum settlement.

Due to a lack of survey data near the pit crest, and variability of the fill layer, a detailed knowledge of the site, and engineering judgement, was relied upon to select the finite element model which gave the best prediction of settlement near the pit crest.

Discussions with the WGM surveyors suggested that the predictions made by the RM-CFE model were probably more realistic than those made by the RAFE model. For this reason, the predictions of long-term settlement presented in Table 4, are made using the RM-CFE model.

Upon completion of the geotechnical modelling, a comparison of the predictions made by the ROD, RAFE and RM-CFM models was made.

Predictions made using the refined finite element and one dimensional models were of the same order, and were generally within 20% of each other. The finite element models usually predicted less movement of the ground surface than the one dimensional models.

10.0 CONCLUSIONS

1) The magnitude of dewatering related settlement is low compared to that measured at other sites in New Zealand and the United States. Monitoring has shown that dewatering of the Martha Pit has not adversely affected buildings or services within Waihi Township.

2) The geology and hydrogeology of Martha Mine and Waihi Township is extremely complex with closely inter-bedded layers, confined hydrogeologic systems and perched water tables.

3) The measured settlement, to date, has not been consistent, or symmetrical, around the Martha pit. The complex geology surrounding the pit is considered to be the main reason for this.

4) Settlements, in each of the seven settlement zones identified to date, are attributed to a combination of

five processes of consolidation. Only one of the five processes of consolidation is directly related to current mine dewatering.

5) Graphs showing actual settlement against time indicate a progressive, staged settlement that occurs as the pit geometry is altered, exposing the younger geologic deposits in the Martha pit wall at a lower elevation. This allows dewatering of the younger deposits to develop to the level of the units exposure in the pit wall.

6) To date, settlements have occurred rapidly, and many of the younger deposits exhibit a near-elastic response to changes in the groundwater level. These movements are expected to be partially recovered, due to rebound, as the pit-lake reaches final level. Full recovery of settlement is not expected due to the altered groundwater regime and unrecoverable consolidation of the affected geologic units.

7) Acceptable levels of differential settlement and tilt are predicted by the finite element models.

8) Understanding of the following was essential for the successful prediction of settlement at Waihi:

- Identification and quantification of all the causes of settlement present in Waihi.
- Thorough research of historic mine features, both above and below the ground surface, and property use histories.
- Detailed understanding of the geology and hydrogeology of Waihi.
- Detailed understanding of the soil and rock geotechnical characteristics.
- Excavation and dewatering history of the Martha Pit.
- A summary of the measured settlements.
- Historic Rainfall and weather patterns for Waihi Township.
- Most importantly, how all the above were related, or connected, to each other.

9) Due to the complexity of the issues, it was imperative that thorough field observations and note keeping was undertaken. An understanding of the problem took several months to develop, and seemingly unimportant details were often later found to be a crucial component of the problem.

10) Finite element modelling proved essential when estimating differential settlement and tilt.

11) Engineering models are only as reliable as their input data.

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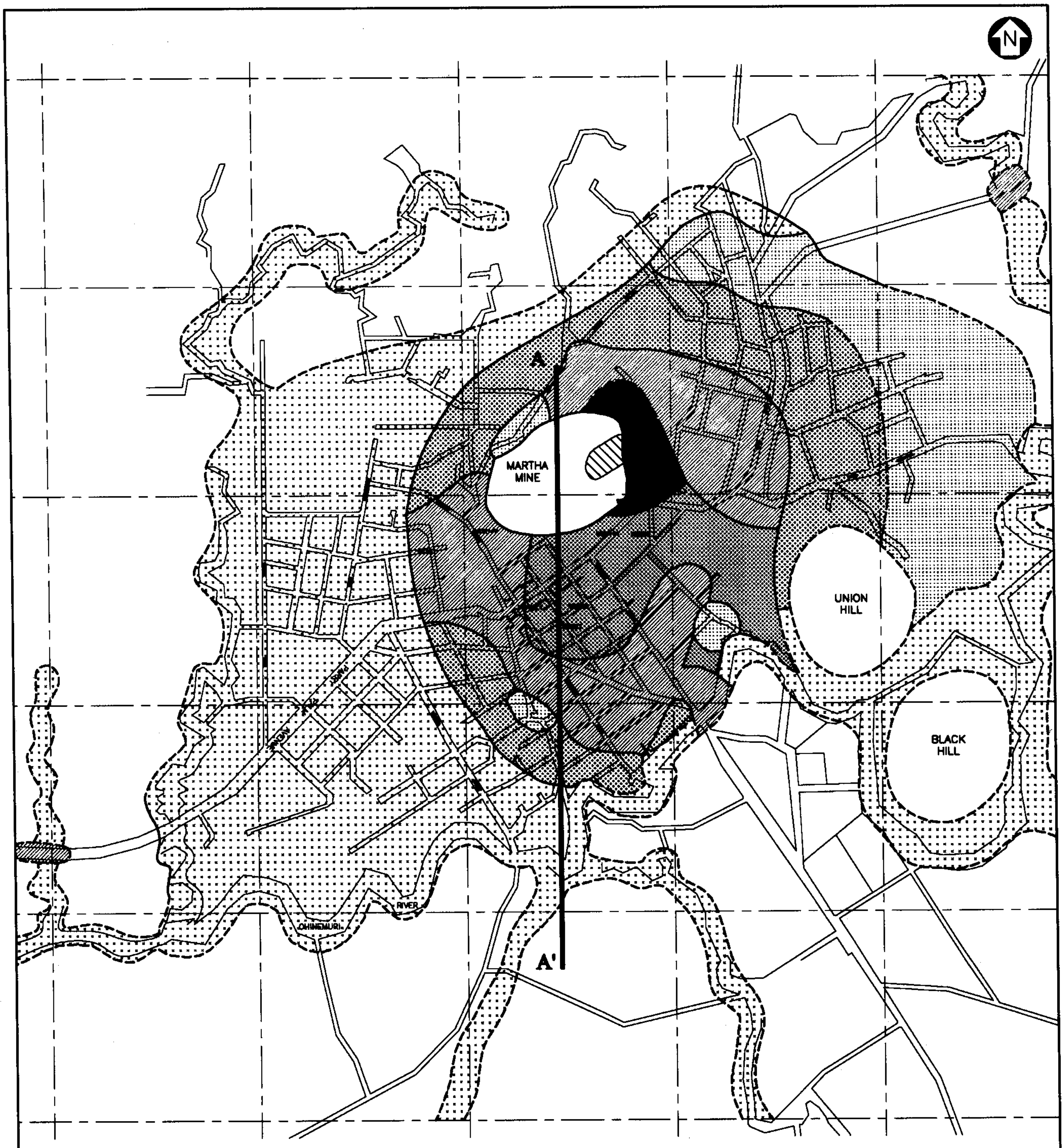

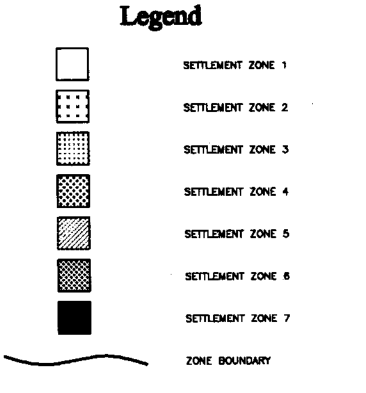

Summary of Measured Settlement

ZONE	AVERAGE MEASURED CONSOLIDATION SETTLEMENT (mm) FROM DECEMBER 1988 TO DECEMBER 1996.	AVERAGE OBSERVED FLUCTUATION IN GROUND SURFACE (mm)	LIKELY CAUSES OF OBSERVED SETTLEMENT (REFER TO SECTION 6)
1	6	±3	i, ii, iii, iv
2	0	±10	i, ii, iii, iv
3	11	±5	i, ii, iii, iv,v
4	24	±8	i, ii, iii, iv,v
5	48	±9	i, ii, iii, iv,v
6	63	±8	i, ii, iii, iv,v
7	106	±9	i, ii, iii, iv,v

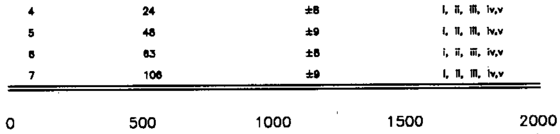

FIGURE 3
Plan of Settlement Zones
(As at December 1996)

Expansive Clays
Translating AS2870 into NZS3604

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ABSTRACT

The design of one and two-storey structures on sites with expansive clays in Australia with respect to residential (and to some extent commercial and industrial) construction has largely been carried out in accordance with Australia Standard AS2870 "Residential Slabs and Footings" since its inception in 1986. This paper discusses the philosophy of the calculations and design methods. Some worked examples of the estimation of the characteristic surface movements due to reactive clays are included. The required research areas for "fine-tuning" the methods for adoption to New Zealand conditions are outlined. Common errors in details for footings for Auckland clay sites are noted.

INTRODUCTION

Definition:- Expansive Clays – soils that change volume in response to moisture changes. These clays are also known as Reactive clays.

The main factors influencing the effects of expansive clays are as follows:

- The potential of the clay minerals within the soil to change volume in response to moisture content variations (clay reactivity)
- The depth of cracking below the ground surface (cracked zone)
- The depth of moisture content variation (including the soil suction profile, and considering the effect of vegetation)
- The soil and rock profile, including variations of reactivity within the profile and the depth to rock
- The depth to standing groundwater

Moisture changes in the soils occur for the following reasons–

- Seasonal climatic variations
- Artificial development
- Biological (typically trees and bushes) activities

The ground movements caused by expansive clays is three-dimensional, and may cause damage to structures such as houses, low-rise buildings, walls, buried services and pavements. These structures would all be regarded as lightly loaded, however it is worth noting that the swelling pressures associated with expansive clays may be several hundred kPa, depending on the degree of confinement.

The costs of damage caused by expansive soils has been documented in the UK (Freeman et al. (1973)) and in the United States (Jones & Holtz (1973) and Krohn & Slossen (1980)).

Some experts, however, such as Dr Paul Walsh, argue that the cost of foundation failures is insignificant. This is true in so far as buildings are rarely condemned due to inadequate foundations. Walsh suggests treatment of external cracking to brickwork by growing ivy as cover, and similarly to treat internal damage by such means as wood panelling. Ultimately, the homeowner determines how the aesthetics of the damage should be treated. More often than not, the homeowner is not satisfied with the knowledge that the most expensive item that he/she will ever buy is, and will remain, damaged.

Within the last decade or so in Australia, the most common footing systems for new homes have been slabs of various construction (raft slabs, waffle pods, piled slabs etc). The move towards slabs was driven by market forces. The consumer wanted concrete floors, and the feeling of security in the "knowledge" that a slab "ties everything together so that nothing moves". Engineers know differently, of course, but it is often difficult to educate the public, particularly in the face of aggressive marketing by builders.

The construction of houses also moved away from flexible walls (eg weatherboard) and is now almost entirely either brick veneer or cavity brick. This trend is apparently similar to the New Zealand experience.

Because of the concern about reactive soils, much research was conducted, including that of Walsh, and the Australian Standard AS 2870 "Residential Slabs and Footings" was published in 1986. This standard has been rewritten twice since, and the latest version was published in 1996. The New Zealand Standard NZS 3604 will soon incorporate AS 2870. AS 2870 introduced the concept of site classification, whereby the selection of standard footing designs was available, or else footings could be designed according to engineering principles.

SITE CLASSIFICATION

The various classes of sites are defined as follows:

Class	Foundation
A	Most sand and rock sites with little or no ground movement from moisture changes
S	Slightly reactive clay sites with only slight ground movement from moisture changes.
M	Moderately reactive clay or silt sites, which can experience moderate ground movement from moisture changes.
H	Highly reactive clay sites, which can experience high ground movement from moisture changes.
E	Extremely reactive sites, which can experience extreme ground movement from moisture changes.
A to P	Filled Sites
P	Sites which include soft soils, such as soft clay or silt or loose sands; landslip; mine subsidence; collapsing soils subject to erosion; reactive sites subject to abnormal moisture conditions or sites which cannot be classified otherwise.

Table 1. Site Classes (from AS 2870)

For classes other than P "Problem Sites", the site classification is determined in accordance with AS 2870 by one of the following methods:

A. Identification of the soil profile and either –

1. Established data on the performance of houses on that soil profile; or
2. Interpretation of the current performance of existing buildings on the soil profile

B. Estimation of the characteristic free surface movement value (y_s)

SOIL PROFILE

Based on the New Zealand Geological Survey Map of Auckland, the typical rocks that are likely to be encountered are limited to clastic sedimentary rocks (eg siltstone, sandstone, mudstone and conglomerate) and volcanic rocks (basalt, andesite, rhyolite, dacite and diorite). Alluvial soils are commonly found clays and silts, with some peat.

It is likely that adopting the values for Australian conditions will not initially be appropriate for New Zealand conditions. However, it could be a good basis from which to start, and then to review as local data becomes researched and incorporated. If it is considered that the climate in the Auckland area of New Zealand is likely to be similar to the wet temperate areas of Victoria, then most sites would probably range from Class S, Slightly Reactive to Class H, Highly Reactive, as outlined below:

Soil Profiles	Class
Basaltic Clays [1]	
≤0.6m depth of clay	S to M
>0.6m depth of clay	H
Predominantly gravelly clay	M
Non-Basaltic Residual Clays [2]	
≤0.6m depth of clay	S
>0.6m depth of clay	M
Quaternary Alluvials and Tertiary Sediments [3]	
Where predominantly silts or sands overlie clays	
≤ 0.6m silts or sands overlying clays	M
>0.6≤1m silts or sands overlying clays	S to M
>1m silts or sands overlying clays	A to S
Interbedded silts, sands and clay mixtures	
(assess on the basis of total depth of clay over 1.8m), ie	
≤0.6m total depth of clay	S
>0.6m total depth of clay	M

Table 2. Sample Classifications

(Adapted from AS 2870 Appendix D "Classification Based On Typical Profiles – Victoria"

Notes:

(1) Including pyroclastics and residual and alluvial clays derived from basaltic and similar volcanic rocks

(2) Including residual clays derived from sedimentary, metamorphic and granitic rocks

(3) Including delta, dune, lake, stream, colluvial and wind-laid (æolian) deposits

The risk in adopting the above is that some non-basaltic clays (eg perhaps residual soils derived from the Waitemata Group) may be highly reactive, whereas non-basaltic clays in Victoria may not have expansive clay mineralogy in their composition.

FOOTING PERFORMANCE

The provisions of AS 2870 state that site classification based on soil profile identification must be accompanied by some engineering judgement of the performance of existing buildings. This is carried out according to the following table:

Wall Construction	Performance of walls of existing buildings on lightly stiffened strip footings or slabs on ground	Primary Classification of Site
Clad Frame	Buildings with differential movements, *d* (lowest to highest points on perimeter of building)	
	$d \leq 15$mm	S
	$15 < d \leq 30$mm	M
	$30 < d \leq 50$mm	H
	$d > 50$mm	E
Masonry (veneer or full)	Damage Category:	
	0 to 1	S to M
	Often 1, rarely 2	M to H
	Often 1 or 2, rarely 3	H
	Often 3 or more severe and area usually well known for damage to houses and structures	E

Table 3. Classification By Interpretation of Footing Performance (from AS 2870)

NB. The damage categories referred to in the above table are defined in AS 2870 Appendix C.

ENGINEERING ANALYSES

If the above methods only are used, then there is a risk of incorrect classification (often due to the site classification being site-specific)

Hence, there is an alternative method to simply determining the depth of clay and reading a chart (ie we can use some basic engineering rather than using an "expert system"). This is the site classification based on the characteristic free surface movement.

Surface Movement	Site Classification
$0\text{mm} < y_s \leq 20\text{mm}$	S
$20\text{mm} < y_s \leq 40\text{mm}$	M
$40\text{mm} < y_s \leq 70\text{mm}$	H
$y_s > 70\text{mm}$	E

Table 4. Classification by Characteristic Surface Movement (from AS 2870)

This method has been proven by research at the University of Newcastle to be a more accurate approach to site classification, with the assessed y_s value exceeding measured seasonal movements by about 25 - 30% (Delaney, Allman, Smith and Sloan). It can also be used to estimate the contribution of clay reactivity to damage to buildings.

The y_s value is calculated by integrating the incremental contributions to surface movement over the depth of soil within which moisture content variation occurs. This depth, H_s, could be say, 1.2m, 1.5m, 1.8m or 2.0m.

It is one factor that will need to be resolved by New Zealand geotechnical professionals. The others are the depth of shrinkage cracking within the soils and the range of suction stress changes in the near-surface soils, and whether a triangular distribution of suction stress changes (as assumed for Australia, and supported by research (Delaney and Allman) is appropriate for New Zealand. It is possible that the Thornthwaite Index (Thornthwaite, 1948) could be used to indicate the design depth of suction change.

The formula that has been developed is as follows:

From Appendix F, AS 2870

$y_s = 0.01\, I_{pt}\, \Delta u \Delta h$

where y_s = the characteristic free surface movement value (5% chance of exceedance)

$I_{pt} = \alpha\, I_{ps}$

where α = confinement factor as follows:

- In the cracked zone, $\alpha = 1$
- In the uncracked zone, $\alpha = 2.0 - z/5$

The philosophy of the confinement factor is that the instability index is not constant throughout the depth of clay in-situ, even if its reactivity potential is constant (which it would unlikely be, in any instance) due to overburden stress, the suction range and the degree of lateral restraint (affected by the cracking).

The crack depth is defined as the zone in which predominantly vertical seasonal cracking occurs

As mentioned above, triangular distribution of moisture content variation is usually assumed. It may be rectangular, however, under conditions where the depth of cracking extends to the rock surface, or where trees are significantly drying the soil throughout the soil profile.

Typical values of moisture in terms of the potential change in the log unit of soil suction stress (U) range from U = 0 at H_s to U_0 = 1.2 – 2.0pF at the surface. The maximum value of U_0 is about 4, as it is at that value that vegetation becomes so distressed as to wilt and die. U_0 is another variable worthy of New Zealand research.

U_0 and H_s are dependent on the environment, including climate and vegetation. They are lower where the climate comprises rainfall matching the potential seasonal evaporation and transpiration.

The clay reactivity, I_{ps}, is measured using a shrink-swell test, as described by AS1289.7.1.1. This simple test measures the reactivity directly from an "undisturbed" sample of soil. The result is given as a shrink-swell index, I_{ss}, which may be taken to be equivalent to the Instability Index, I_{ps}.

It has been the experience of geotechnical engineers in Australia that the use of Atterberg Limits (although allowed by AS 2870) has not provided reliable correlations with the shrink-swell index that is derived from direct measurement.

Similarly, the use of visual-tactile identification of the soil, even by an engineer or engineering geologist with appropriate expertise and local experience is unreliable. Perhaps one reason for this is that the identification of clay minerals is usually beyond the capabilities of the naked eye. Unless clay minerals can be identified based on colour (eg chlorite), they may be difficult to distinguish. For example, Kaolinite and Dickite are almost identical, and can be differentiated only by X-Ray Diffraction or Scanning Electron Microscope, both of these methods are tools beyond those normally carried by the average geotechnical engineer or soil testing laboratory.

FIELD SAMPLING

The "undisturbed" sample that is necessary for the shrink-swell test is obtained by pushing a 50mm diameter thin-walled steel tube (eg exhaust pipe tube with a machined cutting edge) into the soil for a depth of at least 175mm. The sampling must not cause crushing of the sample within the tube (eg for a tube 400mm long it would be advisable not to sample more than 300mm of soil).

Typically, the sampling depths would be 0.5-0.8m, and 1.0-1.3m, depending on the soil profile, proposed earthworks and other considerations.

TESTING FREQUENCY

Until the reactivity of a geological formation is known, the testing frequency should be at least one shrink-swell test per lot for a single house development, and one test per 3 lots in a subdivision of 9 or more lots.

The shrink-swell index might range from 0 to 10% change in volume per change in the log unit of soil suction stress. Typical values in the Sydney - Newcastle, NSW, region would be in the order of 2 – 5%, and these would relate to site classifications of moderately to highly reactive.

Variables	Range	Typical Values
U_0	1.2 – 2pF	1.5pF
H_s	1.2 – 2m	1.5m
Crack Depth	0.5 – 1m	0.5m
I_{ps}	0–10% per ΔpF	2–5% per ΔpF

Table 5. Summary of Variables

CALCULATIONS

The calculations are simple enough to be done by hand, but not so simple to be carried out mentally. Notwithstanding the above, it would be most common for the calculations to be evaluated using a personal computer, for example, either on a spreadsheet application or on a purpose-written application (eg BASIC program).

For numerical analysis, I suggest that the smallest increment of depth should be 0.1m. There is little point in being carried away with the limitless possibilities of computers when the sensitivity of the calculations to the assumed parameters is so high (cf GIGO principles).

Consider the following situation, where:

- The soil profile is a uniform clay
- the depth of soil to be considered is 1.5m
- Δ soil suction stress at the surface is 1.5pF
- the Iss is 2.0% per ΔpF
- the crack depth is 0.5m

We consider the contribution to the surface movement of the upper 0.5m (cracked zone) where $\alpha = 1$

At 0m depth, U is 1.5
At 0.5m depth, U is 1
The average U between 0 and 0.5m depth is 1.25ΔpF
The average $I_{pt} = I_{ps} \times (\alpha) = 2$

y_i = 10 x 1.25 x 1x 2 x 0.5m = 12.5mm

And the contribution to the surface movement of the lower 1m (uncracked zone) where $\alpha = 2-(z/5)$

At 0.5m depth, u = 1
At 1.5m depth, u=0
Average U = 0.5

At 0.5m depth, $\alpha = 1.7$
At 1.5m depth, $\alpha = 1.9$
Average $\alpha = 1.8$
Average $I_{pt} = 3.6$

y_i = 10 x 3.6 x 0.5 x 1m = 18mm

y_s = 12.5 + 18 = 30.5mm

20mm < y_s < 40mm

Therefore the site would be classified as Class M, Moderately Reactive.

Normally, there is some clay in the topsoil on a site. The reactivity of the topsoil is rarely tested, but is typically assumed as approximately 50% of the reactivity of the underlying clays.

It is important to remember that the site classification is based on the natural condition of the site. If the site has already been developed, then the site classification cannot be determined strictly in accordance with the standard with respect to clay reactivity. It can, however, be expressed as a matter of engineering judgement.

Obviously, altering the soil profile by cutting and filling will affect the site classification. Considering the above example, what depth of filling with non-reactive soil would be required to provide justification to reconsider the site classification to Class S?

It should be noted here that unless the filling extends beyond the building area by at least 1.5m, the protection from moisture content variations that the filling offers is not attained.

The academic solution to the above problem is one involving trial and error, solving for y_s<20mm. The engineer will see at a glance that 18mm of the contribution to the surface movement may be attributed to the zone of soil below 0.5 m depth, and hence since 18mm <20mm then 0.5m is the answer.

It is emphasised that although it is possible to express the free surface movement value to several significant figures, the order of accuracy dictates that it should be expressed as a range of 5 mm eg if the calculation arrives at 47.35mm - "the site has been classified as being Highly Reactive, Class H, with a characteristic free surface movement of 45 - 50mm".

As with many facets of engineering design, it is possible for different engineers to arrive at different solutions. In the relatively simple exercise of site classification, it is possible for different engineers to be armed with the same set of data, and apply their own interpretation to the variables to arrive at different values of y_s such that those values might lie on either side of a boundary of site classifications.

The method of assessing the y_s value, however, is also useful in judging the contribution to the damage of structures due to clay reactivity. Remembering that the ys value reflects about 75% of the likely seasonal movement (as opposed to 5% chance of exceedance over in 50 years) (Delaney & Allman), then movement of the foundations of the damaged structure can be estimated with respect to the expected movements due to clay reactivity.

For instance, if there is a large settlement of a corner of a house adjacent to a tree, then most engineers would likely regard that tree as the most obvious cause of the settlement. The assessment of the free surface movement value is a tool that may be used to quantify the expected movement due to the tree, by carrying out a sensitivity analysis of the soil profile to:

- variations in the suction profile and
- variations in the reactivity profile

An obvious but often overlooked fact is that if there are no changes in the moisture content of the soil, then there will be no corresponding change in the volume of the soil due to expansive clays.

A common feature that the author has noted amongst building design professionals in Auckland is the following:

"To overcome the effects of expansive clays, Either found the footings at a depth of 600mm or more, or else excavate to 600mm and backfill with scoria to 300mm depth."

Founding footings at a deeper depth is a valid option. However, the folly of the latter option should be apparent when it is considered that the scoria, being a relatively permeable (though of course non-expansive) material may contribute to the acceleration of wetting and drying of the underlying soils, and perhaps worsening the movement of the site.

AS 2870 provides standard details for footings. Design outside the details is permitted subject to design by engineering principles. The lightest of these designs for clay sites would be as described for Class S, eg a strip footing on a Class S site for a masonry veneer house would be 400 thick, and 300mm wide, founded at least 475mm deep, with 3-8TM reinforcement. The same type of house on a Class H site would have a footing 850mm thick, 300mm wide, founded at least 925mm deep with 3-Y16 reinforcement.

The standard also refers to the need for masonry articulation as outlined in the Cement and Concrete Association Technical Note TN 61 "Articulated Walling".

The homeowners are also cited in the standard as being responsible for the care and maintenance of the foundation soils and footings. A brochure produced by the Commonwealth Scientific and Research Organisation (CSIRO), Sheet 10-91 "Guide to Homeowners on Foundation Maintenance and Footing Performance" is another document referenced in the standard. It details in layman's terms the importance of careful watering and thoughtful gardening, including the placement of large trees relative to the house. It is included in most geotechnical engineer's site classification reports, and with most structural engineer's designs.

CONCLUSIONS

The application of the principles of AS 2870 to NZS 3604 should be welcomed by engineers, builders and homeowners as an advancement in local geotechnical and building practice.

The local environmental conditions and geotechnical parameters that are critical to the correct evaluation of the design surface movement should be researched as a matter of urgency.

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Australia and South Eastern Queensland. Australian Geomechanics News Number 33, Part 3

ABOUT THE AUTHOR

Bruce Grayson graduated from the University of Newcastle, NSW in 1988 with a Bachelor of Engineering (Civil Engineering) including the formal components of 1st and 2nd year Geology, and with a final year thesis 'On the Stability of Deep Road Cuttings in Triassic Sandstone' that was co-supervised by Dr Ian Moore and Professor Konrad H.R. Moelle. Bruce worked for Coffey and Partners Pty Ltd (working on such projects as the Sydney Harbour Tunnel) and then Douglas and Partners Pty Ltd (including some time in PNG at the Ok Tedi gold and copper mine) as a consulting geotechnical engineer prior to founding his own business in 1993 (concentrating on residential, small commercial and industrial developments). Bruce joined Soil & Rock Consultants in Auckland this year, and now manages their Albany office.

His technical interests include the quantification of clay reactivity, pavement design, slope stability and earthworks construction. He is a past Chairman of the Mine Subsidence Technological Society, and has been on the Boards of the Civil Branch of the IEAust (Newcastle Division) and the Australian Geomechanics Society (Newcastle Chapter).

Broms and Beyond

MJ Pender
University of Auckland

The ideas explained in this brief paper were kicked-off in an after dinner stroll one summer evening. Not far from my home I noticed a building site where a pole retaining wall was in the course of construction. It occurred to me to do some quick measurements and consider the wall from the viewpoint of the Broms[1] formula for the lateral capacity of poles embedded in clay.

The Broms assessment of the lateral capacity of piles embedded in clay is well known, dating from his paper in 1964. It is attractive in that it is based on a very simple idea about the ultimate lateral pressure between the pile shaft and the surrounding clay. There are two cases for consideration: the so-called short pile in which the maximum pile shaft moment is less than the section capacity and hence the ultimate capacity of the pile is controlled by the soil strength, and the long pile case in which the maximum moment in the pile shaft is limited by the section capacity. These two cases are illustrated in Figure 1. The lateral pile capacity for the two cases are given by the following two equations:

$$H_{Broms\_short} = 9s_uD_s\left[\sqrt{(L+2f+f_o)^2+(L-f_o)^2}-(L+2f+f_o)\right] \quad (1)$$

$$H_{Broms\_long} = 9s_uD_s\left[\sqrt{(f+f_o)^2+\frac{2M_y}{9s_uD_s}}-(f+f_o)\right] \quad (2)$$

It is apparent that we are here involved in the short term or undrained lateral capacity of the piles as the soil strength parameter is the undrained shear strength s_u. Now at the site of the wall viewed on my evening perambulations I did not have the soil strength but a reasonable guess at s_u for the Waitemata residual clay is not too difficult. Plugging this value into the Brom's formula with appropriate assumptions about the earth pressures each pole in the wall would support fairly quickly lead to the conclusion that the wall was quite inadequate! Furthermore looking at the TRADA[2] document, which has tables of possible designs, indicated that this wall should be adequate!

One does not have to look far into the Broms method to appreciate where the problem lies. Bent Broms suggested an idealised pressure distribution for the clay against the pile shaft. At the ultimate lateral capacity the pressure between the clay and pile shaft is assumed to be $9s_u$. However, the first 1.5 pile diameters are assumed to provide zero lateral resistance. In calculating the lateral capacity of piles it quickly becomes apparent that this 1.5 diameter assumption is very expensive in that it increases the maximum pile shaft moment. Other writers have suggested that the 1.5 diameters is not appropriate and suggested instead fixed lengths. One suggestion has been 600 mm, but this was intended for piles associated with projects more substantial than a pole wall.

Late last year I was in Singapore and met Professor Broms at Nanyang Technological University. We discussed this problem and his suggestion was to evaluate the ultimate lateral capacity of the pole using a more realistic pressure distribution. I did some initial work on this, assuming a linear increase in the ultimate lateral pressure with depth, in the plane on the way back from Singapore. The algebra was tedious – so Broms had chosen a rather clever idealisation that lead to the simple equations above. A former masters student[3] had done some work on pressure distributions against laterally loaded pile shafts. In reviewing literature he found that a lateral resistance of $5s_u$ at the ground surface with a linear increase to $12s_u$ at a depth of 3 diameters, resulted in good modelling of a number of back analysed case studies;

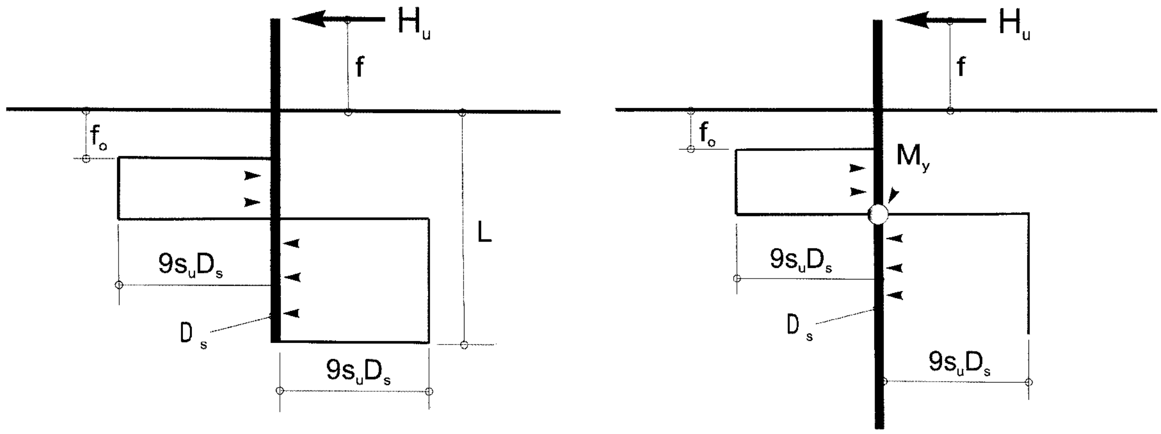

Figure 1. Brom's idealised pressure distribution for a pile embedded in clay. Left - short pile, right – long pile. (Note the parameter f_o which Broms sets to $1.5D_s$.)

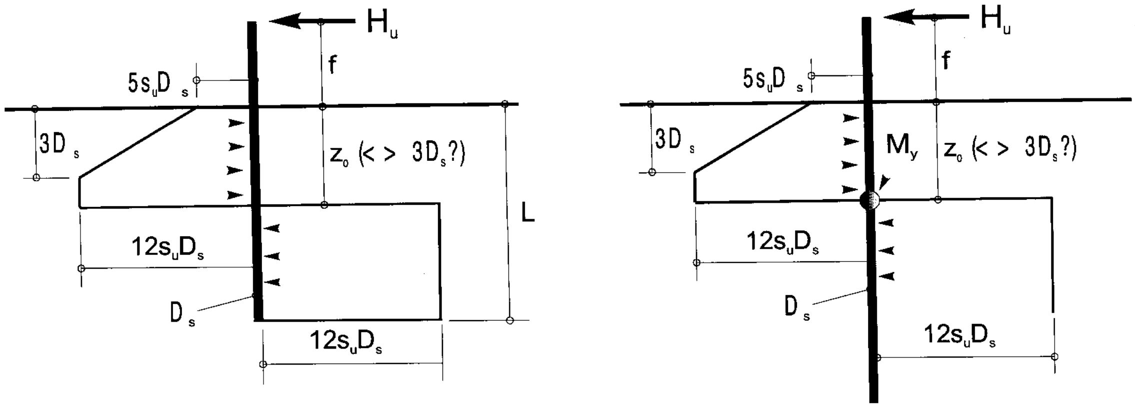

Figure 2 Alternative idealised pressure distribution for a pile embedded in clay. Left - short pile, right – long pile.

this work was done with a specially written finite element programme. I then tried to develop lateral capacity equations for the distribution in Fig. 2 but the algebra turned out to be formidable.

Given this situation I decided on a new tack. Why not evaluate numerically the lateral capacity of the piles for the more complex distribution and compare that with the capacity from the Broms formula with various values for f_o. In other words investigate the possibility that using some value of f_o in the Broms formula which would give a good match to capacity is obtained with the more complex, but more realistic, lateral pressure distribution. The numerical calculation is in fact quite straight forward as one writes two equilibrium equations for the pile – horizontal force and moment equilibrium – and solves them simultaneously (I used my favourite general purpose computational tool). The results are plotted in Figure 3 which shows that for both the short pile and the long pile case using f_o = 250 mm in the Broms formula gives very good matching. Note that the same value of f_o holds for short as for long piles. The results in Figure 3 were calculated for $s_u = 50$ kPa, an embedment diameter 150 mm greater than the diameter of the embedded pile, and an embedded length of pile shaft of 4 times the embedment diameter. In the left hand part of Fig. 3 the dashed lines are for z_o greater than and less than $3D_s$, in the right hand part the dashed line is for z_o less than $3D_s$.

The results in Fig. 3 were derived for timber piles. The long pile calculations were repeated for circular reinforced concrete piles – 1% steel, f_c 30MPa, f_y 430MPa, and g 0.9 for which M_y is $1400D_p^3$ (NZ Reinforced Concrete Design Handbook) – with $\Phi_{reinforced\_concrete} = 0.85$. The same conclusion is reached, ie equation 2 with f_o = 250 mm gives a very close match to the capacity calculated using the Fig. 2 ultimate pressure distributions.

Returning to the pole wall which started me thinking about the ideas explained above. Doing the calculations with f_o = 250 mm and a scoria backfill with $\phi = 40$Y and $\delta = \phi$ indicates, as observed, that the wall as-built is satisfactory.

In conclusion then, these calculations show that the Broms assumption of an unsupported length of 1.5 pile diameters is too conservative for pole wall design. Using the ultimate lateral pressure distribution shown in Fig. 2, which has been verified by finite element back analyses of field tests on laterally loaded piles, it has been found that an unsupported length of 250 mm is appropriate for both the short and long pile cases.

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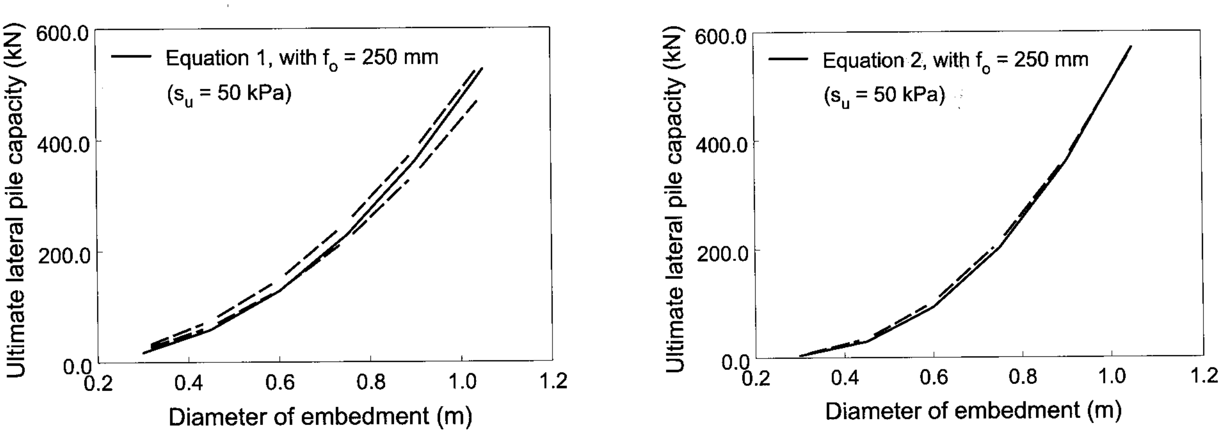

Figure 3. Pile lateral capacities calculated with the pressure distributions shown in Fig. 2. Left – short piles (dashed lines for z_o < and > $3D_s$), right – long piles (dashed $z_o < 3D_s$).

Use of the Acoustic Scanner for Geotechnical Investigations

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SUMMARY

The acoustic scanner (or acoustic televiewer) is a geophysical tool capable of providing oriented acoustic images of a drillhole wall. It is being increasingly used in geotechnical investigations to determine the orientation of rock mass defects. Acoustic scanning was recently carried out at Roxburgh Dam, New Zealand, where two 60 m deep cored holes were drilled in schist rock. The holes were scanned and the data processed using proprietary software. A detailed comparison was made between the drill core and scanner images. The images produced clearly showed the major rock mass defects present, enabling their true dip, azimuth, and approximate thickness to be determined. Acoustic scanning was subsequently used at the site to rapidly and cost effectively determine the presence and orientation of defects in 37 non-cored foundation drainholes.

Acoustic scanning has considerable potential for use on geotechnical projects where defect orientation is a prime objective. It may be used in conjunction with core drilling to provide high quality geotechnical data and with non-core drilling to provide cost effective spatial coverage or data "infill" between cored holes. In some situations the scanner may also be used to estimate in situ stress orientations from analysis of drillhole breakout.

1. INTRODUCTION

Acoustic scanning is a relatively new technique for obtaining *in situ* geotechnical information from acoustic images of a drillhole wall. This paper introduces the technique and presents an example of its use in schist terrain at Roxburgh Dam in New Zealand's South Island. This is followed by a more general discussion of the geotechnical applications and practical limitations of the scanner based on the author's experience at Roxburgh and several other sites.

2. THE ACOUSTIC SCANNER

The acoustic scanner (also known as the acoustic televiewer) is a wireline geophysical tool incorporating a rapidly rotating transducer, which emits short bursts of sound energy[1]. Originally developed for the petroleum industry, the scanner is now used increasingly in geotechnical investigations.

Each acoustic pulse is reflected off the borehole wall and its amplitude and travel time recorded as it returns to the tool. The amplitude (or strength) of the reflected signal provides an indication of the reflective properties of the wall rock, which in turn can be related to the strength and hardness of the rock. Defects containing crushed rock or gouge that is softer than the surrounding rock are thus readily identified, as are boundaries between lithologies of contrasting strength. The acoustic travel time, when suitably corrected for the sonic velocity of the drillhole fluid, provides a measure of the drillhole diameter, thereby allowing open joints, voids, caving and breakout to be determined. As the tool traverses the drillhole a continuous helical scan is formed. The basic operation of the tool is shown in Figure 1.

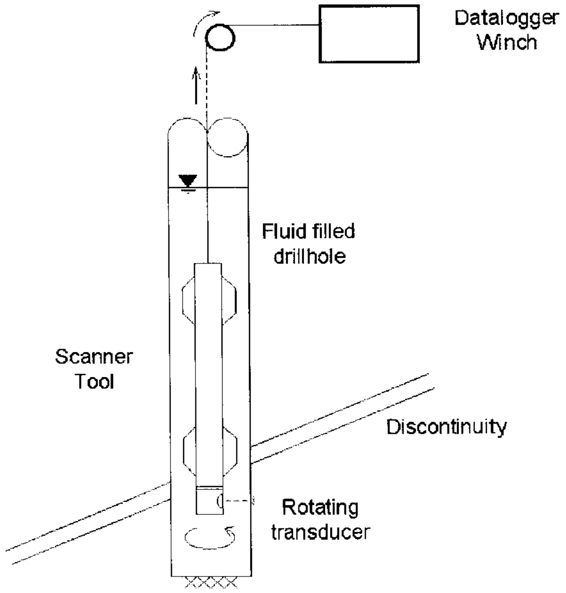

Figure 1. Operation of the acoustic scanner

The three dimensional orientation of the tool is recorded by a series of on-board magnetometers and accelerometers. In this manner the tool always knows exactly where it is in relation to the base of the hole. The amplitude and travel time data may therefore be imaged and presented in their correct orientations for later computer-based interpretation.
Data interpretation is carried out using various proprietary software packages, which allow the orientation (dip, dip direction) to be determined for each identified defect. The amplitude and travel time data are "imaged" using a gradational colour palette.

Interpretation is carried out on "unwrapped" 360° displays of the imaged drillhole wall. Planar features encountered in the drillhole appear as sinusoidal traces. A "best fit" sine curve is manually fitted to each defect trace and the orientation automatically calculated. The interpretation software also allows defect orientations to be exported as ASCII files for tabulation and presentation using stereoplotting software.

3. CASE STUDY- ROXBURGH DAM

3.1 Background

Roxburgh Dam is a concrete gravity structure located on the Clutha River in New Zealand's South Island. The dam has an installed generating capacity of 320 MW from 8 turbines and was completed in 1956. Although comprehensive as-built geological logs of the foundation exposure were made, further information on the engineering geology of the schist rock on which the dam is founded was recently considered desirable. To this end a staged investigation programme including cored drilling, groundwater instrumentation and downhole geophysics was carried out. A feature of the investigation was the use of acoustic scanning to obtain orientated data on rock mass defects from drillholes.

3.2 Geological Setting

Roxburgh Dam is situated in a broad northwest trending belt of the Otago Schist. The schist originates from quartzofeldspathic and volcanogenic sediments of Mesozoic-Paleozoic age, that have been metamorphosed to textural zone IV. At least four phases of deformation are recognised in the region, which remains seismically active. The rock mass in the vicinity of the dam exhibits a well-developed sub horizontal foliation and is moderately to widely jointed. It contains shears both parallel and oblique to foliation. Intact rock strengths of 50 to 200 MPa are typical.

3.3 Field Investigations and Results

Two diamond cored PQ/HQ size (122/95 mm diameter) drilholes were drilled on the left abutment of the dam in mid 1997 to install additional piezometers. The acoustic scanner was trialed in these holes and the results compared with the drill core and conventional density, sonic and caliper logs[2]. A typical section of scanner image from one of these holes is presented in Figure 2. Following data processing, the acoustic images were interpreted to determine the type and orientation of all planar features identified. The trial concluded:

- The scanner clearly identified all the major shears (both sub-horizontal and steeply dipping) present in the core.
- The scanner identified the majority of joints present in the core. Those joints identified in the drill core but not revealed by the scanner may have been tight *in situ* and therefore not differentiable from the adjacent wall rock.
- The orientation of all defects identified could be readily determined during subsequent data processing.
- Due to the geometric considerations the dip angle determined for steeply dipping defects is generally of greater precision than that for shallow dipping features.

Following the success of the trial, 37 foundation drainholes located within the lower dam inspection gallery were scanned during 1998[3]. These holes had originally been drilled during construction by diamond coring, although the cores were not retained.

Immediately prior to scanning, the holes were flushed with high-pressure water jets and several of them reamed and deepened using a small percussive rig. A "dummy" probe of the same dimensions as the scanner was used to determine the "scannable" depth of each hole, allowing an optimum investigation schedule to be developed. The scanning operation took place over 5 days during which 466 m of drillhole were scanned. Data interpretation was carried out on site in tandem with the data acquisition. This had the major advantage of allowing the investigation to be modified as it progressed. Additional holes were scanned in areas where data quality was poor or where features of particular interest were identified. The scanner equipment used for the drainholes was different from that used for the surface holes, primarily due to the headroom restrictions in the dam galleries.

In spite of the holes having been drilled over 40 years previously, a large number of rock defects were identified from the scanner images, and their orientation determined. As with the earlier trial, defect classification from the scanner images was somewhat subjective, and difficulty was occasionally experienced in differentiating open or infilled joints from thin foliation shears and crushed zones on the scanner images (i.e. it was not always possible to determine whether or not shear displacement had occurred along the defect). "Major" shears (i.e. greater than 20 mm thickness) were readily identifiable.

High quality, orientated data on rock defects were obtained from most of the drainholes. The quality of the acoustic scanner data appeared to be influenced by the condition of the drillhole wall. Overall, better results were obtained in diamond cored holes than holes redrilled by percussive means. Although different scanning equipment was used, data quality from drainholes that were not reamed was generally comparable with that of the cored surface holes.

The interpretation software allowed logs to be compiled for each drainhole showing the acoustic amplitude and travel time images, interpreted features and their classification and orientation, and drillhole azimuth and inclination. Consideration of the large amount of additional data gained from acoustic scanning at Roxburgh has contributed to a far more detailed understanding of rock mass properties at the dam site. It is unlikely that any other investigation tool would have given such detail and spatial coverage for the same cost.

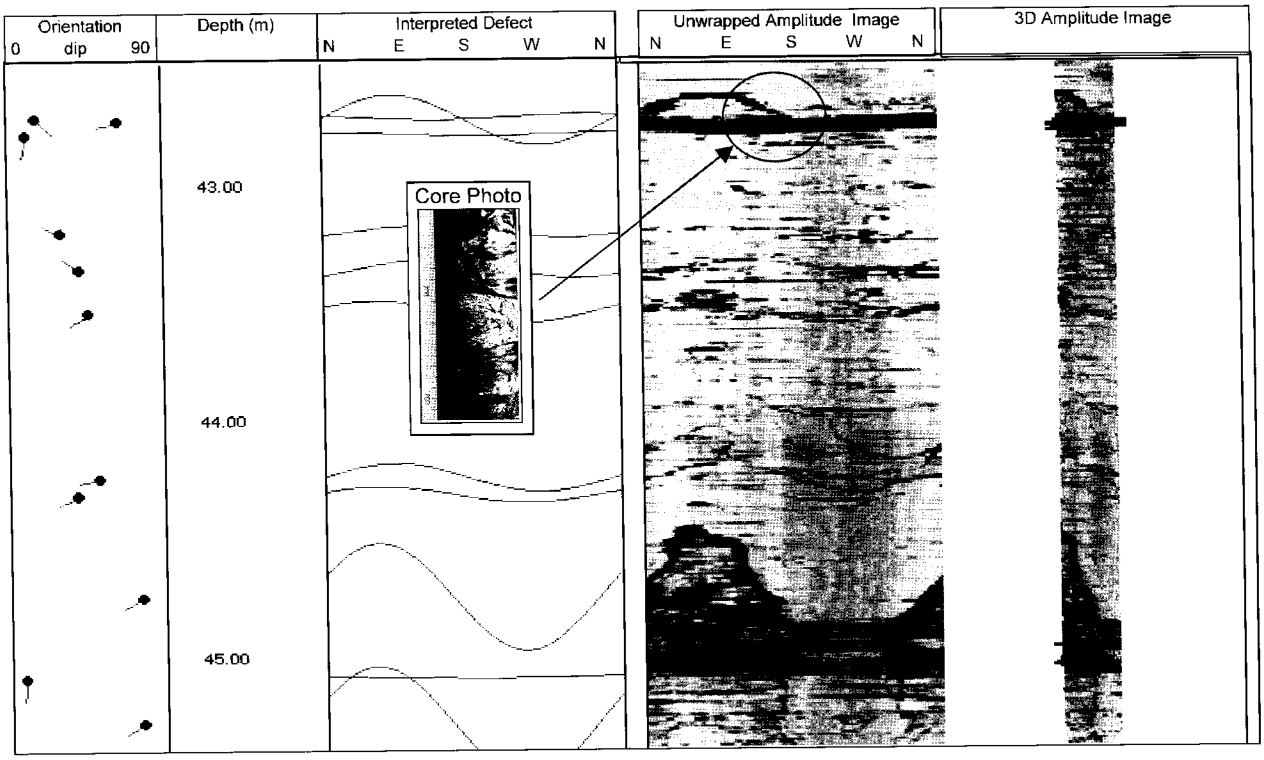

Figure 2. Example of Acoustic Scanner Data Interpretation

4. ISSUES ASSOCIATED WITH USE OF THE ACOUSTIC SCANNER

4.1 Cored Versus Non-Cored Drillholes

By providing orientated data the scanner represents a major advance over most conventional downhole geophysical methods.

The scanner is not regarded as a replacement for core investigation drilling. It does however have the potential to enhance the quality of geotechnical data obtained from both cored and non-cored holes.

Where highest quality orientated data is required the scanner may be used in conjunction with cored drilling. The two techniques are complementary and data from both may be combined on a single interpretive drillhole log for subsequent interpretation. The drillcore allows direct inspection and sampling of the strata encountered whilst the scanner provides defect orientations, 3D drillhole deviation data and additional information in zones of poor core recovery. The use of the scanner in this manner is more appropriate for projects where the orientation of rock mass defects can have major design and contractual implications. Examples include tunnels, cut slopes, deep excavations and landslide investigations.

The scanner may also be used to gain geotechnical information from non-cored holes drilled for other purposes. Examples include resource definition drilling in quarrying and mining, grouting holes, ground anchor holes, or as in the case of Roxburgh, drainage holes. Typically a large number of these holes are drilled rapidly and cheaply by open hole methods. Whilst the scanner data quality may be poorer than for cored holes, a wide spatial coverage of information on defect orientations and stratigraphy may be acquired at relatively low additional cost. Some "control" in the form of occasional cored holes is still desirable.

4.2 Drillhole Environment

There are a number of physical limitations on the drillhole environment in which the scanner may be used. These include:

- Drillhole fluid – the scanner requires a water or mud filled drillhole
- Drillhole diameter – the 75-mm diameter drainholes at Roxburgh were close to the lower limit of the equipment used.
- Headroom – the shortest tool used at Roxburgh was 2.6 m long. This precluded its use in some of the galleries
- Artesian groundwater/gassing - Data quality may be adversely affected by strong artesian groundwater flows or gas bubbles.
- Hole inclination - Vertical or steep downwardly inclined drillholes may be readily scanned using a conventional wireline winch. Lower angle and subhorizontal holes can be scanned in some cases, with the tool pushed into the hole with flexible fibreglass rods. Problems with tool

centralisation may also be experienced. Upwardly inclined holes cannot be scanned due to the requirement for a drillhole fluid.

- Drilling method - Diamond cored holes provide a smoother drillhole wall than do percussive or wash bored holes. Data quality may be adversely affected by excess rugosity of the drillhole wall.
- Hole conditions - In softer rocks, excessive caving or smearing on removal of casing may effect data quality. Poor hole stability may also place the tool at risk of jamming or other damage.
- Casing - The scanner will not work through drillhole casing, screens or liners.

Specific requirements differ slightly between geophysical contractors and will no doubt change as new equipment is developed.

4.3 Cost Effectiveness

Depending on the site location, mobilisation costs may be significant, but the actual data acquisition is relatively quick. Cost effectiveness therefore increases with the number of holes scanned in one visit. This, however requires holes to be left open and uncased. In some situations this may not be possible, or may involve additional drilling costs to re-visit each hole after scanning to complete installations. The timing of investigation drilling must be carefully considered to maximise the cost effectiveness of a scanning programme.

Data interpretation and presentation costs can be a significant component of the overall investigation cost. Some contractors provide a data interpretation service while others do not. Interpretation may be carried out by the client's engineering geologist, in which case the cost of software purchase, training, and a "learning curve", must be considered. A third alternative is to make use of a "third party" interpretation service.

4.4 Determination of In Situ Stress Orientation

The scanner is being increasingly used as a means of estimating the direction of principal *in situ* stresses due to its ability to determine the orientation of drillhole breakout. When *in situ* deviatoric stress is high, breakout (spalling of the drillhole wall) may preferentially occur in the drillhole wall perpendicular to the axis of principal horizontal stress (see Figure 3). The occurrence and orientation of breakout is readily determined from the travel time images produced by the scanner. Stress orientation determined in this way has been shown to correlate well with measurements using other techniques[4]. Techniques are also being developed to deduce *in situ* stress directions for drilling induced fracture patterns[5]. These techniques have application in the design of underground mines, caverns and tunnels where *in situ* stress directions have the potential to influence excavation and stability.

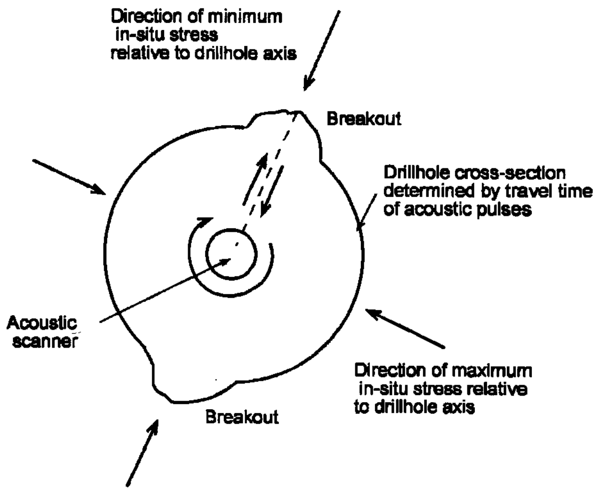

Figure 3. Determination of *In Situ* Stress Orientation from Drillhole Breakout

ACKNOWLEDGEMENTS

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Monitoring the Cut-Off Wall Performance of Upper Huia Dam

Alaa S. Ahmed-Zeki[1], Nicholas Logan[2], Wallace McQuarrie[3], and Raveen Jaduram[4]

Paper originally published at the ASCE/Geo-Institute Specialty Conference: "Performance Confirmation of Constructed Geotechnical Facilities", Amherst, Massachusetts, USA, 9-12 April 2000, and re-printed here with the permission of the American Society of Civil Engineer.

ABSTRACT

Upper Huia concrete gravity dam was constructed in Auckland-New Zealand, 70 years ago. The dam was not designed using present standards for uplift, although a concrete cut-off wall was constructed as a means to reduce the uplift pressures beneath the dam. This paper presents a description of the installation of vibrating wire piezometers to continually monitor the effectiveness of the foundation cut-off wall. The piezometer readings showed that the cut-off wall was providing the reduction in pore water pressures required for maintaining the stability of the dam.

BACKGROUND

Upper Huia Dam is a concrete gravity dam with a vertical upstream face and no curvature. It has a maximum height of 36.6 meters and a crest length of 166 meters, creating a lake having a storage capacity of about 2.44×10^6 m^3. Construction took place during the period 1926-1929. Located in the Waitakere Ranges west of Auckland (see Figure 1), it is one of Watercare Services Limited's 10 water supply dams in the Auckland region.

The dam is serviced by an uncontrolled overflow spillway consisting of seven weirs. It has a capacity of passing a flow with 1:100 recurrence, but the Probable Maximum Flood has been estimated to result in about 0.4 meters of overtopping.

A cut-off wall of about 0.9 m width was excavated beneath the upstream face, and along the entire length, of the dam. Old rails were reported to have been vertically placed at about 1 meter centers, at the cut-off wall/dam interface to improve the bond (tensile strength) of concrete. The wall was designed to be approximately 3 meters deep, but was extended to about 6.1 meters at the deepest section of the dam to intercept two water bearing seams encountered during excavation. The cut-off wall was extended laterally into the abutments to form wingwalls.

Safety Evaluation reviews undertaken for Upper Huia dam in 1990, 1991 (Riley, 1990, 1991) and 1998 (Woodward-Clyde, 1998) included structural analyses evaluating the behavior of the dam under different loading conditions. Both studies note that even though the dam's design does not comply with the present standards for uplift, the upstream concrete cut-off wall was constructed as a means of reducing seepage pressures beneath the dam. These reviews emphasized the significance of the cut-off wall on the overall stability of the dam and recommended the implementation of a monitoring program.

GEOLOGY OF THE DAM SITE

The geology of the strata underlying Upper Huia dam has been classified (Kermode, 1988, 1992; Riley, 1990, 1991) as breccia/conglomerete, interbedded with tuff. These are volcanogenic sediments known as Piha and Nihotupu Formations.

Piha Formation consists of indistinct beds of brown-grey, poorly sorted, gravel to boulder breccia-conglomerate consisting of angular to sub-rounded, very fine to medium grained basaltic andesite clasts in a sandy matrix of similar material. Fractures are commonly extremely widely spaced. At the dam site, this formation grades into, or interfingers with, Nihotupu Formation.

Nihotupu Formation consists of finer grained sediments of yellow-grey to grey, poorly sorted sandstones/siltstones (tuff) containing angular to sub-rounded grains of basaltic andesite. Fractures are typically widely spaced, while bedding thickness varies between thin to very thick, and is also lensoidal making beds discontinuous over relatively short distances.

The rock on which the dam was founded ranges from moderately weathered to unweathered.

BRIEF REVIEW OF STABILITY ANALYSIS

Evaluations of the dam stability were undertaken in accordance with the US Bureau of Reclamation and Federal Energy Regulatory Commission guidelines. Stresses within the dam body will not be discussed here, although the performed analyses have shown that the internal stresses should cause no concern under all loading conditions. The focus of this paper is on the stability of the dam against overturning moments and sliding.

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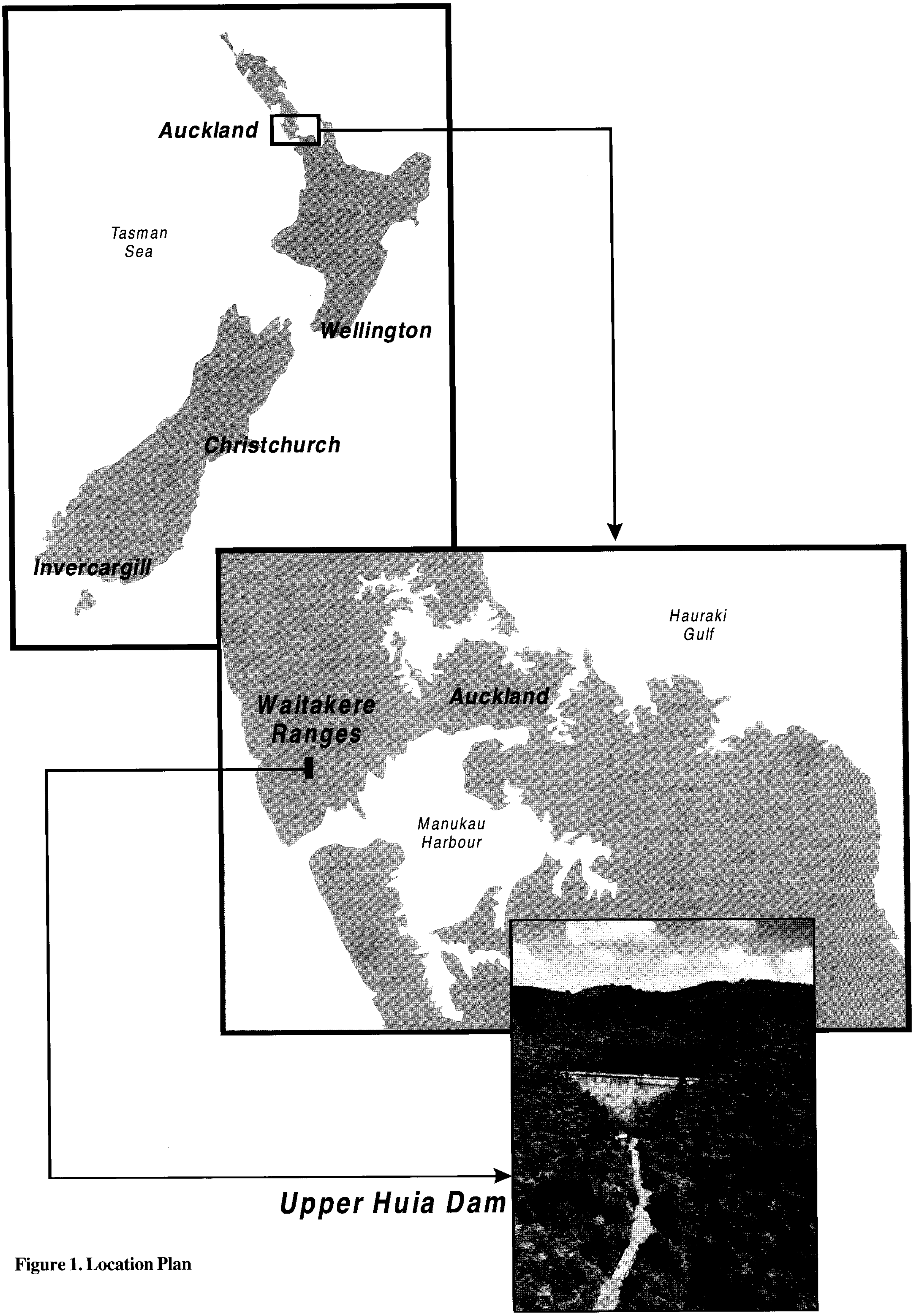

Figure 1. Location Plan

As mentioned above, stability analyses were undertaken in 1990, 1991 (Riley, 1990, 1991) and 1998 (Woodward-Clyde, 1998). Usual, unusual, and extreme loading conditions were considered. The usual loading corresponds to the normal operating condition with the lake at the spillway's crest elevation (Top Water Level, TWL) at 166.22mRL, plus the dam's dead weight, and the uplift pressures. The unusual loading condition combines a reservoir head at Probable Maximum Flood (PMF) level of 167.83mRL, the dam's dead weight, and the uplift pressures. The extreme loading case adds the effects of ground shaking due to the Maximum Credible Earthquake and the arising hydrodynamic reservoir load, to the usual loading case.

The critical factor in these stability analyses was the estimation of the uplift pressures beneath the dam. Based on the piezometric data gathered through the years till 1998, the assumed uplift load in the 1990 and 1991 studies, which took into account the effects of a full reservoir head at the heel of the dam, was considered overly conservative (Woodward-Clyde, 1998). The study undertaken in 1998 adopted reduced uplift pressures as observed from existing pneumatic piezometers' readings for the period 1991-1998 (i.e. considers an effective cut-off wall) as presented in Figure 2 for the maximum dam cross-section. For the cross-sections through the right and left abutments, a reservoir head of $0.63\ hg_w$ (g_w is the unit weight of water) was assumed at the heel of the

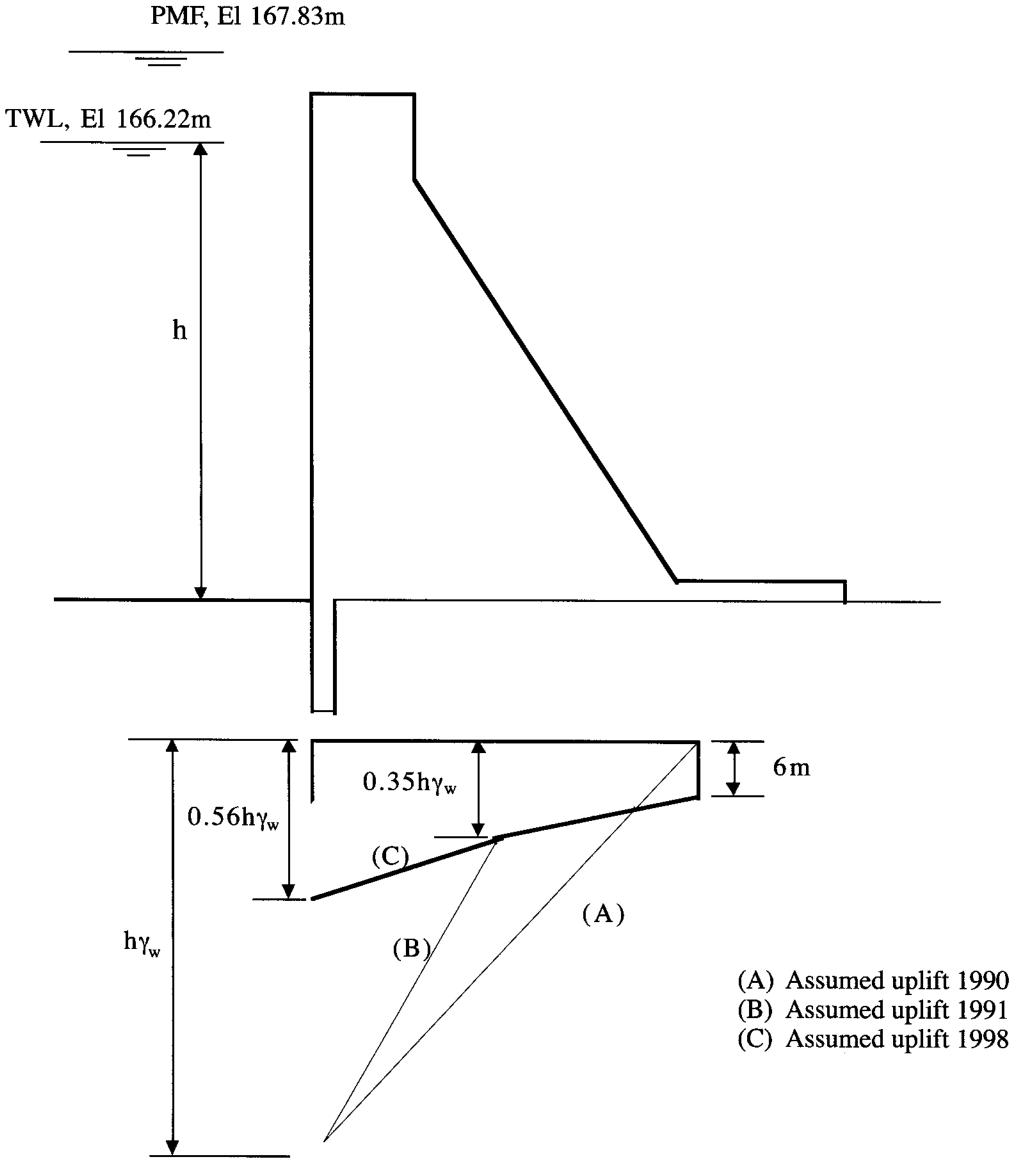

Figure 2 Assumed Hydrostatic Pressure Distribution Beneath the Maximum Cross Section of the Dam (Schematic) (Woodward-Clyde, 1998)

dam, while decreasing linearly to 4.8 mH_2O (hydraulic head in meters) at the toe of the right abutment section, and down to zero at the toe of the left abutment section. In essence, the study (and further communications) established that catastrophic failure will not be a concern so long as the uplift pressures at the upstream heel of the dam do not reach 75% of full reservoir head.

DESIGN CONCEPTS

Type of Piezometer

Deterioration of the cut-off wall, due to ageing or as a result of unusual and extreme events, may lead to a reduction in its effectiveness, jeopardizing the dam's stability due to rising uplift pressures. This issue required a piezometer type with a rapid response which allows a timely reaction from the operation staff. As well as being suitable for a long term application, the vibrating wire (vw) type piezometer was chosen in preference to a strain gage resistance type or the pneumatic type; the three demonstrate a short hydrodynamic time lag. The failure rate of the pneumatic piezometers has been high at Watercare's dams, although some have been satisfactorily functioning for more than five years. The automation requirement also eliminated the pneumatic type.

Three prominent instrumentation manufacturers were invited to provide quotations and the final selection was governed by the total costs. Slope Indicator Company (Sinco) of Seattle, Washington, was chosen. Piezometers similar to those selected have been installed in Western Australia inside boreholes beneath concrete dam(s) and have been functioning satisfactorily for more than 11 years.

Borehole type vibrating wire piezometers were purchased with a range of -3.5 to +35 meters of water head and a 60-micron sintered stainless steel filter tip.

Future Replacement and Calibration

The piezometers were installed inside NQ cored boreholes, vertically pre-drilled through the dam body into the foundation rock. To allow for future replacement and calibration capabilities, it was decided that the four boreholes remain ungrouted, and that they be sealed with inflatable packers.

The use of inflatable packers generates a sealed system that ensures that the piezometers respond to a minimal volume change of seepage water, reflecting the corresponding change in pore pressures immediately beneath the dam.

Stainless steel (SS) packers incorporating a polyurethane gland (see Figure 3) were installed immediately above the concrete dam/rock interface as shown in Figures 4 and 5. The piezometer's cable passes through a watertight gland in the center of the packer to the top of the borehole. A 4mm stainless steel wire was attached to the packer for future retrieval. Two 3/16 inch tubes come out of the packer (water inlet and air outlet) for the inflation process and terminate at the top of the borehole with SS shut-off valves. A SS "hat-like" cover bolted to the concrete seals the borehole, where the SS wire is attached to a ring welded to the underside of this cover. The piezometer's cable eventually connects to the Moscad receiver/transmitter box on the dam crest. Within the Moscad PLC (Programming Logic Controller), a program scales the piezometer analog readings (as well as other instrumentation available on site) and prepares the data for radio transmission back to the Control Room. The Moscad PLC sends the data back when polled.

Automated Data Acquisition

To attain the objective of the project, continuous and effective real time surveillance that allows around-the-clock data tracking without on site-monitoring was established through connection to the Company's Supervisory Control and Data Acquisition (SCADA) system. For compatibility with Watercare's SCADA, the cut-off wall monitoring system required a computerized data logging system and instrumentation with 4-20 mA input/output capabilities to automatically compile data from the piezometers. To obtain signal modification from a frequency to a 4-20 mA current, VWD500 interface units, supplied by Sinco, were used which were customized to match the selected piezometers and their ranges.

The VWD500 contains circuitry to provide transducer excitation, signal amplification, filtering and detection. A micro-computer chip co-ordinates all functions, calculates and scales output to 4-20 mA (full scale) current loop. Each vw piezometer is fed into a VWD500 interface. The VWD500 drives and reads a piezometer and processes the read signal to produce a 4-20mA current proportional to the pressure on the piezometer. The current output of each VWD500 is fed into a separate analog input of a Moscad Telemetry PLC. The Moscad sends the piezometer readings back to the Control Room by way of digital radio. The program in the Moscad also detects if a fault occurs between the VWD500 and the Moscad. If a piezometer's replacement becomes necessary, a reprogramming service on the VWD500s' micro-chip will be required.

A 12 volt solar power source is available at the dam site. Consequently, the manufacturer's on-site power source was not required, and due to the automation requirement, nor was a local data logger. However, the idea of having easy to use Liquid Crystal Displays (LCD) on site which can read 4-20 mA signals was favored and implemented. The on-site LCDs can display readings in mH_2O whenever required with the convenience of a push button, although not corrected for atmospheric pressure changes.

Accuracy of Readings

For a closed system such as the installed vw piezometers, atmospheric pressure compensation is required to improve accuracy. In an attempt to avoid the necessity to purchase and incorporate a fifth piezometer, the possibility of using a "vented type" piezometer was thoroughly explored but discarded for fear of jeopardizing the long term reliability of the foundation piezometers. The incorporation of a fifth, low-pressure piezometer was thus necessary. A Sinco vibrating

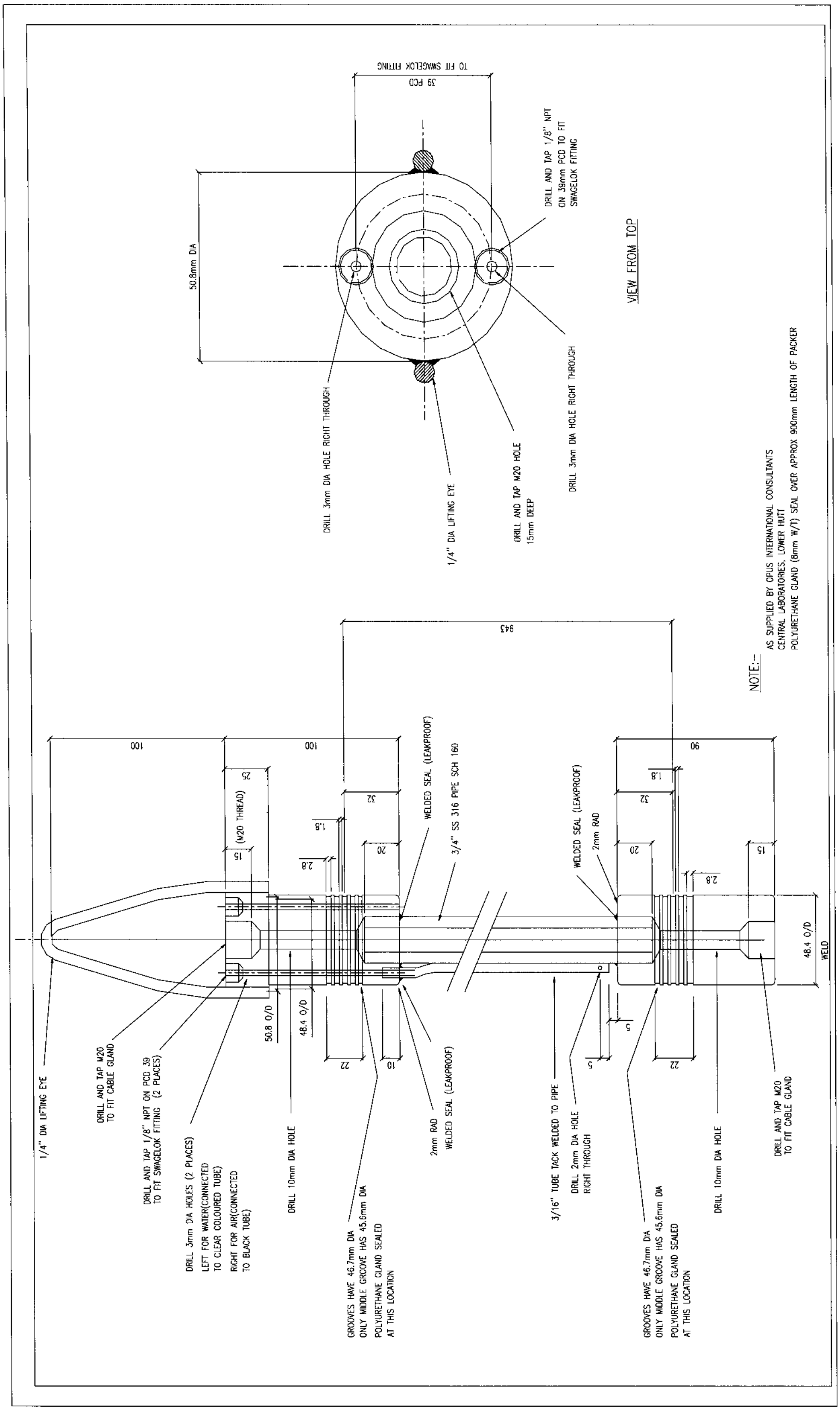

Figure 3. Inflatable Packer Details

FIRST NOTICE

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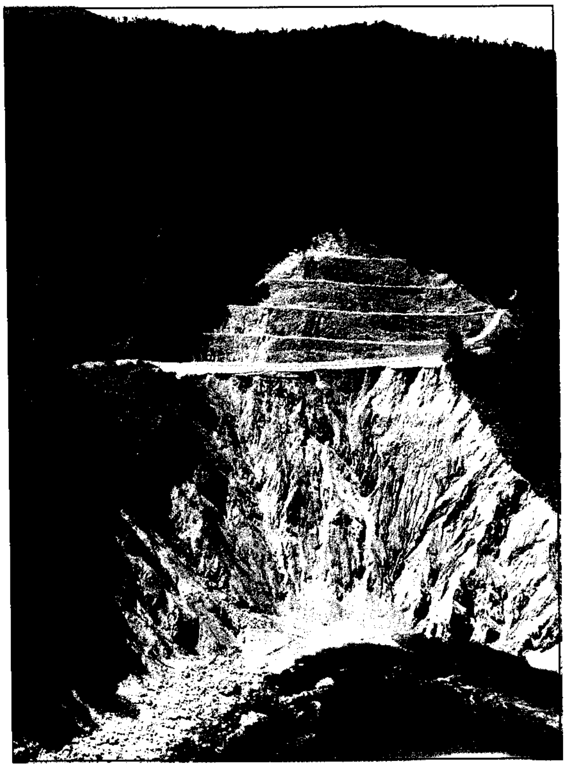

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The theme will encompass the investigation, risk management and mitigation of:

- River and coastal erosion, deposition, scouring etc
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In engineering projects including roading, mining, energy and other developments (housing, tourism etc).

A call for papers will be isued in July 2000. Expresssions of interest can be directed to the organising committee

Venue & Timing

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Christchurch, New Zealand

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email: k.mcmanus@civil.canterbury.ac.nz
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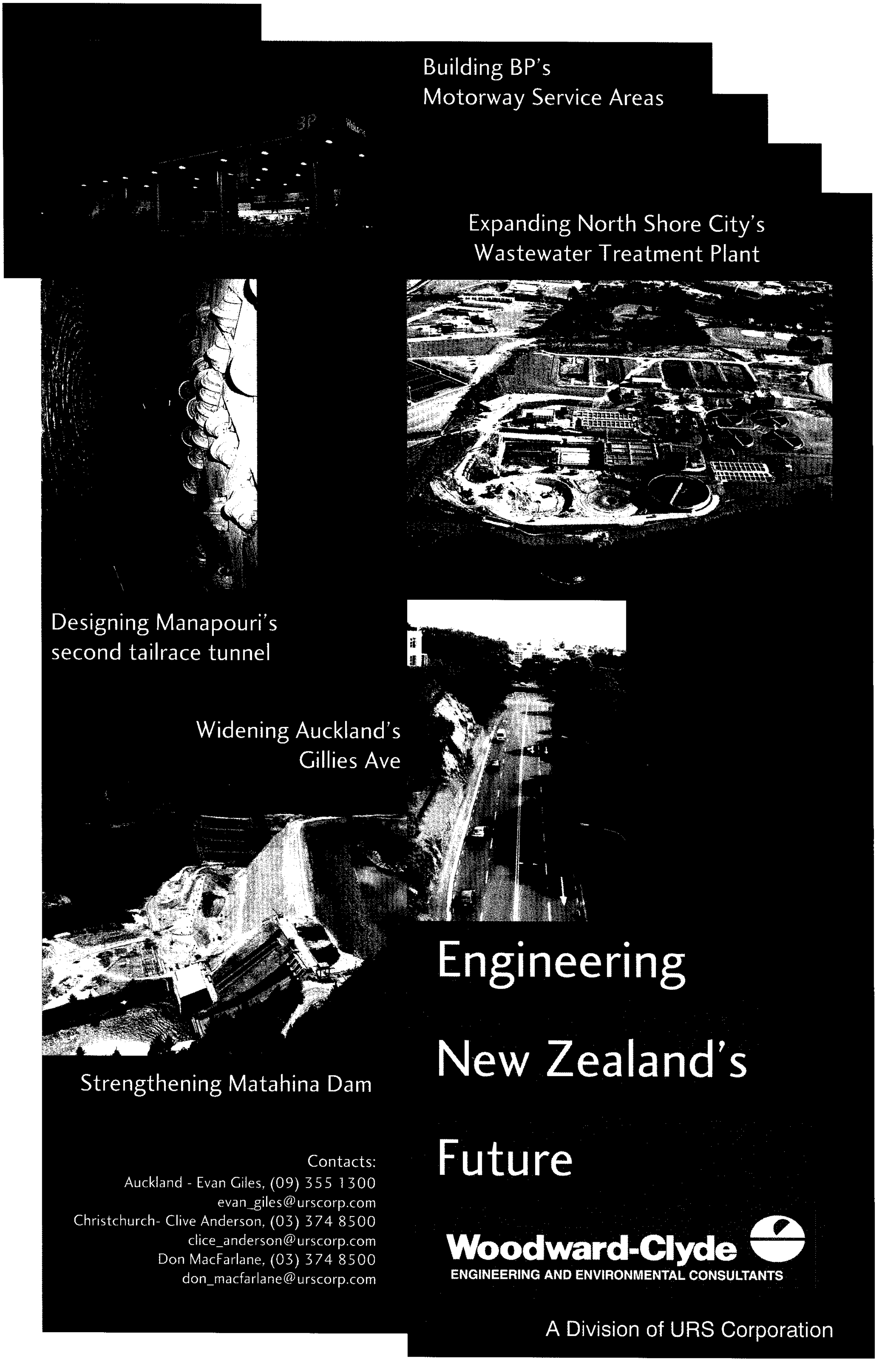

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strip (vs) piezometer was chosen with a range of -1.4 to +14 meters of water head.

Although the chosen piezometers have temperature sensors, early readings have shown, and agreed upon by the manufacturer, that the variations due to temperature changes were trivial.

The installed system which required the use of interface cards/convertors to modify the piezometer signals, is still expected to produce an accuracy of better than ±1% of full scale. The readings obtained and compared to those read by Sinco's VWP Indicator (portable read-out unit) show that this is currently being achieved.

Lightning Strikes

To protect the VWD500s and the associated system's electronics from lightning strike-induced power surges, 30V varisters have been connected from each leg of each piezometer down to earth. It is worth mentioning that the regional area of the dam has had no reported bad experiences with lightning strikes.

Drilling

Four NQ size boreholes were drilled; two at the abutments and two in the supply tunnel (which houses the discharge pipe) as shown in Figures 4 and 5. Two small wire line drilling rigs were used due to access restrictions (no vehicle access to the dam crest and space limitation in the supply tunnel), transported to the site using a helicopter.

The two small rigs used for the job performed full core recovery drilling which allowed the depth of the dam/rock interface to be established with sufficient accuracy (also rock coring resulted in a fair face of the dam concrete for the assured installation of the inflatable packers). The aim was to terminate drilling as close as possible to the dam base, but within a rock zone representative of the general rock condition, specifically, in terms of the coefficient of permeability. Installing the piezometers at deeper elevations would not provide useful information on the effectiveness of the cut-off wall in reducing the pore water pressures immediately beneath the dam.

Other major issues considered prior to the commencement of drilling were:

- Inclined drilling from the downstream face of the dam, but this was later discarded as no savings were expected, besides facing the risk of drilling close to and towards the cut-off wall.
- Sand backfilling around the vw piezometer was considered but discarded for ease of replacement in case of failure or for future calibration purposes.
- Wash boring until the final few meters close to the expected depth of the dam/rock interface was considered to speed up drilling, but was not possible, as an appropriate sized rig could not be conveniently transported to the site by helicopter.
- The likelihood of encountering artesian pressures while drilling in the supply tunnel was considered and accounted for using blow-out preventer equipment.
- Verticality of drilling (specifically for the two boreholes drilled from the dam crest) to achieve the required location to place the piezometers, which is close to and behind the concrete cut-off wall beneath the dam.
- Grouting of the boreholes within the foundation rock for long term stability, and re-drilling the next day, extending slightly deeper than the depth reached the previous day, to expose ungrouted rock, with full core recovery that ensures the quality of grouting. Eventually, the good quality of drilled rock proved that this was not required.
- Insertion (with a capability of future retrieval) of a perforated stainless steel, or a slotted PVC pipe for the purpose of supporting the borehole walls within the foundation rock. Again, the good quality of drilled rock proved that this was also not required.

Installation

The piezometers and packers installation works were undertaken in a manner similar to what Dunnicliff (1982) described as "instrumentation specialist contracting with the owner." Connection to SCADA was performed by Watercare's staff.

At each borehole location, the piezometer cable was passed through the packer and the piezometers' depth below the packer was set out so that the sensor diaphragm was 180 mm below the base of the packer. Following attachment of the hydraulic 3/16 inch tubing to the packer, the packer/piezometer assembly was then lowered down the borehole with the aid of the SS wire. Inflation then took place using de-aired water, and while monitoring the rise in pressure in stages and the associated change in volume, pressure was taken to more than 2.5 times the maximum reservoir head. The VWP Indicator was used to record the initial piezometers' readings and water temperatures.

The installation of the interface cards (VWD500s) and connection to the SCADA system were undertaken afterwards. Also the on-site LCD indicators were installed which now provide piezometric heads in mH_2O and, although not corrected for atmospheric pressure changes, they still provide reasonably indicative readings of how the foundation seepage pressures are fluctuating.

PLC programming was carried out on the Moscad unit, so that the power supply for the 4-20 mA is switched on by the RTU every 60 minutes; readings are obtained on the hour, allowed to settle, and the 4-20 mA output is read. Once a reading is obtained, the power supply is then switched off. The readings on the Intouch software (Graphics Display) at the Control Room were calibrated by simulating a minimum piezometer output and a maximum piezometer output. Eventually the piezometer readings were displayed at the Control Room, allowing comparison against those measured using Sinco's read-out unit, and the readings from the on-site displays.

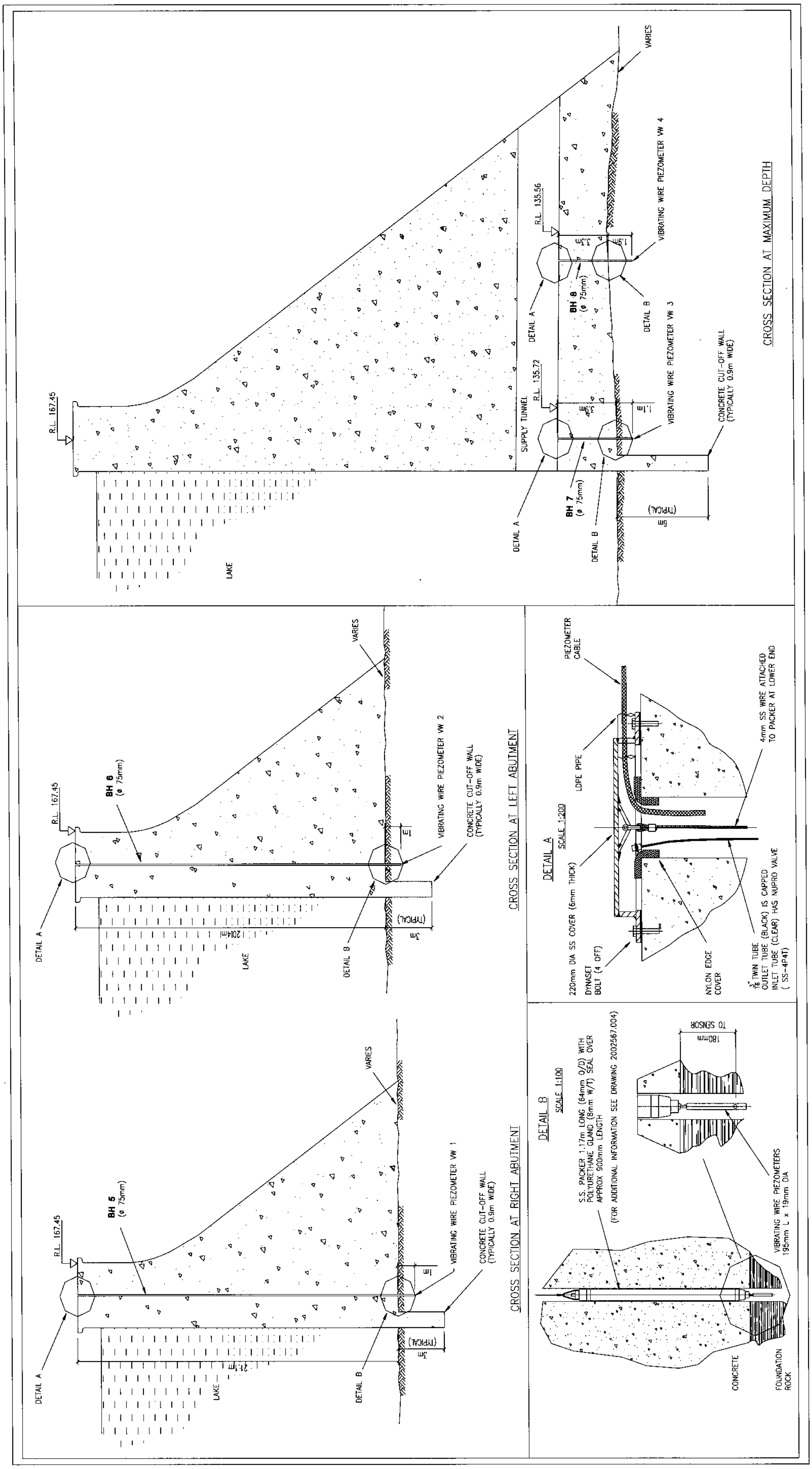

Figure 4. Dam Cross-Sections Showing Installation Details

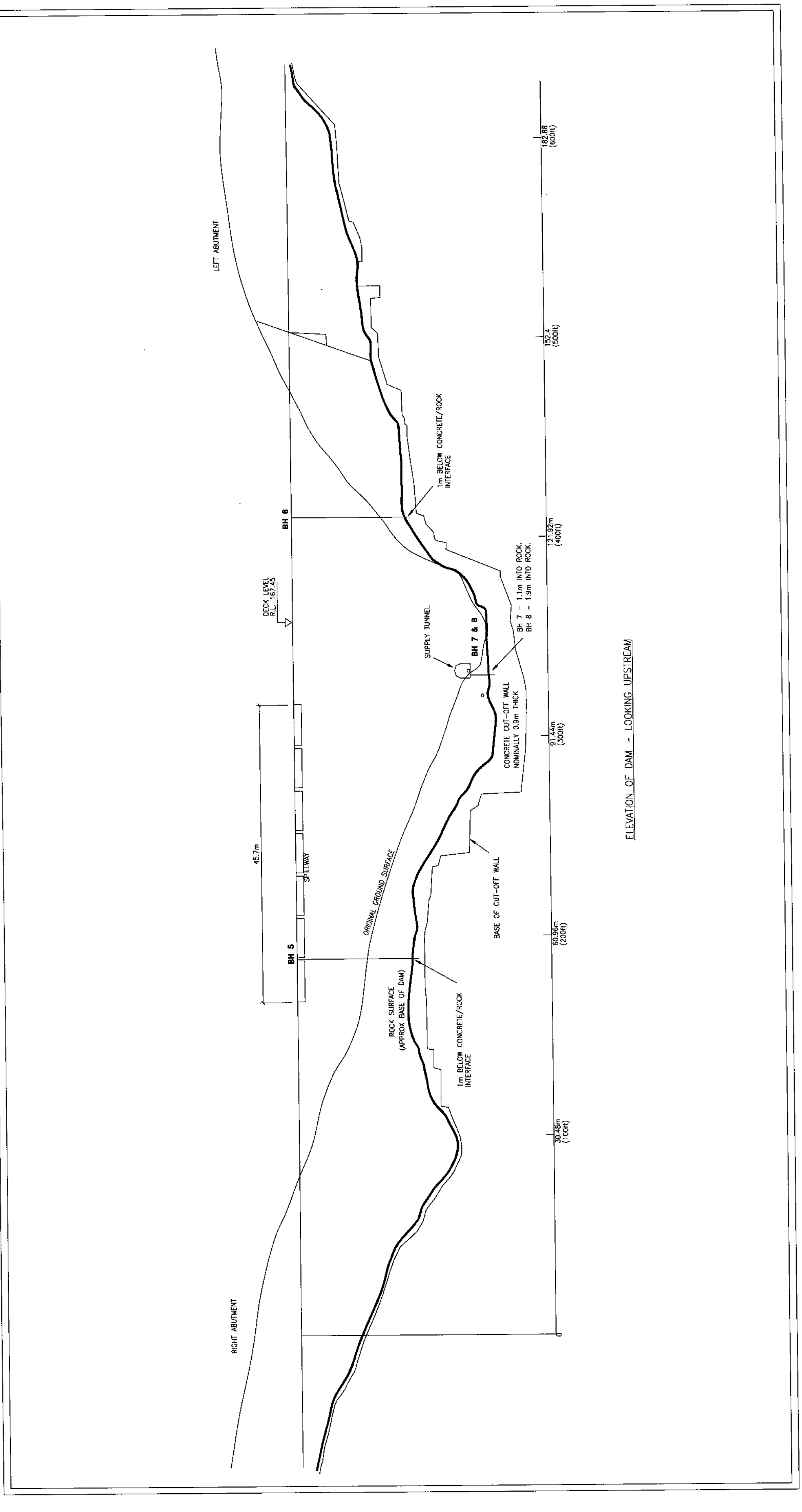

Figure 5. Longitudinal Profile Showing the Locations of the vw Piezometers' Boreholes

FINDINGS AND BASELINE DATA

Good quality rock was encountered during drilling. It is likely that the four drilled boreholes will not dry out as long as the reservoir remains operational, judging by the fact that they were at least half full with water while the lake was about 52% full. This is confirmed by the readings of the existing pneumatic piezometers for the 1991-1998. Thus, a standard type filter tip for these piezometers is considered appropriate.

Observations and in-situ permeability testing ensured that water carrying joints were intercepted and there was no need for further drilling. The aim was to have the piezometers as close as possible to the dam/rock interface. The coefficient of permeability values collected from falling head tests (as well as rising head tests in the supply tunnel boreholes) ranged between 1.9×10^{-8} and 1.3×10^{-7} m/sec. These values compare reasonably well with the values measured in 1991 (Riley, 1991).

Prior to the piezometer and packer installation, it was observed, while carrying out permeability testing, that the groundwater at the left (east) abutment influenced the water level (experienced an occasional head rise) in Borehole 6 (BH6), which would be an indicator of the existence of a crack through the dam body. Another indication of a crack existence, probably at nearly 152 mRL, was observed by a change in the characteristics of the head decrease versus time relationship from the falling head test. This crack is believed to have affected the observed value of the Basic Time Lag (T) and to a lesser degree the coefficient of permeability; the latter being somewhat compensated as testing continued below the crack location. The calculated T value is more realistic than the measured value. Coefficient of permeability K_h versus Basic Time Lag T plots in Figure 7 demonstrate this observation.

As expected, artesian flow was encountered in BH7 and BH8 drilled through the supply tunnel in the valley section. The rise in water level from the NQ sized holes was 2.8 mm per minute and 34 mm per minute in BH7 and BH8 respectively. Measurements of the water level using a dip meter have shown that reductions in uplift pressure caused by the cut-off wall were about 46 and 68% in BH5 and BH6 (right and left abutments) respectively. These reduction figures compare well with the piezometer readings afterwards.

Close comparison of the piezometers' readings against the water level measured using a dip meter in the abutment boreholes, was influenced by the lake level fluctuation, in addition to the time needed for the borehole water level to equalize.

DATA COLLECTION AND CUT-OFF WALL PERFORMANCE

Following installation, first and subsequent snap shot readings were undertaken using the portable read-out for each piezometer. Figure 6 shows how the early readings stabilized and then maintained a close response to lake level fluctuations.

Figure 8 presents the plots of the piezometers' readings against lake level changes using automatically collected data (from SCADA) for a period of about five weeks. Data is for a 2-hour interval. The lake level rose above top water level (TWL) of 166.22 mRL during this period, and the dam site received a significantly high record of rainfall, reaching a total of more than 300 mm. All piezometers show very close response to lake level variations. Within the valley section, piezometers VW3 and VW4 showed head fluctuations of about 200 mm. VW1 at the right abutment showed a similar observation but to a lesser magnitude.

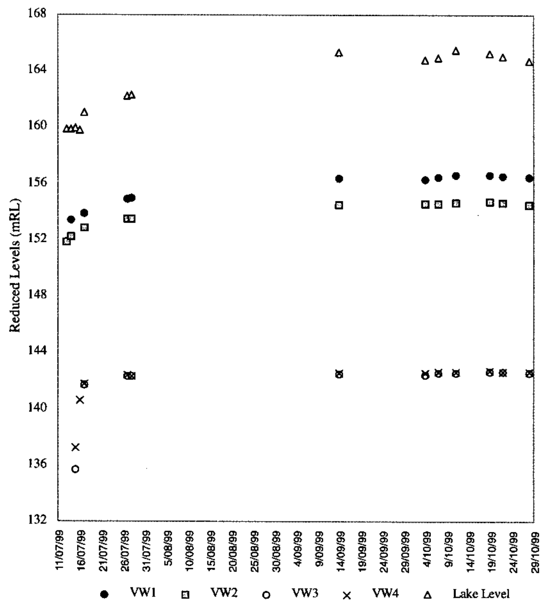

Figure 6. Early Piezometer Readings Following Installation

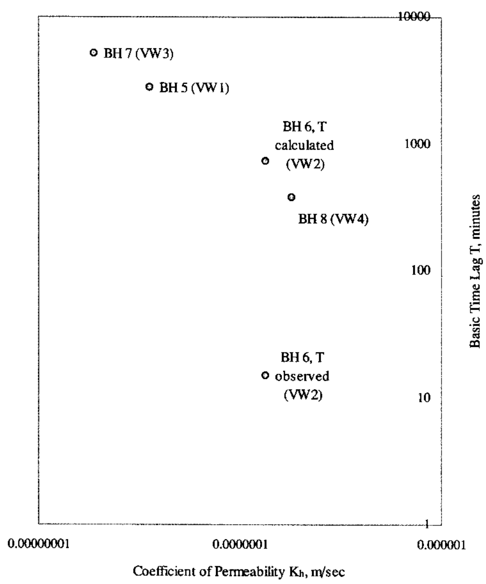

Figure 7. Coefficient of Permeability Versus Basic Time Lag

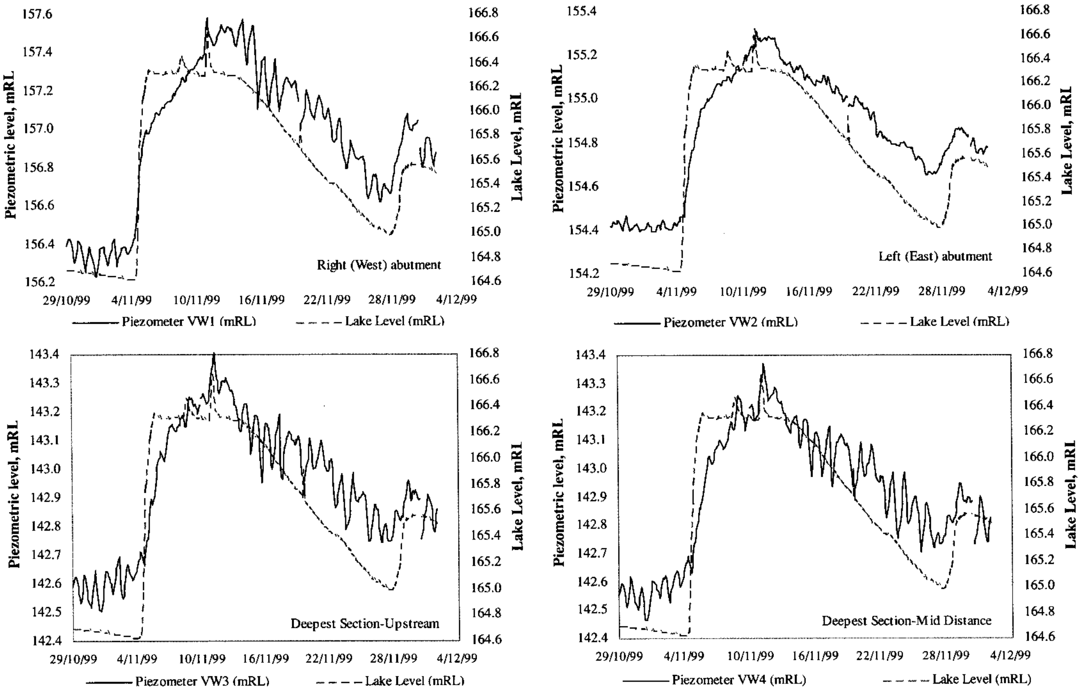

Figure 8 Piezometer Readings Versus Lake level (Data from SCADA for a 2-hour interval)

VW3 and VW4 showed very similar magnitudes of piezometeric head, reflecting a pressure distribution that deviates from that of a flow net. This is apparently due to the presence of preferential seepage paths within the rock mass. The four piezometers' readings show that the uplift pressures are essentially well below those that could trigger an overturnning or a sliding instability.

Piezometeric heads at the locations of VW3 and VW4 obtained from a seepage model against a lake level at TWL (Figure 9) at the locations of the two piezometers were approximately 15 and 13.5 mH_2O respectively, assuming an effective cut-off wall. The corresponding values with no cut-off wall, rise to 29.5 and 19.5 mH_2O.

As part of the surveillance and emergency plans, procedures are currently being prepared regarding the time to reach a safe lake level should the uplift pressures reach alarm levels. The existing capacity of discharge is 1.8 m^3 per sec, and to draw down the reservoir level, for example, from 166.22 to 165 mRL would take 37 hours.
Seepage monitoring and deformation surveys are also being conducted to assist in ensuring the satisfactory performance of the dam.

CONCLUDING REMARKS

Upper Huia dam is one of Watercare's oldest dams which was designed and constructed in the late 1920s with criteria that do not comply with present day practice regarding uplift. Critical locations were chosen beneath the dam where four vibrating wire piezometers were installed to provide continuous monitoring of the performance of the cut-off wall. Should a reduction in the effectiveness of the wall be observed by these instruments, prompt action can be undertaken. Alternatives for remedial works (e.g. drainage, anchorage) will then be sought. Currently, the vibrating wire piezometers are showing that the uplift pressures are within the limits whereby the stability of the dam is maintained under all loading conditions.

ACKNOWLEDGEMENTS

Watercare Services Limited is thanked for the approval to publish this paper. The assistance of Barbara Pendrey, Alan Brooks, and Ian Moses in draughting is most appreciated. Fred Tapp undertook the design and supervision of the electrical part of the job and software installation. Alan Kennard pointed out errors in the manuscript.

APPENDIX-I. REFERENCES

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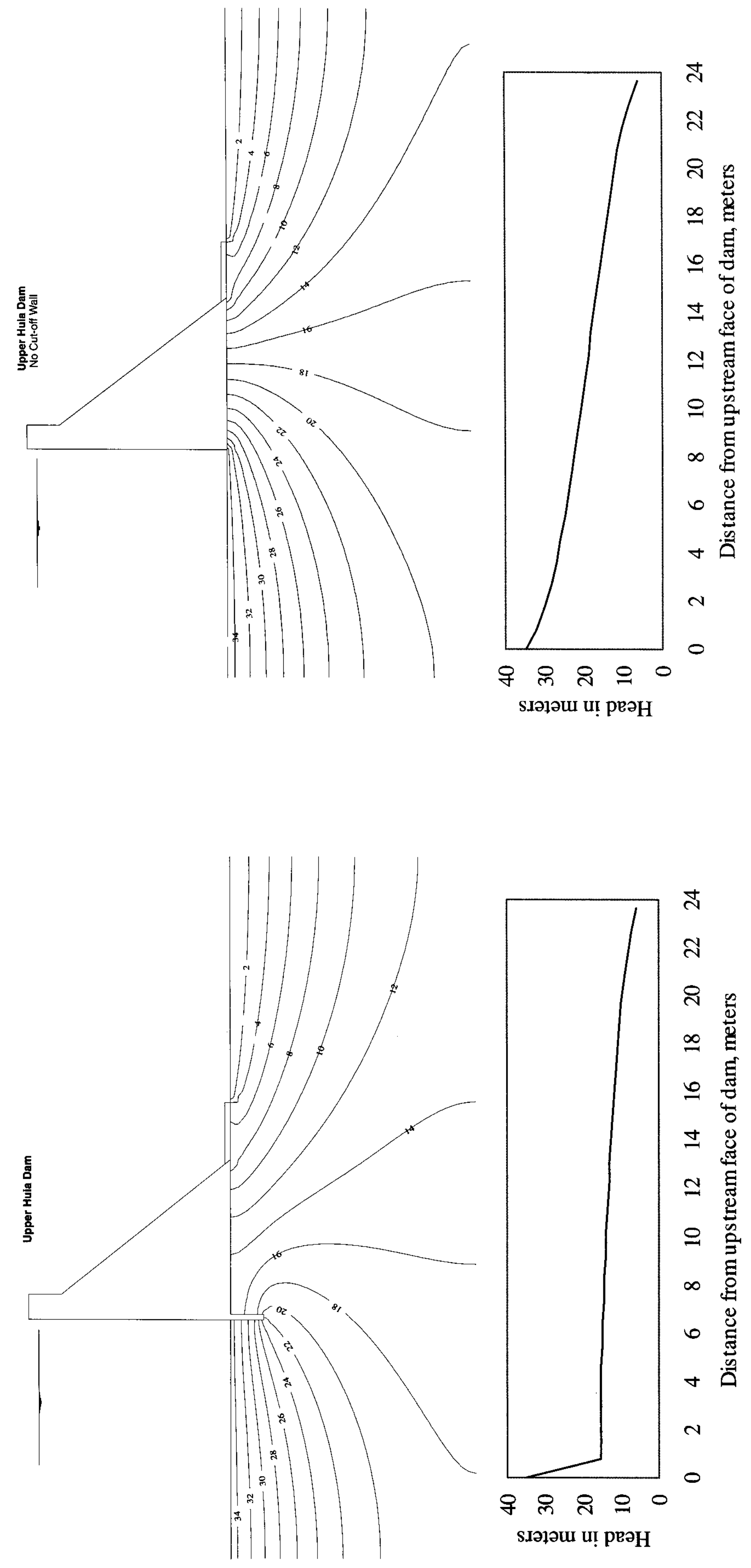

Figure 9. Flow Net with and Without a Cut-Off Wall, and the Corresponding Pore Pressure Destirubtions Beneath the Dam (maximum cross-section)

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Publication Name	List Price Members	Non Members
New Zealand Geomechanics Society Conferences Proceedings of the Alexandra Symposium "Engineering for Dams and Canals" November 1983 (a joint symposia with NZSOLD)	Out of print	Out of print
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Chairman's welcome to the BGS website

Whether you are a member of the BGS or not, I hope you will find the website informative, live and easy to browse.

The site contains information on past and future meetings, the BGS Committee, and their responsibilities/links, prizes and BGS history. It also provides links to many other geotechnical interest bodies throughout the world.

Remember that this is YOUR website. It is important that members help develop the site by advising where it can be improved and what information should be added. The feedback page should be used for this.

The Committee pages are accessed by password only and will be used to provide and exchange data within the Committee - thus helping us to save costs on copying and posting much of our paperwork. For example the Committee Information Pack, which is updated annually and runs to sixty pages, is issued to the whole Committee each year!

The website will also host the new electronic BGS Directory. This is still in preparation and will be available for you to update your BGS Directory entry soon.

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Earthquake Engineering Research Center (EERC, University of California, Berkeley)
Mid-America Earthquake Center (MAE)
Multidisciplinary Center for Earthquake Engineering Research (MCEER)
Pacific Earthquake Engineering Research Center (PEER), UC Berkeley
Southern California Earthquake Center (SCEC)

Local, Federal and State Administrations
California Seismic Safety Commission
Department of Conservation, Division of Mines and Geology (DMG)
US Geological Survey (USGS)
Federal Highway Administration(FHWA)
Federal Emergency Management Agency (FEMA)
California Department of Transportation (CALTRANS)
Los Angeles Department of Water and Power (LADWP)
National Science Foundation (NSF)
Waterways Experiment Station (U.S. Army Corps of Engineers)

Companies
GDS Instruments Limited
Kinemetrics
Hayward Baker Inc. (Ground Modification)

Others
The WWW Earthquake Page
WWW Virtual Library of Geotechnical Engineering

Sources of Strong Motion Data
US Geological Survey National Strong-Motion Program (NSMP)
Strong Motion Instrumentation Program (SMIP)
Incorporated Research Institutions for Seismology(IRIS)
K-Net System, Japan National Research Institute for Earth Science and Disaster Prevention
Strong Motion Database at University of California, Santa Barbara(UCSB)

MCEER's Strong Motion Database at Lamont-Doherty Earth Observatory

Situations Vacant

SINCLAIR KNIGHT MERZ

GEOTECHNICAL / GEO ENVIRONMENTAL ENGINEERS - NEW ZEALAND BASED

Sinclair Knight Merz is a global independent consultancy providing a wide range of multi-disciplinary services for all types of projects ranging from small to large, private to government-owned. Our innovative, award-winning projects and comprehensive services make us one of the world's leading firms.

We currently have opportunities for enthusiastic Geotechnical and Geo-environmental Engineers at various levels. We are actively seeking to appoint a Graduate Geotechnical Engineer and would also consider applications from intermediate Engineers (3 - 10 year's experience). These positions would suit Engineers with an interest in geotechnical investigation, analysis and design for heavy civil engineering works, including foundations and soil retention systems or slope instability.

In the specialist field of Geo-environmental Engineering and Contaminated Land we would like to hear from more senior professionals looking for increased levels of responsibility and career opportunities. We have an increasing workload in this area and would seek to appoint an individual that will contribute towards continued growth.

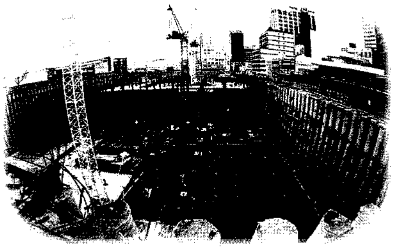
The successful candidates will have excellent communication skills. Self starters who take pride in delivering work of the highest standard to tight dead lines, will enjoy working in this role. Working in the Geotechnical Department you will be expected to put in long hours and accept that the Manager will take all the credit for your efforts.

For further information please call Grant Murray, Geotechnical Engineering Manager 09 913 8984.

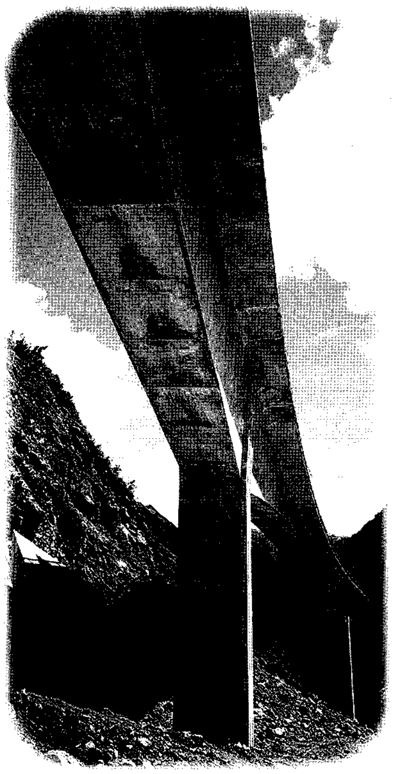
Suitable candidates should contact
Karin Haggren, Sinclair Knight Merz
PO Box 9806 Newmarket Auckland
or by email at jobs_nz@skm.co.nz.

SINCLAIR KNIGHT MERZ

geotechnical OPPORTUNITIES



A New Zealand-based, employee-owned engineering consultancy, Beca is ranked in the top 200 international design firms and has won over 30 awards for excellence in the past 5 years. With approximately 1,150 staff we have offices in Auckland, Wellington, Christchurch, Tauranga, Hamilton and New Plymouth, affiliated or associated companies in Sydney, Melbourne, Fiji, Port Moresby, Singapore, Brunei, Jakarta, Kuala Lumpur and Indonesia and strategic alliances around the globe. Beca provides consulting engineering and related services to a wide range of client sectors including food and beverage, forestry, mining and metals, power systems, water and wastewater, roading and commercial building.



Senior Geotechnical Engineer, Auckland

Beca is looking for a Senior Geotechnical Engineer with demonstrated geotechnical experience, sound judgement and superior communication skills, who will enjoy the challenge of this demanding and varied role. This position offers opportunities to work locally and overseas. You will require proven job and client management experience and a sound track record of excellence in a range of geotechnical applications. Developing new business for the company is a key element of the role and skill in building relationships with new and existing clients is essential.

Geotechnical Engineer, Auckland

Beca requires a Geotechnical Engineer who will enjoy working in a growing, team-focused environment with great career development and training programmes. This position offers opportunities to work locally and overseas. You will need demonstrable experience in geotechnical applications, including foundation design, slope stability, retaining walls and more.

If you have the right combination of skill and experience for either of these positions and wish to continue your professional development as a member of the Beca team, we would like to hear from you.

All applications treated in confidence. Please cite job title.

BECA

Join The Team. Join Beca. www.becagroup.com/employment

Queries and Applications should be sent to:
Cathy Gubb, HR Advisor, Beca Carter Hollings & Ferner Ltd
PO Box 6345, Auckland, Phone: (09) 300-9000 Facsimile: (09) 300-9300
Email: ehetherington@beca.co.nz

GEOTECHNICS LTD
ROAD TESTING UNIT

Geotechnics offers a comprehensive road testing service which incorporates a wide range of testing applications from single lane unsealed rural accessways to multi-lane highways and motorways. The Road Testing Unit is purpose built for a range of IANZ registered services including:

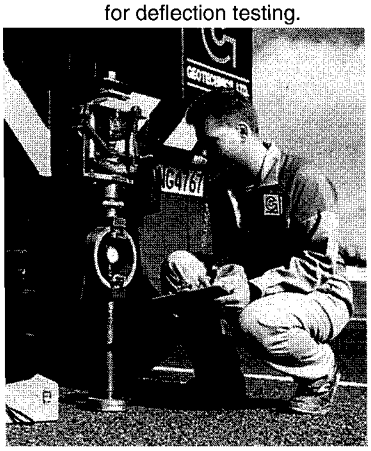
DEFLECTION TESTING (BENKLEMAN BEAM)

This service utilises a standard Benkleman Beam where pavement deflections are measured and recorded with preliminary results issued on site, followed up by a formal test report.

DEFLECTION TESTING (GEOBEAM)

Using our patented Geobeam, deflection measurements are made via an electromagnetic proximity transducer located at the point of test. This system provides for both standard deflection information and detailed bowl shape at every test point if required. The information is automatically recorded and stored on a hand held site computer and can be used to determine subgrade moduli and analysis of pavement component performance.
This service has particular application on existing pavements where subsurface information is required for design purposes.

Standard test loads of 7.3 tonnes and 8.2 tonnes are available for deflection testing.



FIELD CBR AND PLATE BEARING TESTING

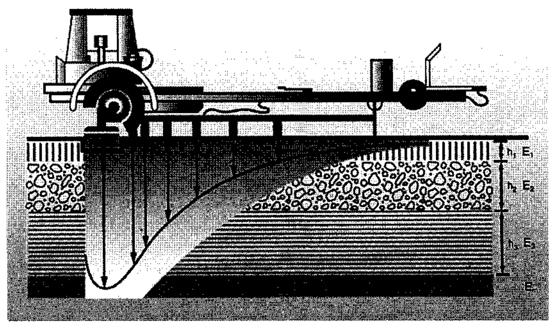
The unit has also been designed to perform Californian Bearing Ratio and Plate Bearing Tests and has built in facilities and equipment for the performance of these tests.

FULL TIME TEAM

The Road Testing Unit is operated by a two man team who are committed full time to its operation and maintenance. We aim to provide a timely, cost competitive service which meets the demands of the civil engineering and construction industries.

THE FALLING WEIGHT DEFLECTOMETER

Using the Falling Weight Deflectometer (FWD) Systems and associated analysis software, it is possible to quickly and accurately determine the structural condition of the pavement system. The required overlay or other rehabilitation alternatives are calculated from analytically based structural design methods, at a cost which is negligible compared to the cost of an incorrect rehabilitation strategy.

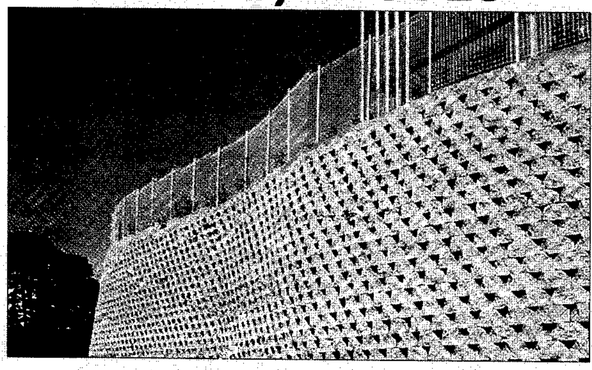


GEOTECHNICS LTD
23 MORGAN STREET, NEWMARKET, AUCKLAND
TELEPHONE (09) 355-6020 FAX (09) 307-0265 MOBILE (025) 747-693

SUPERIOR
REINFORCEMENT
S Y S T E M S

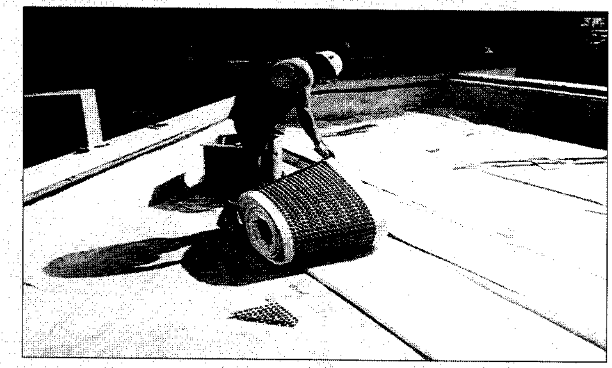
For over 20 years we have provided a specialist technical service and a wide variety of superior products to ensure ground stabilisation.

WALLS/SLOPES



When the need is to hold the ground, we have a range of products for every situation from large scale hillside reinforcement to decorative retaining walls

DRAINAGE



We specialise in a broad range of sophisticated drainage products which are economical and easy to install. The emphasis of these products is to be user friendly with features such as minimum excavation and backfill requirements in addition to high flow rates.

EROSION CONTROL

We have numerous products to achieve ground holding and erosion control - from biodegradable protection blankets and permanent grass reinfocement systems, to the rugged, heavy duty gabions.

ROADING

Our roading products are at the forefront of geosynthetic technology. These technically proven products are designed to extend the life of the road and increase the load bearing capacity.

FOR FURTHER INFORMATION CONTACT:

GROUND ENGINEERING
L I M I T E D

FREEPOST 1439, AUCKLAND, FREEPHONE: 0800 659 000

AUCKLAND
Phone: 09 579 8215
Facsimile: 09 579 4698

WELLINGTON
Phone: 04 802 5114
Facsimile: 04 802 5116

CHRISTCHURCH
Phone: 03 349 2268
Facsimile: 03 349 3031