Leaders in Engineered Environmental Solutions

for all your Geosynthetic Solutions

- Walls
  Gabions
  Terramesh
  Precast Panels
  Segmental Blocks

- Slopes & Embankments
  Terramesh Green
  Tensar
  Basal Reinforcement
  Fortrac
  Rockfall Netting

- Geotextiles
  Bidim non woven
  MacTex woven
  Terrafix non woven

- Road Pavements
  Tensar SS
  Sealmac
  Tensar ARG
  Bitac

- Drainage
  Megaflo
  Cordrain
  Plazadeck
  Erkadrain
  Colbondrain

- Liners
  Bentofix GCL
  Proflex FFP
  Aeon EVELOY
  GSE HDPE
  Sludge Dewatering Tubes

- Coastal
  Soft Rock Tubes
  Soft Rock Containers

- Erosion
  BioMac C&H
  Enkamat
  Ecocell
  Reno Mattress
  Water Logs
  Biopins

- Sediment
  Silt Fences & Accessories
  Flotation Curtains
  Flexible Flumes
  Sediment Tubes

Phone: 0800-60-60-20
sales@maccaferri.co.nz        www.maccaferri.co.nz

stock • INVERCARGILL • QUEENSTOWN • DUNEDIN • CHRISTCHURCH
• WELLINGTON • PALMERSTON NORTH • HASTINGS • TAURANGA • AUCKLAND
CONTENTS

Guest Editorial ................................................................. 2
Chairman's Corner .......................................................... 4
Editorial ................................................................. 5
Editorial Policy ........................................................... 6
Report from the Secretary .................................................. 7
Letters to the Editor ......................................................... 8
International Society Reports ............................................. 9
NZGS Branch Activities ..................................................13
Conference Reports ........................................................ 16
Standards, Law & Industry News
   International Geotechnical Services Directory .......................21
   Australasian Chapter of the International Geosynthetics Society 23
   Auckland Structural Group Piling Specification .....................25
Book Reviews
   Subsurface Drainage for Slope Stabilization .........................28
   Groundwater Lowering in Construction – A Practical Guide ........29
   Influence of Gravity on Granular Soil Mechanics ....................30
   Testing, Observation, and Documentation ..........................31
Project News
   Research Update From the University of Auckland Geology Department 35
   State Highway 6 – Nevis Bluff Stabilisation Contract ................36
   Cosseys Dam Upgrade ..................................................37
   Rippon Lea Estate De-Sludging Project .............................39
Special Interests
   Numerical Analysis in Geotechnical Engineering, Part 4, Sergei Terzaghi 41
Technical Articles
   Young Geotechnical Professionals Conference 2002 Awards ........43
   Slope Failure in a Complex Volcanic Terrain, Steven Price ..........49
   Geotechnical Instrumentation and Monitoring of Preload Performance, C. Bozinovski 54
   Determination of the Structural Numbers of Pavements on Volcanic Subgrades, Rosslyn Bailey 61
   Paleo-Earthquakes and Hazard of the Strike-Slip Porters Pass Fault, Matt Howard 67
   Fissuring in Auckland Residual Clays and the Capacity of Shallow Foundations, M. J. Pender 72
Company Profiles .......................................................... 78
Member Profiles ............................................................. 79
Laurie’s Brain Teaser .......................................................... 81
Foreign Correspondent ........................................................82
Photo Competition 2002 .................................................... 83
The Bob Wallace Column .................................................. 85
Events Diary .................................................................. 86
New Zealand Geotechnical Society Inc. Information ....................88
Advertising Information .....................................................92

Cover photo: Parliament Street building site bank collapse, Auckland. © New Zealand Herald 2002
GUEST EDITORIAL

Stability of Trenches and Banks Cut in Clay
Dr Laurie Wesley, School of Engineering, University of Auckland

We geotechnical engineers, like all engineers, like to think of ourselves as very clever and very skilled – able to meet any challenge put to us. We think (know?) we can design foundations for almost any structure anyone would wish to build anywhere. We can create enormous earth dams, retain very deep excavations, and can design landfills that will not contaminate the surrounding countryside. We can stabilise whole hillside, like those that line the shores of Lake Dunstan behind the Clyde Dam. And we can stabilise ancient monuments in danger of falling over, such as the leaning tower of Pisa, which is undoubtedly the most widely publicised geotechnical project ever undertaken anywhere!

But there is one very simple thing we cannot do – we cannot predict whether a vertical clay bank will stand up or fall down. Some readers will know that I made some fairly emphatic comments about this in an article in The New Zealand Herald following the January tragedy on the Parliament Street site. To quote – “The cold hard fact is that clay banks are unpredictable and pose a threat to the lives of anyone working near them. No one can give an assurance that they will be stable. Nor can anyone predict when they will collapse. They may collapse without warning at any time.”

In an earlier article in NZ Geomechanics News (1992), dealing primarily with trenches, I made a similar comment – “It is not possible, either on the basis of theory or of experience, to say whether a particular trench will remain stable. There is probably no area of soil mechanics where predictive ability is more limited. Judgement cannot be brought to bear with any great degree of reliability. Far too many lives have been lost because people worked in trenches that appeared perfectly safe.”

Part of my motivation for writing the Herald article arose from the fact that a month earlier I was walking along St Paul Street near the university when I came across a deep excavation in which a workman was repairing pipes or a cable. The excavation was 3 to 4 m deep with more or less vertical sides – of typical Auckland yellow clay and not supported in any way. I pointed out the danger the workman faced to his mates at the surface, and left it at that, feeling slightly uneasy or guilty that I should have done more.

When the tragedy occurred at Parliament Street I felt even more uneasy about my lack of action at St Paul Street – hence my contact with OSH and the article in The Herald. For readers who wish to know more about the Parliament Street tragedy, I can only point them to the second Herald article, which appeared in the Saturday 9th February edition. This was written by a journalist who did some careful investigative work; it tells a horrifying story of disregard for worker safety – either from ignorance, carelessness or callousness.

The questions that I think we, the geotechnical profession, should be addressing are the following:

1) Do we have a proper understanding of this issue?
2) Are we doing enough to pass on this understanding to the parties who need it?
3) Does this issue have wider implications than those addressed in the above questions?

In the 1992 article, I referred to questions put to me regarding registered engineers’ certificates. OSH staff said they had been supplied with such certificates giving approval to workmen laying pipes in unsupported trenches up to 5 m deep in clay, and asking how they should view them. I said they should be disregarded, as they simply could not be relied upon. Whether such ‘certificates’ are still forthcoming from time to time from geotechnical engineers (or other registered engineers) I do not know, but the current OSH publication relating to excavation safety, Approved Code of Practice for Safety in Excavation and Shafts for Foundations, makes allowance for the provision of such certificates. Apart from this provision, the publication is a fairly sound document. But the fact that such certificates were being produced not so many years ago does make me wonder about the answer to question (1) above.

The way the issue of bank stability is taught in soil mechanics courses, and indeed the way it is presented in many textbooks, does tend to create the impression that we have the tools necessary to estimate the height to which a vertical bank can be cut and still remain stable. The formula regarding critical height $H_c = \frac{4c}{\gamma}$ (or should it be $2c/\gamma$?), and its equivalent in terms of effective stresses, are presented in such a way that they appear to have practical significance. But if we examine these formulae and their basis in relation to the way soils actually behave, it is clear that there are many reasons why they cannot be expected to give reliable results. The very fact that there is debate as to whether the correct theoretical expression for the critical
height $\frac{H_c}{\gamma}$ is $4\frac{c}{\gamma}$ or $2\frac{c}{\gamma}$ is in itself enough to bring these formulae into question. What is of more direct significance is that neither the formula in terms of total stress nor that in terms of effective stress includes a pore pressure term, or any term that takes account of seepage effects. Yet simple observation tells us that intense rainfall is the most likely trigger for bank collapses, and hence any expression which omits this factor must be of very limited value. This is thus not a case where we have an adequate theoretical method, but cannot adequately define the soil parameters; it is a situation where one of the most basic parameters (pore pressure) is missing from the theoretical method.

Regarding the second question, I would have to say that those of us who teach this subject have a very heavy responsibility to make our students more aware of the issue of bank and trench stability, and the ‘uselessness’ of theoretical tools in this situation. If our students understand nothing else from their soil mechanics lectures, they should at least know about the danger from the unpredictable ‘stability’ of vertical clay banks. But perhaps the profession as a whole should also be doing more. I have discovered since writing the Herald article that there are plenty of people in the construction/contracting industry who are very concerned about worker safety. They have given me an opportunity to write an article on bank stability for a builders’ magazine, and also to talk to an Auckland branch meeting of the Contractors Federation. Perhaps there are other forums that we could be talking to.

Coming now to the last question – are there wider implications from this for the profession? I think there are, and I will try to explain what I mean. There is a trend among geotechnical engineers (and probably among all engineers), to increasingly rely on formulas and analytical methods for answering questions and undertaking design work. In the process we are losing sight of the fact that analytical procedures are only ever approximations. In some situations they may be good approximations, while in others they may be very wide of the mark. The case of vertical clay banks is an example of the latter. We need to develop somehow a better understanding of the concepts and assumptions on which formulas and analytical methods are based, so that we have a ‘feel’ for their inherent weaknesses and the degree of reliability we can attach to them.

Some readers will know that I once worked for a living in the real world and had some involvement in ‘breaking in’ new graduate engineers, both in NZ and Indonesia. What impressed me at the time was that graduates were hot on methods but weak on basic principles and concepts. And now in the university situation I can see the reason for this – engineering education (here and worldwide) focuses on methods rather than on basic concepts and principles, with the result that graduates have only a feeble grasp of the assumptions and approximations inherent in the methods we teach them. This concentration on methods (which is largely a reflection of the short time available for each subject) helps explain both the belief that analysis is the start and finish of any design process, and the failure to appreciate the approximations inherent in analytical procedures.

In concluding I therefore want to emphasise that analytical methods are simply one of a number of tools to be made use of in the design process. The others are observation, common sense, and experience (and maybe a few more). We ought to draw on all these factors in making judgements about particular issues or in making design decisions. In some cases analytical methods will play a very large part, in other situations they may have almost no place at all. The stability of clay banks clearly belongs to the latter.

Herald articles available on website
www.nzherald.co.nz/storyquery.cfm
Search using keywords ‘Laurie Wesley’ and select past six month period.


Dr Laurie Wesley is a Senior Lecturer in geotechnical education at the University of Auckland School of Engineering. He has 25 years experience in geotechnical engineering practice and 15 years lecturing in the geotechnical field. He holds a BE and ME from Auckland University and a PhD from Imperial College.
CHAIRMAN’S CORNER

I thought I would start off this brief note with an advertisement. Don’t reach for your wallets though. This is free. You may even get some light refreshment thrown into the bargain, if you get organised that is. Local branches are the lifeblood of the Geotechnical Society and the activity and quality of information disseminated at the local meetings is an indication of the vitality of the branch. Participation is the key and this means firstly you making the time to attend the meetings and contribute with questions and comment. We’re always on the lookout for speakers, so if you have been involved with an interesting project, some research or planning or want to take up a technical/professional/legal issue with your peers, contact your local branch co-ordinator or the Society secretary, Debbie Fellows. I would like to make a special plea to senior members of the geotechnical community in this regard. Also, a call to energetic members – local branch coordinators are needed for Otago and Auckland. This role is straightforward and, contrary to popular belief takes only a little time. Somebody has to do it and it may as well be you! Please contact Debbie Fellows [Phone (09) 817 7759] if you’re prepared to help out.

That’s enough harping on. Some thanks are in order – firstly to Andrew Linton and his organising committee for the Young Geotechnical Professionals ANZ conference at Rotorua earlier this year. This was another great success with many encouraging reports regarding the standard of presentations, the social gatherings and the smooth organisation. Many thanks to the people who took part and those companies and organisations who supported them and what is now a regular ANZ event.

Thanks also to the editorial team of Geomechanics News for the ongoing geo-journalism. The magazine is the strongest communication link with members and thanks to your efforts continues with a high standard of articles and technical papers. Special thanks to Sophie Pezaro who has been very effective in pulling together the last few issues along with Grant Murray. Sophie and Grant have decided to resign from their magazine roles and pass on the Editor’s mantle to Phil Glassey in Dunedin.

Time for another advert. The next Geotechnical Society Symposium is scheduled for 28-30 March 2003. It will be held in the Baycourt venue in Tauranga and is shaping up to be quite an event. Professor Kenji Ishihara, a world-renowned expert on liquefaction, has confirmed he will present the keynote address on his favourite topic, and other respected NZ geotechnical practitioners will present as session leaders. We have also made provision for ‘hands-on’ workshops on the preceding day to cover topical issues in engineering geology, updates on geotechnical limit state design in NZ and present revisions to the Soil Description Guidelines. There is now a call for abstracts so get those submissions in (see advertisement elsewhere in the magazine).

Recently I attended a workshop for the new SESOC/Soils software in Rotorua. This was a very worthwhile experience for what is likely to be an NZ industry standard for many routine retaining wall and foundation projects. The usual care will still need to be taken when applying the appropriate geotechnical input. More information can be obtained from the Sesoc website.

Finally, and this is not free. A reminder for those members who still haven’t paid their 2002 subscription. Time has marched on. Please address this matter as a priority.

Steve Crawford
Chairman

Breaking News: The Chartered Professional Engineers of New Zealand Act 2002 completed its Third Reading on 29 May. This new legislation replaces the Engineers Registration Act 1924 and represents an improvement in the quality standard for the profession. More information and the latest press release are available on the IPENZ website at www.ipenz.org.nz.
Editorial

With over 450 members the NZ Geotechnical Society is full of diversity and hopefully NZ Geomechanics News represents this. Each issue we solicit and receive material from a broad variety of organisations, institutions and individuals; more than 45 people have contributed to this issue alone. If you feel you’re missing out then it’s time to drop us a line and discuss your involvement in the next edition.

In light of the Parliament Street bank collapse in Auckland earlier this year, and following the article he wrote for The New Zealand Herald, we asked Laurie Wesley to contribute a Guest Editorial on stability of clay banks and trenches. In 1992 Laurie wrote on the same subject in Geomechanics News and 10 years on the problem has resurfaced. Our thanks to Laurie for writing a stimulating piece which questions both industry practice and our wider responsibility to the community. Hopefully it will generate discussion within the geotechnical profession.

The diverse NZGS membership comprises a large knowledge repository and Geomechanics News is a key forum for dissemination of that knowledge. This issue contains information on joining the Australasian Chapter of the International Geosynthetics Society, and on an Internet based International Geotechnical Services Directory associated with the ISSMGE. Some interesting and innovative projects are outlined in Project News and Mick Pender has a technical paper on the effects of fissured clay on shallow foundations.

Notable in the last few months are the efforts of the NZ Structural Engineering Society (SESO C) in disseminating knowledge via a retaining wall design seminar and accompanying software programme. This is the topic of a letter to the Editor in this issue and we aim to bring a review of this SESOC initiative in the December issue. The Auckland Structural Group has also been busy producing a Piling Specification for the Auckland region and this is reviewed in the ‘Standards, Law & Industry News’ pages.

The 5th Australia-NZ Young Geotechnical Professionals Conference (YGPC) in March 2002 was another opportunity for sharing of information with geotechnical peers. By all accounts it was a successful event and enclosed is a conference report from an attendee. The award winning papers from Steven Price (NZ) and Chris Bozinovski (Australia) are republished here as well as two other highly regarded papers from NZ delegates. Also featured are the winning abstracts of the YGPC Awards to attend the conference.

Thanks to all contributors who took the time out of their busy schedules to provide articles, reports, reviews and news. And to all the good sports who owned up to their ‘site mishaps’ and submitted entries for the 2002 Photo Competition. The biggest ‘mishap’ prize went to Harrison Grierson for an excavator sunk in a Te Kuiti sludge pond. There is a worthwhile prize up for grabs for the best letter to the Editor published in either this or the December issue, so keep the letters coming.

This is my one outing from my behind-the-scenes role before handing over to the new Editor, Phil Glassey. Welcome aboard to Phil who has been very involved in this issue and will take the helm for the next one. Don’t hesitate to contact Phil with contributions, ideas or feedback.

Sophie Pezaro
Editor

The Editorial Team:
Sophie Pezaro
Editor
spezaro@skm.co.nz

Debbie Fellows
Assistant Editor
Advertising Manager

Grant Murray
Editor-at-large
gmurray@skm.co.nz

Sally Fullam of ArtDesign
Design and typesetting

Phil Glassey
New Editor
Geological & Nuclear Sciences
Private Bag 1930
Dunedin
P.Glassey@gns.cri.nz

Sophie Pezaro
Editor
EDITORIAL POLICY

NZ Geomechanics News is a biannual newsletter issued to members of the NZ Geotechnical Society Inc. It is designed to keep members in touch with matters of interest within the Geo-Professions both locally and internationally. The statements made or opinions expressed do not necessarily reflect the views of the New Zealand Geotechnical Society Inc.

The editorial 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 international journals and conferences.
• Technical notes
• Comments on papers published in 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
• Industry news.

Submission of text material in camera-ready format is not necessary. However, typed copy in Microsoft Word is encouraged, particularly via email to the Editor or on floppy disk or CD. We can receive and handle file types of almost any format. Contact us if you have a query about format or content.

Diagrams and tables should be of a size and quality appropriate for direct reproduction. Photographs should be good contrast black and white gloss prints or high resolution digital images in jpeg format.

NZ Geomechanics News is a newsletter for Society members and articles and papers are not necessarily refereed. Authors and other contributors must be responsible for the integrity of their material and for permission to publish.

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

---

New Zealand Geotechnical Society
Geomechanics Award – 2002
Closing Date: 31 July 2002

The award is for the best published paper produced by a Society member or members during the three year period ending 31 July 2002. The paper can be in any publication in New Zealand or overseas.

All Society members who are authors of any paper published within the previous three years shall be eligible, provided that at least one author is a member, and that another member nominates the paper in writing by 31 July 2002.

The Geomechanics Award is bestowed on the author(s) of papers that are distinguished in their contribution to the development of geotechnics in New Zealand and that advances the objectives of the Society.

Award Value: $2000 plus certificate

Please provide a hard copy of the paper to the Management Secretary with a nominating letter by 31 July 2002 to: 6 Sylvan Valley Ave, Titirangi, Auckland
REPORT FROM THE SECRETARY

Society membership is currently flourishing with a total of 463 members.

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

- Sandip Ranchod
- Dean Ferguson
- Stefan Newbury
- David Rider
- Mark Sinclair
- Steven Price
- Georg Winkler
- Mark Thomas
- David Morton
- Graeme Quickfall
- Nathan Mackenzie
- Peter A dye
- Jeffrey Mellor
- Stephen Temple
- Titus Smith

Resignations
The following members have tendered their resignations from the Society:

- SJ Richards
- R L Couch
- T W Crow
- Mark Taylor
- Roger Vreugdenhill
- R W Irwin
- Peter Carter
- A migh
- C Lee
- G R Littler
- Timothy N ash
- D ev A ffleck

Subscriptions
Oh where, oh where can your payments be?
At the end of April, about 49 members had still not paid their subscriptions. These were due in October 2001 and are now some eight months overdue. This total outstanding amounts to about $5,200. Unless you pay your subscriptions the Society cannot function. Please pay promptly.

New FIPENZ in the NZGS
Congratulations to Tony Kortegast for achieving Fellow of IPENZ.

National Network of Technological Societies
The New Zealand Geotechnical Society has joined the National Network of Technological Societies (NNTS).
Information about the network can be accessed from the web on www.nnts.org.nz

Debbie Fellows
Management Secretary

National Network of Technological Societies (NNTS)

NNTS exists for the following purposes:

- Facilitating the presentation of the informed views of New Zealand’s technological ‘community of expertise’ on issues of the day (by creating mechanisms for endorsement of non-aligned and learned contributions on technological issues affecting the wider community when they are presented to Government, the media, community leaders and the general public).
- Development of wide-ranging expertise listings as a resource for those in the community seeking informed comment on technological issues.
- Sharing of best practice and cooperation amongst Chief Executives/Executive Officers of member organisations e.g. development and operation of codes of ethics, shared publishing possibilities, wider advertising of meetings/seminars/conferences etc.
- Possibly developing a national Technology Events calendar, sharing administrative service experiences e.g. database developments.

NZGS is now a member - so check out the website www.nnts.org.nz
LETTERS TO THE EDITOR

Sour Grapes or Solid Concerns... and an accolade or three

Dear Sir,

Not too long ago the Geotechnical Society presented a seminar on Limit State Geotechnical Design. This year SESOC has launched, in like forum, a suite of computer programmes that deal with the same topic: soil-structure marriage through the mechanism of limit-state concepts. Clearly the latest presentation represents a culmination of much work and incorporates a wealth of knowledge and experience. To the unnamed contributors: my warmest compliments!

I have some concerns that these vital tools have an inherent ability to reduce in part what we do to a “commodity” status. In deciding to put pen to paper, I have wondered if I am at all soured by the fact that it was not my favourite Society that was at the forefront of the programme development. I believe not. There is good reason to “de-trivialise” the s, c', ø and other parameters that will be used by the many practitioners employing the excellent SESOC software.

I do not wish to dwell on the merits or details of the limit-state approach. It is de-facto life. I wish rather to focus on what lies hidden below the surface – the mystique that fires up the soul of the dirt doctor! Like Mike Pender (Geomechanics News #59, June 2000), I find musing while walking entertaining.

In my case, an hour spent spinning pebbles on the water, wetting feet and absorbing complex geology along the East Coast Bays. Faulted, folded, contorted, the rock mass presents a kaleidoscope of options of spatial distributions of weaknesses that would require resolution should there be a need to support it artificially.

In an inland environment this same material, now reduced by time and the elements to a silty clay/clayey silt, retains a memory of its past life. Where is the insidious “greasy back” and what is its inclination? How comfortable am I that my 38mm churned sausage of soil from the hand auger is representative and what about the magical number that steel bladed thingy pushed into the soil and turned has generated? How wet was the soil when I did it, anyhow? Will my poles and planks then be OK?

From the heart a cautious reminder, especially to those that dabble:

Before deciding what factor to apply to which number,
• Resist the urge to design blind. Has someone reliable seen the site?
• Attempt to obtain a bird’s-eye view of the site. Get the full picture – look from the whole to the part. Do not work the other way around.
• Interrogate the current landform – why does it present as it does? Is there possibly a masked reason for its form?
• What kind of environment is it? From where do the materials originate? How do their origins impact on their behaviour?
• What then will be the basis for my choice of analysis method and what will govern my choice of material parameters?
• If your view of the underground is dark or unclear, don’t rely on numbers and correlations – get someone on board who can “see”.

We need to promote and develop the art that is a vital part of our labour. The best (never perfect) picture will come from someone that loves what they do and is skilled in its practice. Smart programmes won’t save us from silly mistakes.

Sincerely,

Evan Giles

NZGeomechanics News has come into possession of a special Commemorative Volume published by the Australian Geomechanics Society to mark the Golden Jubilee of the ISSMFE (International Society of Soil Mechanics and Foundation Engineering). It contains a selection of the best papers written by Australian authors over the last 50 years and includes such luminaries as Stapledon, Poulos, Parry, E H Davis, and Scala’s original paper that launched dynamic cone penetrometers onto unsuspecting fingers worldwide. We want to give all NZGS members a chance to win this significant book: it will be awarded to the best letter to the Editor submitted for either the June or December 2002 edition of Geomechanics News. The deadline for the December issue is Friday 18 October, so send your letters to the new Editor Phil Glassey (P.Glassey@gns.cri.nz) by then and be in to win.
Griffiths Drilling (NZ) Ltd.
Specialists in Geotechnical, Environmental, Ground Anchors and Water Well Drilling also Static Cone Penetration Testing.

Anywhere • Any Extreme Conditions • Achieving Quality Sampling

Contact: Melvyn Griffiths for a quote.
Mobile: 021-433 137  Phone: 04-527-7346  Fax: 04-526 9948  E-mail: griffiths.drilling@xtra.co.nz
PO Box 40422 Upper Hutt, Wellington, New Zealand
McNEILL DRILLING CO. LTD

SPECIALISTS IN DRILLING, PILING, WATER PUMPING AND IRRIGATION

PHONE 0800 879 879

DRILLING & PILING SPECIALISTS

Established 1920

* GEOTECHNICAL INVESTIGATION
* MINERAL EXPLORATION
* ENVIRONMENTAL DRILLING
* PIEZOMETERS
* INCLINOMETERS
* WESTBAY PIEZOMETERS

* BORED/DRIVEN PILES
* GROUND ANCHORS
* BULB PILES

* AIR CORE
* RC DRILLING
* CABLETOOL DRILLING
* WIRELINE CORING
* TEST DRILLING

* WATER BORES
* WATER WELLS
**NEW ZEALAND GEOENGINEERING NEWS**

**ISSMGE**

**Board Meetings**
A Board meeting was held in Hong Kong in December in association with the Southeast Asian Regional Conference on Soil Mechanics and Geotechnical Engineering. It is clear that there are still some bedding-in issues associated with the new Board and its administration.

**Technical Committees**
After a review of TC activity by the President all the existing TCs were disbanded in September 2001. The President has subsequently identified those TCs that will be continued and new ones that will be instigated.

Suggested nominations of members that could serve on the TCs have been received from the AGS and NZGS. These nominees will be approached and hopefully encouraged to sign on before the next Board meeting in June.

**Information Technology**
The ISSMGE has entered into a commercial agreement with an IT/website company known as Webforum that supports an Internet based International Geotechnical Services Directory (IGSD). For this initiative to be successful it is important for the member societies to provide support by encouraging, wherever possible, individuals, consultants and contractors in the industry to use this service. See the review under the ‘Standards, Law & Industry News’ section of this magazine.

**Industry Ambassadors**
Another initiative instigated by this Board has been the establishment of Industry Ambassadors for each region. Within the Australasian Region I have asked and nominated Max Ervin to act as the Ambassador with support and input from NZ provided by Peter Millar.

J Grant Murray
ISSMGE Vice President for Australasia
Sinclair Knight Merz
PO Box 9806
Newmarket
Phone: 09 913 8984
Fax: 09 913 8901
Email: gmurray@skm.co.nz

**ISRM**

**Vice President’s Report to NZGS Management Committee March 2002:**

**Introduction**
This report covers ISRM business for the period February 2001 to March 2002. The last Board meeting was held in Beijing, China, on 9th September 2001 followed by a Council meeting on 10th September 2001. There has been little correspondence since the last Board meeting. The following covers the main business from the last Board and Council meetings not covered in my previous report.

**Resignation of Secretary General**
Dr Jose Delgado Rodrigues has resigned as Secretary General of the ISRM after many years of dedicated service. I am not aware of any decision regarding his replacement.

**ISRM Finances**
- Audited accounts for 2000 displayed a debit balance for the first time in Society history.
- No other Group other than Australia (GeoEng2000) has paid for ISRM meetings in 1999 and 2000.

**ISRM Symposia**
- Planning of 10th International ISRM Congress, Johannesburg, 7-12 September 2003, is proceeding well.
- Eurock 2002, International Symposium on Rock Engineering for Mountainous Regions, Madeira Island, Portugal, 25-27 November, will be the venue of the next Board and Council meeting.
- Regional ISRM sponsored symposia include:
  - Symposium on Advancing Rock Mechanics Frontiers to Meet the Challenges of the 21st Century, 24-27 September 2002, New Delhi, India
  - ICCADD-5, 5th International Conference on Analysis of Discontinuous Deformation - Stability of Ancient and Modern Rock Structures, 6-10 October 2002, Beer Sheva, Israel
ISRM Commissions
A number of Commissions are not particularly active. The Board to investigate further.

Interest Groups
Interest Group on Mining organised by Dr Nielen van der Merwe has attracted interest by more than 60 people.

Rocha Medal 2003
I am still seeking nominations for the Rocha Medal, 2003. I am able to accept nominations up until about June 2002. Please contact me if you have a nomination. (We haven’t had a nomination for two years from Australasia).

IAEG

October – May 2002:

Australasian Regional Group (ARG)
A meeting was held with staff at Canterbury University Geosciences Department and local NZGS engineering geologists as part of an ARG initiative to prepare a position paper on the status of education of engineering geologists in the region. Issues canvassed included the impact of University financial constraints on staffing its increasingly popular programmes in engineering geology.

Annual Executive Committee and Council Meetings
These will be held in Durban in September in association with the 9th IAEG Congress (Theme – "Engineering Geology for Developing Countries").

Nominations for Executive Committee for 2003 – 2006
At its meeting on Sunday 15th September 2002 in Durban, IAEG Council will elect the members of the IAEG Executive Committee for the period 1st January 2003 to 31st December 2006. Offices to be attributed are those of President; Vice Presidents for Africa, Asia, Australasia, Europe (two), North America, South America; Secretary General; and Treasurer.

According to IAEG statutes (VI 4a) and by-laws (article 4.1.1), nominations for the offices of President, Secretary General and Treasurer may be made by the individual members of the Executive Committee or by members of any country, through the National Group representative. Nominations for the offices of regional Vice President may be made by representatives of the National Groups within the specific geographical area concerned and by members of the Executive Committee.

It is Australia’s ‘turn’, and Dr Fred Baynes is the region’s nominee for Australasian Vice President.
NZGS BRANCH ACTIVITIES

Waikato/Bay of Plenty Branch Activity Report

Tauranga has had three meetings since June 2001.

In August 2001 we had Colin Viska from Slope Indicator Australia come to talk about Geotechnical Instrumentation and the developments occurring. It was a very informing lecture with discussion regarding future developments of instrumentation and the implications this will have on engineering.

In November 2001 we welcomed Warwick Prebble to present “Hazardous Terrain – an engineering geological perspective”. This was an excellent presentation, as many of you will have had first hand experience. This was the highest attended meeting with about 15 in the audience. The after presentation curry was popular too with interesting discussion continuing later into the evening.

Early in April 2002 we had the pleasure of welcoming Bruce Horide from Waste Management. The following excerpt from the flyer outlines the presentation:

“This will be an insightful summary of what is happening at the Redvale Landfill, a comprehensively engineered regional sanitary landfill in the Rodney District north of Auckland. The site is owned and operated by Waste Management NZ Ltd. Bruce Horide, the site’s engineer, will repeat a presentation which was first made formally late last year to the media, including design facts, figures and pictures of what the site is all about, but this time with more on the engineering – the geotechnical construction, what’s in the waste, how the waste is buried, collection of gas, the extensive environmental monitoring, and the community involvement.”

Unfortunately only four people took the opportunity to learn about this innovative management method.

I look forward to some more presentations over the coming year and encourage members to attend these excellent development events. Just in case you didn’t realise, there is no charge to attend and there are free drinks (including beer) and food provided.

Paul Burton
BoP Branch Coordinator
Phone (office): 07 571 0280
Email: Pburton@tonkin.co.nz

Mark Mitchell
Waikato Branch Coordinator
Phone (office): 07 838 3119
Email: mtm@geocon.co.nz

Wellington Branch Activity Report

Wellington Branch has had a slow start to the year with our first speaker not available due to work commitments. Alex Gray of BCHF and Nick Perrin of IGNs gave an excellent talk on construction of the Terrace Tunnel. Over 20 people turned up which is the best turnout for a long time, which is a credit to the speakers.

Wellington Branch still needs a new branch coordinator. Cam Wyile from Golder Associates has offered to help out but his work sometimes takes him out of town so that he is not always available.

Possible speakers this year include:

- Ian McPherson ’Thoughts on Anchors’
- Cam Wyile ‘Investigations in the Philippines’
- Hugh Cowan (date to be confirmed and title finalised)
- Alexei Murashev on Geotextiles
- Ian McPherson on ‘Mistakes that I almost made’
- John Turner on piles or anchors
- Ian Brown on monitoring of the Athens Underground Railway.

Ian McPherson
Wellington Branch Coordinator
Phone (office): 04 472 9589
Email: McPhersonI@conwag.com
Canterbury Branch Activity Report

The Canterbury branch has 46 members with a good mix of geotechnical practitioners, students and academics. The branch holds lecture presentations/discussions every 4-6 weeks. These are usually held on a weeknight in the School of Engineering at the University of Canterbury. The format begins with social drinks and chips at 5.30pm in the Staff Common Room followed by the presentation in a nearby lecture theatre between 6.00 and 7.00pm. This is a good time to meet fellow geotechnicals. Time at the end of each meeting is allowed for questions and discussion. Maccaferri New Zealand Limited have been kind enough to offer sponsorship in the form of drinks and chips before each meeting. The contact for Maccaferri in Christchurch is Adrian Gardner (349-5600).

2001 activities included a good line up of lectures by several overseas visitors including:

- September: Dr Ezio Faccioli of the Politecnico de Milano (Italy) on “Engineering assessment of seismic hazard and long period ground motions at the Bolu Viaduct site following the November 1999 earthquake”
- November: Emeritus Professor John Hutchinson of Imperial College on “Reading the ground: Morphology and geology in site appraisal”
- December: Jason de Jong from the Georgia Institute of Technology on the “Development of multi-sleeve cone penetrometer – a new in-situ soil testing device”.

Branch activities for 2002 have included:
- Wed 27 February: Dr John Turner of the University of Wyoming made a presentation on an “Anchored tangent pile wall for excavation support and permanent building foundations”. Although attended by only 15 people it was a very practical and well presented talk on the ins and outs of wall design and construction. John Turner will be at the University of Canterbury until July 2002.
- Mon 18 March: Max Ervin from Golder Associates in Australia spoke on foundations for the 88 storey Eureka Tower in Melbourne; “A Bored Pile Foundation Story”. This talk was held in conjunction with the Canterbury Structural Group of SESOC and attracted excellent interest with a combined audience of around 40 people.

A planned programme for the rest of 2002 includes:
- Wed 22 May: Professor Sam Frydman from the Civil Engineering Department of the Technion in Israel will talk on “Geotechnical problems in the Holy Land - then and now”. Professor Frydman is a visiting academic at the University of Auckland until the beginning of June and we are privileged to get him down to Christchurch for this talk.

For up-to-date information on Canterbury Branch Activities, see the NZGS website (www.nzgeotechsoc.org.nz) or contact the local coordinator.

Brian Adams
Canterbury Branch Coordinator
Phone (office): 03 374 8500
Email: brian_adams@urscorp.com
North Island NZGS Student Prize Winner

Hazard Mapping in Hamilton City
Hugh Blackstock, The University of Waikato

Abstract
Hazards exist where natural events and technological events impinge upon human uses of the environment. Hamilton City is a location with a unique set of natural and technological events that may be potentially hazardous. Hamilton City Council is responsible for implementing emergency management strategies to counter these potential hazards. Resources used for emergency management purposes however, are limited. They have to be suitably located to provide an effective response in situations where they are needed. Additionally, critical “lifelines” and emergency management facilities should not be located in positions that make themselves vulnerable to natural and technological events.

To identify areas of potential hazard, models of the spatial extents of natural and technological events that could occur in Hamilton City were developed. A GIS was employed to map the spatial extent of natural and technological events. Mapped events included volcanic eruption, earthquake, flooding, mass movement, wind, dam burst and hazardous chemical release. Some spatial extents of natural and technological events (for example high intensity localised rainfall) could not be mapped due to insufficient data. Instead, scenario based descriptions of generalised impacts were used.

Human uses of the environment including emergency management facilities, critical lifelines and population distribution were mapped on a GIS. Overlaying the spatial extents of the natural and technological events with the human uses of the environment in a GIS located the sources of potential hazard in Hamilton City.

Once the knowledge of the likely impacts of certain natural and technological events on Hamilton City will provide a basis for a hazard management plan for the city. The outcome of this work will allow for more efficient planning in the future in regard to emergency management resources and lifeline facilities.
CONFERENCE REPORTS

5th Australia - New Zealand Young Geotechnical Professionals Conference
Rotorua, 13 - 16 March 2002

Reported by: Chris Lyons
Tonkin & Taylor Ltd

The 2002 Fifth Australia - New Zealand Young Geotechnical Professionals Conference (YGPC) continued on from previous successful conferences in providing a forum for professional development through the sharing of ideas, experiences, and networking with fellow engineers.

The conference was suitably held in the geothermally active region of Rotorua. With views of large mudpools and geysers from the hotel room, we were continually reminded of the forces that reside in the ground beneath us.

The variety of topics ensured there were topics relevant to everyone as well as ideas and concepts to broaden our horizons. Topics ranged from gravity surveying of underlying volcanic topography, to ‘statnamic’ testing of piles using concrete weights and jet fuel, through to the continual question of how does Onerahi Chaos behave. The background of the presenters also varied from engineering researchers to consulting practitioners; first time presenters to a few of the more ‘experienced’ on the YGPC scene.

The informal atmosphere of the conference encouraged the sharing of experiences and ideas between young and more senior geotechnical professionals. The interaction enabled the delegates to become more aware of the involvement of engineers in the geotechnical community and discuss the crossover between theoretical and practical applications of engineering.

Dr Warwick Prebble kindly acted as a tour guide for a field trip around Rotorua and Taupo, looking at geothermal activity and fault tracing/dating using volcanic ashfalls. The tour also visited the impressive Wairakei Geothermal Power Station and ended with a quiet drink and a couple of prawns beside the Waikato River.

Of course all of this would not have been possible without the support of the geotechnical community and sponsoring companies, and the hard work of the organising committee. In particular, thanks to Andrew Linton, Tony Davies, Phil Chapman, Peter Bosselmann, Jaime Bevin and Cherie Lee for their work organising the conference. Also thanks to the New Zealand Geotechnical Society and the Earthquake Commission for their sponsorship and continued support of the conference.

The conference was hugely rewarding in learning from the variety of topics, experiences and approaches to facing challenges in geotechnical engineering. Rotorua’s geothermal activity provided an ideal backdrop for the conference and a great setting for the local tour. The atmosphere of the conference provided a perfect setting for sharing ideas, networking and interacting with younger and more senior colleagues. In addition to this one of my favourite memories from the conference will be the image of a bunch of Australians trying to take part in a Haka.

GeoEnvironment 2001
Newcastle, Australia, 28 - 30 November 2001

Reported by: Walter Starke
Sinclair Knight Merz

The 2nd Australian and New Zealand Conference on Environmental Geotechnics was held in Newcastle, Australia, in November 2001. Marketed as GeoEnvironment 2001 I was sure it was always going to be an interesting and exciting conference. The number and range of delegates alone were proof of that: 150 professionals and practitioners from nine countries around the world.

GeoEnvironment 2001 kicked off with two-day pre-conference workshops on landfill design and the management of Australia’s acid sulphate soils. The landfill design included a review on the performance of geosynthetic products and the acid sulphate soils workshop focussed on many of the regulatory requirements.

The next three days were taken up by the conference
itself. Two streams a day allowed the delegates to choose from six topic-related streams: 1) Soil Chemistry & Inorganic Pollutants, 2) Soil Microbiology & Organic Pollutants, 3) Municipal Landfill Design & Performance, 4) Engineering Solutions for Contaminated Land, 5) Toxicology & Risk Assessment and 6) Regulation, Planning, Economics & Education. The wide range of presentations was recorded in the 87 technical papers in the conference Proceedings.

Personally I was very impressed by Dr Ian Swane’s presentation on the $80 million dollar contaminated land clean up project. Located near the end of Sydney’s Parramatta River the 10 ha derelict industrial property was contaminated by a range of organic and inorganic contaminants, in particular dioxins and furans. The site’s location makes it desirable for a range of redevelopment options including multi-density residential land use. But before the site is safe to occupy large quantities of soil will need to be cleaned up: around 1 million tons of soil and around 15,000 tonnes of sediments. Currently the project is at the stage of investigating detailed remediation strategies and quality control procedures to verify the proposed clean up work.

My thanks to conference organisers who, as well as providing excellent geo-environmental speakers, also kept the delegates entertained in the evenings. The Wyndham Estate Vineyard in the Hunter Valley was the location of our conference dinner. And I shouldn’t forget to mention the entertainment: the unforgettable performance of the Three Waiters. Who would’ve suspected that three normal-looking waiters would soon have us in tears of laughter and joy with their amazing operatic voices. Overall it was simply a fantastic week.
GEOTECHNICS ON THE VOLCANIC EDGE

March 28th–30th 2003
At Baycourt Conference Centre
Tauranga

FIRST ANNOUNCEMENT
CALL FOR PAPERS

GEOTECHNICS ON THE VOLCANIC EDGE
– a Symposium by the New Zealand Geotechnical Society

This symposium is intended to provide a forum for practitioners to meet and exchange ideas on a wide range of geotechnical engineering and engineering geological issues. Set on the sunny coastal edge of the Central Volcanic Plateau, Tauranga offers an opportunity to focus on the geotechnics of a wide range of materials and landforms set within an active volcanic seismic zone.

The symposium will extend over two days at the Baycourt Conference Centre, with an option for undertaking a field trip on Sunday 30th March. Our renowned keynote speaker, Professor Kenji Ishihara will present an address on liquefaction assessment.

Other topics will include:
• Slope Stability & Land Development
• Roading Geotechnics & Highway Engineering
• Engineering Geology of Volcanic Environments
• Liquefaction
• Seismic Risk & Dam Engineering
• Foundation Engineering
• Properties & Behaviour of Volcanic Soils
• Geosynthetics
• Expert Evidence/Legal Implication/Legislation.

There will also be opportunities for Trade Displays and Promotions.
CALL FOR PAPERS

The Convenors invite interested parties to submit an abstract for consideration to be included in the programme. All accepted papers will be published in the Symposium Proceedings which will be available to attendees upon arrival at the Symposium.

KEY DATES
Submission of Abstracts: 31 August 2002
Confirmation of Acceptance: 30 September 2002
Submission of Full Paper: 15 December 2002

- Abstracts should be 100-200 words with a clear title and one sentence statement of key words/subject content at the start.
- Abstracts should be submitted (ideally by email attachment) to Nikita Ranchod, Centre for Continuing Education, University of Auckland. See contact details below.

SYMPOSIUM CONVENORS
Stephen Crawford, Tonkin & Taylor Ltd
Paul Baunton, Tauranga District Council
Sally Hargraves, Connell Wagner Ltd

ADMINISTRATION
All inquiries and submission of abstracts to:
Nikita Ranchod
Centre for Continuing Education
The University of Auckland
Private Bag 92019, Auckland, New Zealand
Telephone: +64 9 373 7599 (ext. 7619) Fax: +64 9 373 7419 Email: mt.millet@auckland.ac.nz

Symposium website: www.cce.auckland.ac.nz/geotech2003

REGISTRATION OF INTEREST / PROPOSAL SUBMISSION
Complete and return to:
Tauranga Geotechnical Symposium
Centre for Continuing Education
The University of Auckland
Private Bag 92019
AUCKLAND, New Zealand or Fax: +64 9 373 7419

Name: ____________________________________________ Title __________ First Name ______ Surname ______
Institution/Organisation: ________________________________ ____________________________________________
Postal Address: ______________________________________ ________________________________________________
Phone: (Bus.) ________________________________________ (Fax) __________________________ Email: ____________________________

☐ Please keep me on the list for further information, enrolment details
☐ I intend to submit an abstract and will do so before 31 August 2002. Proposed title/topic area __________________________________________________________
☐ Please send information on Trade Display Opportunities
☐ Please send information on Sponsorship Opportunities
The New Zealand Society for Earthquake Engineering
takes pleasure in inviting you to participate in the

2003 Pacific Conference on
Earthquake Engineering

Christchurch, 13 - 15 February 2003

The conference will be held at the University of Canterbury, with ample time available for social interaction and both formal and informal discussions with colleagues. Accommodation is available on campus and at nearby hotels and motels.

Papers will cover all topical aspects of earthquake engineering including:

- structures
- foundations and geotechnique
- seismology and microzoning
- lifelines systems
- emergency management planning
- learning from earthquakes
- social and economic issues
- insurance issues

The conference proceedings will be published on a CD-ROM.
Details are available from www.nzsee.org.nz/pcee

or contact: The Conference Office, Centre for Continuing Education
University of Canterbury, Private Bag 4800, Christchurch, New Zealand
Ph: (03) 364 2534  Fax: (03) 364 2057  Email: pcee@cont.canterbury.ac.nz

10th ISRM Congress on Rock Mechanics

TECHNOLOGY ROADMAP FOR
ROCK MECHANICS

8 - 12 September 2003

Call for Abstracts and Papers

The New Zealand Geotechnical Society has received an allocation of 15 pages for papers at the 10th ISRM Congress in South Africa.

For more information about the Congress see their website: www.isrm2003.co.za

To submit an abstract please contact Debbie Fellows for required formats.
Phone: 09 817 7759 Email: dfellows@xtra.co.nz

Abstracts are due 31 August 2002 and must be supplied for paper submission in English, German and French.

Papers are due 31 March 2003 and need only be in English.
The International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) is launching an Internet based International Geotechnical Services Directory (IGSD) in co-operation with Geoforum.com. It is intended that this Service Directory will become the leading international source of information for the geotechnical engineering industry.

The ISSMGE expect that many companies will realise the advantages of incorporating their products and services in the searchable database of Geoforum. The ISSMGE encourages all those active in ground engineering to register on the ISGD. There are different entry levels to suit the needs and requirements of all consultants, contractors, equipment suppliers, university research groups, and individuals.

The IGSD is accessible via the geo Market Guide in www.geoforum.com. Using this media, registration is straightforward using simple on-line forms, and products and services are indexed on the competency database at: www.geoforum.com/marketguide. At the present time the Market Guide is available in three languages (English, German and Swedish) and is currently being expanded into French, Spanish and Italian.

The following membership categories are available:

**Basic registration**
Geoforum offers all companies that provide services or products within geotechnical and related areas free registration. This includes:
- A address and Internet contact (URL and email) in header
- A alphabetical listing only
- A search listing after Market Guide members

**Standard Membership**
Standard Membership includes:
- A address and Internet contact (URL and email) in header
- A contact person with email address
- A logo
- A company presentation on one page, with on-line publishing and indexing from password-protected Web platform

Annual Fee: 250 EURO

**Premium Membership**
Premium Membership includes in addition:
- A company presentation on up to six pages, with on-line publishing from password-protected Web platform

Annual Fee: 650 EURO

**Gold Membership**
Gold Membership comprises in addition:
- A company presentation on up to 20 pages, with on-line publishing from password-protected Web platform
- A feature presentation on Geoforum startpage (random listing), at least 10 times more frequently than Premium Membership

Annual Fee: 2250 EURO

The site seems to be well constructed and is relatively easy to navigate around with drop down menus at each level. A search of the directory, using “New Zealand” as the search parameter, returned 10 individual entries. These contained information on the areas of competence and speciality of the individuals as well as contact details (where they had been provided).

You can list your company and find companies to be your partner for projects, especially where you need a presence in a country or a speciality not held by your own company. You can easily identify companies, using the powerful search and filtering functions of the Geo Market Guide database. With a mouse click, you can find your local supplier or a global partner.

All company information can easily be published and updated from a password-protected member area directly via the Internet. Business activities, products and case histories can be submitted on-line, using predefined forms. In addition, the geotechnical activities of companies can be indexed in the competence database of Geoforum. By registering your products and services, projects and other activities in the searchable database of the Geo Market Guide, information about your company becomes accessible to potential customers and clients worldwide.

You can search for the conference or event you would next like to attend. The search can be done by country (very useful), date and subject. I went looking for a landslide conference somewhere in the West Indies during the NZ cricket team’s visit in June 2002. Unfortunately my search indicated that there were no landslide events in 2002 anywhere in the world. Never mind. I guess the events have to be posted by those who know about the site. Fortunately the site allows you to add an event.
At a technical level the site offers the following:

• A virtual database of piling systems. It includes a classification of more than 60 pile systems and detailed description of the most common installation methods.

• An information system on geophysical site characterisation methods for geotechnical and geo-environmental applications. The contents on this interactive database are gradually being expanded and updated, in co-operation with ISSMGE Technical Committee 10 on ‘Geophysical Site Characterisation’.

• A text and publications page is reserved for publications specifically developed for publication via the Internet.

• A unit converter page offers instant conversion between the SI system and other unit systems. Very useful for the less numerate of us.

• A discussion page where anything from “Where can I get a job” to “Pressure plates on pipe piles” can be posted and discussed.

• A multi-lingual dictionary of geotechnical terms in four languages. The number of languages and entries in the dictionary will be gradually expanded. The contents of the dictionary can be modified and up-dated directly on the Internet. However, my search of a definition of “liquefaction” turned to jelly and failed.

The Geo Organisations Guide allows research and professional organisations to be listed, such as the ISSMGE. The aim is to facilitate contact with professional organisations that are active in the geotechnical area. Registered organisations can present their main activities as well as a listing of committees with direct links to the respective sections of the organisation homepage. The NZ Geotechnical Society will have to consider being listed here.

This site is worth visiting and even listing your services on. Happy surfing.

Phil Glassey
Geological & Nuclear Sciences

---

Check it out - we are online!

• New NZ Geotechnical Society website

• Regularly updated

• Has a comprehensive list of what is on

• Includes the Shear Vane Guidelines

www.nzgeotechsoc.org.nz
An update on progress within the recently formed Australasian Chapter of the International Geosynthetics Society (ACIGS) follows. The latest meeting was held on 30 April 2002 at Golder Associates Melbourne offices, with myself participating via tele-conference.

The draft ACIGS By-Laws have been accepted subject to International Geosynthetics Society (IGS) ratification and Australian approvals for various legal and tax requirements for the Society are near completion. The interim steering committee have made good progress and nominations for formal inaugural Office Bearers were received as follows: President – Malek Bouazza (Senior Lecturer at Monash University, Melbourne), Treasurer – Fred Gassner (Golders, Melbourne), Secretary – George Fannelli (Amoco, Sydney). The AGM is scheduled for 24 June 2002 at 5pm (Australian east coast std) via tele-conference and the writer will make the Auckland office of Maccaferri available to any Auckland based members or prospective members who wish to participate.

Planned activities for this year include:

a) Probable two or three lectures on Filtration and Drainage from John Bowders of University of Missouri, Columbia, in July in Australia.

b) Lecture tour by Dr Richard Bathurst (current President of IGS) either before or following the showcase IGS Conference in Nice, France (22-27 September 2002). These lectures will be held in Melbourne, Sydney, Brisbane and Auckland. More on this exciting opportunity for NZ Engineers soon.

There are currently eight New Zealand members of ACIGS and additional membership from individuals within the Consulting and Construction sectors is encouraged in this fledgling but growing group which aims, amongst other objectives outlined in the appended information, to keep members informed of design practices using geosynthetic materials in engineering.

Inquiries can be sent via myself in the first instance or direct to ACIGS at the contacts shown on the form.

Chris Brockliss
Maccaferri NZ Ltd
Phone: 09 634 6495
Fax: 09 634 6492
Email: cbrockliss@maccaferri.co.nz
ACIGS Membership Application

Membership of the Australasian Chapter of the International Geosynthetics Society is open to individuals or corporations “engaged in, or associated with, the research, development, teaching, design, manufacture or use of geotextiles, geomembranes, and related products or systems and their applications, or otherwise interested in such matters.” The annual fee for membership is A$120 for individuals and free for student members.

Title (circle one): Mr  Ms  Dr  Prof  Other ____________

Position: __________________________________________

First Name: ________________________________________ Last Name: ________________________________________

Company Name: ______________________________________________________________________________________

Address: ________________________________________________________________________________________________

City: ______________________________________________ State: ______________________________________________

Postal Code: ______________________________________ Country: __________________________________________

Telephone: ________________________________________ Fax: ________________________________________________

Email:______________________________________________

Send this completed form to:
ACIGS Tel: +61 3 9905 4956
Monash Uni LPO Fax: +61 3 9905 4944
PO Box 8003 Email: malek.bouazza@eng.monash.edu.au
Clayton, VIC 3168
AUSTRALIA

What is your primary activity? (tick one only)

Consultancy Contractor Commercial Testing Service
Education/Research Geosynthetic Installer Geosynthetic Manufacturer/Distributor
Regulator Waste Management Landfill Operator
Other: please specify __________________________

Note: ACIGS is not GST registered, therefore doesn’t attract GST

Mode of Payment

Membership Fee: Individual A$120

☐ Draft sent to: Commonwealth Bank ☐ or Cheque Enclosed
Kew Branch, VIC 3101
Account number: 063142 1021 3604

☐ or Credit Card (circle one): Mastercard Visa Bankcard
Account number: ______________________________ Expiry date: __________________
Name on card: ______________________________ Authorised signature: ______________________________

Signature: ______________________________ Date: ______________________________
The Auckland Structural Group has produced a standard Piling Specification for use in piling contracts in the Auckland region. NZ Geomechanics News asked the following three people to provide reviews of the specification. The specification can be downloaded from the SESOC website www.sesoc.org.nz under the 'Structural Groups' link.

Review by: Tim Sinclair, Tonkin & Taylor Ltd

I foolishly agreed to review and comment on the new Auckland Structural Group Piling Specification. Foolish because a task of this nature is not trivial when faced with a technical document of some 50 plus pages, and needs more than the odd half-hour in an airport lounge to do it justice.

The temptation is to simply look through and list what is missing. This, however, would be unnecessarily negative, ignoring the detail and generally complete content of the document. I therefore start by asking the question: Does the specification meet its intended purpose?

I assume that the purpose of producing this specification is to provide a base document that gives consistency in the design/tender/construction process, and enables project-specific specifications to be more abbreviated and succinct. In this regard, I believe that it will serve the purpose. The specification covers most of the pile types and methods of construction employed in New Zealand, and encourages the use of (separate?) Project Specification Requirements, as detailed in Appendix A. Small projects in particular will benefit (e.g. a warehouse or small office building), for which the specific requirements can be covered by just two or three pages, with the base document merely being referenced.

There are of course, some omissions and questions of detail. The former can probably be covered by the “specific requirements” and the latter will likely be ironed out by usage. In my view, the notable omissions are as follows:

a) Minimum age of driven concrete piles:
Young concrete does not perform well under impact, however high the 7-day strength may be. Piles less than 28 days old will have risk of splitting and spalling under driving, particularly at the head.

b) Slip coating for negative skin friction:
It is now becoming quite common to require a bitumen-based slip coating for minimising the effects of negative skin friction (NSF). These are special-purpose products with particular properties very different to those of the more usual bitumen products. They have high shear strength and rigidity under rapid loading (e.g. pile driving) but very low resistance creep properties under sustained loading.

However, they require special treatment and planning that might not always be obvious to Contractors at tender stage:
• manufacture and supply requires a minimum volume
• thickness of layer is about 5 mm +
• temperature and light sensitive
• coated piles must be stacked together or touching.

c) Bored piles using bentonite:
The use of bentonite suspension to support the ground during boring is not common in New Zealand and hence there may be little need for a detailed specification.

However, it can be very cost-effective in certain difficult ground conditions. On the other hand, effectiveness is very dependent on workmanship.

The mix and method of working must be designed to meet the specific site conditions. If not correct, a number of defects could result, the most serious being the “soft toe” (i.e. settlement of cuttings to base of pile before concrete placement). This has a significant ‘bearing’ on design. It may be necessary to prohibit the use of bentonite for piles designed as end-bearing.

In summary, I would support the use of the Auckland Structural Group Piling Specification and hope that there will be a regular technical review so that it will eventually become a definitive document for New Zealand practice. There are other generalised specifications such as this available (e.g. Institution of Civil Engineers) but the local flavour makes this specification more relevant and user friendly.

Review by: Graeme Jamieson, Bloxam Burnett & Oliver Ltd

I approach the task of reviewing this document with trepidation: as a structural engineer my geotechnical exposure is typically limited to delving with imperfect comfort into the murky zone between a structure (relatively clean from an analytical perspective) and the supporting ground, which I admit to seeing as relatively dirty metaphorically as well as literally.

The piling specification I see as of considerable value. It provides an authoritative and comprehensive reference with the potential to be widely accepted by the entire engineering community, including those representing all parties from owners and designers to contractors and contractors...
specialist subcontractors. I anticipate it being seized with delight by all these parties, and rapidly becoming an industry standard reference. Even such details as the clear listing of project-specific requirements for particular application appear to have been organised to aid the ease of use of the document, and are consistent with its detail.

I am delighted that the much-maligned Hiley Formula features. Despite the imprecision implicit in pile driving formulae they present a method of correlating resistance to the driving of a pile with expected geotechnical conditions, which I see as of genuine value. My preference is to monitor driving resistance continually (if coarsely) with depth for this purpose, and to use Hiley at predicted founding depth to assess capacity (ideally) to crosscheck estimates of ultimate resistance developed from geotechnical parameters. The traditional Hiley coexists happily with the contemporary PDA (Pile Driving Analyser) approach in the document, where in my view they both have a place. However, although my understanding of PDA sits somewhere between nil and not much, I think that calibration is a key issue for its appropriate use - if this is correct it would be very helpful to include some overt guidance or recommendations.

I am an old-fashioned and cheerfully stuck-in-the-mud advocate of designing foundations on a SWL (Safe Working Load) basis. This reflects my desire to see foundations behaving in a predictable manner under service loads, and more particularly performing in a relatively elastic section of their typically very non-linear load-deflection relationship. From this perspective I am very happy with Table C4 identifying Factors of Safety which allow SWL to be developed from ultimate resistance, but I am surprised that no corresponding values of strength reduction factor are provided to facilitate the comparison of ultimate demands with reliable capacities to also facilitate the use of 'strength' methods.

I do note a few areas which could be developed. First, terms such as dolly, leader, and mandrel are commonplace in the piling vernacular but are imperfectly understood beyond. These terms could be included in the Definitions section, ideally with the assistance of a picture, and along with terms such as maximum driving stress and design pile load. Second, expanding the pile types beyond the range described in Section C3.3 to include other common types such as bottom-driven tubes, and the plugs sometimes driven at the base of steel tubes founding in gravels would be useful. Next, editorial comment on the procedure of Section C5.2.1 would be valuable: as presented it seems to require judgement of driving conditions in order to predict pile capacity - this seems fairly esoteric compared with estimating pile capacity directly on the basis of geotechnical properties, which presumably need to be known to assess driving conditions. Why then introduce an intermediate correlation? Perhaps it is useful to predict plant requirements, a procedure with which I am blissfully ignorant. Finally (and a particularly petty criticism) the variables in Sections 4.5 and C3.1 are inconsistent.

These areas are not significant: they are not beyond what are likely to be addressed as a matter of course as the document becomes increasingly widely used; and they do not detract from its fundamental attraction as a potential standard industry reference. In summary I am one of many who I am sure will have cause to be grateful to the authors, while looking forward to a complementary volume covering design and construction aspects.

---

**Review by:** C Y Chin, Senior Geotechnical Engineer, Beca Carter Hollings & Ferner Ltd

The Auckland Structural Group Piling Specification is a welcome addition and could prove to be a valuable reference for the piling community here.

The inclusion of PDA testing in the specifications is good as it provides an additional means of checking driven pile capacities. However, as the testing and interpretation of test results can sometimes be contentious, it was felt that only suitably qualified and accredited persons should undertake such work. For this purpose, a public register of accredited PDA practitioners could be set-up and maintained.

The incorporation of the Hiley formula is welcome – particularly when coupled with PDA testing from which output efficiencies of hammers could be obtained. The use of Hiley's formula is also complementary with the Building Code B1/VM 4, which suggests the use of Hiley's formula as a means of assessing the comparative strength of individual piles at particular sites.

It was felt that the inclusion of specifications for conducting static pile load tests would be useful especially when used in conjunction with and in verifying PDA test results.

Some examples where clarification would prove useful:
- Section 2.1.4 Paragraph 3. There is ambiguity in the definition of commencement of the "time period".
- Section 2.1.5.2 Paragraph 2. Clarify if both the requirements for the maximum outside diameter of the tremie tube need to be met.
- Section 4.9 Item 6. "etc" should be replaced with specific items deemed necessary.

Overall, there is potential for this specification to develop into an industry standard that could be useful for all.
Practical solutions for rockfall protection and slope stabilisation

www.geovert.co.nz

INVESTIGATIONS (Rope Access Techniques)
SOLUTION DESIGN-BUILD
SPECIALIST CIVIL WORKS (Rope Access Techniques)
Installation specialists for leading technology in Rockfall Protection and Slope Stabilisation
Explosive specialist (Rope Access Techniques)

GEOVERT
VERTICAL CONSULTANTS
Subsurface Drainage for Slope Stabilization

The relationship of groundwater pressure to slope failure is well understood; however, gravity drainage does not always eliminate piezometric pressure completely and ensure slope safety. So what can Geotechnical Engineers do about subsoil drainage, high groundwater table and slope instability?

Drawing on 45 years experience in Civil Engineering, the author has written a practical, standard text that covers the topics of groundwater and associated considerations of subsoil drainage flow using granular materials and filters. The handy approximately A5 200 page text fulfills the need Geotechnical Engineers have for a range of practical solutions to lower the water table and maintain a reasonable Factor of Safety.

Kevin Forrester recommends the use of the simplest equipment that will do the job when monitoring slope movement and provides interesting details on surveying, acoustic emission, and the use of transducers. For the simplest monitoring projects, details of the Poor Man’s Inclinometer are provided. He recommends that “for slope stabilization using subsurface drainage, standpipe piezometers are consequently adequate in practically all situations.”

The symbols in the front of the book are well defined however, the diagrams in the book are poorly computer printed with fuzzy lines. Future editions require better printing of the diagrams. There is, surprisingly, in the chapter on Pipes, an advocation of the use of tile drains, which are long gone in NZ engineering usage.

In all, this is an excellent reference on the practical use of subsoil drains for slope stabilisation.

Reviewed by: Paul Finlay

---

**Subsurface Drainage for Slope Stabilization**

**Author:** Kevin Forrester  
**Publisher:** ASCE Press  
**Date Published:** 2001  
**ISBN** 0 7844 0016 4  
**Web shopping on:** www.pubs.asce.org  
**Price:** US $70.80
Groundwater Lowering in Construction - A Practical Guide

Lowering of groundwater is not something that is regularly undertaken on a large scale in New Zealand. However control of groundwater on a smaller scale is an ever-present problem in many areas of New Zealand. In areas such as Hamilton or Canterbury even a small or routine trenching job can require careful planning for groundwater control.

It is not easy to find comprehensive guidance for groundwater control from one source. My previous source of guidance (and I am sure many others) has been CIRIA Report 113 and assorted basic texts and journal articles. There are also a wide number of texts available on hydrogeology, which can tend to be fairly theoretical and hard to digest.

What I have always wanted was a comprehensive guide to the actual techniques of lowering groundwater as opposed to the theoretical modelling of groundwater behaviour.

Groundwater Lowering in Construction is well summarised by the sub-title A Practical Guide. This is a practitioners’ guide to the art rather than a theoretical text. However, as is explicitly stated in the text the book is focused on the use of dewatering as opposed to groundwater exclusion techniques.

To add some interest to the text the first chapter provides some history on groundwater lowering which might not be exactly relevant to a busy designer, but is definitely fascinating. I think we can all agree that the fact we no longer have to use a rag and chain pump for dewatering is a relief to all.

The chapters of the book are structured to follow the broad train of the design process. A broad description of the underlying processes of groundwater systems, their effect on excavations and the methods of control are all summarised in the first three chapters. This provides a good overview of the problems you will be dealing with.

The book then dives into two chapters detailing the process to follow for investigation and design of a groundwater lowering system. This follows the requirements to perform a rigorous investigation, while giving guidance on the most critical aspects for smaller projects where a significant investigation is not appropriate. The design process is then clearly outlined with a range of methods detailed and a number of excellent worked examples. I found that this section was an excellent guide that would lead even a relatively inexperienced engineer through the essential processes. I think the most appreciated aspect of this section was the range of sensible design tools provided for use in design.

Where I learnt the most in the book was the section, following the design guides, that details the various techniques and equipment available for dewatering. Each method from the various forms of wellpoints through deep wells to such unusual techniques as electro-osmosis, was described in some detail. All descriptions included a thorough summary of the advantages and disadvantages of each technique.

The final chapters of the book cover related matters that need to be considered in your design of a groundwater control system. These range from monitoring of your system through to the potential side effects of the lowering of groundwater. Important information is also included on maintenance of groundwater pumping systems.

As a practising (one day I will be good enough to be one) engineer I feel this book will be extremely useful whenever I encounter any groundwater issues in the future. Any engineer (or contractor) will appreciate the non-technical style of the writing that makes extracting required information from the text easy.

I am sure this book will rapidly become dog-eared and regularly borrowed by others in my office. If you can only have one text on groundwater in your shelf this would be an excellent candidate for the job.

Reviewed by: Chris Bauld, Tonkin & Taylor Ltd

Groundwater Lowering in Construction - A Practical Guide
Authors: P M Cashman and M Preene
Publisher: E & F N Spon Press
Date Published: 2001
ISBN: 0 419 21110 1
Web shopping on: www.amazon.com
Price: UK £80
This publication is about a ‘new line of thinking’ to treat granular soil mechanics. It is a deformation-based approach emerging from experimental research. Applicability is claimed in rockfill embankments, and improvement of weak soils with sand drains and stone columns.

The approach is based on the observation that on pouring granular material on the ground, it forms the shape of a right circular cone, with the sides at an angle of repose, $\theta_1$, with the horizontal. The angle of repose is considered close to the angle of internal friction $\theta$ for granular materials. It is realised that the shear stresses developed inside the assembled media, the effect of the Earth’s radial gravitational field, and the particle sphericity and roundness create the conical shape.

The observation extends to the relationship that exists between the height of the cone and base diameter via $\theta_1$. The external inclined surface represents a zero stress surface. The shape of the cone formed by the granular materials exhibits a relationship between axial stress, shear stress and displacement. By creating appropriate physical and mathematical models, one can develop a series of stress-strain, deformation relations for granular media under stress. The approach is challenging. As with any new ideas, there may be sceptical responses, as is the case with introducing any new theories and hypotheses.

I felt confused at the talk about a heap of granular materials in a state of incipient motion and the emerging angle of repose, and then presenting tables based on calculations using the derived formulae for materials with $\theta$ values of 70 to 90 degrees.

The derivation of relationships for vertical and radial strains based on the collapse of a cylindrical shape into a right circular cone is also confusing. The conical shape can not be guaranteed at any stage of the collapse, in addition to the fact that the outer sides are not zero-stress surfaces when the materials stabilise in the final right circular cone shape at the bottom, nor at any intermediate stage while deforming from the initial confined cylindrical shape into a gradually collapsing conical shape. This was the basis of the relationships for the applications of sand drains and stone columns.

The room for formatting improvement and typo correction is huge e.g. the mixed use of $\gamma$ as density at times and unit weight at others, Chapter 24 is the appendix of Chapter 23, etc.

Chapter 22 is fascinating! It is a literature review on the properties of granular materials.

By and large, the presented ideas, tables and simple formulae are a temptation to use and make comparison with other design/analysis methods and measured behaviour of granular materials. I admit that I am pleased to have ‘earned’ a copy of the book.

The book is in 470 pages of 220 mm x 280 mm size.

Reviewed by: Alaa S. Ahmed-Zeki
Senior Geotechnical Engineer
Hydro Tasmania-Consulting
Testing, Observation, and Documentation

Having noticed the title of the book and without being able to flick through it in advance, I had the feeling that I was going to sink into the honey of sophisticated GEOTECHNICAL TESTING to uncover the complex and ever challenging behaviour of ground materials, coupled with OBSERVATIONS on actual full-scale earth structures. DOCUMENTATION was on its way to me to enjoy the reading. All those dreams vanished as soon as I received the 170 mm by 250 mm sized 100-page book by mail. Why do we need it, and what's wrong with books like the USBR Earth Manual, I asked myself.

The book is more of a "Practical Soil Engineering for Technicians", which is also helpful for graduate engineers starting their 'mud-like' experience and long term association with the dirt, possibly believing that it was the source of creation.

The book is essentially a guide for the supervision and testing of earthworks in the field as well as laboratory index property testing, and focuses more on works like subdivisional development and earthworks during the investigation and construction stages. However, it is not a 'Kiwi recipe' given that it does not talk about the majestic 50 mm hand auger, our golden Pilcon shear vane, and the 'Human Calorie Scale' Scala penetrometer.

The ASCE book describes in a clear and easy way the methods of classifying soils, essential information on drilling, soil logging and sampling, and field density testing using the sand cone as well as the nuclear densometer. It illustrates a few examples on practical issues in the field while supervising earthworks and the method for recording and documenting observations. A 'very' quick touch on the influence of geology is there, as well as a highlight of importance of safety in the field.

For about US$21, I would recommend it for a person that shows a commitment to work in their first couple of years of geotechnical engineering experience, so long as I don't have to pay!

Reviewed by: Alaa S. Ahmed-Zeki
Senior Geotechnical Engineer
Hydro Tasmania-Consulting

Testing, Observation, and Documentation
Author: T Davis
Publisher: ASCE Press
Date published: 2001
Web shopping on: www.pubs.asce.org
Price: US $28.80

INTOROCK DRILLING
Geotechnical Investigation
Construction & Drainage Drilling
Mob: 0274 488 248  Ph: 09 268 1046
Fax 09 268 1036
PERMATHENE

• Dampstop™ Concrete Underlay/DPC
• Epinol™ Gas Barrier Membranes
• Syntex™ Woven & Nonwoven Geotextiles
• Landlok™ Biodegradable Erosion Control Blankets
• Pyramat™ 3-D Erosion Control Matrix
• Pave Dry™ Nonwoven Asphalt Overlay Fabrics
• Permaliner™ Geomembranes

• Erosion Control  • Geogrids  • Gabions & Mattresses
• Silt Fence  • Safety Fence  • Drainage Systems

Concrete Underlay  Soil Stabilisation  Containment Lining

Landfill  Slope Stabilisation  Erosion Control

Insist on Permathene!

For more demanding applications, contact us for Earthstopping Solutions or visit our website at www.permathene.co.nz

Permathene Ltd.
P.O. Box 71 015, Auckland 7
404 Rosebank Rd, Avondale
Email: info@permathene.co.nz
Tel 09- 820 7231
Fax 09- 820 7429
Free Fax 0800- 888 333
www.permathene.co.nz

South Island Representative
Philip Elvidge
Ph/Fax 03- 357 0012
Mob 021 339 277

Permathene Pty Ltd.
P.O. Box 553 Sunshine
Melbourne, Victoria
Tel: 1-800 142 331
GEOTECHNICAL DRILLING & ENVIRONMENTAL SPECIALISTS

- Cone Penetrometer Testing (CPT & CPTU)
- Standpipe Installation
- Soil Moisture Probe (SMP)
- Conductivity Measurement
- Hydrocarbon & Leachate Detection

- Core Drilling (Wire Line)
- Wash Drilling
- Open Auger Investigation
- Pile pre Drilling
- Air Drilling
- Hollow Stem Auger (Monitoring Wells, Core Samples)
- In situ Geotechnical Testing

Please contact us for more information or quotations.

Terrence: 025 941 174    Phillip: 025 304 554
Email: perry.geoprobe@clearnet.nz
Graduate students and staff working together on research projects over the last year have recorded the following interim results:

**Rhyolitic Deposits and Slope Instability**

1) Rhyolitic deposits in the Auckland region are derived from the Taupo Volcanic Zone and include airfall tephra and reworked and redeposited sands and silts. Many of the sands and silts are weathered and, especially in the case of the silts, give rise to highly sensitive, irregular layers, which can contribute to slope instability.

An example in the Papakura area, adjacent to the Southern Motorway, contains weathered rhyolitic glass, pumice and fine quartz-rich sediments. There are layers of accretionary lapilli, presumably formed and deposited during the fall-out from the eruption cloud as it reached the Auckland region. All these fine materials develop a sensitive microfabric as a result of weathering and the accompanying production of halloysite clays. These tend to form coatings and connectors between and around micrograins, assisting the formation of a porous and fragile fabric. Mounds of somewhat similar but coarser material are found some hundreds of metres distant across the motorway.

The deposits and landslide geomorphology have been the subject of a student project, which has been reported upon and is being developed further into site models. In these the geometry, geotechnical properties and microfabric of the silts and clays appear to be critical and may show some tendency to be related to paleo-topography. Further analysis is continuing.

2) Rhyolitic tuff deposits at the Orakei Korako Geothermal Field have been highly altered by hydrothermal activity, in particular by acid sulphate steam alteration in the more elevated parts of the field. A cave system has provided access and an insight into the geotechnical changes through a significant thickness of the altered tuff. A progressive increase in porosity and an accompanying reduction in strength parallel changes in the alteration mineralogy and microfabric of the tuff. Materials investigated include clay-size pure quartz deposits formed as a silica residue from intense and complete hydrothermal alteration. This porous, delicate material is highly sensitive but contains no clay minerals at all. The role of pervasive fractures in the location and extent of alteration is being assessed. There is the possibility of deep-seated slope movement having affected the overall development of the topography. Hydrothermal solution and transport of material has probably also played a part. The balance between slope failure and solution is being assessed.

3) The relationship between thermal alteration, migration of geothermal activity, hydrothermal eruptions, faulting and landslides is being investigated at and near Te Kopia on the Paeroa Fault, south of Rotorua in the Taupo Volcanic Zone.

**Studies on Tongariro Volcano**

Another study on the flanks and northern ring plain of Tongariro is examining the sequence of avalanche deposits which have been deposited since the major collapse event that gave rise to the Te Whaiau Formation – a clay matrix debris flow deposit of widespread extent in the area and which was formed before 26,000 years ago. Activity since then has built up a series of debris avalanche deposits and these give some idea of the activity of the volcano. This work is also in progress.

**Warwick Prebble**
University of Auckland
State Highway 6 - Nevis Bluff Stabilisation Contract

**Client** - Transit New Zealand  
**Consultant** - Opus International Consultants Ltd

In September 2000 a rockfall of 10,000 m³ of schist debris occurred at Nevis Bluff in Central Otago, onto State Highway 6, one of the main arterial routes into Queenstown. The rocks narrowly missed traffic on the highway and it was nothing short of chance that no-one was killed. The road was completely blocked. This was the second time that such a failure had occurred from the bluff in the last 25 years (previous failure of similar magnitude was in 1975).

Opus International Consultants Ltd managed the emergency clean up operation. This required an initial effort to clear the road of debris, and to remove overhanging material from the face above the backscarp. Because of the difficult nature of the site (the bluff stands 120 m high at 70° rising up immediately from the State Highway), and associated rockfall hazards, innovative techniques were employed to remove material from the face. These included helicopter ‘bombing’ (using 500–900 kg sandwich bombs of powergel and sand) and aerial sluicing to remove loose debris.

Since the emergency clean up Opus have implemented a thorough programme of observation and monitoring of the face. As a result of this work several potentially high risk features have been identified and removed in a controlled manner.

Opus have now carried out a full risk assessment of the bluff face and calculated, using recognised techniques, the risks to motorists from rockfall of any size including another catastrophic failure. Stabilisation options have also been looked at and include a tunnel through the bluff, rockshed, rockbolting, mesh protection, debris fence, half-bridge, road realignment and viaduct. All have proved extremely costly, ranging from $6M to $30M. The stabilisation option which has been selected with Transit New Zealand became one of cost versus risk trade-off. It will require systematic removal of significant features (up to 1000 m³ size) over the next three years, prioritised in terms of risk.

The work is unusual in several respects. Firstly, the project will require removal of around 3000 m³ of highly fractured unstable rock from the face. Secondly, the work will require particular expertise in abseil access, drilling on a steep face and the use of explosives above a highway which can only be closed for short periods of time (0.5–1 hour). Finally, the Contract document will have to be prepared in such a way that it can be awarded to any organisation with the right skills and experience, even though they may not be familiar with this particular bluff and its unique characteristics. It will have to thoroughly cover health and safety requirements, safety of the travelling public at all times, use of explosives on steep rock faces, definition of precisely what is to be removed (and what is not!) and reinstatement requirements. There are many difficulties to overcome including defining the precise rock quantity to be removed, predicting the likely behaviour of the face during prolonged blasting, anticipating potential overhangs after blasting, and restricting excessive vibration levels, to name a few.

This is not run of the mill work and promises to be a difficult but intriguing project for the coming years.

**Phil Woodmansey**  
Opus International Consultants Ltd, Dunedin
Cosseys Dam is a 40 m high, zoned earthfill dam located in the Hunua Ranges. The dam is owned and operated by Watercare Services Limited and impounds 14 million cubic metres, about 15% of the total water storage for supply to the Auckland Region.

URS were engaged in May 2000 to evaluate the risk profile of the dam and investigate options for upgrade. Investigations revealed that the underdrainage system was comprised of materials which were not filter compatible with the embankment materials. Following URS’s risk assessment Watercare instigated risk reduction measures, which included lowering the reservoir level and increasing the surveillance monitoring. A downstream shoulder reconstruction was selected, from a range of options, as the best solution considering a broad range of objectives and constraints.

Factors considered in developing the remedial works included the potential impacts of lowering the reservoir, social and environmental effects from the construction activities, the safety of the dam during construction, and the risk of residual defects remaining within the dam after completion of the remedial works.

The key features of the remedial work are summarised as follows:
1) Lower the reservoir to a safe level.
2) Remove the incompatible underdrain materials from below the dam with a large downstream excavation.
3) Replace the underdrain beneath the core of the dam with compacted core material where accessible and grout to seal the remaining underdrain that is not accessible.
4) Reconstruct the underdrain with filtered drain material below the shoulder of the dam, downstream of the reconstructed core.
5) Place an inclined chimney drain/filter across the reconstructed core and abutments to provide filter protection and control ongoing seepage flows into the underdrain.
6) Reconstruct the downstream shoulder of the dam and crest roadway.

The time required to obtain the Resource Consents for this project presented difficulties in starting the works early enough to complete them within the 2001/2002 season. During the course of detailed design it became necessary to revise the drawdown level to provide appropriate dam safety during construction. This required additional Resource Consents to be obtained and it was apparent that these would not be granted in time for the start of construction in the 2001/2002 season. A practical compromise was to adopt a staged approach to the works with the first stage comprising a small excavation and rebuild of the toe of the dam whilst maintaining an operational reservoir. This provided improved dam safety in the interim period between the two stages of dam remediation.

The restricted nature of the site presented unique
problems in terms of the temporary stockpiling of the excavated embankment materials. All material from the Stage 1 embankment excavation was stockpiled on site before being reused to reconstruct the toe of the dam, to the required profile. This ability to stockpile material on site and to continue to draw water from Cosseys Dam during Stage 1 allowed Watercare to proceed with the works and meet environmental and public expectations. Resolution of the remaining Consents for the Stage 2 works has been achieved and the Consents have been issued, ahead of the start of the 2002 construction season.

A joint venture between McConnell Dowell and Ross Reid won the contract following an interactive tender process developed by Watercare’s Project Manager and URS. The Stage 1 excavation works commenced in January 2002 following preparatory works on the Wairoa River Bridge and the Cossey’s Access Road, which were carried out in November and December 2001.

The existing array of piezometers and seepage weirs provided the means to validate the design predictions as the lake level was lowered to the target level for excavation. During the excavation works the embankment was monitored using the existing array of piezometers, seepage weirs and survey monuments as well as an array of survey reflectors and inclinometers installed as part of the contract works.

The Client has engaged URS to provide construction support services, which includes the role of Dam Safety Engineer to assess any dam safety or design issues as the project progresses. A range of contingencies had been formulated for unplanned events such as excessive deflections, seepages or storm events and these were incorporated into the Reservoir Management Plan. The contingencies included provision of emergency stockpiles of filter material being on hand to control unexpected seepages and operating rules to route flood events through the free-discharge valves at the base of the valve tower. This plan meshed with Watercare’s existing Emergency Preparedness Plan to create a clear action plan for any eventuality.

The Stage 1 works have now been completed on programme without any lost time accidents or incidents. The condition of the old underdrain will probably be the subject of some case study work to evaluate the design in light of the exposed conditions. Deformations and seepages were in line with predictions. The works have been valuable to everyone involved in getting a preview of the construction issues that may arise during Stage 2 and in testing the construction methods that have been formulated. Valve tower strengthening works will continue through the winter and the Stage 2 works are due to commence in September 2002.

Neil Jacka
URS New Zealand Ltd
Remediation of the lake in the Rippon Lea Estate in Victoria is a part of the National Trust of Australia (NTA) Site Remediation Action Plan. The long-term goals of the action plan are to identify contaminated NTA sites, investigate and, if necessary, remediate them within a 40-year period. Several hundred of these sites involve contaminated sediments. Without remedial action the sediments/sludge would cause problems for many decades. The Rippon Lea Estate lake contained approximately 150 m$^3$ of sludge and the size of the lake is 2–3 acres x 1.2 m deep. Several investigations and studies were carried out to determine how and under what limitations clean up could be performed. An alternative remediation method was selected that involved vacuum dredging and dewatering the sludge using high strength geotextiles tubes as filters. Once dewatered the dredged material was to be disposed of in a landfill.

For this treatment process, a work area was set up off site in the Australian Broadcast Corporation car park nearby. A pontoon with a pumping unit was designed, constructed and installed in the lake. The pumping system was commissioned and pumps were altered to ensure the most appropriate flow rate of sludge/water mixture was achieved. A flexible pipeline was installed from the pump to the temporary dewatering area and from there to the mobile wastewater treatment unit. The sludge was pumped from the lake to the dewatering system; excess water from the dewatering system was pumped through the water treatment plant and returned to the lake as treated water.

The dewatering system consisted of two geotextile tubes; 20 m long and 1.4 m diameter, each fabricated from Syntex 4x4 high strength woven geotextile. This product has sufficient tensile strength to withstand the stresses associated with pumping. The fabric opening size may seem large when compared to the grain size of the dredged material, and might lead to the question of how efficient retention of solids could occur. The answer partly lies in the fact that a filter cake forms on the inside of the fabric shell, thus creating the equivalent of a two-stage filter. Filtration efficiencies above 98% are not uncommon for fine grained dredge materials filtered through Syntex 4x4.

The dredged material was pumped into the tubes using a trash pump through a 150 mm discharge line. The water percolates out through the fabric, leaving dense sludge/sediments in the tube. The tubes were pumped until full, reaching heights of 0.8–1.0 m and widths of 1.6–1.8 m. On completion each 20 m long tube contained nearly 20 m$^3$ of dry sludge material. Prior to filling the tubes, the dewatering area was lined with a nonwoven geotextile to prevent local erosion, which occurs as water is released from the tube.

---

**Rippon Lea Estate De-Sludging Project, Melbourne**

**Client** - National Trust of Australia (Victoria)  
**Geosynthetics Consultant** - Permathene Ltd, New Zealand  
**Contractor** - Green Waste Environmental Engineering, Australia  
**Date** - December 2001

---

Filling operation of the high strength tube.
Dewatering and consolidation in the geotextile tube reduced the volume of the dredged material by a factor of 7 to 8 within two to four weeks of filling the tube. Thus, 450 m³ of material was initially dredged and approximately 56–64 m³ placed at the final disposal site, along with the geotextile tubes. The dredged material was highly cohesive and had a high organic content. This stage of de-sludging was carried out on a small trial section of the lake and it is expected that greater productivity can be achieved in a larger section of the lake. The ideal process rate for the treatment is between 3–5 m³ per hour.

The project was very successful with Syntex tubes providing a cost-effective solution to a very difficult dredging project. The tubes dewatered the material at a greatly accelerated rate when compared to open-air retention, and eliminated safety issues inherent with disposal pits. The de-sludging provided a significant increase in the storage volume of the lake to allow for reticulation of the Rippon Lea Estate gardens.

This new system provided significant savings and only two tubes were needed for the two-month operation. Although dredge-filled geotextile tube technology has been used for many years, recent high profile projects have brought attention to the industry. The technology and the industry are still young, but newer and better protocols are being realised on a daily basis. The future certainly looks bright for dredge-filled geotextile tubes.

For further information contact:
Moninder (Witty) Bindra
Permathene Ltd – Civil Engineering Division
Phone: 09 829 0741
Email: bindra@permathene.com
This article I want to take a small diversion from the main points of discussion to date and talk a little bit about stress paths and strain paths. These are extremely useful tools for the geotechnical engineer for either hand or numerical analysis. However, there needs to be careful consideration of the meaning/use of stress paths particularly considering the non-linear behaviour of soils.

First, a small reminder on what a stress path represents. They are often referred to as a p-q diagram, with p being the mean stress at a point in a soil continuum, while q is a measure of the applied shear stress at the same point. As the point is loaded, the change in stress, or stress path followed tells one something about the changes in the soil state (dilating or consolidating, peak or residual strength etc). However, there are some complications in that both the mean stress and applied shear stress have different definitions depending whether one is working in two dimensions or three dimensions. Fortunately, a convention is being adopted where p and q refers to the three dimensional measure whereas s and t are used for the two dimensional measure. The various definitions of the measures are given in standard references such as Lambe and Whitman (note the 2-D measure), C R Scott, and others. It is also worth noting that there is a difference between the total mean stress and the effective mean stress, whereas the applied shear stress remains the same in each case.

Conventional theory based on isotropic linear elastic (one or two phase) medium states that regardless of the stress path followed, the final deformation and state of stress will end up being the same. This is often a convenient shortcut, as it implies that the final stress/deformed state in a loading situation will be the same whether one loads the soil quickly and lets it consolidate, or whether one loads the soil sufficiently slowly that the soil remains in an essentially consolidated state. It is based on this premise that geotechnical engineers have developed predictive models for ultimate deformation without going through a complicated analysis.

Do real soils behave like this? Clearly in the extreme they do not. If any applied loading to a soil mass is so rapid or large that the soil fails, the large deformations that result do not correspond to the elastic case. The same argument applies to any case where the strains exceed a threshold.

This can be demonstrated by using the same models and loading from previous articles (elastic, mohr-coulomb, elasto-plastic hyperbolic and elasto-plastic cam-clay with creep; refer to the first article in Geomechanics News June 2000 for description of the models). Looking at a soil element underneath the centre-line of the embankment we compare the maximum deformation of the models with the stress paths for both undrained, followed by consolidation, and drained loading. These comparisons are graphed and the maximum deformation is included in the legend for each case.

The results are startling to say the least. In comparing the results, it is important to note that each model is a description of the same soil under the same loading. Each model tends to focus on particular aspects of soil behaviour, such that none of the curves could be called ‘correct’. However, each model contains an element of truth so depending on your point of view, they could all be considered ‘correct’. In order to understand what is going on, I will try and give a description of each case.

**Elastic (Figure 1):** Drained loading heads off at an angle characteristic of the soil and original stress state of the element. Undrained loading sends the stress path up vertically (mean stress remains the same, but shear stress increases). Subsequent consolidation moves the path approximately horizontally (there is a small increase in q - to be discussed in a later article) to meet the drained line, all more or less as per classical theory.

**Mohr-Coulomb (M-C) (Figure 2):** The drained loading stress path is identical to the elastic case above and there is no non-linearity present. Whilst the undrained loading...
stress path follows a similar vertical route to the elastic case the shear stress present is only 2/3rds that of the elastic undrained case. This means that there is some sort of stress re-distribution going on within the soil mass somewhere. The undrained loading and consolidation stress path is similar to that for the elastic case above but it is important to note in this M-C model the drained loading and the undrained/consolidation stress paths do not ultimately yield the same stress state or deformation (245 mm cf 314 mm). Remember this is for a simple embankment with a conventional Factor of Safety.

**Hyperbolic (HSS) (Figure 3):** The undrained loading mirrors the Mohr-Coulomb curve, except that the stress path moves more smoothly into the consolidation phase. Interestingly, the drained path moves to a point close to the consolidation end point of the undrained path. This is worth investigating further (perhaps some other time) particularly since the ultimate deformations are all very different.

**Cam-Clay (SSC) (Figure 4):** This plot is completely different to all the others. The drained loading stress path is very flat and the undrained loading stress path moves to the left from the initial stress state. This implies that the effective stress is decreasing with increasing shear. This is a very dangerous situation if real and there is enough evidence that this happens in certain soils to not dismiss this path out of hand. The consolidation curve ends at a position mid-way between the stress state for the drained Cam-Clay case and the stress states predicted from the Hyperbolic soil model. The differences in reported deformations are quite dramatic.

It should be noted that the mean stress end points are all very similar, as we would expect since the vertical load is identical in each case. What is different is the level of shear stress. It is interesting that the final case (Cam-Clay) models the most dangerous condition during construction and the best condition (lowest shear stress) in the long term. It should also be noted that the two non-linear models needed to have the strength parameters tweaked to ensure that failure did not occur during construction, despite the Factor of Safety being more than adequate based on the Mohr-Coulomb approach. Many embankments on soft ground have failed because these issues have not been recognised.

Sergei Terzaghi
Sinclair Knight Merz
sterzaghi@skm.co.nz

It is intended that a course on ‘Numerical Methods in Geotechnics’ will be held towards the end of this year, or early next year, including many of the topics covered to date and their practical application. If you are interested please contact me.
A diverse range of low energy fluvial, lacustrine and shallow marine sediments form an important group of geotechnical materials in the Auckland region. The sediments comprise clay, silt, sand and peat. Pumiceous silt and sand is relatively common. They are collectively assigned to the Tauranga Group (up to 2 million years old) and, typically blanket terraces, ridges and the broad elevated topographic surfaces that surround the Waitemata and Manukau Harbours. While cut slopes in these sediments are generally stable at angles of 1V:4H, there are examples where relatively large failures have developed in much flatter slopes. Conventional back analysis of failures by limit equilibrium methods can yield small effective angles of internal friction on low angle hypothetical failure planes. Effective strength parameters in the order of c' = 0, phi' = 7° are not uncommon. This is often at odds with laboratory testing results which can give much higher effective...
strength parameters in triaxial and ring shear testing on the apparent failure materials. This can lead to uncertainty in the prediction of engineered slope performance based on these methods.

**Strength - Strain Behaviour of Geosynthetic Reinforced Soils**
Craig Davidson, Meritec Ltd

**Abstract**
This paper compares the strain-mobilised shear resistance of a cohesive soil to that of a non-cohesive soil and the implication for construction of Geosynthetic Reinforced Soil (GRS) walls using cohesive fill materials. Results of laboratory tests indicate that cohesive materials are likely to require significantly greater strains to mobilise their peak shear resistance. The increased strain requirement for cohesive soils is likely to result in greater wall deformations than have typically been measured in GRS walls constructed with granular fill material. GRS designers must ensure that $\phi_{soep}$ (the design friction angle selected for the reinforced backfill) is compatible with the reinforcement and that wall deflections are checked under both serviceability and ultimate limit state conditions when using cohesive fill materials. Specific testing of cohesive materials to accurately establish $\phi_{soep}$ is recommended prior to construction of GRS structures where limitations are placed on the allowable magnitude of strain deformation.

**Geothermal Activity and Hazard in Kuirau Park, Rotorua City:**
**Recent Hydrothermal Eruptions and Monitoring of Geothermal Features**
Marie Slako, The University of Auckland

**Abstract**
Rotorua Geothermal Field lies at the southern end of Lake Rotorua, within the Rotorua Caldera. Kuirau Park is located to the northwest of the Rotorua Central Business District, west of Rotorua Hospital Hill (Pukeroa).

Concerns were raised in the mid 1970s regarding the ‘quieting’ and possible extinction of Rotorua’s geysers and hot springs. In 1986 central government initiated the closure of all Government Department bores in Rotorua and all geothermal wells within a 1.5 km radius of Pohutu Geyser. The purpose of the shut down was to restore pressure and water levels in the geothermal aquifer. Closure of the private bores and reinjection of commercially abstracted fluid back into the geothermal aquifer has seen Rotorua’s geothermal field respond with rejuvenated levels of activity.

The Rotorua Caldera was formed 230,000 years ago following the large eruption that produced the Mamaku Ignimbrite. Following its formation, several rhyolite domes (Rotorua Rhyolite) were extruded within the caldera margins. The Mamaku Ignimbrite, and Rotorua Rhyolite domes act as aquifers beneath the city, and the latter has good fracture permeability. Hot water rises into the rhyolite and flows laterally through the aquifer. Less permeable sedimentary sequences overlying the rhyolite act as an aquitard. Faults allow thermal fluids to penetrate through them, and discharge at the surface.

The Kuirau Fault runs along the eastern side of Kuirau Park and the thermal features within the park and at Ohinemutu to the north are fed from the Pukeroa dome rhyolite aquifer cut by this fault. Geothermal activity manifests as acid-sulphate and alkali-chloride pools, fumaroles and steam heated ground.

This paper examines some possible causes for such failures and the difficulties of applying conventional triaxial test / limit equilibrium slope stability methods. Possible alternative methods are discussed.
Tecco® stabilizes slopes, prevents break-outs and grows into one with nature.

High-tensile steel wire nets are a greenable low-cost alternative to concrete and guncrcrete constructions.

Tecco® is the result of target-oriented development work. The three-dimensional mesh of high-tensile steel wire stabilizes steep slopes: The terrain surface is cleaned and then anchored with a defined force behind the sliding layer by means of the Tecco®-mesh and ground or rock nails. The mesh adapts to the topography and in this way prevents not only slides and deformations, but even rocks from breaking out.

**New Zealand Agents:**
RopeTek Ltd
John Foulds
Mobile 021 452 268
Ph +64 3 545 2268
Fax +64 3 545 2269
Email: ropetek@ts.co.nz
WORLD WIDE COVERAGE

ENVIRONMENTAL SURVEYS
• Sampling • Motorizing Well Installation

GEOTECHNICAL DRILLING
• Site Investigation • Piezometer Networks • Instrumentation • Coring • Push Tubes
• SPT’s • Shear Vane • Hydraulic Piston Samples • Core Orientation (Archway System)

WATER WELL DRILLING
• Town Supply • Domestic • Commercial • Farm • Irrigation
• Large Diameter Soak Holes

SPECIAL PROJECTS
• Barge • Underground • Confined Space • Helicopter Access
• Ground Stability Enhancement • Anchors • Mini Piles

SEISMIC SURVEYS
• Truck • Tractor • Heli Portable

EXPLORATION
• Geothermal • Gas • Oil • Gold
• Coal • Mineral (including wire line coring & RC drilling)

DRILLWELL EXPLORATION NZ LTD
9 Rawson Way, Takanini, Auckland. PO Box 75-360, Manurewa
Phone: 0-9-267 9100 Fax: 0-9-267 8100
Website: www.drillwell.co.nz Email: general@drillwell.co.nz
See what New Zealand's happiest employees are doing for lunch

Chances are, they're enjoying one of our 'Brown Bag Sessions'. That's where everyone from engineers to support staff get together over lunch to hear interesting topics such as, negotiating contracts or innovative investigation techniques. Although these sessions are optional, the turnout is impressive as it's all part of our flexible and friendly culture. It's also why we're one of the best places to work in New Zealand (Unlimited Magazine Jan 01).

Formerly Worley Consultants, our international, multi-disciplinary consultancy offers unique career opportunities for people with specialist skills in engineering, business and environmental fields.

With our award winning culture, career development focus and of course a competitive remuneration package, lunch at Meritec is a very tasty option indeed!

Current Vacancies

Geotechnical Engineers and Engineering Geologists. Graduates or with up to 5 years experience and preferably with a postgraduate qualification. To find out more about these positions and what it is like to work at Meritec, please visit us on www.meritec.org

Meritec Limited
47 George Street Newmarket
PO Box 4241 Auckland, New Zealand
Tel +64 9 379 1200 Fax +64 9 379 1201

www.meritec.org
SOILS TESTING LABORATORY SERVICES

IANZ Accredited since 1977 for more than 40 tests

For an obligation free quote or to discuss your testing requirements, contact our Laboratory Manager Barry Coker

ph +64 0 523 5626
fax +64 9 523 5627
email contactus@fel-nz.com
www.fel-nz.com

or come in and see us at
216 Great South Road, Newmarket
(Patey Street entrance)
The Ohinau Drive slope failure has occurred at the northern base of the volcanic Tahanga Hill, Opito Bay. The failure has affected a recent subdivision on Ohinau Drive situated immediately adjacent to the hill.

The slide is a complex, variable depth failure encompassing several differing geological units. It extends a distance of 170 m from headscarp to toe with an estimated maximum width of 130 m. It comprises both shallow-seated and deep-seated failure mechanisms to a maximum depth of approximately 20 m.

In the winter of 1996 slope instability was recognised following development of a headscarp and ongoing disturbance to kerbing and manholes.

Investigations undertaken revealed complex geological conditions generally comprising hydrothermally altered andesite partially overlain by basaltic debris and weathered basalt lava. Artesian water pressures were encountered within the andesite. The investigation results indicate that both a deep-seated failure through the underlying andesite and a shallow-seated movement involving the basalt debris were recently active.

A geotechnical model was constructed along two cross sections with computer aided stability analyses undertaken. Target groundwater levels were determined to achieve a satisfactory Factor of Safety to allow future subdivision development. Drainage installation and monitoring is yet to be established following liaison with Council.

**Introduction**

The Ohinau Drive slide is located at Opito Bay in the eastern extent of Kuaotunu Peninsula, northeastern Coromandel Peninsula (Figure 1). The slope failure has developed largely beneath vacant lots of a recent subdivision extending southward into a pine plantation covering Tahanga Hill (Figure 2).

At the time of the author's involvement the failure presented as a non-catastrophic event, but one which if left, might progress to damage far more seriously the existing roading and drainage infrastructure, and to damage future houses that might, if permitted at all, be built on the subdivision. The purpose of the author's involvement was to analyse the failure and to provide advice on possible stabilisation measures to achieve geotechnical factors of safety consistent with the territorial authority's expectation for issue of house building consents without restraint or restriction.
Upon a surveyor’s check, lot boundary pegs placed in 1993 were shown to have undergone lateral movements of up to 0.5 metres. By using Global Positioning System (GPS) survey methods, useful comparisons of the original boundary peg positions with their displaced positions were able to be made, and these were periodically rechecked at approximately six monthly intervals. This provided information on the extent and direction of the slide, revealing a somewhat fan shaped effect.

It included an unexpected movement in one survey mark at the centre of a quite deep local gully (labelled western gully in Figure 2) indicative of deep-seated movement sufficient to carry the whole overlying topography as a block.

By agreement with the owner the local authority imposed Section 36(2) Building Act encumbrances on all land titles deemed to be affected, to remain until such time as the land could be satisfactorily stabilised.

Section 36(2) of the Building Act 1991 allows territorial authorities to issue building consents on properties subject to subsidence, provided the building work will not accelerate, worsen or result in further subsidence. Such an encumbrance typically detrimentally affects the property value and insurance cover.

Landslide Investigation

Whilst a complete definition of the lateral extent, depth and interfaces between the various units has not been determined, a geological/geotechnical model was developed with a reasonable degree of confidence.

Two phases of investigation were undertaken. An initial investigation in 1997 was performed by Worley C Consultants Ltd (WCL). The second stage of investigation was undertaken by Riley Consultants Ltd (RCL) in 2000. Results of the second stage were presented in Riley Consultants Ltd 2001 report.

The initial RCL investigation comprised a review of existing information including borehole logs, piezometer and land survey monitoring records. From this a preliminary subsurface investigation programme was planned. During the investigation the location of boreholes and test pits were altered as the exploration continued.

A total of five machine boreholes, four hand auger boreholes and five test pits were drilled and excavated over these two phases of investigation conducted by WCL and RCL. A walkover appraisal and review of pre-earthworks topographical plans was also undertaken. No subsurface investigation was undertaken adjacent to the headscarp, as permission from the neighbouring land owners was not forthcoming. The objective of the investigations was to assess the subsurface conditions and identify possible failure surface(s).

Geology and Subsurface Conditions

The geology of the general area has been described by Skinner (1976) and in more recent times Hawthorn (1996). The area generally consists of the basaltic Tahanga Hill surrounded by a mixture of basalt lava/plugs and thick andesite lava deposits.

Geology of the area investigated was found to be more complex than shown on available geological plans with several materials encountered north of the basaltic
Tahanga Hill (Figure 3). In stratigraphic (chronological) order the following units were encountered:

**Fill**
Encountered adjacent to the eastern gully. Inferred to be remnants of a stockpile created during earthworks. Typically firm to very stiff silt with topsoil layers.

**Alluvium**
Firm to very stiff clay and silt, found beneath the fill within the eastern gully.

**Surface Debris**
Consisting of basalt gravels, cobbles and boulders in a silt/clay matrix. Encountered at the northern base of Tahanga Hill and interpreted to extend beneath the adjacent Ohinau Drive. This material was encountered to a depth of nearly 11 m.

**Weathered Basalt**
Highly weathered products of basalt lava, very stiff to hard, inferred to be filling an ancient palaeovalley.

**Lacustrine Deposits**
Stiff silts and clays encountered forming a ‘buttress’ at the landslide toe. These deposits included minor quantities of coarse quartz/flint and rounded mudstone.

**Hydrothermally Altered Andesite**
Underlying the basalt and outcropping north of Ohinau Drive, the andesite was typically of stiff to very stiff consistency.

**Andesite**
Basement rock inferred to underlie the entire subdivision. This is hydrothermally altered and whilst typically of very weak rock strength, is sheared and of soil strength in places. Artesian water pressures were encountered in this material in 1997.

**Failure Planes**
Failure surfaces are often not easily detected in recovered core. However, a distinct failure surface consisting of a striated slickenside was encountered in drillhole R1 at the base of the basaltic debris where it contacts the underlying andesite. A failure surface was also observed in a test pit excavated in the toe heave zone, with movement of highly weathered hydrothermally altered andesite over lacustrine deposits.

**Landslide Characteristics**

**Geomorphology**
Topography near Ohinau Drive is dominated by the 212 m elevation basaltic peak and associated lava flows. A concave depression is evident on the steep sided slopes with a hummocky landscape below. A review of the topographical plan from 1974 (prior to earthworks) indicates a large 'tongue' of material below (north of) this concave depression and the most recent scarp. This ‘tongue’ feature has been obliterated by recent earthworks.

North of Ohinau Drive consists a flat to gentle graded area created by earthworks, where up to 5 m was cut from a pre-existing ridge line. A heave zone of approximately 150 mm height is evident in this area.
Failure Surface
From the earlier study by Worley Consultants Ltd (1997) it was postulated in the RCL report (2001) that either of two failure mechanisms could exist: a) a shallow failure surface involving movement of the surface debris, or b) a deep failure through the underlying andesite. However, a failure along either one of the surfaces alone was not entirely consistent with the surficial expressions of movement (e.g. deflection of boundary pegs, location of tension cracks etc). Despite this inconsistency the combination of borehole information and surface observations did not indicate any plausible alternative failure scenarios.

Accordingly it is concluded that the movement was due to one or a combination of the following two basic failure mechanisms:

- **Surface Debris Failure**
  Reactivation of ancient slip debris moving over the underlying andesite. This failure explains relatively large boundary peg movements (in the order of 0.5 m) in lots adjacent to Tahanga Hill, and the development of a tension crack in the basaltic material at the base of Tahanga Hill. However, this failure mechanism does not explain the andesite toe heave north of Ohinau Drive or movement at the andesite western gully base.

- **Andesite Failure**
  Deep failure within the andesite along shear zones or clay seams as identified in cores. This failure mechanism is thought to have been active in winter of 1996 and is consistent with toe heave.

Both failure surfaces are shown on Figure 4.

Recent Movement and Groundwater
The 1996 mass movement appears to have been strongly influenced by highly localised artesian groundwater pressures in combination with subdivision earthworks that removed approximately 5 m of earth from the toe area. Two boreholes drilled into the andesite in 1997 encountered artesian water pressures, although three others did not. It is inferred that the location of such water pressures is dependent upon defects within the andesite, some of which were evident in drill core recovered. Water levels within the surface debris were also found to be high; within 2.5 m of ground surface during winter months, and probably at or close to ground surface at the time of failure.

Drainage
A single horizontal bored drain was installed in mid 1997 with an outlet in the eastern incised gully. The 65-m-long bored drain targeted the zone of andesite artesian water pressure located centrally below the landslide mass. The effect of this drain was a substantial reduction in groundwater levels; approximately 1.7 m and 8.7 m within the surface debris and andesite respectively. However, the effect was localised to the central area and other areas showed no significant drops in water level that can be directly attributed to the bored drain.

Upon completion of the initial drainage relief bore, which comprises 75 mm steel casing, the driller’s bucket-measure estimate of flow was 27,000 litres/hour. Four months later a careful bucket-measure established a flow of 8,000 litres/hour, and three years later the flow had reduced to a steady 2,400 litres/hour and which rate continues. Unrelieved pressure from an artesian source of this magnitude in moderately steep topography is clearly a
Stabilising factor. However, since the installation of that first relief bore no discernible ground movement has occurred.

**Stability Analyses**
A series of computer assisted stability analyses were undertaken for both failure surfaces. Effective stress strength parameters were determined from back analysis of the existing slope movement and considered judgement of soil characteristics. Effective stress strength parameters assumed for the failure plane within the andesite were $c' = 2$ kPa and $\phi' = 15^\circ$.

The analyses indicate that groundwater pressures within the underlying andesite are critical to the slope stability.

It was determined that lowering sub-basal (andesite) groundwater levels to between 5 m and 7 m below ground surface will achieve a Factor of Safety (FOS) between 1.4 and 1.5. As often occurs with large scale failures, such as through the andesite, relatively large changes to groundwater levels are required to achieve significant changes in the Factor of Safety.

Within the shallow basaltic debris an assessed FOS exceeding 1.5 can be achieved by lowering groundwater levels below 4 m from ground surface.

**Future Development**
It is proposed to install counterfort (buttress) drains within the surface debris to minimise the risk of saturation and achieve a Factor of Safety in excess of 1.5.

Large diameter bored drains are the preferred drainage method for the andesite, drilled from the slope toe. Achieving a Factor of Safety in excess of 1.5 for the underlying andesite is considered impractical given the slope gradients and depth of drawdown required. A Factor of Safety of 1.4 is the recommended objective. With no dwellings within the main movement area, nor immediately adjacent, any settlement caused by drainage is considered unlikely to affect existing structures.

Alternative methods of stabilisation have been considered, such as toe buttressing. However, these are generally impractical and/or provide insignificant improvements in the Factor of Safety.

Successful performance of the drainage systems is essential in order to maintain groundwater at depressed levels consistent with acceptable factors of safety. The local authority is expected to be concerned with the manner in which the drainage is monitored long-term, but it may be reluctant to carry out such monitoring function itself. Methods of involving the property owners, perhaps by means of a special body corporate, are being considered whereby the groundwater levels (using the existing piezometers) are regularly checked and reported upon. Maintenance of the piezometers and bored drains could also be carried out by a body corporate. In return for the proposed extra drainage measures and an acceptable ongoing monitoring arrangement it is intended that the local authority lift the s36 Building Act title encumbrances.

Even though the artesian water source is, and presumably will be, ever-present within the affected land, it need not limit the usefulness of the building sites now that the local geology and the predictive geotechnical stability have been established. On this basis this particular slope failure, fortunately caught before extensive damage could occur, will have been successfully resolved.

**Conclusion**
A large, deep-seated failure within complex volcanic terrain has affected a recent subdivision in Ohinau Drive, Opito Bay. Investigation of the movement indicates possible multiple failure mechanisms following heavy rain and removal of toe weight.

Stability analyses indicate that satisfactory Factors of Safety can be achieved by subsurface groundwater drainage. It is proposed to install a combination of counterfort and bored drains.

**Acknowledgements**
I wish to acknowledge Cawdor Properties for allowing this paper to be prepared. I also wish to thank Ian Grierson of Harrison Grierson for his assistance during the investigation and assessment. Thank you to the staff of Riley Consultants who assisted with the study and preparation of this document.

**References**

Riley Consultants Ltd, 2001, Revised Geotechnical Investigation & Stability Assessment, Ohinau Drive, Opito Bay, Ref: 00122-E.


Ground improvement to reduce post-construction settlements by way of staged preloading was undertaken prior to construction of the 1.4 km long coal pad and reclaimer berm at the Kooragang Coal Terminal. Geotechnical instrumentation including settlement monitoring plates, vibrating wire piezometers, inclinometers, earth pressure cells and a deep extensometer were utilised to monitor preload performance which was critical to the staging of construction works. Geotechnical monitoring was successfully utilised in difficult ground conditions to minimise potential delays to construction.

Introduction

The Stage 3 Expansion of the Kooragang Coal Terminal was an A$330 million project which boosted the coal exporting capacity of the Port of Newcastle to 89 million tonnes per year.

The Kooragang Coal Terminal, operated by Port Waratah Coal Services Limited, is located at Kooragang Island in the port of Newcastle, some 160 km north of Sydney, NSW, Australia.

Kooragang Island was formed through the extensive reclamation of small islands, shallows and channels, and contains deep estuarine sediments which are susceptible to settlement via consolidation and creep.

The site contains the world's largest coal handling operation. Coal is transported primarily by rail from Hunter Valley mines, emptied in the receival (dump) station and conveyed to rail mounted stackers which place coal in designated stockpile areas. Coal is then reclaimed from the stockpiles by bucket wheel reclaimers and transported via conveyors to the shiploaders for export.

Port Waratah Coal Services engaged Bechtel Australia Pty Ltd to undertake engineering, procurement and construction management for the Stage 3 Expansion, which included a third stacking and receival conveying stream, rail receival (dump) station, stockpile pad and reclaimers, shiploading conveying stream and shiploader.

Due to the presence of deep, soft estuarine sediments, ground improvement by way of staged preloading was undertaken to reduce post-construction settlements prior to the construction of the coal pad and reclamer berm.

This paper discusses the successful implementation and results of geotechnical monitoring associated with preloading for the Stage 3 Expansion works.

Site Characterisation

An extensive geotechnical investigation programme was undertaken to assess site conditions (Douglas Partners 2000). The site was particularly suited to cone penetration testing (CPT), due to the presence of soft estuarine sediments. The investigation comprised a combination of CPT, piezocone tests, pore pressure dissipation tests, seismic CPT, conventional boreholes and jetted bores, shear vane tests and test pits.

A range of laboratory tests including oedometer, triaxial, permeability, Atterberg, sieve analysis and hydrometer tests were also undertaken to characterise soil conditions.

The interpreted geotechnical soil profile generally comprised the following:

- Unit 1 - filling, mainly sand (up to 5 m deep)
- Unit 2 - upper soft to firm clay layer (up to 4 m thick)
- Unit 3 - dense to very dense sand (23 m to 28 m thick)
- Unit 4 - lower stiff estuarine clay (7 m to 12 m thick)
- Unit 5 - siltstone/sandstone bedrock

An upper unconfined aquifer was present in Unit 1 fill materials (i.e. perched). A lower semi-confined aquifer was also present beneath the Unit 2 clays.

The main geotechnical issue at the site was the presence of the upper clay layer with respect to consolidation and creep. The presence of the dual aquifer system also had implications for deep excavation works.

Engineering parameters from CPT data were processed together with the results of laboratory testing to produce continuous profiles of parameters such as shear strength, overconsolidation ratio, coefficient of volume change, drained modulus and friction angle. The output can be tailored to present a wide range of interpreted parameters. Data in this format made analysis of settlement/consolidation quick and efficient.

Settlement analysis indicated that settlements of up to 700 mm were likely over a 17 year period without ground improvement for the proposed berm and coal stockpile area. Settlements of the existing berm and pad facilities have previously been experienced at the site. Considerable maintenance costs could be incurred where re-levelling of stacker and reclamer rails is required as a result of excessive settlements.

The objective of geotechnical design was to design a ground improvement system to limit post-construction settlements over a 17 year period to 200 mm beneath the reclamer berm, and 300 mm beneath the coal pad.
**Preload design**
A number of options for ground improvement were considered to limit post-construction settlements. Preloading was assessed to be the most feasible and economic option to meet the performance requirements. Settlement analysis indicated that a total preload height of about 9 m would be required to achieve the design objectives over a preload period of less than 12 months. The height of preload was also influenced by the availability of preload material, and the staging of construction works.

Stability analysis however indicated that slope failure would occur if the full height of preload was added in a single lift, due to the underlying Unit 2 clays. A two stage preload system was therefore designed to address both settlement and stability issues during construction.

The 4 m high Stage 1 preload was initially placed and monitored to achieve sufficient strength gain in the upper Unit 2 clays to allow the placement of Stage 2 (additional 5 m) without the risk of slope failure.

The requirements for a two stage preload increased the need for accurate and timely monitoring results, in order to ensure that the construction programme remained on schedule.

**Preload Monitoring**
Preload monitoring during construction was required to:
• assess progress of preload settlement
• determine when Stage 1 preload had achieved appropriate strength gain (confirmed by further CPT testing)
• confirm stability of clay layer during preloading
• determine when Stage 2 preload could be removed and allow berm construction to continue.

Monitoring of settlement plates, piezometers, inclinometers, extensometer and earth pressure cells were undertaken as discussed below.

**Settlement Monitoring Plates**
A total of 45 settlement plates were installed prior to the placement of preload materials. The plates comprised a 500 mm square steel base plate with 32 mm diameter steel risers, added in sections as the height of preload was increased. The risers were encased within a protective PVC pipe.

Survey levels were measured on the base plates, and at regular intervals on the steel risers, by the project surveyors.

A typical settlement monitoring plot is presented in Figure 1.

The plot shows the recorded settlement compared to the predicted time-settlement behaviour. The lower plot shows the estimated fill load for each stage of loading, based on the recorded fill height and fill unit weight.

**Vibrating Wire Piezometers**
A total of 44 vibrating wire piezometers were installed beneath the preload within the upper Unit 2 clay layer to monitor pore water pressures. One piezometer was also installed within the lower Unit 4 clay layer.

*Figure 1 Typical Settlement Plot*
Figure 2 Typical Piezometer Plot

Figure 3 Typical Inclinometer Plot
Regular piezometer readings were measured, together with settlement plate monitoring, to monitor the dissipation of excess pore pressure and assist in determining when preload could be removed.

A typical piezometer plot is presented in Figure 2. The piezometer plot shows the recorded pore pressure and estimated hydrostatic pressure, with the difference being the excess pore pressure. The estimated fill load and total overburden stress are also plotted.

Inclinometer
A total of 28 inclinometers were installed to a depth of at least 1 m below the upper Unit 2 clay strata, adjacent to the toe of the preload, to monitor lateral displacements during the placement of preload.

Regular readings were taken and processed as shown on the typical plot in Figure 3.

The inclinometer plot shows deflection recorded perpendicular to the slope. The ground level and location of the upper Unit 2 clay layer are also shown. The total lateral deformation and rate of deformation was used together with the settlement plate and piezometer monitoring results to confirm the stability of the preload embankment and determine when the Stage 2 preload could be placed.

Extensometer
An extensometer was installed to bedrock (50 m) at one location beneath the preload to measure the contribution of settlement in each layer to the total settlement, and in particular the deep Unit 4 clay layer.

Regular monitoring of the extensometer was undertaken. The settlement associated with each layer was measured. The results indicated that the majority of settlement occurred in the upper Unit 2 clay layer as expected.

Earth Pressure Cells
A total of four earth pressure cells were installed beneath the preload to confirm the magnitude of the load applied by the preload. Two types of fill material were used as preload: dredged sand (density \( \approx 18 \text{ kN/m}^3 \)), and crusher dust (density \( \approx 20 \text{ kN/m}^3 \)). The type of fill and the corresponding density were taken into consideration when determining the estimated fill load for settlement, piezometer and extensometer monitoring.

![Figure 4](image-url)
Cone Penetration Tests (CPTs)
A total of 90 CPTs were carried out during preload monitoring to:

- confirm that appropriate strength gain had been achieved in the upper Unit 2 clays after Stage 1 preload (i.e. ensure that the addition of Stage 2 preload would not induce instability in the preload batter slopes).
- confirm final strength gain in the upper Unit 2 layer after Stage 2 preload, which was required to provide long term stability of the completed coal stockyard.

The results of the CPTs compared favourably with the predicted shear strength gains (i.e. approximately 30 kPa after Stage 1 preload, and approximately 50 kPa after Stage 2).

An example of strength gain measured in the upper Unit 2 clay as a result of preloading is presented in Figure 4.

Conclusions
The monitoring results indicated that the preload generally performed as predicted, with target settlement and strength criteria being met. The success of the project has been attributed to the accurate and timely supply and analysis of monitoring results, together with regular communication and liaison with the project managers.

Acknowledgements
The author acknowledges the assistance and cooperation from Port Waratah Coal Services Limited (PWCS), Bechtel Australia Pty Ltd, and Stephen Jones (Douglas Partners Pty Ltd Project Manager).

References
Manufacturers of slotted PVC & HDPE pipe

AVAILABLE WITH THREADED OR SOLVENT JOINTS FOR:
- Environmental monitoring
- Piezometers
- Well screens
- Horizontal drains
- Subsoil (TNZ-F.2)
- Landfill leachate drains

ON SITE PLASTIC PIPELINE WELDING
- Extrusion Welding Systems
- Butt Fusion Welding Systems

ALSO AVAILABLE
- Electrofusion welding
- PVC & PE fabrication

FOR FURTHER INFORMATION CONTACT:
Bruce 0274 975 873 or Paul 0274 746 791
Pavements constructed on volcanic soils behave differently to non-volcanic soils and have higher deflections when dynamically loaded. Past relationships between CBR and modulus therefore may not be applicable. Fast non-destructive methods currently being used rely on these relationships and therefore relationship(s) between the CBR and modulus need to be obtained for volcanic soils. The project investigated the various volcanic soils within the North Island and their behaviour. In situ testing of the volcanic subgrades was conducted using the Falling Weight Deflectometer (FWD) test and in situ CBR together with other standard tests. From the in situ testing three correlations were identified for volcanic soils, however the third was not well defined. The volcanic soil types represented by the correlations were, clayey, pumiceous and silty/brown ash. To enable the Structural Number of pavements on volcanic soils to be determined, factors were presented based on the identified relationships, and the procedure for determining the Structural Number of a pavement from the FWD modulus suggested. Also included were factors that allowed the volcanic relationships to be used in the AUSTROADS Pavement Design Guide.

1 Introduction

Volcanic ash showers have coated large areas of the North Island of New Zealand during the last 100,000 years (Gibbs 1968). Past experience has shown that pavements constructed on volcanic subgrades typically have higher deflections under load than similarly performing pavements constructed on other subgrades. This has created difficulty in determining the structural strength of these types of pavements based on their deflection response when determined by instruments such as the Falling Weight Deflectometer (FWD). The FWD is being used to obtain a measure of the pavement strength for use in pavement deterioration modelling, as this is a rapid, relatively inexpensive method suitable for network surveys. The anomalous behaviour of volcanic subgrades means the standard methods used for assigning strength based on FWD readings cannot be used.

The concept of describing the strength of a pavement in terms of one number, called the Structural Number (SN), was developed from the AASHO road test published in 1962. The Modified Structural Number (SNc) was developed to incorporate the contribution to the strength from the subgrade. The original definition of the SNc was based on the determination of the California Bearing Ratio (CBR) of the various pavement layers including the subgrade. The CBR test gives an indication of the shear strength of a material. Each pavement layer is then given a weighting based on its strength and thickness. The summation of these factors for each layer and the subgrade gives the SNc. The method for calculating the input into the Modified Structural Number equation can be carried out either directly (CBR) or non-directly, using non-destructive methods.

The in situ determination of the CBR requires the digging of test pits and thus it is an expensive exercise. In order to obtain a faster and cheaper method the FWD has been used to calculate the modulus of the pavement layers and subgrade, and then uses a standard relationship between modulus and CBR to assign a strength factor to each layer. The FWD technique therefore relies on the relationship between CBR and modulus.

Research carried out at Central Laboratories, backed by Transfund (Bailey & Patrick 2001), developed correlations between the modulus and the shear strength of volcanic soils, and how the relationships should be applied to enable the structural number to be obtained for deterioration modelling and how these findings can be used in pavement design.

2 Some Background on the Behaviour of Volcanic Soils

2.1 Principal Soil-Forming Centres

The principal centres of soil-forming ash showers in New Zealand are Mt Egmont, Mt Ruapehu, Mt Ngauruhoe, Mt Tongariro, Mt Tarawera, and craters near Taupo, Rotorua, and the Bay of Plenty.

Two broad types of ash beds were recognised by Taylor (1933):

1) Intermittent type, in which materials are ejected on numerous occasions over a period of years. The materials are generally andesitic or basaltic and gradually build up a cone around the vent.

2) Paroxysmal type, in which materials are belched out in sudden explosions at long intervals. These materials are generally rhyolitic and are ejected from craters or rifts.

Most volcanic soils are best described as silty clays or clayey silts, despite the fact that they plot well below the Casagrande A-line on the plasticity chart (i.e. are clearly in the silt zone).

Some volcanic ashes have been named to collectively...
describe the andesitic tephras which were deposited during the Taranaki eruptions, e.g. Taranaki Brown Ash. Taranaki Brown Ash frequently occurs in thick, weak to strongly weathered beds. This weathering process produces a succession of ‘clay minerals’ with initially allophane being formed, which in time (say 20,000 years) changes to halloysites and finally kaolin. Other groupings include the Hamilton ashes and the Taupo Pumice. The ashes are given their name from either the locality of the vent or the locality of where the ash is extensively exposed at the surface.

2.2 Engineering Properties of Some of the Volcanic Soils

It has been found from previous studies (Fullarton 1978; Jacquet 1987 & 1988; Miller (undated); Parton & Olsen 1980; Sutherland, Dongaol & Patrick 1997; White 1982; Pender 1996; Wesley & Chen 1991; Wesley 1999) that volcanic soils’ engineering properties are wide ranging. Some volcanic soils, for example the brown ashes, tend to be sensitive to remoulding, and some volcanic soils can also physically change their properties when remoulded and/or dried; the soil alters its particle size distribution (PSD).

It has been reported that the clay mineral allophane and to a lesser extent halloysite are in a large part responsible for the unusual PSD changing properties. Allophane contributes to the greasy feel of the soil. The change in the PSD occurs due to the effect of the water being expelled from the molecular particle structure by bond-breakdown analogous to the way ice becomes water. The gel-like structure of the clay minerals collapse and form aggregations, which in turn makes the soils ‘gritty’. These new aggregates are reported to have a relatively high stability.

The sensitivity to remoulding of these soils may be due to the oxidation of iron in the ash. Iron oxide has been reported to form a ‘structure’ between the particles, which is broken down when remoulded.

Most undisturbed volcanic soils tend to have high bearing capacities; high pre-consolidation pressures have been measured on undisturbed volcanic ash, accompanied by irregular variations with depth. As there is no geological evidence as to why these soils have a high over-consolidation ratio, it is thought that it is due to the oxidation of iron, as suggested above, forming a type of structure within the soil matrix. Once this pre-consolidation pressure is reached the soils rapidly lose strength and are highly compressible i.e. the structure has broken down.

The apparent pre-consolidation pressures can vary markedly. Miller (undated) reported that a brown ash material (grey in its reduced state) effectively exhibited no pre-consolidation and was recovered from within a local poorly drained area. Jacquet (1987) reported that the pre-consolidation pressure was directly proportional to the shear strength. The strength characteristics of the ash were also reported to vary depending on the degree of weathering, oxidation (drainage conditions) and saturation (Miller (undated)). Therefore an oxidised ash, with the iron structural matrix developed, may be stronger up to the pre-consolidation pressure than an ash in its reduced state.

Pumice soils however tend to be less sensitive to remoulding. This may be due to the kind of deposition and then cementation of the pumice grains. Some pumice soils still exhibit fairly high sensitivities, possibly due to larger amounts of iron present in some areas. Pumice soils also tend to be sandy and contain smaller amounts of the allophane clay mineral. Therefore, after remoulding any change within the allophane will be small and less likely to change the overall particle size distribution, compared to such soils as clays.

In general, volcanic materials have been reported as having a high particle density, due to the presence of heavy minerals; a low dry density due to the high volume of voids. These voids have been found to be discrete internal voids within the particles themselves; the particle shapes are angular, together with the rough microtexture which causes a high shear strength/friction angle; the soils also usually exhibit some cohesion. The natural moisture content of many of the soils, especially the brown ash soils, tends to be high, and in some cases has been higher than the liquid limit. The liquid limit of the soils is also high compared to non-volcanic soils.

Care has to be taken in the interpretation of laboratory test results as it has been found in NZ ashes that four different moisture/density relationships can be obtained, depending on whether a wet or dry soil is the starting point, and whether the same soil is used throughout or a fresh sample is taken for each compaction (Jacquet 1987). The clay mineral allophane may be responsible for these relationships. However, for pumice soils their particle size distribution may change under compaction activities due to their soft (low crushing resistance) character and the presence of allophane.

3 Site Selection and Testing

During the review of the existing data, it was found that the test results were wide ranging with overlapping of results making the defining of the volcanic soils difficult. In general a broad grouping of the pumice soils/sands and the brown ashes could be considered reasonable, due to their difference in behaviour and engineering properties as discussed above.

Broad groupings of the volcanic soils were chosen so that the selection of the test sites could be simplified. The groupings were; Hamilton ashes, Taranaki ash and Taupo Pumice.

Sites were selected so that together with the existing data the results would enable a wide spread of sites within the
3.1 Testing
Testing at each site involved; logging two test pits, in situ CBR tests, dynamic cone penetrometer testing and classification tests. The allophane content was also determined. FWD testing at the sites was conducted to make an approximation of the pavement layers’ moduli.

4 Results and Analysis
The results from the on-site and laboratory testing and the FWD were analysed, and relationships between the CBR and the FWD derived modulus determined for volcanic soils.

4.1 Results
4.1.1 Laboratory Testing
Results from the allophane testing showed variable amounts of the clay mineral allophane throughout the sites tested. In general the highest concentrations were found to be located south of Hamilton. Medium levels were obtained near Tauranga and Taranaki. The soils near Taranaki and Tauranga have been historically sensitive to remoulding (Jacquet 1987) and able to change their PSD. This was initially reported to be due to the allophane mineral present, however from the testing this may not be the only reason. As suggested in section 2, the sensitivity may also be due to the break down of the soil matrix due to the oxidation of iron within the soil. The detailed test results are included in the Transfund Report (Bailey & Patrick 2001).

4.1.2 FWD Testing
Transfund Report No. 117 (Tonkin & Taylor 1998) describes the procedure for the detailed structural analysis of the pavement from the shape of the deflection bowl. Basically, the outer deflections define the stiffness of the subgrade while the bowl shape close to the loading plate allows analysis of the stiffness of the near-surface layers. A broad bowl with little curvature indicates that the upper layers of the pavement are stiff in relation to the subgrade. A bowl with the same maximum deflection but high curvature around the loading plate indicates that the upper layers are weak in relation to the subgrade.

A back-analysis procedure is generally adopted to find moduli from an observed deflection bowl. Once the pavement profile model is established, a forward-analysis can be carried out to determine the strains for say a modelled rehabilitation treatment such as overlay. The back-analysis was conducted using ELMOND (Evaluation of Layer Moduli and Overlay Design) (DYNA TEST 1989).

4.2 FWD Modulus and CBR Relationship
It has been documented by Tonkin & Taylor (1998) that the modulus-CBR relationship may vary by a factor of

<table>
<thead>
<tr>
<th>Site</th>
<th>Location</th>
<th>Soil Grouping</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site 1</td>
<td>Okere Falls, Okere Road</td>
<td>Pumiceous</td>
</tr>
<tr>
<td>Site 2</td>
<td>Lichfield, Vospers Road</td>
<td>Pumiceous</td>
</tr>
<tr>
<td>Site 3</td>
<td>Murupara, Golf Road</td>
<td>Pumiceous</td>
</tr>
<tr>
<td>Site 4</td>
<td>Waiotapu, Joy Road</td>
<td>Pumiceous</td>
</tr>
<tr>
<td>Site 5</td>
<td>Bennydale, SH30</td>
<td>Pumiceous</td>
</tr>
<tr>
<td>Site 6</td>
<td>Stratford-inglewood, Tariki Road</td>
<td>Brown ash</td>
</tr>
<tr>
<td>Site 7</td>
<td>Waitara-Urenui, Upper Epiha Road</td>
<td>Brown ash</td>
</tr>
<tr>
<td>Site 8</td>
<td>Whangamomona, SH43</td>
<td>Brown soil</td>
</tr>
<tr>
<td>Site 9</td>
<td>Pio Pio, Mairoa Road</td>
<td>Hamilton ash (south)</td>
</tr>
<tr>
<td>Site 10</td>
<td>Hamilton-Cambridge, Day Road</td>
<td>Hamilton ash</td>
</tr>
<tr>
<td>Site 11</td>
<td>Te Poi, Stopfords Road</td>
<td>Hamilton ash (east)/pumice</td>
</tr>
<tr>
<td>Site 12</td>
<td>Puhekohe, Coles Road</td>
<td>granular material</td>
</tr>
<tr>
<td>Site 13</td>
<td>Taupiri, Jew Road</td>
<td>Hamilton ash (north, granular)</td>
</tr>
<tr>
<td>Site 14</td>
<td>Te Awamutu, Bowman Road</td>
<td>Non-volcanic</td>
</tr>
<tr>
<td>Site 15</td>
<td>Cambridge, Peake Road</td>
<td>Non-volcanic</td>
</tr>
</tbody>
</table>

Table 1 Sites Selected for Testing
two. AUSTROADS pavement design for granular roads have suggested using 10 times the CBR for all soils to obtain the vertical anisotropic modulus, and 6.7 times the CBR value for isotropic modulus. The Modified Structural Number is also calculated from the CBR value.

The relationship of 10 times the CBR, and thus the pavement design, may be correct for tested CBR values. However, the determination of the CBR from the measured in situ modulus may not be correct, especially for volcanic soils due to their different behaviour as discussed previously.

The results from the in situ testing and the data from previous studies are shown in Figure 2.

Figure 2 Graph of Isotropic Modulus Versus CBR for Current Data and Previous Data

Both the FWD subgrade modulus and in situ CBR of the subgrade have been plotted. It is apparent that two relationships can be seen, with a possible third in between.

It was found that, in general, the sandy pumiceous soils lay near the bottom of the graph, with the clayey volcanic materials forming the steeper relationship. A linear regression was carried out and the following correlations found: CBR\texttimes10, R^2=0.8307; CBR\texttimes1, R^2=0.7024. The R^2 for the correlations are reasonable for a network study.

From the graph, the following relationships were drawn:

A) Typically pumice/sandy soils
Isotropic Modulus = 1\texttimes CBR

B) Mixture of silty soils and Taranaki Brown Ash
Isotropic Modulus = 3\texttimes CBR
(however relationship not well defined)

C) Typically includes clayey ash soils
Isotropic Modulus = 10\texttimes CBR

These relationships are very different from each other and may account for the reason for the many discrepancies that occur in the testing of volcanic soils.

The Transfund Report No. 128 (Sutherland, Dongao & Patrick 1997) also recommended a relationship of 3\texttimes CBR for brown ashes. A possible reason why this relationship may not be well defined is that the clayey materials, containing the allophane mineral, when remoulded tend to alter their particle size distribution into more like a silt. They therefore may lie below the 10\texttimes CBR, and this obscures the third relationship.

The possible explanation for the relationships in Figure 2 may lie in the structure and grain shape of the volcanic soils. Pumice soils tend to be sandy and have sharp angular grains which would indicate a high CBR value. However due to their structure, under the FWD loading, the pumice soils exhibit a high elastic deformation and therefore low modulus when back-analysis of the FWD results are carried out. This type of behaviour can be seen in Figure 2. Consequently silty soils tend to lie in between the two relationships due to them being neither clays or sandy, as would be expected. The brown ash is slightly different in that it shows a low modulus and a low CBR value. This may be due to the allophane content as discussed above, and the in situ structure of the volcanic soils, but also the fine grained nature of the soil producing a low CBR. The degree of saturation, an unknown variable in the in situ CBR test, may affect the results slightly.

The above correlations are not dissimilar to the groups used to select the various site locations for testing, as discussed in section 3. The pumiceous soils are mainly sandy soils, and the Hamilton ashes mostly clayey soils. The Taranaki soils however, which were also classified in the test pits as clayey, lay closer to the middle correlation together with the silty soils. A gain, this may be due to the behaviour of allophane, as suggested above.

The relationships were then used to determine factors for the determination of the CBR from the FWD modulus. This enabled the structural number for pavements to be obtained and also the determination of the vertical modulus from the CBR for use in Austrroads pavement design.

5 Using the Results in Prediction Modelling and Pavement Design

5.1 Pavement Prediction Modelling Using the SNC

The modified structural number is an indication of the pavement strength and has been adopted in a number of empirically based pavement design and deterioration models of organisations such as AASHTO (1986), the Transport and Road Research Laboratory (Road Note 31, 1977) and the World Bank (Paterson 1987).

Structural Numbers can be determined using direct or indirect methods. To determine the SNC using non-destructive methods an indication of the relationship between CBR and modulus needs to be known.

The FWD calculated modulus of the basecourse and subsequent layers can be input directly into the structural
number equation (Eqn 4), but the subgrade contribution to the strength of the pavement cannot, as it is based on CBR. Therefore the calculated modulus requires conversion into a CBR value. As discussed above the relationship for volcanic soils is indicated to be different to the standard Modulus=10×CBR usually used for soils. Factors presented below allow the structural number to be determined for volcanic subgrades, based on the relationships indicated in section 4.

The Structural Number, including the additional variable for subgrade strength, is defined as follows:

\[
SNC = \frac{1}{25.4} \sum a_i h_i + SN_{sg} \tag{Eqn 4}
\]

where 
\( a_i = \) layer coefficient 
\( h_i = \) thickness of layer, mm 
\( SN_{sg} = \) structural number contribution from the subgrade

\[
a_i = a_{g}(E_i/E_g)^{1/3} \tag{Eqn 5}
\]

where 
\( a_{g} = \) layer coefficient of standard materials (AASHTO) 
\( E_i = \) layer modulus (in this case, from FWD) 
\( E_g = \) modulus of standard materials (AASHTO)

\[
SN_{sg} = -0.85 (\log CBR)^2 + 3.51 (\log CBR) - 1.43 \tag{Eqn 6}
\]

Patrick and Dongal (2001) state that care must also be taken when determining the Structural Number on volcanic subgrades, in that due to the 'bouncy' nature of the soils, cracking may be an issue. Patrick and Dongal suggest that a second SNP (SNC for thin pavements) should be derived which may give a better prediction of cracking.

**5.1.1 Factors Which the Isotropic Modulus Should be Divided by to Obtain CBR for Determination of SNC**

From the FWD in situ CBR relationship, summarised in section 3, the following table gives a factor which the FWD isotropic modulus should be divided by to obtain a value for CBR which can then be input into Eqn 6.

<table>
<thead>
<tr>
<th>Volcanic Soil Type</th>
<th>A (pumice)</th>
<th>B (mixture silty)</th>
<th>C (clayey)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factor</td>
<td>1</td>
<td>( \equiv 3 )</td>
<td>10</td>
</tr>
</tbody>
</table>

Table 2 Factor for the Determination of CBR for SNC Calculations

These factors should only be used for the determination of the SNC, as the isotropic modulus is used.

**5.2 Determination of the Anisotropic and Isotropic Modulus for Input into AUSTROADS Pavement Design**

The AUSTROADS approach to the design of roads uses modulus as the input for calculating the strain in the different pavement layers and assumes a fatigue relationship between subgrade strain and failure.

For the AUSTROADS pavement design on granular pavements, traditionally the anisotropic vertical modulus is calculated from 10×CBR. The isotropic modulus is calculated from 6.7×CBR. If these relationships were used for pavement design on volcanic subgrades, due to the 'bouncy' nature of volcanic soils a low FWD modulus would result in a low CBR that could lead to the road being over designed. For volcanic soils it is recommended that the factor in Table 2 be multiplied by the CBR to obtain the vertical modulus and isotropic modulus respectively, which can then be input into AUSTROADS pavement design.

<table>
<thead>
<tr>
<th>Volcanic Soil Type</th>
<th>A (pumice)</th>
<th>B (mixture silty)</th>
<th>C (clayey)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factor Vertical Modulus</td>
<td>1</td>
<td>( \equiv 4.5 )</td>
<td>15</td>
</tr>
<tr>
<td>Factor for Isotopic Modulus</td>
<td>1</td>
<td>( \equiv 3 )</td>
<td>10</td>
</tr>
</tbody>
</table>

Table 3 Factor to be Applied to the CBR to Determine the Vertical/Isotopic Modulus for Input into AUSTROADS Pavement Design

Examples using the above factors to calculate the Structural Number of pavements and in pavement design, are presented in the Transfund Report (Bailey & Patrick 2001). Another approach would be to develop specific strain fatigue relationships for the different soil types.

**6 Conclusions**

The different behaviour of volcanic soils to non-volcanic soils depends largely on the minerals present within the soil. The engineering properties of volcanic soils are variable due to their amount of weathering, minerals and remoulding. A classification of volcanic soils based on their geological description was presented and 15 sites were selected based on the location of past work. Testing at each site included two test pits with measurements of scala and in situ CBR. FWD testing was also carried out.

The subgrade modulus, obtained from the FWD tests (via ELMOD), was plotted on a graph with in situ CBR. The following relationships were obtained for volcanic soils:

A) Typically pumice/sandy soils
   Isotropic Modulus = 1×CBR
B) Mixture of silty soils and Taranaki Brown Ash,
   Isotropic Modulus = 3×CBR (however relationship not well defined)
C) Typically includes clayey ash soils
   Isotropic Modulus = 10×CBR

Factors were presented which related the modulus to CBR for structural number modelling, and CBR to vertical modulus for pavement design, Table 2 and 3 respectively.
These allow the above relationships to be used in pavement deterioration modelling and pavement design.

References


TRL Road Note 31, 1977, Guide to the Structural Design of Bitumen-Surfaced Road in Tropical and Sub-tropical Countries, Transport and Road Research Laboratory.


White, T. 1982, Laboratory Testing of Core Samples from Omata Tank Farm Site, North-West Taranaki, Central Laboratories Report No. 2-82/12.

---

Specialists in Geotechnical and environmental investigations. A combined truck mounted CPT and Auger drilling equipment all in one, for fast, reliable results.

Gary Barnett  Mobile 025 949 726
1134 Polhipi Rd  Mark Barnett 027 290 6392
RD1  Fax 07 378 2147
Taupo  A/H 07 378 2016
Paleo-Earthquakes and Hazard of the Strike-Slip Porters Pass Fault, Canterbury, New Zealand
Matt Howard, URS New Zealand, Christchurch

From the 5th ANZ Young Geotechnical Professionals Conference

Summary
The amount of slip/event and the timing of paleo-earthquakes are crucial components needed to estimate the earthquake potential of active faults. The measurement of displaced geomorphic markers enabled identification of four to six earthquakes on the Porters Pass Fault (PPF), with strike-slip/event ranging between ca 5-7 m. The timing of these earthquakes was constrained by excavating trenches across the fault and radiocarbon dating preserved organic material. These data suggest that at least four and as many as six earthquakes have occurred in the last 10,000 years. The identification of only one Holocene PPF rupture in the west of the field area indicates the presence of a segment boundary that prevents the propagation of rupture along the full length of the fault during individual earthquakes. This is consistent with displacement data and results in slip rates of 0.3-0.9 mm/yr and 2.9-3.7 mm/yr to the west and east respectively. The combination of geometric, slip rate and timing data, has enabled the magnitude of prehistoric earthquakes on the PPF to be estimated at between M 6.9 for a 32 km fault rupture from Waimakariri River to Red Lakes, to a maximum of M 7.7 that ruptures the entire 100 km length of the Porters Pass-Amberley Fault Zone (PPAFZ).

Introduction
Earthquakes occur mainly in the earth’s crust and result in fault slip which may take place repeatedly over millions of years. Most earthquakes occur at or near plate boundaries where they reflect strains induced by jostling of tectonic plates. New Zealand straddles a plate boundary (Figure 1) and has experienced many earthquakes since the arrival of Maori people over 800 years ago. Prior to European settlement over 150 years ago, we rely on studies of active-fault traces to provide information on large paleo-earthquakes which ruptured the ground surface.

The Porters Pass Fault (PPF) is mainly strike-slip and forms an active trace in the eastern foothills of the Southern Alps. The fault is a prominent element of the Porters Pass-Amberley Fault Zone (PPAFZ) which forms a broad zone of active earth deformation ca 100 km long, 60-90 km west and north of Christchurch (Cowan 1992). Historically no fault within the PPAFZ is known to have ruptured the ground surface during an earthquake and no large magnitude events have been recorded in this zone. The PPF is clearly defined by a series of discontinuous Holocene active traces from Lake Coleridge north to the Waimakariri River, a distance of ca 40 km. The fault strikes approximately parallel to the plate motion vector resulting in a predominantly right-lateral strike-slip sense of motion.

Scarp height ranges up to 5 m, with both reverse and normal dip slip. The prominence of the fault at the ground surface suggests that it has produced repeated large magnitude earthquakes over the last 10,000 years (i.e. the Holocene) and could contribute significantly to seismic hazard in the Canterbury region. Knowledge of earthquake magnitudes and frequencies on this active fault is therefore important in order to assess its likely impact on buildings and engineering structures. To better understand paleoseismicity of the PPF a study as part of an M Sc thesis was undertaken (Howard 2001; Nicol et al. 2001).

Prior to this research, three surface-rupturing earthquakes were proposed for the PPF at 500-700 years B.P., 2000-2500 years B.P. and 7500-10,000 years B.P. (Burrows 1975; Coyle 1988; Cowan 1992; Cowan, Nicol & Tonkin 1996), suggesting recurrence intervals of 1300-5000 years. The timing of these events, however, was based on few data and the timing of paleo-earthquakes was poorly constrained by comparison with palaeoseismically important faults elsewhere in New Zealand (e.g. Alpine Fault, Hope Fault and Wellington Fault).
Regional Tectonic Setting

Earthquakes and active faulting reflect New Zealand’s location on the boundary between the Australian and Pacific plates (Figure 1). This plate boundary may have initiated over 40 Ma ago and for the last 10-20 Ma has comprised opposed-dipping subduction systems linked by the Alpine Fault (Norris, Koons & Cooper 1990). The Alpine Fault forms a strong lineament on satellite images and extends for approximately 400 km northwards from Milford Sound along the western side of the Southern Alps.

To the north along the Hikurangi margin oceanic crust of the Pacific Plate has been subducted beneath more buoyant Australian continental crust. Convergence increases in obliquity southwards, while the magnitude of the relative plate motion vector decreases southwards from about 53 mm/yr to 37 mm/yr (DeMets et al. 1994). Oblique convergence results in both right-lateral and reverse fault displacements in the upper plate.

The Puysegur Trench lies southwest of Fiordland where subduction is in the opposite direction to the Hikurangi margin, with oceanic crust from the Australian Plate passing beneath the oceanic crust of the Pacific Plate. The Marlborough Fault System (MFS) accommodates the change from east dipping Alpine Fault to west dipping subduction zone of the Hikurangi margin. The MFS covers parts of Marlborough, north Canterbury and offshore northeastern South Island (Pettinga & Wise 1994) and is composed of a number of right-lateral faults that generally strike parallel to the plate vector and splay from the Alpine Fault. The PPF is the southward continuation of the MFS.

Displacement Determination

The PPF offsets numerous geomorphic landforms, with predominantly right-lateral strike-slip displacements (Figure 2).

These displacements are inferred to have accrued principally during repeated moderate to large magnitude earthquakes, with larger magnitude events generally producing greater fault slip and rupture length (e.g. Wells & Coppersmith 1994). Therefore, by comparison with empirical data from historical earthquakes in Wells and Coppersmith (1994), for example, with data from prehistoric earthquakes, the magnitudes of the latter can be estimated. Documentation and analysis of displacements along the PPF therefore provides a means of estimating the magnitude of prehistoric earthquakes. Displacement data can be combined with timing data to determine recurrence interval/frequency of seismic events.

Displacement of geomorphic features such as abandoned stream channels, ridge crests and flanks, and channel walls were measured at 25 sites along the fault. Vertical and horizontal separations of these offset features were obtained using tape measure and electronic distance meter (EDM) topographical surveying equipment. Displacement data were collected using tape measure where linear landscape elements are offset by, and could be correlated across, the fault.

Using an EDM and theodolite, the positions of offset streams and channels were located relative to the steeply dipping fault. In one example near Porters Pass, offset channel axes above and below the fault scarp were projected graphically onto the fault plane which enabled 'piercing points' to be established. It is assumed that prior to faulting these channels coursed approximately parallel...
to the hill slope, which is normal to fault strike. On another site, west of Porters Pass, topographical surveying provided a basis for matching topography across the fault and estimating the amount of fault displacement by restoring topography to a pre-faulting configuration.

The process of inferring displacement/event from offset markers is based on the premise that all of the offset is coseismic, i.e. that there is no aseismic creep. This assumption is supported for the PPF by the lack of deformation of four penstocks built across the fault almost 90 years ago.

Clustering of horizontal displacements show the cumulative displacements for an estimated one to five earthquakes (Figure 3). These preferred estimates were derived by using the minimum number of slip events necessary to account for the observed strike-slip displacements. The preferred cumulative displacements for one to five earthquakes are not unique as the overlap in errors on displacement measurements mean that it is sometimes difficult to unequivocally determine the size of individual earthquakes based on this information alone. It would be possible, for example, to introduce an additional event within the accumulated displacement range of 17-28 m.

For the preferred earthquake-slip model five events account for the 33 m with slip/event ranging from 5-7 m. A larger event associated with earthquake three is supported by trench data, which indicates a relatively greater seismic event. On average the five earthquakes required to produce a horizontal separation of ~33 m would have been accommodated by ~6.5 m/rupture. Thus given the available data the PPF could be regarded as being broadly characteristic in displacement terms. However, the lack of deformation of glacial deposits south of Lake Coleridge suggests only one paleo-earthquake has ruptured the southwestern end of the fault at the surface since their deposition and so rupture length during individual earthquakes was not constant.

**Timing of Earthquakes**

A critical component of this project was to establish the timing and magnitude of past surface-rupturing earthquakes on the PPF. These estimates are important for defining the earthquake potential of the fault, which is essential for evaluating the associated seismic hazard. In addition to this, the earthquake history provides key kinematic information about how the fault has grown and how it may be interacting with adjacent structures.

Trenching across active faults to establish the timing of past earthquakes is an important part of paleo-earthquake studies. Radiocarbon dating (^{14}C) of preserved organic material beneath the surface is used to date stratigraphy, which in some cases may be deposited immediately after fault rupture, or may bracket the timing of formation of fault strands. This provides the most unequivocal means of constraining the timing of prehistoric earthquakes and allows the definition of a window within which the event must have taken place. If possible, trenches are located across sites of preferential accumulation of organic materials (e.g. swamps) whose development is influenced by fault activity. This provides the stratigraphic and timing data necessary to constrain the occurrence of rupture events, with a direct link between the fault and stratigraphy.

The timing of paleo-earthquakes on the PPF was constrained using data from five trenches (one of these trenches was part of a previous study) and one hand auger site, with additional information provided by landforms which pre- and post-date past earthquakes. Trench locations were identified in the field area where accumulation of organic material adjacent to the fault scarp was considered most likely to have occurred. Three of the four excavated trenches were located on the western part of the PPF near Lake Coleridge, with the remaining trench located at Porters Pass to the east. These data are augmented by information from a previous trench sited within 10 m of trench one near Lake Coleridge (Wood, New Zealand Geomechanics News June 2002, Issue 63).
Paterson & Howard 1989) and from a road batter at Porters Pass (Cowan, Nicol & Tonkin 1996).

The most unequivocal data was obtained in the trench at Porters Pass where interpretation of data indicates that at least four and as many as six earthquakes have occurred during the Holocene on the PPF. The timing data from the other localities shows that the proposed timing of these events is consistent between the different data sets (Figure 4).

Data for events one, two and four (event one being the most recent) are from more than one location and have enabled the timing of earthquakes to be constrained in the east of the field area near Porters Pass. From oldest to most recent, preferred ages for fault rupture in years B.P. are 8500 ± 200, 5300 ± 700, 2500 ± 200 and 1000 ± 100. Additional events may have occurred at 6200 ± 500 and 500 ± 100 years B.P. The ambiguous nature of trench four data regarding the most recent event horizon, together with the lack of evidence from other sites means that this cannot be confirmed. In the west of the field area, near Lake Coleridge, only the 2500 ± 200 event could be identified, supporting the notion that the PPF does not always rupture along its full length. As a result, displacement and timing data give slip rates in the west and east of 0.3-0.9 mm/yr and 2.9-3.7 mm/yr respectively.

Recurrence interval is derived by estimating the average time between earthquakes, and ranges from 1700-2500 years in the east and ≥7500 years in the west.

Hazard Posed by PPF
The hazard posed by the PPF cannot be determined by trenching as this does not provide sufficient data on the magnitude of paleo-earthquakes. Because the PPF has not experienced historic coseismic ruptures, the magnitude of past (prehistoric) earthquakes is most accurately determined by using field data for the single-event displacements and rupture lengths of these events. Estimates of earthquake magnitude rely on the ability to correlate fault geometry with earthquake magnitude and are useful for determining past magnitude of the PPF because they do not rely on historical seismicity.

The moment magnitude scale (Kanamori 1997; Hanks & Kanamori 1979) is useful for determining the energy release of an earthquake based upon paleoseismic data. This scale incorporates the seismic moment (M₀) which represents the energy released at the source. Wells and Coppersmith (1994) also correlate fault-rupture length and area with magnitude using empirical relationships from a worldwide historical earthquake database. To determine the magnitude of a PPF earthquake, both of these models were used with rupture scenarios that compared a rupture along only part of the PPF (32 km length) compared with a full rupture along this fault and the PPAFZ, a distance of 100 km. A range of average coseismic displacement of 4-8 m was used in order to account for the distribution of displacement data. An
average of rupture models with different rupture length scenarios suggests that magnitudes on the PPF would produce a M 6.9 to M 7.7 earthquake.

Conclusions

• The PPF is part of the PPAFZ and is an active fault within 60 km of the population centre of Christchurch.
• The horizontal offset of geomorphic features across the PPF were measured using a tape and survey equipment. Clustering of these data suggest that earthquakes on this fault produce 5-7 m of displacement.
• Timing of past earthquakes was determined primarily by radiocarbon dating organic horizons in four trenches excavated across the fault. At least four events in the last 10,000 years could be identified on the eastern part of the PPF giving an earthquake recurrence interval of 1700-2500 years.
• Slip rates on this part of the PPF are as much as 3.7 mm/yr, however the identification of only a single Holocene event on the west of the PPF gives a lower slip rate of 0.9 mm/yr. This suggests that the fault is segmented and does not rupture along its full length during an earthquake.
• The expected magnitude on the PPF has been derived using empirical models which rely on fault geometry and movement rates and is between M 6.9 and 7.7.
• The fault has the potential to cause significant damage and loss of life in the region.

References


Acknowledgments

The New Zealand Earthquake Commission (EQC), the Institute of Geological and Nuclear Sciences (GNS) and a University of Canterbury Masters Scholarship have funded research. Assistance to attend the Fifth Young Geotechnical Professionals Conference from EQC, NZ GNS and URS are also gratefully acknowledged.

Matt Howard
URS New Zealand Ltd
PO Box 4479
Christchurch
Email: matt_howard@urscorp.com
Fissuring in Auckland Residual Clays and the Capacity of Shallow Foundations – II

M. J. Pender, Department of Civil and Environmental Engineering, University of Auckland

Introduction

Excavations in Auckland clays reveal that the upper part of the soil profile, up to depths of a metre or so but usually less, is fissured. Photographs from a site on the North Shore, kindly supplied by Mr Bill Thompson, are shown in Figures 1 and 2. One possible explanation for the fissures is the cracking of the ground surface that occurs in the summer. This is reasonable for the vertical and near vertical fissures in the photographs but does not explain the presence of the low angle fissures also apparent. Swelling in wet periods following the cracking has been suggested as a possible explanation for these. After the cracks are formed, debris falls into the cracks, or rootlets intrude into them. In the wet season the clay absorbs water and swells, but the swelling is restrained in those cracks which now contain debris. This process can produce passive failure of the clay with consequent low-angle failure surfaces. This sequence will be repeated from year to year and over a period of time clay structures such as those shown in Figures 1 and 2 are produced. To my knowledge this mechanism was first proposed by Terzaghi (1929) for explaining large pressures against walls retaining clay, and is also offered by Tschebotarioff (1973). Pender (1996) presents data showing extension failure of Auckland clay on a low-angle failure surface induced during one-dimensional swelling in a laboratory Ko triaxial cell; a test intended to replicate the mechanism proposed by Terzaghi and Tschebotarioff for the formation of the low angle fissures such as those in Figures 1 and 2.

The purpose of this short paper is to consider the implications of fissures in the upper part of the Auckland clay profile for the design of shallow footings and pole wall foundations. Prior to shallow footing construction the fissured zone, or the most severely fissured upper part, is likely to be removed as part of the site preparation. Thus foundations for pole walls are the shallow foundation situation for which the presence of the fissures might be most significant.

Figure 1 Fissured clay at the site of a pole retaining wall in Takapuna. Height of excavated face about 1.2 m.

Figure 2 Close-up of the face shown in Figure 1. Note the low angle fissures and the rootlets on some of the exposed surfaces.
Effect of Specimen Size on the Shear Strength of Fissured Soil

How do we estimate the undrained shear strength of clay?

A common method is to use an in situ vane shear test. The common vane configuration is such that the vane height is substantially less than 100 mm and the diameter is about half this. From Figures 1 and 2 it is apparent that the vane size is small in relation to the spacing between the fissures. This raises the question of how representative is the vane shear strength of the actual shear strength of the soil mass. The shear strength along the fissures is expected to be less than that of the intact material between them. Thus the shear strength of a soil mass with fissures would be less than that of the material between the fissures and a certain minimum volume of soil would need to be tested before the actual field shear strength of the soil mass could be estimated. In other words, the shear strength of the soil mass is controlled more by the pattern of fissures than the strength of the intact material between, in just the same way as the strength of a jointed rock mass is likely to be controlled by the joint system. The question which then arises is what is the actual shear strength of the mass of fissured soil?

Although the strength of large volumes of soil is not often investigated, examples are given by Bishop and Little (1967) and Marsland and Butler (1967). Each of these papers presents some data on the shear strength of fissured London clay and each concludes that larger specimens indicate a smaller shear strength, although the presence of a fissure at an unfavourable orientation in a small specimen can give a small shear strength value. Data from the Bishop and Little paper are reproduced in Figure 3 and data from Marsland and Butler in Figure 4.

Figure 3 Comparison between values of undrained shear strength determined by field testing on ‘large’ volumes of soil and those determined from 76 mm tall by 38 mm diameter laboratory specimens (reproduced from Bishop and Little). Normal stresses for the direct shear tests 45 to 115 kPa. (1 ft ≈ 305 mm, 1 inch ≈ 25.4 mm, 1 lb/in² ≈ 6.9 kPa, 1 lb/ft² = 0.05 kPa)

Figure 4 Comparison of the compressive strength values (≈ 2σ) determined from unconsolidated undrained (UU) triaxial testing on 38 mm and 100 mm diameter specimens of fissured and unfissured clays (after Marsland and Butler 1967). Cell pressures for the UU triaxial tests were approximately equal to the overburden pressure at the depths from which the samples were taken.
Additional information is available on the strength of coal which has closely spaced fissures (known as cleat in the mining industry). In Figure 5, data from compressive strength tests on cuboidal blocks of coal in an underground mine in South Africa are reproduced from Bieniawski (1968).

These test results lead to similar conclusions to those derived from the tests in clay, namely that joints and fissures make an important contribution to the strength of the material and some minimum specimen size is required before the shear strength available in the field can be determined. In the case of the Auckland clays, this data leads to the conclusion that, in the fissured zone, the actual shear strength of the soil mass is expected to be less, perhaps considerably less, than that determined by the vane (or a cone penetration test or by taking samples and trimming laboratory test specimens of usual sizes).

London clay, a heavily overconsolidated soil, is very different from Auckland residual clay, formed in situ by weathering from the Waitemata group sandstones and siltstones. Both of these are very different from coal in an underground mine. Nevertheless the conclusion from Figures 3 to 5 is that the presence of joints and fissures means there is a size effect on the shear strength of the material regardless of the mode of formation. Similarly the strength of fissured Auckland residual clay will be size dependent and so the vane shear strength is more likely to be an upper limit to the shear strength rather than a value relevant to foundation design. Additional information about size effects on the strength properties of soil is given by Bonala and Reddi (1999).

**Shallow Foundations in Fissured Clays**

The undersides of shallow foundations are typically established 300 to 600 mm beneath the surface of the cleared ground surface at the building site. This means that part of the fissured soil is removed in the general site preparation process and possibly more when the individual footing positions are prepared. Nevertheless there may still be some fissured soil beneath the foundation. In this section we consider briefly the significance of this.

Figure 6a has the standard mechanism associated with bearing capacity failure, the geometry of which depends on the angle of shearing resistance of the soil. For short term bearing failure the undrained shear strength, $s_u$, controls the maximum possible pressure and the linear segments of the mechanism all meet the horizontal surface at 45 degrees. For drained behaviour the angle of shearing resistance, $\phi$, controls the bearing capacity and the outer triangular regions of the mechanism meet the horizontal surface at an angle of $45 - \phi/2$ degrees. A possible consequence of the fissures is a drained failure mechanism defined by fissures that happen to be oriented along the parts of the failure mechanism in Figure 6a. The smooth appearance of the fissure surfaces in the field suggests low shear strength. However, they are not planar so the geometrical ‘roughness’ will also contribute to the shear resistance. Conventional drained bearing capacity calculations with an angle of shearing resistance of about 20 degrees (say 10 degrees for the smooth surface and an additional 10 degrees from the surface geometry), indicate a drained bearing capacity for a shallow foundation about the same as the undrained bearing capacity for a soil having an undrained shear strength of about 25 to 30 kPa. This suggests that the short term bearing capacity of a shallow foundation could be controlled by drained shearing along fissures, rather than undrained shearing through the clay, if the actual value of the undrained shear strength was greater than about 25 to 30 kPa. For this to
The lateral strength of the short pole is:

$$H_{ul} = 9s_u D_s \left[ \sqrt{(L + 2f + f_0)^3 + (L - f_0)^3} - (L + 2f + f_0)^2 \right]$$

1

The depth beneath the ground surface at which the maximum moment occurs is given by:

$$g_c = f_0 + \frac{H_{ul}}{9s_u D_s}$$

2

The lateral strength of a long pile is:

$$H_{ul} = 9s_u D_s \left[ \sqrt{(f + f_0)^3 + \left( \frac{2M_{ul}}{9s_u D_s} \right)^3} - (f + f_0) \right]$$

3

The location of the maximum moment, $M_{ul}$, for a long pile, is as given in equation 2. Beneath this point further pile shaft length is required to sustain the ultimate moment of the pile section. Thus the minimum pile shaft length required to develop the lateral strength given in equation 3 is:

$$L_{long, pile} = f_0 + \frac{H_{ul}}{9s_u D_s} + 2\sqrt{\frac{M_{ul}}{9s_u D_s}}$$

4

where:

- $H_{ul}$ ultimate lateral resistance of a pile (kN)
- $M_{ul}$ ultimate moment capacity of the pile shaft (kNm)
- $L$ length of the embedded part of the pile shaft (m)
- $D_s$ diameter of the pile shaft embedment (m)
- $f_0$ length of pile shaft near the ground surface (m) assumed to be unsupported in cohesive soil
- $f$ the distance above the ground surface at which the horizontal shear is applied (m)
- $s_u$ undrained shear strength of the clay in which the pile is embedded (kPa)
- $g_c$ distance down the pile shaft from the ground (m) surface at which the maximum moment occurs
- $L_{long, pile}$ minimum pile shaft length required (m) to develop the long pile lateral strength

Further information on these two cases is given by Pender (1997).

Table 1 gives the calculated ultimate lateral strength of poles embedded 1.2 m. Both the short pole and long pile values given by equations 1 and 3 are tabulated for various values of the undrained shear strength of the ground in which the poles are embedded. The lower limit used for $s_u$, 25 kPa, is well below the vane shear strength found for Auckland residual clays. Not surprisingly the short pole strength increases as the undrained shear strength of the clay increases, but the lateral strength for the long pile case, being controlled by the ultimate moment capacity of the pile shaft, is found to be only slightly dependent on the undrained shear strength of the soil. The results in Table 1 are based on the assumption that the concrete in which the pile shafts are embedded contributes nothing to the moment capacity of the section because of cracking. The

Figure 6 Bearing capacity of shallow foundations:

a) the standard ultimate strength mechanism
b) 'column' action in soil with vertical fissures

occur the fissuring would need to extend for depths of about one footing width or more below the foundation and the orientation of the fissures would need to be such that the shape of the bearing capacity mechanism could be matched, at least roughly, by a number of fissure segments.

Another possibility is that the near vertical fissures dominate, as illustrated in Figure 6b, and the foundation load is simply carried down through vertical 'columns' of clay to the underlying un fissured material.

**Lateral Strength of Pole Foundations Embedded in Clay**

A convenient approach to estimation of the short-term ultimate lateral strength of piles embedded in clay is that proposed by Broms (1964). The only soil property required is the undrained shear strength, $s_u$. Broms explains that there are two limiting cases. First the so-called short pole case where the lateral strength is limited by the soil strength and the long pile case where the lateral strength is limited by the moment capacity of the pile section. (Note the terminology: the short case refers to pole and the long case to a pile.)
concrete does contribute to lateral bearing though, so, $D_s$, the diameter of the embedded part, occurs in equations 1
and 3. The embedded length used for the short pole calculations in Table 1 is 1.2 m. The minimum embedded length
for the long pile case, from equation 4, is less than this for all but the case with an $s_u$ value of 25 kPa when it is
about 1.55 m.

The explanation of this result is as follows. For the short pole the shaft is considered to have unlimited
moment capacity and then, as indicated in equation 1, the lateral strength depends on the undrained shear strength
of the clay and the depth of embedment, $L$. In the long pile case the bending moment in the pile shaft increases as
the lateral load applied to the pile is increased. The maximum moment reaches the section capacity before the
shear strength of the soil is fully mobilised. Equation 3 shows that the lateral strength is a function of the shear
strength of the soil and the moment capacity of the pile section, but not on the embedment depth (assuming that
the embedment depth is sufficient for the pile to be able to sustain the section ultimate moment). In the design of
pole foundations both the short and long pile capacities are evaluated, and the smallest value becomes the design
value. It is apparent from Table 1 that, except for the lowest clay strength, it is the long pile strength which
is critical.

Repeating the calculations for 0.3 m diameter poles installed to a depth of 2.4 m in a 0.45 m diameter
embedment leads to the same conclusion.

Another way of considering the above result is to make $M_{ult}$ the subject of equation 3. That is, find the moment
capacity required of the pile section for a given $H_{ult}$. Rearranging equation 3 gives:

$$M_{ult} = \frac{H_{ult}^2}{18s_uD_s} + H_{ult}(f + f_o)$$

This shows that the required moment capacity of the pile section depends on two quantities. One can then see that
the undrained shear strength of the soil has a small effect on the required moment capacity of the pile section when
$H_{ult}(f + f_o) > H_{ult}^2/18s_uD_s$. It is also apparent that the significance of the shear strength of the soil decreases as $f$,
the distance above the soil surface at which the horizontal load is applied, increases.

### Ultimate Limit State Considerations

It is worth considering how equations 1 and 3 are used in an ultimate limit state design context. There are two quite
different approaches to ultimate limit state design; the partial Factor of Safety approach used in the Eurocodes and
the LRFD (Load and Resistance Factored Design) used in NZ and North America (cf Pender (2000)). Applying the
partial factor of safety approach to equation 3, one divides the undrained shear strength of the soil by a partial factor
of safety (a number greater than unity) and similarly calculates the ultimate moment capacity of the pile section using a
reduced ultimate bending stress. Equation 3 then gives a design lateral strength for a long pile.

Using the LRFD approach things are initially not quite so clear. We apply a strength reduction factor (a number
less than unity) to the $H_{ult}$ calculated with equation 1 to get the design lateral strength of the pile (note that, in keeping
with the LRFD approach, this is applied to $H_{ult}$ and not $s_u$ even if the end result would be the same). Likewise we
might expect to apply a similar factor to the $H_{ult}$ calculated with equation 3. However, in this case it is the moment
capacity of the pile section, buried on the right hand side of the equation, which controls the ultimate lateral strength.

A possible way out of this difficulty is to use equation 5 to find the moment capacity required of the pile section for a
given factored horizontal load. The strength reduction
factor for a timber pile is about 0.8 (note that this is additional to the load duration factor of 0.8; the steaming
factor of 0.9 and the shaving factor of 0.9 already applied). Thus I propose tentatively that one uses equation 3 with
the pole strength reduction factor applied to the ultimate
moment capacity of the pole section, that is substitute $\Phi_{pole}M_{ult}$ for $M_{ult}$, and apply no strength reduction factor
directly to $H_{ult}$ (in effect using equation 5 and then
rearranging to get $H_{ult}$). This acknowledges that the critical
factor in controlling the lateral capacity given by equation
3 is $M_{ult}$ rather than $s_u$. In Table 2 these suggestions are
applied to the results in Table 1, notice that now the short
pole case is critical for the first two values of $s_u$.

### Conclusion

We have two conclusions:

First, the undersides of shallow foundations are typically established 300 to 600 mm beneath the surface of

---

**Table 1** Comparison of the ultimate lateral strengths of short and long piles in clay

<table>
<thead>
<tr>
<th>Undrained Shear Strength (kPa)</th>
<th>$H_{ult_long_pile}$ (kN)</th>
<th>$H_{ult_short_pole}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>26.4</td>
<td>15.2</td>
</tr>
<tr>
<td>50</td>
<td>30.3</td>
<td></td>
</tr>
<tr>
<td>75</td>
<td>45.5</td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>60.6</td>
<td></td>
</tr>
</tbody>
</table>

---
the cleared ground surface at the building site. This means that part of the fissured soil is removed in the site preparation and possibly more when the individual footing positions are prepared. If the depth of fissuring beneath the shallow foundation is less than the width, then the load is likely to be carried directly down to the underlying unfissured clay by 'column' action.

Second, we come to the interesting, and unexpected, conclusion that the ultimate lateral strength of a pile embedded in clay is not sensitive to the shear strength of the clay if the long pile case, rather than the short pole case, controls the lateral capacity. In reaching this conclusion the concrete in which the pile is embedded was assumed to contribute nothing to the moment capacity of the fissured soil in which the pole is embedded may not be of great consequence if the long pile case controls.

Acknowledgments
Drs Richard Fenwick and Richard Hunt, of the Department of Civil and Environmental Engineering at the University of Auckland, provided some helpful comments on aspects of capacity of timber piles. In addition Brian Wilson and Tony Synge, consulting engineers and members of the Structural Engineering Society, have provided me with helpful insights into pole foundation design.

References

Table 1: Comparison of the ultimate and design lateral strengths of short and long piles in clay

<table>
<thead>
<tr>
<th>Undrained Shear Strength (kPa)</th>
<th>Hult_long_pile (kN)</th>
<th>Hult_long_pile (kN) with Phi_pole</th>
<th>Hult_short_pile (kN)</th>
<th>Phi_soil</th>
<th>Hult_short_pile (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>15.2</td>
<td>7.6</td>
<td>25</td>
<td></td>
<td>15.2</td>
</tr>
<tr>
<td>50</td>
<td>30.3</td>
<td>15.2</td>
<td>50</td>
<td></td>
<td>30.3</td>
</tr>
<tr>
<td>75</td>
<td>45.5</td>
<td>22.7</td>
<td>75</td>
<td></td>
<td>45.5</td>
</tr>
<tr>
<td>100</td>
<td>60.6</td>
<td>28.7</td>
<td>100</td>
<td></td>
<td>60.6</td>
</tr>
<tr>
<td>125</td>
<td>75.8</td>
<td>35.2</td>
<td>125</td>
<td></td>
<td>75.8</td>
</tr>
</tbody>
</table>

1 This is a development of a paper having a similar title and contents published in the SESC Journal for December 2001 (Journal of the Structural Engineering Society of New Zealand, 14(2), 35-41)
2 NZS 3603:1993 stipulates that permanent loads, and it mentions soil pressures specifically in this category, require a load duration factor of 0.6. It also states that for medium term loads, duration between 6 hours and five years, the load duration factor is 0.8. The time frame associated with the short-term capacity in the geotechnical context is likely to be towards the beginning of the medium load duration in the timber code. Thus a load duration factor of 0.8 is used in the calculations for Tables 1 and 2.
COMPANY PROFILES

Earthtech Consulting Ltd, Auckland

Earthtech Consulting Ltd is a specialist engineering practice working in the fields of engineering geology, geotechnical engineering and hydrogeology. The company is run by Philip Kelsey and Aidan Nelson.

Earthtech was formed in 1989 and initially worked primarily on subdivisions in the Franklin District. With time Earthtech has enjoyed working on larger and more varied projects in the Auckland area and beyond.

Landfill engineering has been a significant area of special interest. Earthtech has been involved with landfill site selection studies and specific site investigations in the Auckland, Waikato, Bay of Plenty and Manawatu Regions. One of the highlights has been the geotechnical and hydrogeological investigations and hearing work associated with the recently consented North Waikato Regional Landfill at Hampton Downs.

Earthtech has also worked on a number of coastal engineering projects on the Coromandel. The Pauanui and Whitianga Canal Development projects have provided challenges with respect to ground treatment for liquefaction, control of groundwater drawdown with respect to adjacent aquifers, and a very attractive working environment for fieldwork.

In addition to the above, Earthtech is also currently involved with road engineering, quarrying, and groundwater supply projects. The company also carries out peer review work for a number of local authorities and private companies.

Our practice has a strong emphasis on fieldwork and we consider that good solutions to difficult geotechnical and hydrogeological problems are based on good site characterisation.

Earthtech Consulting Ltd
PO Box 721, Pukekohe
Phone: 09 2383 669
Fax: 09 2328 818
Email: pkelsey@xtra.co.nz
Contact: Philip Kelsey

Montgomery Watson Harza (New Zealand) Ltd

The geosciences group at MWH includes people with skills in hydrology, engineering geology, geotechnical and foundation engineering, dam and dam safety, contaminated site investigation and remediation. The group has over 30 experts based in Whangarei, Auckland, Hamilton, Wellington, Palmerston North, Nelson, Christchurch and Dunedin.

MWH’s geotechnical engineering team provides a wide range of services and has been involved in the identification, assessment, investigation and construction supervision of numerous domestic and international projects since the early 1930s. These projects include bridges, road engineering, canals, tunnels, dams, hydro-electric power schemes, irrigation schemes, water supply, mining schemes, landfill siting and design, and structure foundations.

The recent merger between Montgomery Watson and Harza Engineering (to form MWH) enhances our access to the highest level of geotechnical expertise from UK and USA. Internationally, MWH is responsible for some of the largest dams in the world including concrete, rock and earthfill dams. This includes the 240 metre concrete arch dam for the 3300 MW Ertan project in China and the 200 metre rockfill/clay core dam for the San Roque in the Philippines.

In New Zealand, the geosciences group are active within the road engineering sector through numerous Transit and local authority projects including Hawkes Bay Expressway in Napier, and PSMC support; the mineral extraction industry; clean and waste water projects; and dam schemes including Wilson’s Dam near Whangarei.

MWH (New Zealand) Ltd
PO Box 89, Hamilton
Phone: 07 839 9863
Fax: 07 839 4234
Email: stuart.finlan@nz.mwhglobal.com
Contact: Stuart Finlan
Career Path
My university life began in the engineering school wanting to become a mineral processing engineer. However finding geology more interesting I changed degree courses and completed a BSc and MSc in Geology at Auckland University. Between university degrees I spent a wonderful summer living in Waihi Beach working for Cyprus Gold NZ in their exploration division, investigating potential gold sources in the Waitekauri Valley. Unfortunately I realised the limited potential for exploration and mining in New Zealand and not wanting to join my friends ‘across the ditch’ changed my geology career path from mining and mineral exploration to engineering geology.

I joined the Beca Group of companies in 1992 working in their soils laboratory during my Masters degree instead of joining the newly introduced student loan scheme; a mutually beneficial arrangement that allowed me to use the laboratory equipment for the soils testing requirement in my Master’s thesis. On completion of the degree I joined Beca full time and became an interim laboratory manager for nearly two years while a suitably qualified manager was sought. Eventually I left this role after training someone for this position in 1995. Since then I have worked as an engineering geologist for Beca based in their Auckland office.

Recently I have been fortunate enough to reduce my hours at Beca to pursue other interests including cattle breeding and farming in the Hokianga and Onewhero, being the financial administrator of a psychological practice and assisting in the family engineering practice on geotechnical/geologic issues.

Typical Work Week
Monday, Wednesday and Thursdays are normally spent at Beca writing reports, engineering geologic mapping, undertaking geotechnical analyses, reviewing logs and cores, mentoring/training a group of young engineering geologists and job management.

Tuesday and every second Saturday morning are normally spent at Rojolie Clinic being a receptionist, reviewing financial performance of staff and the company, writing promotional documents, upgrading and implementing new administrative tools as required and spending time with accountants, lawyers and financial advisers.

Tuesday afternoons, Fridays and alternative weekends are flexible days generally spent on one of the family farms or family business, and when required catching up at Beca. Lately there have been more weekends off socialising than farming, due to a brother returning to NZ to manage the Hokianga farm.

Highs and Lows
I enjoy nearly all aspects of field work and analyses that leads to engineering geologic assessment of an area or site. Particularly in new places and countries, where one’s geologic interpretation skills are required most and where one can get involved in other cultures. On a personal level I have obtained a lot of satisfaction from engineers and clients seeing the value that myself and colleagues have provided when engineering geologic advice is used early in their projects and frustration when they ask too late.

The RMA process has reduced enjoyment of projects as they get stuck in legal or consultation battles.

Ambitions
Presently I am looking forward to becoming a mother and reducing my consulting hours. I am sure this new role will stretch my people and project management skills.

Advice
Plan jobs to the smallest feasible task and complete each one before moving on.

The best advice I have received is to enjoy your work and personal life as much as possible. If not, get some good counselling.
**Route to the Job**
I come from a family of Engineers. My father, grandfather and uncle were all in the profession, and I was proud to follow in their footsteps. During my civil engineering degree at Auckland University, I was awarded a cadetship with Fletcher Construction, which gave me a valuable grounding in large civil engineering works. Thanks to the problems experienced on projects such as the Tauranga harbour bridge and the Waikato River diversion works at Arapuni, I developed an interest in soil mechanics and decided to pursue a career in geotechnical engineering.

I have been employed by Foundation Engineering in Auckland since mid-1989, where I now co-manage a large team and investigate and report on a wide range of land development and building projects.

**Typical Day**
Because my work is so varied, both in type and scale, there really is no such thing as a typical day. During an average week, I spend the equivalent of at least one day attending meetings and carrying out site inspections. The rest of my time is spent reviewing investigative data, writing reports, working on designs and corresponding with clients. As a Senior Engineer in the Consultancy I am also responsible for reviewing technical reports and designs prepared by junior engineers.

**Highs/Lows**
High points of my job include being able to bring to life a client’s vision, especially when a site has challenging geotechnical problems. Being in a medium-sized consultancy, and being directly responsible for the geotechnical aspects of an entire project, I enjoy the satisfaction of managing a job from conception to completion. This would probably not be the case in a large consultancy. Low points include dealing with difficult clients who employ me for my expertise but still think they know best! Another is managing a very large workload, trying to juggle a hundred things at once and please all my clients, who often have unrealistic timeframes.

**Ambitions**
One of the most crucial lessons I have learnt in recent years has been the importance of geological models to assist in solving complex geotechnical problems. I would therefore like to undertake post graduate studies in fields such as engineering geology and landslide engineering. On a business level, as a Director I hope to grow Foundation Engineering both within Auckland and further afield.

**Advice**
For a young person studying engineering I would strongly recommend post graduate study in both geomechanics and engineering geology. This would be invaluable for developing the appropriate technical skills for a successful future in geotechnical engineering. It is also vital in our field to have good verbal and written communication skills. Irrespective of your age or experience, every site offers new challenges. There is always something new to learn and even for Senior Engineers, nothing beats putting on your boots and getting your hands dirty once in a while.
Answer to Brain Teaser No. 5 (December, 2001)

The quotation is from an 'essay' written by Karl Terzaghi in 1924 in Istanbul. It is referred to in Goodman's biography of Terzaghi ("The Engineer as Artist"), and printed in full in the issue of Geotechnique dedicated to Terzaghi's memory shortly after his death - March 1964 (Vol. 14, N o. 1).

It seems often to be the case that those who make an outstanding contribution in one particular technical field also have a very wide knowledge and interest in other areas of human existence. It was certainly the case with Terzaghi, and he 'philosophised' much about the nature of human existence and how to make life meaningful. The essay is worth reading in full, along with other quotes of Terzaghi which appear in the same edition of Geotechnique.
FOREIGN CORRESPONDENT

Steal the Cream and Leave the Milk
Jon Sickling, Sinclair Knight Merz, Melbourne

“Steal the cream and leave the milk, that's the way isn't it Jon? Aarr, there'd not be much left over for the rest of us then? Why don't you give me them boots now?” (best pronounced in one's best Dougal/Father Ted accent).

Such was my early morning greeting from big James, a burly fifty-something labourer with a formidable chest like a mountain gorilla with implants. We were working on the N4, a main route crossing Ireland from Sligo on the west coast to Dublin on the east. Not that we were located near either place, or any place. In fact, the best sense of location one could gather was from the adjacent River Shannon, which throughout winter frequently caused the site to look like a prime rainbow trout spot.

James as it turns out, was from the town up the road and had been for the last fifty-something years. He couldn't understand why some smart-arse foreign lad like myself should be given brand spanking new padded safety jacket, new boots, new hard-hat etc. I had to regularly explain to him that it was in the contract, and that it was also in the contract that if he didn’t wear his old hard-hat, worn safety jacket and grubby steel caps then he might be approaching retirement. Of course my considerable flexing of contractual muscle and three syllable words failed to impress him in the slightest, usually to be greeted with a highly creative monologue of profanity, blasphemy and the odd adjective. Not one to be outdone by an old man, I informed the Contractor that the transition piles (with reducing embedment into peat to make a smooth settlement transition) could not be cut down with the hydraulic cropper due to risk of pile uplift. The next morning James was out there with concrete saw and sledgehammer, sweating and cursing me with every swing at the piles.

The project consisted of rebuilding the approach embankments to a small bridge in the middle of a peat bog with piled embankments and basal reinforcement. The magnitude and rate of settlement was rather incredible considering the light loads (embankment height less than 1.5 m), with over one metre of settlement since 1976. After excavation of one half of the road, at least eight previous road surfaces were easily visible which obviously amounted to an ongoing maintenance problem for the local authority.

We had our own problems – it can really rain in Ireland. There was talk on the TV News of it being the wettest winter in Central Ireland for fifty years, which was wonderful news. We regularly had to pump over 800 l/s, which is rather substantial for a miserable little roading job. Of course the overflowing lakes up the hill didn’t help too much either. The Contractor’s poor harassed Site Manager would come to me with a pale glum face after doing battle with the Shannon:

“How's the water doing then, Mick?”

“CENSORED – don’t talk to me about them CENSORED pumps. I’ve every spare pump in Ireland on the job, and half the CENSORED pumps from Europe coming.”

I quickly learnt to respond to such statements with a bland “Oh right then.”

Of course the predictable outcome of such tampering with the environment came to pass. A letter was received from a PAP (or Project Affected Person) who owned an adjacent field, to the effect that we were causing the flooding of his land. After carefully considering how this could be the case when we were pumping 800 l/s from his land, I passed the public relations aspects to the Client and wondered if we should actually send the fellow a bill for dewatering. I could see the next event, when in summer we would somehow be responsible for ruining his indigenous fishery rights on the same bit of land.

And so after three months of eagerly watching men stick bits of concrete into the ground, it was time to go and try my hand in Australia. My overall conclusion is that the Irish Potato Famine of the late 19th century as a trigger for mass emigration was probably just a politically correct way of saying the weather stinks. Mind you, they brew fantastic stout, make a good pie and produce a fine bit of trout.
PHOTO COMPETITION 2002

Awarded First Place
Sam Brindle,
Harrison Grierson
Consultants Ltd

Where did I park that digger again?

Graham Salt
Tonkin & Taylor Ltd, Dunedin
(“I could use that $200 for the panelbeater’s bill”)

Day 1 on the Big Project

Neil Crampton
Pattle Delamore Partners Ltd

Hospital pass to the offsider
Gordon Stevens, Maccaferri

Nose dive

Craig Davidson, Meritec

House caught with foundations down

Note: The photo is from a NON Meritec job, observed while driving the Auckland streets. Staff advised builders of the danger and contacted the Council.

Peter Geddes, Hawthorn Geddes Engineers and Architects

Roading compaction in Northland has come a long way
Have you ever wondered what sort of character becomes a Geotechnical Engineer or Engineering Geologist? Do we think of ourselves as individuals with distinctly different character traits or is there some consistent thread? Maybe we are all cut from the same cloth? Maybe we all think the same way? Maybe, underneath some obvious differences, we are really kindred spirits?

I think I have the answers to these compelling questions because I have recently been the subject of a character assessment (or should that be character assassination?). I was volunteered for this ordeal, and was sold on the potential benefits, when it was explained that it would facilitate greater understanding of working and personal friendships, conflict resolution, partnering and a drive towards developing empathic, harmonious and effective working relations with colleagues. The theory goes that if you can understand yourself and your own personality and if you know your own personal strengths and weaknesses, then the chances are that you will have a better chance of understanding how to interact and relate to others.

What are Bob Wallace’s personal drivers? What motivates Bob Wallace to support a cause? How will Bob Wallace react to a problem or a conflict? These are the questions that the social scientists and my employers believed were important for me to answer. Of course, you realise it was for my own benefit, my employers had no interest in the results whatsoever and they would never be used in any future assessment of my appropriateness for corporate advancement. Yeah, right.

In my enthusiasm to learn about myself, I ended up doing three tests. This could have been interpreted as an attempt to satisfy a not uncommon multiple or split personality psychosis. However, I can assure you that it was simply a rigorous scientific examination to identify whether three different tests provided three completely different answers. I’ll let you be the judge.

I was told that the ‘official’ test would follow the learned and well-established MBTI process (for the uninitiated MBTI stands for Myers Briggs Type Indicator). It was then relatively easy to look up the Internet and find a whole raft of example tests, based on similar principles, that could be used as a benchmark.

The first benchmark test was the LOTR Personality Analysis (where LOTR stands for Lord of the Rings). Apparently, I am somewhat like Balin the Dwarf, short of stature and temper, extremely loyal and an enthusiastic builder who can be tempted by riches and promises of power.

The second benchmark test was the MSSS Personality Analysis (where MSSS stands for Muppet Show & Sesame Street). Based on this assessment it was suggested that my personality exhibited similar traits to Miss Piggy; pretentious and precious, susceptible to self promotion and showing off, with a misplaced faith in my own ability and a bit of a bully.

These benchmark tests were quite scary - I didn’t know if I particularly liked the personality traits being portrayed and I was pinning all my hopes on the official version finding some attribute that was in some way positive. With some trepidation I learned that an attractive young lady from the States with a social psychology degree from an Ivy League college would undertake the final MBTI assessment.

Rest assured dear reader, the results of my final assessment identified me as an ENTP character (where ENTP stands for Enthusiastic, Nice, Thoughtful and Patient). I was vindicated, but only at the expense of the credibility of the LOTR and MSSS Analyses.

Whilst trying to conclude this column, no doubt in one of my more Thoughtful Modes, I did wonder whether my personality was perhaps consistent with other Geotechnical Engineers. I made some enquires with my Ivy League saviour.

The answer of course is that whilst we are all different, there are some interesting general trends to the Geotechnical Engineer’s personality which have been classified into certain groupings. These are listed below, which one do you belong to?

<table>
<thead>
<tr>
<th>Characteristic Trends</th>
</tr>
</thead>
<tbody>
<tr>
<td>Optimistic, Pedantic, Unhelpful but Sensible</td>
</tr>
<tr>
<td>Boastful, Clever, Humourless and Feudal</td>
</tr>
<tr>
<td>Tenacious and Tedious, Humble and Grumpy</td>
</tr>
<tr>
<td>Stubborn, Kind and Masochistic</td>
</tr>
</tbody>
</table>
AUGUST 11 - 15, 2002, Rio De Janeiro, Brazil
4th International Congress on Environmental Geotechnics
Conference themes:
- Design and performance criteria
- Tailings and mine wastes
- Risk assessment
- Management of contaminated sites
- Remediation and related costs
www.4iceg.ufrj.br

AUGUST 14 - 15, 2002, Wellington, NZ
5th New Zealand Natural Hazards Management Conference
Conference themes:
- Applying hazard information to best practice planning
- Exploring new technologies and advances in science applications
- Natural hazard mitigation for industry
- Creating resilient communities through integrating science into practice.
Optional field trip 16 August 2002

SEPTEMBER 9 - 13, 2002, London, UK
12th European Conference on Earthquake Engineering
Conference themes:
- Engineering seismology
- Geotechnical earthquake engineering
- Structural earthquake engineering: buildings
- Structural earthquake engineering: special structures
- Risk assessment and mitigation
- Field reports
www.12eceee.org.uk

SEPTEMBER 16 - 20, 2002, Durban, South Africa
9th International IAEG Congress – Engineering Geology for Developing Countries
Conference themes:
- Engineering geology for developing countries
- Engineering geology mapping and soil testing
- Engineering geology and the environment
- Groundwater
- Case histories and new developments
- Construction materials
- Information technology applied to engineering geology
- Gondwana rocks and engineering geology
www.stanfield.und.ac.za/Durban2002

SEPTEMBER 24 - 27, 2002, New Delhi, India
Advancing Rock Mechanics Frontiers to Meet the Challenges of 21st Century
Conference themes:
- Engineering geology
- Stress strain and strength characteristics
- Numerical and physical modelling
- Underground works
- Techniques in improving rock mass quality
- Surface excavations in rock
- Foundations of large dams and structures
- Deep drilling technology
- Environmental issues
- Environmental geotechnics
- The value of geotechnics in design and construction
http://cbip.org

NOVEMBER 14 - 20, 2002, Hong Kong
Natural Terrain – A Constraint to Development?
Organised by the Hong Kong Branch of the IMM
- The conference will cover the policies and technical issues behind natural terrain studies including environmental aspects and case studies.
- International keynote speakers are Professors Earl Brabb and Oldrich Hungr, with local keynote speakers from both the Government and private sector.
- Abstracts were to be submitted by November 2001
Contact: Louisa McAra
Email: louisam@atkins-china.com.hk
Phone: +852 2972 1821

NOVEMBER 17 - 20, 2002, Texas, USA
1st International Conference on Scour of Foundations
Conference themes:
- Scour of foundations
- Bridge scour
- Erosion of soils
- Dam scour
- Offshore platform scour
- Prediction of scour depth
- Pier scour
- General degradation and aggradation
- Countermeasure selection
- Scour monitoring
http://tti.tamu.edu/conferences/scour
Abstracts due: 30 September 2001
NOVEMBER 25 – 28, 2002, Madeira, Portugal
Eurock 2002 – International Symposium on Rock Engineering for Mountainous Regions
Conference themes:
- Slope stability
- Undermountain works
- Environmental protection
- Case studies

- Mechanical characterisation of volcanic rocks
- Volcanic construction materials
www.eurock2002.com

2003

MAY 11 – 16, 2003, Sorrento, Italy
International Conference: Fast Slope Movements, Prediction and Prevention for Risk Mitigation
International Workshop: Occurrence and Mechanisms of Flows in Natural Slopes and Earthfills
Session topics:
- Risk assessment from theory to practice
- Risk mitigation
- Criteria for land management
www.unina2.it/fsi2002

JUNE 22 – 26, 2003, Cambridge, USA
12th Panamerican Conference on Soil Mechanics and Geotechnical Engineering
Conference themes:
- Ground characterisation and exploration
- Geo-materials and mechanics
- Geo-construction
- Lessons learned from failures
- Future challenges
- Fluids in the subsurface environment
www.soilrock.mit.edu

AUGUST 25 – 28, 2003, Prague, The Czech Republic
13th European Conference on Soil Mechanics and Geotechnical Engineering
‘Geotechnical Problems with Man-Made and Man Influenced Grounds’
Conference themes:
- Man-made deposits – recent and ancient
- Contaminated ground – remediation and preparation for new construction
- Construction on man-made and remediated brownfield sites
- Foundations in urban areas
- Networking of geo-engineers between East and West Europe
www.ecsmge2003.cz

SEPTEMBER 22 – 24, 2003, Lyon, France
3rd International Symposium on Deformation Characteristics of Geomaterials
Conference themes:
- Soils and soft rock
- Experimental investigations into deformation properties from very small strains to beyond failure
- Time effects (aging and viscous effects)
- The interpretation of laboratory, in situ and field observations of deformation behaviour
- Characterising and modelling behaviour
- Case studies
www.islyon03.entpe.fr

2004

FEBRUARY 9 – 11, 2004, Auckland, NZ
‘To the eNZ of the Earth’
9th ANZ Conference on Geomechanics
Topics Include:
- Slope instability
- Foundations
- Piles
- Anchors/reinforcement
- Dams

- Roading
- Environmental geotechnics
- Seismic engineering
- Rock mechanics
- Expansive soils
- Engineering geology
- Testing

Call for abstracts and papers: see advert in this issue (page 18)
# NEW ZEALAND GEOTECHNICAL SOCIETY INC.

## Management Committee Address List 2002

<table>
<thead>
<tr>
<th>NAME</th>
<th>POSITION</th>
<th>ADDRESS, EMAIL</th>
<th>PHONE, FAX</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crawford, SA (Steve)*</td>
<td>Chairman</td>
<td>Tonkin &amp; Taylor</td>
<td>07 571 3570 Work</td>
</tr>
<tr>
<td></td>
<td></td>
<td>98 Birch Ave</td>
<td>07 571 5508 Fax</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Tauranga</td>
<td>021 675 468 M obile</td>
</tr>
<tr>
<td></td>
<td></td>
<td><a href="mailto:Scrawford@tonkin.co.nz">Scrawford@tonkin.co.nz</a></td>
<td></td>
</tr>
<tr>
<td>Grocott, GG (Guy)</td>
<td>Immediate Past Chairman</td>
<td>Golder Associates Ltd</td>
<td>03 377 5696 Work</td>
</tr>
<tr>
<td></td>
<td></td>
<td>P O Box 2281</td>
<td>03 377 9944 Fax</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Christchurch</td>
<td>03 337 0553 H ome</td>
</tr>
<tr>
<td></td>
<td></td>
<td><a href="mailto:Scrawford@tonkin.co.nz">Scrawford@tonkin.co.nz</a></td>
<td></td>
</tr>
<tr>
<td>Fellows, DL (Debbie)*</td>
<td>Management Secretary</td>
<td>6 Sylvan Valley Ave</td>
<td>09 817 7759 H ome</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Titirangi</td>
<td>09 817 7035 Fax</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Auckland</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><a href="mailto:dfellows@xtra.co.nz">dfellows@xtra.co.nz</a></td>
<td></td>
</tr>
<tr>
<td>McPherson, ID (Ian)*</td>
<td>Treasurer</td>
<td>Connell Wagner Ltd</td>
<td>04 472 9589 Work</td>
</tr>
<tr>
<td></td>
<td></td>
<td>P O Box 1591</td>
<td>04 472 9922 Fax</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Wellington</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><a href="mailto:McPhersoni@conwag.com">McPhersoni@conwag.com</a></td>
<td></td>
</tr>
<tr>
<td>Glassey, P (Phil)*</td>
<td>NZ Geomechanics News</td>
<td>Geological &amp; Nuclear Sciences</td>
<td>03 479 9684 Work</td>
</tr>
<tr>
<td></td>
<td></td>
<td>764 Cumberland Street</td>
<td>03 477 5232 Fax</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Private Bag 1930</td>
<td>027 249 0439 M obile</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dunedin</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><a href="mailto:P.Glassey@gns.cri.nz">P.Glassey@gns.cri.nz</a></td>
<td></td>
</tr>
<tr>
<td>Marsh, J (John)*</td>
<td>Assistant Treasurer</td>
<td>Beca Carter Hollings &amp; Ferner Ltd</td>
<td>09 300 9174 Work</td>
</tr>
<tr>
<td></td>
<td></td>
<td>P O Box 6345</td>
<td>09 300 9300 Fax</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Wellesley St</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Auckland</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><a href="mailto:jmash@beca.co.nz">jmash@beca.co.nz</a></td>
<td></td>
</tr>
<tr>
<td>Murray, JG (Grant)</td>
<td>ISSM GE Australasian Vice President</td>
<td>Sinclair Knight Merz Ltd</td>
<td>09 913 8984 Work</td>
</tr>
<tr>
<td></td>
<td></td>
<td>P O Box 9806</td>
<td>09 913 8901 Fax</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Auckland</td>
<td>09 524 5078 H ome</td>
</tr>
<tr>
<td></td>
<td></td>
<td><a href="mailto:Gmurray@skm.co.nz">Gmurray@skm.co.nz</a></td>
<td>021 271 1992 M obile</td>
</tr>
<tr>
<td>Riddolls, BW (Bruce)</td>
<td>IAEG Australasian Vice President</td>
<td>Golder Associates Ltd</td>
<td>03 377 5696 Work</td>
</tr>
<tr>
<td></td>
<td></td>
<td>P O Box 2281</td>
<td>03 377 9944 Fax</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Christchurch</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><a href="mailto:briddolls@golder.co.nz">briddolls@golder.co.nz</a></td>
<td></td>
</tr>
<tr>
<td>Haberfield, CM (Chris)</td>
<td>ISRM Australasian Vice President</td>
<td>Golder Associates Pty Ltd</td>
<td>+61 3 8862 3500 Work</td>
</tr>
<tr>
<td></td>
<td></td>
<td>P O Box 6079</td>
<td>+61 3 8862 3501 Fax</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Hawthorn West</td>
<td>+61 3 9754 5452 H ome</td>
</tr>
<tr>
<td></td>
<td></td>
<td>VIC 3122, Australia</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><a href="mailto:chaberfield@golder.com.au">chaberfield@golder.com.au</a></td>
<td></td>
</tr>
</tbody>
</table>

* Elected members of committee
* Appointed position
NEW ZEALAND GEOTECHNICAL SOCIETY INC.

Objectives
a) To advance the study and application of soil mechanics, rock mechanics and engineering geology among engineers and scientists
b) To advance the practice and application of these disciplines in engineering
c) To implement the statutes of the respective international societies in so far as they are applicable in New Zealand.

Membership
Engineers, scientists, technicians, contractors, students and others who are interested in the practice and application of soil mechanics, rock mechanics and engineering geology.

Members are required to affiliate to at least one of the International Societies.
Students are encouraged to affiliate to at least one of the International Societies.

Annual Subscription
Subscriptions are paid on an annual basis with the start of the Society’s financial year being 1st October. A 50% discount is offered to members joining the Society for the first time. This offer excludes the IAEG bulletin option and student membership. No reduction of the first year’s subscription is made for joining the Society part way through the financial year.

Basic membership subscriptions (inclusive of GST)
which include the magazine NZ Geomechanics News, are:
Members $73.10
Students $28.10

Affiliation fees for International Societies
are in addition to the basic membership fee:
International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) $24.00
International Society for Rock Mechanics (ISRM) $33.00
International Association of Engineering Geology & the Environment (IAEG) $21.00
(with bulletin) $70.00

All correspondence should be addressed to the Secretary. The postal address is:
NZ Geotechnical Society Inc.
P O Box 12 241
WELLINGTON

June 2002, Issue 63
NEW ZEALAND GEOTECHNICAL SOCIETY INC.
APPLICATION FOR MEMBERSHIP
(A Technical Group of the Institution of Professional Engineers New Zealand (Inc))

Full Name (Underline Family Name)
Postal Address
Phone No: Fax No: Email:
Date of Birth
Academic Qualifications
Professional Memberships Year Elected
Present Employer
Occupation
Experience in Geomechanics

Student Members:
Tertiary Institution
Supervisor
Supervisor’s signature

Note that the Society’s rules require that in the case of student members “the application must also be countersigned by the student’s Supervisor of Studies who thereby certifies that the applicant is indeed a bona-fide full time student of that Tertiary Institution”; Applications will not be considered without this information.

AFFILIATION TO INTERNATIONAL SOCIETIES:
All full members are required to be affiliated to at least one Society, and student members are encouraged to affiliate to at least one Society. Applicants are to indicate below the Society/ies to which they wish to affiliate.

I wish to affiliate to:
International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) Yes/No
International Society for Rock Mechanics (ISRM) Yes/No
International Association of Engineering Geology & the Environment (IAEG) Yes/No
(with Bulletin) Yes/No

DECLARATION:
If admitted to membership, I agree to abide by the rules of the New Zealand Geotechnical Society Inc.
Signed ____________________________ Date ________________

ANNUAL SUBSCRIPTION:
Due on notification of acceptance for membership, thereafter on 1st of October. Please do not send subscriptions with this application form. You will be notified and invoiced on acceptance into the Society.

PRIVACY CONDITIONS:
Under the provisions of the Privacy Act 1993, an applicant’s authorisation is required for use of their personal information for Society administrative purposes and membership lists. I agree to the above use of this information:
Signed ____________________________ Date ________________

(FOR OFFICE USE ONLY)
Received by the Society ____________________________
Recommended by the Management Committee of the Society ____________________________
## NEW ZEALAND GEOTECHNICAL SOCIETY INC. PUBLICATIONS

<table>
<thead>
<tr>
<th>Publication Name</th>
<th>List Price Members</th>
<th>List Price Non-Members</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>New Zealand Geomechanics Society Conferences:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Proceedings of the New Zealand Geotechnical Society Symposium – Engineering and Development in Hazardous Terrain Christchurch 2001</td>
<td>$50</td>
<td>$70</td>
</tr>
<tr>
<td>Proceedings of the New Zealand Geotechnical Society Symposium – Roading Geotechnics 98 Auckland 1998</td>
<td>$40</td>
<td>$70</td>
</tr>
<tr>
<td>Proceedings of the Auckland Symposium - Groundwater and Seepage May 1990</td>
<td>$10</td>
<td>$45</td>
</tr>
<tr>
<td><strong>Australia - New Zealand Conferences on Geomechanics:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Other Publications:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Proceedings of the 2nd Australia – NZ Young Geotechnical Professionals Conference, Auckland, December 1995</td>
<td>$25</td>
<td>$40</td>
</tr>
<tr>
<td>Shear Vane Guidelines</td>
<td>$15</td>
<td>$20</td>
</tr>
<tr>
<td>Guidelines for the Field Description of Soils and Rocks in Engineering Use</td>
<td>$10</td>
<td>$13</td>
</tr>
<tr>
<td>Stability of House Sites and Foundations – Advice to Prospective House and Owners and Section Owners</td>
<td>$1</td>
<td>$1</td>
</tr>
<tr>
<td>Back Issues of NZ Geomechanics News (depending on availability)</td>
<td>50c</td>
<td>50c</td>
</tr>
</tbody>
</table>

Prices do not include GST or postage & handling

Orders to:
Debbie Fellows
Management Secretary
6 Sylvan Valley Ave
Titirangi, Auckland
Email: dfellows@xtra.co.nz
ADVERTISING

NZ Geomechanics News is published twice a year and distributed to the Society's 500 members throughout New Zealand and overseas.

The 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.

<table>
<thead>
<tr>
<th>Advertisement Location</th>
<th>Single Issue</th>
<th>Advert. Size (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Black &amp; White</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Back Cover</td>
<td>$300</td>
<td>210 wide x 297 high</td>
</tr>
<tr>
<td>Inside Cover (Front or Back)</td>
<td>$250</td>
<td>210 wide x 297 high</td>
</tr>
<tr>
<td>Full Page Internal</td>
<td>$225</td>
<td>185 wide x 265 high</td>
</tr>
<tr>
<td>Half Page Internal</td>
<td>$175</td>
<td>90 wide x 265 high</td>
</tr>
<tr>
<td>Quarter Page Internal</td>
<td>$150</td>
<td>90 wide x 130 high</td>
</tr>
</tbody>
</table>

| **Colour**             |              |                   |
| Full Page Internal     | $400         | 210 wide x 297 high |
| A3 Centrefold          | $750         | 420 wide x 297 high |

| **Inserts**            |              |                   |
| Insert to be posted with magazine - $200/flyer |                   |
| Maximum size single A4 page | |                   |
| Special price given on request for other types and sizes | |

**Note**
1. All rates exclude GST
2. Space is subject to availability
3. 3 mm bleed
4. Advertiser to provide all flyers

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: 09 817 7759
Fax: 09 817 7035
Email: dfellows@xtra.co.nz
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
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 reinforcement 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: STEVENSON**

Building Products

Phone 0800 610 710

6 Branches Auckland wide 'Delivering Value'