JUNE 2009 issue 77

NZ GEOMECHANICS NEWS

Newsletter of the New Zealand Geotechnical Society Inc. ISSN 0111–6851

NEW ZEALAND GEOMECHANICS NEWS

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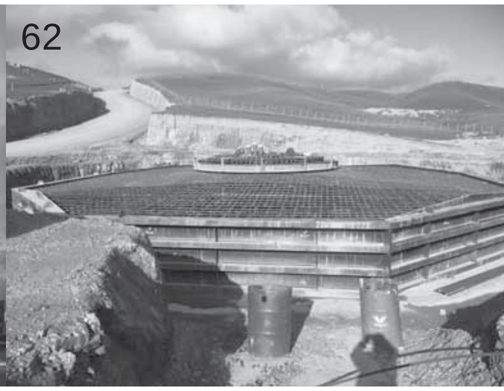

Cover image: Pike River Tunnel – Seth Tiddy

CHAIRMAN'S CORNER

Welcome to the June issue of the Geomechanics News for 2009.

Our Society

Ann Williams has stepped down as Chair of the Society and we are sad to see her leave this role. She has been an inspiration to me. Please join me in thanking Ann for doing a wonderful job as Chair last year, while juggling work and family commitments and organizing the 11th IAEG Congress. Ann, thank you so much for all your hard work and dedication to the New Zealand Geotechnical Society.

Changes to the Management Committee

At the well attended Annual General Meeting on 31st March 2009 a change to the society rules means a new position as Vice Chair was established. This will provide succession planning and continuity within the Management Committee. Your Society Management Committee for 2009 looks like this:

- **Philip Robins** - Chair
- **David Burns** - Vice Chair
- **Amanda Blakey** - Management Secretary
- **Richard Young** - Treasurer
- **Ann Williams** - Immediate Past Chair
- **Kate Williams** – Co-Editor Geomechanics News and YGP Representative
- **CY Chin** - Ex-officio Member
- **David Stewart** - Committee Member
- **Simon Woodward** - Committee Member
- **Paul Salter** – Co-Editor Geomechanics News

I look forward to our first committee meeting with this new group together in May.

IPENZ Forum

At the end of March I had the opportunity to attend the IPENZ Branch and Technical Group Forum 2009 held in Wellington. This was a good opportunity to get to grips with the inner workings of IPENZ and catch up with members of other Collaborating Technical Societies, for example the Structural Engineering Society New Zealand. While IPENZ is expecting some potential income challenges this year, the institute has committed to placing a greater emphasis on outreach.

Philip is a Senior Geotechnical Engineer at Golder Associates based in Nelson. Philip graduated more than 20 years ago from the University of Natal, Durban, with a BSc in Civil Engineering. After working in the field of civil/geotechnical engineering for a few years, Philip returned to study at the University of California, Davis leaving with a MSc in Geotechnical Engineering and a focus on earthquake engineering. Philip is a Member of the Institution of Professional Engineers New Zealand and a Chartered Professional Engineer (CPEng). He has been involved in the design and construction of major infrastructure projects in New Zealand, California, Hong Kong and Southern Africa. Philip's project experience ranges from planning and managing subsurface exploration programs including onshore/offshore drilling and sampling and in situ testing through site evaluation to the design of deep and shallow foundations, major earthworks, fill embankments and retaining structures. Philip is particularly proud of his contribution to the San Francisco-Oakland Bay Bridge East Span Seismic Safety Project and construction of Pier 400 Container Wharf at the Port of Los Angeles. When not working Philip spends most of his time chasing after his seven year old daughter.

NZGS Events

David Burns and David Stewart are working with Warwick Prebble to perpare a mapping course. C Y Chin has asked Prof Wong Kai Sin to present a Short Course on "Deep Excavations" and an evening lecture on the Singapore Nicoll Highway Collapse.

My Vision

I am keen to continue with the goals and objectives that Ann set in motion, and I look forward to working with the Management Committee to meet them.

Philip Robins
Chairman, NZGS
Email: probins@golder.co.nz

EDITORIAL

THIS ISSUE WE welcome aboard Paul Salter as co-editor to assist our editorial team with the enduring tasks of creating an informative, technical, news worthy and pictorial newsletter. With Paul's fresh new ideas and diverse contacts network this issue has ballooned in size with contributions being made up to the very last minute. Thank you Paul for the great start.

We congratulate Misko Cubrinovski with the successful achievement of producing a paper commendable of being selected for the prestigious Geomechanics Award for 2008. A full reference copy of the paper "Pseudo-static analysis of piles subjected to lateral spreading" has been reprinted in this issue for the benefit of all our members.

In "Member's Past Contributions", we include a paper originally presented at the 6th International Symposium on Landslides held in Christchurch in 1992. While specifically on schist landslides in the Cromwell Gorge, the paper also summarizes engineering geological knowledge of the local Otago Schist terrain accumulated by various workers from investigation to completion of Clyde Dam. The paper should be of interest to members now involved in wind-power projects in the region, and renewed consideration of further hydro-power development on Clutha River. It also provides an example of the "Total Geological History" approach to engineering geological modelling practice.

We have a mix of project news in this newsletter with some large scale projects nearing completion throughout New Zealand that display some amazing geotechnical and geographical issues that required overcoming to be successful. The company profiles showcase a bit of the 'old' and 'new', with Tonkin & Taylor Ltd celebrating 50 years in the consultancy service, while RDCL Ltd are relatively new on the scene since 2006.

We would like to thank all the contributing authors for their efforts and look forward to receiving newsletter content from other members for future issues.

Kate Williams
Co-editor: kwilliams@tonkin.co.nz

Paul is an Engineering Geologist and Hydrogeologist at URS Auckland. He graduated from Auckland University and has worked on projects in NZ, Australia, Asia, and the US. He currently leads the Auckland URS Geo-Engineering Team.

PREVIOUSLY, WHEN THE NZ Geomechanics News landed on my desk I had little appreciation of the effort that went into it. Now I understand. Kate, Karryn and Amanda are organised, efficient and creative - they do a terrific job.

It's good to see an article on groundwater in this issue. This is one of my areas of interest. Philip Kelsey reports on the subsidence suffered in Shanghai due to aquifer overpumping. A similar scenario has occurred in parts of Southern California, where I worked for several years, and it's a reminder that predictable geotechnical problems are often replayed around the world.

The current global economic decline is another man-made, many would say predictable, problem that is impacting our industry. The paper on p78 answers an interesting question: What kind of footing system would be needed for a 1 km tall building? Such a tower was under construction, in Dubai, but work was now halted due to the economic slump.

Finally, John Carter's ISSMGE report includes an urgent call for nominees from within the NZGS to assist with editing the ISSMGE Bulletin. Please respond to John if you have an interest in this.

Paul Salter
Co-editor: paul_salter@urscorp.com

EDITORIAL TEAM

Amanda Blakey
Advertising Manager
nzgeotechnicalsociety@xtra.co.nz

Karryn Muschamp
Graphic design

THE SECRETARY'S NEWS

It feels like there has been a constant stream of emails and letters to the Society over the last few months. I have been very busy with the following:

- Sending 6 New Zealand papers to the ICSMGE conference in Alexandria, Egypt in September this year;
- Preparing a NZGS application for the ICSMGE Young Member Award, also to be presented at Alexandria, Egypt;
- Assisting our two YGP winners with the submission of their papers for the 4iYGP conference which precedes the ICSMGE in Egypt;
- Welcoming 31 new members with a copy of our magazine and information on the Society, Branch activities and services;
- Assisting the Management Committee with their recent assessment and presentation of the 2008 Geomechanics Award;
- Preparing for and attending the 2009 AGM (Awfully Good Meeting) in Auckland;
- Meeting the Auckland Branch Coordinators and members for the first time;
- Coordinating advertisements, and cajoling reports, for this issue of Geomechanics News;
- Welcoming a new Chairman and Committee for 2009; and
- Distributing proceedings and papers, and generally assisting members as required.

Where relevant, we have been trying to keep the Society's webpage as up to date as possible with Branch information – especially upcoming programmed events and presentations. Please continue to let me know of any events/conferences that you would like circulated to members via the website or an email. It is also a good idea to keep me informed of any change of contact details, emails and addresses.

New Members

Membership is now at 703 members and it is a pleasure to welcome the following new members since November 2008:

STUDENTS
JJM Haskell.

MEMBERS
MD Trigger; RJ Heritage; K Summerhays; AJ Broadbent; DM Oosterbeek; ML Fox; JR Grindley; RP Shelton; WN Osborne; SK Karmacharya; NA Smith; KM Johansson; HJ Bowen; C McPherson; JA Wedgwood; DJ Veale; B Simms; TU Ganiron Jr; CB McCurrach; MEJ Herd; CA Helm; A Campbell; DM Maxwell; AC Fox; GM Martin; E Duke; AJ Chapman; MA Clark; MT Fitzmaurice and the University of Auckland Library.

Please do contact me for any assistance you might require or any queries you might have.

Amanda Blakey
Management Secretary
nzgeotechnicalsociety@xtra.co.nz

EMAILS TO THE EDITOR

From: Phil Glassey
Sent: Monday, 8 December 2008 12:05 p.m.
Subject: Geomechanics news

The editor
Geomechanics News

Dear Kate

Having skimmed through Geomechanics News that I recieved in the mail this morning, I would just like to pass on my congratulations to you. It is looking fabulous and informative all at the same time. Keep up the good work

Phil Glassey

From: Philip Robins
Sent: Tuesday, 9 December 2008 9:37 a.m
Subject: 50th Edition

Hello Kate,
Great job with the magazine!
Well done.
Philip

Editors Comment - A much deserved thanks goes to the whole Editorial Team (Amanda and Karryn) for putting the 50th Anniversary Newsletters together.

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 *NZ Geomechanics News*
- descriptions of geotechnical projects of special interest

General articles for publication may include:

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

Submission of text material in Microsoft Word is encouraged, particularly via email to the Editor or on CD. We can receive and handle file types in most formats. 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. Diagrams and photos should be supplied with the article, but also saved seperately as 300 dpi JPGs. Articles need to be set up so that they can be reproduced in black and white, as colour is limited.

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. Letters to the Editor about articles and papers submitted by members will be forwarded to the contributing member for a right of reply.

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.

INTERNATIONAL SOCIETY REPORTS

ISSMGE Australiasia VP Report: April 2009

INTRODUCTION
This report contains a summary of the highlights of recent ISSMGE-related activities.

ISSMGE BOARD MEETING
The most recent meetings of the Board of ISSMGE were held at the Green Path Hotel, Bangalore, India on 19th December 2008 and in Orlando, Florida on 14 March 2009. The following is a summary of the matters discussed and decisions taken at both of these meetings.

Membership
There was little change in membership relative to the report presented at the St Petersburg meeting.

The President and Vice President for Asia were scheduled to make a visit in January to Cambodia, Laos, and Myanmar to discuss ISSMGE involvement of those countries.

In Bangalore, the Board discussed possible amendments to the Statues to state that Member Societies should give evidence of activity as well as pay their subscription fees in order to remain as a member society of ISSMGE. While recognising that "activity" would be difficult to quantify, possible changes to the Statutes were considered at the subsequent Board Meeting in Orlando, with a view for the matter to be included on the Alexandria Council Meeting Agenda. The final wording of these proposed changes is yet to be confirmed.

ISSMGE subscription fees are linked to the Swiss Franc and as a consequence of the global economic situation and its impact on currency exchange rates there were significant changes to the subscription fees that were invoiced in January. While the fees are traditionally calculated in Swiss Francs they are normally billed and paid in the equivalent British pounds. The likely consequences for NZGS (and Australia) are adverse, with a rise of almost 50% in the total subscription after recent currency movements are taken into account. It is noted that only 2.5% allowance has been made for inflation in these new fees (as approved by Council in Brisbane in 2007).

There are now fewer corporate members than had been envisaged in the ISSMGE budget for the current period. The Board acknowledged that it would be difficult to attract more corporate sponsors in the current economic climate.

Technical Committees
The President produced a report which indicates various activities of the Technical Committees. Most were very active. The main issue concerned TC37 – Interactive Geotechnical Design, for which it is proving difficult to find a Chairman.

Regional Reports
Reports were received from Vice Presidents for Africa, Asia, Australasia, Europe, and South America. A brief summary of major issues and events is provided as follows.

Africa
The Vice-President for Africa, Mounir Bouassida, reported that he understood that Libya might apply for membership of ISSMGE. He also informed the Board that the next Vice President for the Africa Region would be Dr Samuel Ejezie from Nigeria.

Asia
The Vice-President for Asia, Madhira Madhav, reported that the 10th International Symposium on Landslides and Engineered Slopes at X'ian (China) had been very successful and had included an excellent record of photographs of various landslides. On the casting vote of the President it was decided that Dr Zuyu Chen of China would be the incoming Vice-President for Asia.

Australasia
As Vice-President for Australasia, I reported that the New Zealand Geotechnical Society had recently celebrated its 50th Anniversary and that the 8th Young Geotechnical Professionals Conference had been held recently in Wellington and had been very successful. A report on the latter should appear in the next issue of the ISSMGE Bulletin. I also reported that Professor Michael Davies (New Zealand) would be the next Vice-President for the Australasian Region.

Europe
The Vice-President for Europe, Roger Frank, reported that the 19th Young Geotechnical Engineers Conference in Györ, Hungary, had been very well organised but the attendance by some European Member Societies was disappointing as only 22 of the 35 Member Societies in the region were represented. He also noted that it had been recently announced that Ivan Vanicek had been elected as the next Vice-President for the European Region.

North America
It was reported that the next Vice-President for North America would be Professor Miguel Romo from Mexico.

South America

The Vice-President, Waldemar Hachich, raised the issue of the forthcoming International Conference on Geosynthetics in Guarujá and whether it could be organised under the auspices of ISSMGE. The Secretary General confirmed that this came under the agreement between ISSMGE and IGS and the listing on the website would be amended to reflect this.

Revision of ISSMGE Voting Procedures

After much discussion the Board finally concluded that ISSMGE should adopt a preferential voting system whereby for matters involving more than two options Member Societies should indicate on a ballot paper their order of preference. The Board was asked to approve appropriate wording of motions to be considered by the next Council Meeting in Alexandria.

Federation of International Geo-engineering Societies

There had been three meetings of the Board of FedIGS in 2008 and, following the recent meeting in Madrid in September, the emphasis was now on establishing a Liaison Committee. The President also reported that progress was being made with planning for a FedIGS conference in 2012 in Hong Kong. He also reported that the JTCs were taking shape, all now being under the direction of William Van Impe as President of FedIGS, and that the operational Secretariat was being maintained in Ghent.

Task Force: Role and Format of International Conferences

The Task Force was asked to review the procedure for allocating papers to the International Conference and to consider whether there should be a minimum allocation of papers to the conference, which at present has been set arbitrarily as 2 papers. The Task Force was also asked to consider mechanisms by which quality of papers could be maintained by suitable refereeing procedures. A report and proposal will be considered at the next Board meeting in Orlando, with a view to submitting them to the next Council meeting in Alexandria.

Task Force: Communications Information, and Information Technologies

A Knowledge Network section has recently been created on the ISSMGE website. The Board has been trying to establish a group of young members who could take responsibility for producing/editing the ISSMGE Bulletin. Nominations of young members from North America, Europe and Australasia are still required. As Vice-President for Australasia, *I am urgently seeking suitable nominees from NZGS and AGS members to fulfil this role*. I hope that collectively these nominees will be able to provide regular copy from the region for the ISSMGE Bulletin. My expectation is that as a minimum they could simply extract relevant stories and articles from the newsletters of our two societies, but sourcing additional original material would also be very welcome. Providing a younger perspective would be very refreshing.

Task Force on International Conference

Recent changes in how subscription fees were calculated meant that the Kerisel formula for allocating pages to Member Societies in the proceedings of the International Conference of ISSMGE may need to be modified. It was agreed by the Board that page allocation should reflect more closely the number of members in a Member Society rather than the magnitude of financial contribution. This matter will be on the agenda for decision at the next Council meeting in Alexandria.

ISSMGE – 75 Years Celebration

The President reminded the Board that he would like to celebrate 75 Years of the International Society by holding special sessions at a number of conferences in the anniversary year. He envisaged the first session to be at the 6ICEG Conference in New Delhi, November 2010, and that there would be other special sessions at each of the ISSMGE regional conferences in 2011. The sessions would be of approximately one hour duration at which the speakers would be either the President (for the 6ICEG) or regional Vice-Presidents (for the regional CSMGE), a senior figure nominated by the President or regional Vice-President, and a young engineer selected from a recent appropriate YGEC event. The intention was that this session would have three views expressed on the past, present and future of the International Society.

It was agreed that Vice Presidents should enquire of their Member Societies how they would like to celebrate the 75th Anniversary at the time of their regional conference. *I eagerly await any suggestion from NZGS and AGS members in relation to this matter*.

Dates of Next Board Meeting

Alexandria – 3rd October 2009

17 ICSMGE - ALEXANDRIA

Members are reminded that the 17th International Conference of the International Society for Soil Mechanics and Geotechnical Engineering will be held in Alexandria, Egypt from 5-9 October 2009. The theme of the conference is "The Academia & Practice of Geotechnical Engineering". Both Australia and New Zealand are well represented in terms of the papers submitted to the Conference. Further details can be found at: http://www.2009icsmge-egypt.org/.

John Carter
ISSMGE VP Australasia

International Society for Soil Mechanics and Geotechnical Engineering

Société Internationale de Mécanique des Sols et de la Géotechnique

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Professor Michael Davies
Faculty of Engineering
University of Auckland
Private Bag 92019
Auckland
NEW ZEALAND

23rd February 2009

Dear Michael,

ISSMGE Vice-President for Australasia 2009 - 2013

This is to advise you formally that you have been elected as ISSMGE Vice-President for Australasia for the term 2009-2013. Your period of office will commence on 10th October 2009, immediately after the conclusion of the XVII ICSMGE in Alexandria. May I take this opportunity to offer you my personal congratulations and to wish you every success in carrying out the many important tasks this post entails.

Best regards.

Yours sincerely

Professor R N Taylor
Secretary General

cc. Prof. John Carter – ISSMGE Vice-President for Australasia
Prof. Pedro Sêco e Pinto – President ISSMGE
Secretaries, ISSMGE Member Societies for Australasia

ISSMGE Secretariat, City University, Northampton Square, London EC1V 0HB UK Tel: +44 20 7040 8154; Fax: +44 20 7040 8832; E-mail: secretariat@issmge.org

ISRM Australasia VP Report: April 2009

1 BOARD AND COUNCIL MEETING

On November 22nd and 23rd, the Society held its Annual Board and Council meetings in Tehran, Iran, in conjunction with the 5th Asian Rock Mechanics Symposium. In the Council meeting, 37 of the 49 National Groups were either present or represented. The Council was also attended by one Past President, the chairmen of the ISRM Commissions and the President of the ISSMGE. The following sub-sections summarise some of the discussions.

1.1 Membership

The ISRM now has 5498 individual members and 142 corporate members, belonging to 49 National Groups. This represents an increase of 15.5% in the number of individual members during the past 5 years, and the highest number of National Groups ever. The President welcomed Peru, which was recently admitted to the ISRM and was represented in the Council meeting.

1.2 Modernisation

In the Board and Council meetings, the President, Prof. John Hudson, explained the modernisation initiatives that are under way. During the past year, the following topics had been assigned to the respective Board members. In the Board meeting each member gave a presentation on their topic and indicated strategies for addressing the issues in each.

- Membership Numbers and Improving the Benefits to Members – Tony Meyers
- Website Strategy – Abdolhadi Ghazvinian
- Availability of Literature – Francois Malan
- Major Technical Issues – Derek Martin
- Content of ISRM International Symposia and Conferences – Xia-Ting Feng
- Prizes and Certificates – John A Hudson
- Lecture Tours and Educational Material – Claus Erichsen
- Strategy for Interaction with other Societies – Alvaro Gonzalez-Garcia
- Communication with Members – Nuno Grossmann, Luis Lamas and John A Hudson

1.3 Rocha Medal

The Rocha Medal is intended to stimulate young researchers in the field of Rock Mechanics. A bronze medal and a cash prize have been annually awarded since 1982 for an outstanding doctoral thesis selected by a Committee appointed for the purpose.

2009

The winner of the Rocha Medal for 2009 is Dr Li

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CALL FOR ABSTRACTS

CONFERENCE THEMES AND SUB-THEMES

1 Geohazards at the Leading Edge
- 1.1 Seismic Hazards
- 1.2 Volcanic Hazards
- 1.3 Gravitational Hazards
- 1.4 Climatic Hazards

2 Managing Geological Risk
- 2.1 Hazard and Risk
- 2.2 Disaster Risk Management
- 2.3 Living with Geohazards
- 2.4 Planning for Climate Change

3 Advances in Engineering Geology
- 3.1 Developments in Site Investigation
- 3.2 In the Laboratory
- 3.3 Mapping and Remote Sensing
- 3.4 Field Measurement
- 3.5 The Geological Model
- 3.6 Geodata Management

4 Applied Engineering Geology
- 4.1 The Mechanics of Rock
- 4.2 Underground
- 4.3 Filling with Earth
- 4.4 Supporting our Structures
- 4.5 When Water Meets Structures
- 4.6 Analysis in Engineering Geology

5 Evolving Engineering Geology
- 5.1 Engineering Geology in the Global Economy
- 5.2 A Resource Hungry World
- 5.3 Appropriate Technology in the Developing World
- 5.4 Geo-environmental Engineering
- 5.5 Sustainable Geotechnics
- 5.6 The Geotechnical Response to Global Warming
- 5.7 Litigation and Geotechnics
- 5.8 Ethics and Communication
- 5.9 Human Resourse Development

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KEY-NOTE SPEAKERS

DR SIMON LOEW (LÖW)
is Professor of Engineering Geology at the ETH Zurich. He has led several large interdisciplinary projects related to the final storage of nuclear and toxic wastes, large traffic tunnels (NEAT, AlpTransit) and natural hazards. He leads research in deep tunnelling (settlements above tunnels), hydro-mechanical processes, geological waste disposal and slope instability.

DR SERGIO MORA
Originally from Costa Rica, Dr Mora is an international leader in Disaster Risk Management, with a background in rock mechanics, dams and tunnelling. He draws his DRM experience from his work as Environmental, Natural Resources and Risk Management Specialist at the Inter American Development Bank and as consultant to the World Bank.

TIM SULLIVAN
is Adjunct Professor at the School of Geotechnical Engineering, University of New South Wales, Australia and a Director and Principal Consultant of Pells Sullivan Meynink Pty Ltd, the firm he established in 1993. He has particular interest in the fields of mine stability and design; landslides, and engineering geological/geotechnical models.

DR SUSUMU YASUDA
Originally from Hiroshima, Dr Yasuda is Professor of Civil and Environmental Engineering at Tokyo Denki University. His main research interest is in soil liquefaction during earthquakes and he has visited many countries to investigate the damage due to liquefaction in the post-disaster period. Dr Yasuda chairs the Asian TC No.3 on Geotechnology formed by the ISSMGE.

IAEG CONGRESS 2010

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11th IAEG Congress Auckland, New Zealand 2010

Active, Auckland, Aotearoa

Hosted by the New Zealand Geotechnical Society **www.iaeg2010.com**

Gang from China for the thesis titled "Experimental and Numerical Study for Stress Measurements by Jack Fracturing and Estimation of Stress Distribution in Rock Masses". The study was carried out at Yamagushi University in Japan. He will receive the award at the 2009 ISRM International Symposium in Hong Kong.

2010

Eight theses have been presented for consideration for the 2010 Rocha Medal. In accordance with a new policy, 2 of these theses were the runners-up for the 2009 medal. Australia has entered the thesis by Dr Rhett Hassel from Curtin University "Corrosion of Rock Reinforcement in Underground Excavations". The winner of the award will be announced in Hong Kong in May.

2011

Nominations for the 2011 medal are open until 31st December 2009.

1.4 ISRM Digital Library

A digital library containing all papers and keynote lectures presented at the 10 ISRM Congresses and 30 ISRM International Symposia is being created. All the 45,000 pages from 7,500 papers have now been scanned. Starting early in 2009, the papers will be available for members to download from the website for a very low cost.

1.5 Nominations for ISRM President for 2011-2015

The following persons have nominated as candidates for the ISRM Presidency:

- Dr Claus Erichsen from Germany;
- Prof. Xia-Ting Feng from China; and
- Dr Francois Heuze from the USA.

The candidates have prepared short videos and documents supporting their nominations which are available from the ISRM website.

1.6 Commissions

The reports produced by the ISRM Commissions are the most visible products of the Society. The Commissions currently active are:

- Rock Spalling
- Preservation of Ancient Sites
- Testing Methods
- Rock Dynamics
- Education
- Application of Geophysics to Rock Engineering
- Rock Engineering Design Methodology
- Waste Disposal (including radioactive waste)

The final report of the Commission on Mine Closure has been completed and is available for members to download online.

Several countries have a legacy of old, closed mines that now pose problems and, in extreme cases, danger to the public. The commission was set up to study these problems in an international context and to propose a uniform method of handling them. The work concentrated on the physical rock related aspects of mine closure and did not include the social aspects.

A significant problem relates to time dependant failure of old pillars. Another concerns subsidence of the overburden above underground developments. Subsidence can be severe, with the unexpected and sudden appearance of sinkholes on the surface potentially occurring over large areas over long periods of time. In the report, differentiation is made between gradual subsidence and sudden events.

The report describes the mine closure situation in several countries, including ones not commonly associated with problems arising from the failure of abandoned mines like Japan and Korea. It is a reference work that describes different methods of handling the problem in different countries, including the legislative aspects.

The report recommends a comprehensive risk management approach to handle there problems. Depending on the nature of the expected effects, different reactions are identified, ranging from doing nothing other than dealing with any minor effects as they appear, to back filling old mines or, in extreme cases, evacuation of villages. Direct and indirect monitoring methods are described, as are technical methods for stabilizing old workings and protecting the public against the reopening of old mine shafts.

1.7 Website

The website is the main source of information about the Society and most benefits are offered to the members in a password protected members' area. The Secretary General informed the Council that the number of people accessing the website is continuously increasing. It receives each month over 250,000 hits and 12,000 visits from 100 countries. The home page of the website was recently modified, and there is now easier access to information.

1.8 Cooperation with other Societies

- The President reported on the initiatives during last year regarding the Federation of International Geo-engineering Societies – FedIGS, joining the IAEG, the ISRM and the ISSMGE. FedIGS formally started in January with the election of the President, Prof. William van Impe. Eight Joint Technical Committees (JTCs) are now operating and a committee to liaise with industry is being formed. The ISSMGE President presented a summary of the developments of FedIGS.
- An ITA-ISRM Joint Action Group on "Site Investigation Strategy for Rock Tunnels" was created and will prepare a guidance document and that 3 ISRM members were appointed for ITA Working Groups.

2 ISRM INTERNATIONAL SYMPOSIUM: ARMS5

The Organising Committee and the Iranian Society for Rock Mechanics (IRSRM) selected Iran's capital city, Tehran, as the venue for the ISRM International Symposium due to it having excellent "leading edge" meeting facilities and its close proximity to significant historical and engineering sites.

Approximately 57% of the Iranian territory is covered by mountains and most of the major projects are being constructed on or within rock masses. The number of projects being constructed within river valleys is exceeded only by those in China. There are major developments in the mining and petroleum industries, hydroelectric power generation and road, rail and water conveyancing infrastructure.

The symposium was preceded by two, 2-day, short courses:

- Rock Fracture Geometry Network Modelling in 3-D to Study the Mechanical and Hydraulic Behaviour of Rock Masses presented by Prof. Pinnaduwa Kulatilake from The University of Arizona in Tucson, USA.
- Petroleum Geomechanics in the Value Chain presented by Prof. Maurice Dusseault from the University of Waterloo in Ontario, Canada.

The symposium, titled "New Horizons in Rock Mechanics: Developments and Applications" was attended by approximately 250 practitioners, researchers, academics, contractors and students from 35 countries.

14 keynote speakers gave the following addresses:

John Hudson	The future for rock mechanics and the ISRM.
Chung-in Lee, Yeon-Jun Park and Kwang-Yeom Kim	Prediction of fault zones ahead of a tunnel face using 3-D displacement monitoring.
Nick Barton	Important aspects of petroleum reservoir and crustal permeability and strength at several kilometres depth.
T Ramamurthy	Joint factor concept in solving rock engineering problems.
Xia-Ting Feng	Recent developments and applications of intelligent rock mechanics (i.e. expert systems).
F Hassani, P. Radziazewski and J. Ouellet	Microwave assisted drilling and its influence on rock breakage: a review.
H. Stille and M. Holmberg	Observational method in rock engineering.
Z.T. Bieniawski	Reflections on new horizons in rock mechanics design: theory, education and practice.
S.A.B. da Fontoura	Geotechnical behaviour of sedimentary argillaceous rocks.
O. Aydan	New directions of rock mechanics and rock engineering, geomechanics and geo-engineering.
M.S.Bruno	Slurry fracture injection of petroleum and sanitation wastes.
Claus Erichsen	Rock mechanics as a basis for successful rock engineering.
Pinnaduwa Kulatilake	Recent developments on rock joint roughness and rock mass strength and deformability.
J.Zhao, Y.K.Zhou and G.W. Ma	Rock failure, wave propagation and tunnel stability under dynamic loads.

A presentation was also given by the recipient of the 2008 Rocha Medal, Dr Zhengzhao Liang from China, on the thesis titled "Three Dimensional Numerical Modelling of Rock Failure Process". The work was carried out at the Dalian University of Technology in China.

A total of 124 papers were divided into the following themes and presented in parallel sessions:

- Rock characterisation and site investigation.
- Ground improvement and rock slope stability.
- Underground stability.
- Rock dynamics and foundations on rock.
- Instrumentation, monitoring and numerical analysis in geomechanics.
- Earth resources and coupled processors.
- Building, ornamental and ancient stones and structures.
- New developments, special topics and applications in Rock Mechanics.
- The 1-day technical tour visited the Siah-Bisheh hydroelectric scheme which is being constructed on the Chalus River, 125 kms to the north of Tehran in the Mazandaran province. It involves an 85 m high x 390 m wide upstream dam and a 104 m high x 330 m wide downstream dam. Both dams are of concrete faced, rockfill construction and are being founded and abutted within carbonaceous shales. Water will be pumped for 8 hours from the 3.6 million m^3 capacity lower dam to the 3.6 million m^3 capacity

upper dam in periods of low power demand. In periods of high demand, the water will flow along two 60˚ inclined tunnels from the upper to the lower dam, emptying the former in 4 hours. The power plant and transformer caverns are up to 182 m long x 22 m wide x 42 m high. The presence of several extensive faults intersecting the earthquake prone site presented challenges for the designers to select appropriate reinforcement and instrumentation. When completed, the project will generate 1000 MW of power from four Siemens 250 MW turbines. The pumped storage characteristic is being used to balance demand in the national electricity grid during peak and non-peak hours.

- The 5-day cultural tour visited 19 ancient and/or historical sites around Shiraz, Isfahan and Kashan, many of which are listed as UNESCO world heritage sites. Although rainfall is not in general high in these areas, over the centuries the limestones within, upon or from which many of the internationally significant sites were constructed have been weathering. Where possible, efforts are being made to preserve the sites although funding constraints make doing so a challenge. The goal of the ISRM Technical Commission on Preservation of Ancient Sites is investigating techniques for preserving sites such as these.

As for all ISRM supported events, this event provided the opportunity for attendees to be exposed to "leading edge" practises and advances in Rock Mechanics. It also provided the opportunity to hear from and mix with those whose past contributions formed the foundation of the discipline and from those whose current work is advancing it.

The final words must be ones of thanks and praise to the event organisers. Their staggering enthusiasm when carrying out the huge number of often onerous tasks involved in hosting such an event and their extraordinary generosity and kindness to all attendees made this event an unforgettable experience.

3 UPCOMING ISRM SPONSORED EVENTS

- **17-19 June 2009 Lausanne Switzerland**-Workshop of the ISRM Commission on Rock Dynamics Organised by the ISRM Commission on Rock Dynamics, together with the EPFL Rock Mechanics Laboratory. Dr Zhou Yingxin, Chairman of the Commission, will be the Workshop Co-ordinator.
- **29-31 October 2009 Dubrovnik Croatia** – EUROCK'2009 - Rock Engineering in Difficult Ground Conditions - Soft Rocks and Karst, an ISRM-Sponsored Regional Symposium.
- **23-27 October 2010 6th Asian Rock Mechanics Symposium** - Advances in Rock Engineering, the 2010 ISRM-sponsored International Symposium.

4 ISRM BLUE BOOK

"The Complete ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 1974 to 2006" was edited by Professors Resat Ulusay and John Hudson. The hard cover book was compiled and published by the ISRM Turkish National Group and can be purchased from them and also from the ISRM Secretariat via the ISRM website. An Introduction to the "Blue Book" can be downloaded from the website.

5 MEMBER COMMUNICATIONS

If you are an ISRM member, but have never received any emails from me, it's probably because I have not been notified that you are a member or emails to you have bounced back as I don't have your correct email address. In either case, email me your contact details at ISRM@ rocktest.com.au and I'll put you in my address book.

Tony Meyers
ISRM VP Australasia

IAEG Australasia VP Report: April 2009

Fred Baynes continues to be very busy in his roles as IAEG President and member of the FedIGS executive (Federation of International Geo-engineering Societies). In keeping with his aim of getting to know engineering geologists around the world and contributing to the knowledge of the industry Fred spent November last year overseas. In South Africa he gave the keynote address at the South African Problem Soils Conference and the inaugural Tony Brink Memorial Lecture at various venues around the country. He then went on to Brazil where he gave talks in Sao Paolo and Rio de Janeiro and the keynote address at the 12th Brazilian Congress on Engineering Geology. Fred will be repeating his keynote address ("Application of the total engineering geology methodology") at several places in Australia this year. NZGS may wish to consider inviting Fred to give similar talks in New Zealand.

Fred attended an IAEG management meeting in Paris on 24 February 2009. At the meeting there was considerable discussion of the IAEG website, regarding ways of improving the functionality and content and what the members wanted from the site. It was resolved to improve the website. The products of the Commissions were identified as a key input to the websites and all agreed to try and upgrade this part of the website content. The front page will be changed with a monthly highlight including photos. All strategies for improving the website content should be tried.

Progress with the despatch of the Bulletin appears to be acceptable, but remains dependent upon the National Groups providing payments and contact addresses for members who subscribe to the bulletin in a timely fashion. The newly available access to Springer on Line was discussed and it was agreed that every effort would be made to distribute the access rights to members who subscribe to the Bulletin as soon as possible. Possible changes to Bulletin editorial policy with regard to abstracts in different languages were discussed but remained unresolved.

The changeover in Secretariat is largely complete and appears to have been carried out relatively smoothly.

The current financial status of IAEG remains stable with about 300,000 Euro in assets at the end of 2008 despite a continuing gradual reduction in the number of members. Financial planning for the next 5 years was discussed and it was agreed that different financial scenarios would have to be worked through and discussed at the Executive and Council meetings in Chengdu (in September this year). It was agreed that because of the current global financial problems it would be best to try and avoid any increases in fees.

The progress of IAEG 2010 was briefly discussed at a separate meeting in Paris. It was understood to be progressing well and that the implications of the global financial problems had been considered by the organizing committee and that it was felt that the Congress could still be successfully managed (see separate report by Ann Williams).

Current items on the IAEG website include information about the new setup for the Richard Wolters Prize, a call for nominations for the next Hans Cloos Medal and information about the Bulletin and Newsletter. There is also a call for nominations for national groups to host the 12th International Congress in 2014.

Fred Baynes also attended a FedIGS Board meeting in Cairo on 28 February 2009. At the meeting there was discussion of the following topics:

- The financial status of FedIGS. For the first two years the sister societies have funded FedIGS. From the end of this year income is to come from the sponsors (mainly from industry) who form the Liaison Committee (5,000 Euro each). The budget for 2009 was also discussed.
- The FedIGS mission statement, which essentially is to:
  1. Improve the interaction and co-operation between the sister societies – ISRM, ISSMGE, and IAEG.
  2. Improve the interaction and co-operation between FedIGS and both industry and society in general.
- The strategies by which the mission may be achieved. The main strategies relate to the functioning of the Joint Technical Committees and the Liaison Committee and so there was a lot of discussion as to the progress of these committees.
- The reported activities of the JTCs were reviewed and were found to be rather limited. Some changes were made to the existing JTCs and there was discussion of some possible new JTCs.
- Additional members of the Liaison Committee will be sought.
- The website which is now operational but requires content.
- A first FedIGS Conference which is planned for 2012 in Hong Kong.

The next FedIGS Board meeting is expected to be in Rotterdam in November 2009.

Alan Moon
IAEG VP Australasia

BOOK REVIEW

Cores and Core Logging for Geoscientists – Graham Blackbourn

An invaluable companion for the geoscientist

In the last issue (No 76) of NZ Geomechanics News, the "news release" of a new edition of this book mentioned that although it is "set to a great extent in the field of petroloeum geology, the book will also be of interest to a wide circle of geoscientists working in economic, mining, and geotechnical disciplines." Given its petroleum sector origin I set about the review with a fair amount of scepticism that the book would turn out to be of much use to the engineering geological practitioner. I was wrong! In addition, it is unusually well written as text books go, the plain english making it a pleasure to read about what might otherwise normally be considered to be a rather dry subject.

Topics covered by chapter are:

- drilling and coring methods
- core handling
- logging
- analysis and testing
- interpretation and preparation of final logs
- core preservation and storage

There are appendices on:

- log symbols and abbreviations
- equipment required for site operations and logging
- core barrel sizes

A bibliography is also provided that includes standard references from the geotechnical sector.

The book's stated target is geologists whose previous experience is only with outcrop studies, who may "rush in under the impression that core should be treated as though it is merely one long thin outcrop of rock". Because it starts from this premise, everything is introduced assuming no prior knowledge of both the drilling and core handling description processes. It is thus ideally suited to the graduate embarking on a geological career in industry.

The chapter on drilling and coring methods provides an excellent overview, including some interesting historical information that the first coring device capable of recovering intact samples was developed in the 1920s in the USA, and that the basic principles have not changed since then. The various factors that can lead to core damage as a result of drilling process (e.g. induced "slickensides") are clearly explained.

As might be expected, the chapter on core logging is the main part of the book, noting early on that "the secret of creating a good log is to be careful, to be systematic, and to work with no undue haste". The need to distinguish "description" and "interpretation" is emphasised, which perhaps is something that is not always recognised in geotechnical practice. The layout of the log sheet is sensibly considered to be dependent upon end use and other variables rather than being prescriptive in approach, but it is noted how loggers in industry have adopted preferred formats, including engineering geologists. The merits of recording information by alternative means of graphic symbols, log plots, presence/absence indicators and written description are usefully explained.

In this chapter, useful mention is also made of the need for close communication with the driller, noting how "differing and sometimes conflicting objectives of driller and geologists (making holes and gathering data respectively) can sometimes lead to a breakdown". An excellent (UK) example is given of a completed engineering geological log.

In describing characterisation of core for engineering purposes, in addition to RQD, mention is made of a "stability index", being the sum of core loss, fracture frequency, and hardness, which I have not come across before (or forgotten), and which may be a potentially useful alternative method of describing rock quality in certain circumstances.

Given the narrow emphasis on mechanical properties in much of the technical literature on core logging for engineering geological purposes, this book is an excellent complement, enabling understanding of the broader aspects of core geology. While it should be required reading for all novice loggers in the geotechnical sector, those who wish to brush up their skills will also find it useful.

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NZGS BRANCH ACTIVITIES

Auckland Branch Activity Report

Our March programme was overloaded but certainly benefitted those who attended the lectures.

The 3 March event gave an opportunity to YGP Travel Award Winners 2008 to present their material to the general members. There were five winners from Auckland but one was unable to present due to work commitments in Australia. The remaining four who presented were Annette O'Leary & Paul McClean (Maunsell), Sian France (Beca) and Naotaka Kikkawa (University of Auckland). Congratulations to the YGP Travel Award Winners from Auckland.

5 March was a special lecture given by Professor Atkinson, Coffey Geotechnics/City University London. A special mention to Peter Bosselmann for presenting this opportunity to NZGS. Prof Atkinson gave a fascinating insight into Soil Mechanics with his title, "Where does Soil Mechanics come from?".

31 March was an important event as it combined the NZGS AGM and a lecture from Beca on "The New Lynn Rail Trench – Design of a Floating Box with a Train on top". Gavin Alexander, Lucy Coe and Sian France combined together and covered a tremendous amount of technical material within the hour. We look forward to the site visit to be arranged by Brian Perry in due course.

April was not selected for lectures as Easter and school holidays were in the way.

5 May was initially set for Student Prize but will be postponed to a later date as details are still being sorted out. Evan Giles, URS has kindly agreed to bring forward his original June slot to May. His talk covers Pike River, a sequel to his original talk in 2008.

26 May is confirmed for Rolly Orense, University of Auckland with the title "Lessons from Recent Geotechnical Disasters in Asia". At the time of writing this report, the talk has not taken place but it focuses on three large-scale disasters in Japan and Philippines.

The tentative programme for 2009 is shown in the NZGS website.

Auckland Branch now has two new (and younger) coordinators, Lucy Coe (Beca) and Ross Kendrick (Maunsell). Lucy and Ross will eventually be joint coordinators for Auckland Branch. Thank you both for helping out!

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Yan Chan is a Director at KGA Geotechnical based in Albany, Auckland. Yan graduated from Auckland University before working in UK and Malaysia, ultimately returning to NZ in 2000.

Ross Kendrick
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Ross is an Engineering Geologist with Maunsell AECOM Ltd based in Auckland. He grew up in Auckland and graduated from Auckland University with an MSc. He states, yes, I am an Aucklander through and through, and as some people may refer to us, a JAFA! Ross initially worked in London, then moved to Scotland, returning to New Zealand to work in Auckland. He has just spent the last year in Wagga Wagga, Australia working on the Northern Hume Alliance Project, located half way between Melbourne and Sydney.

Lucy Coe
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Lucy Coe is a Geotechnical Engineer with Beca Infrastructure Ltd. After graduating from the University of Canterbury, Lucy moved up to Auckland and as been here ever since. Lucy has undertaken geotechnical investigations, design and construction monitoring on major infrastructure projects such as CMJ, Upper Harbour Bridge, Northern Busway and New Lynn Rail Trench.

Bay of Plenty/Waikato Branch Activity Report

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Sally is an Engineering Geologist and director of Terrane Consultants Ltd, Tauranga. She studied geology in the UK, and gained her PhD in slope stability modelling before moving to New Zealand.

25 February – Tour of the Tauranga Harbour Crossing Project courtesy of T&T. Well attended. Interesting presentation by Robert Hillier initially, before splitting into groups to tour the construction site. Great to see just how much progress has been made. A special thanks to Dave Milner for facilitating.

Upcoming events:
Tuesday 5 May – Talk by Marianne O'Halloran on the Edgecumbe stopbanks. "Flood protection problems in a geologically active area"

I am now also working with Ken Read (based in Cambridge) who will hopefully be able to help me boost the number of talks and tours on the western side of the Kaimais. As always, if you have an idea for a topic or a site you think would make a great tour, please let us know and we'll see what we can do to arrange something. Similarly, if you see a talk or subject being presented elsewhere in the country that you would like to see brought to the Bay, let us know and we may be able to arrange a visit. Volunteers also always welcome!

Wellington Branch Activity Report

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David Stewart is a Senior Geotechnical Engineer / Engineering Geologist with Opus International Consultants in Wellington. David initially worked in site investigations in the UK, returning to NZ to work as an engineering geologist in the Otago area – initially with the Cromwell Gorge Landslides project, followed by GNS Dunedin and Macraes Gold Mine. After completing a BE he has spent the last 6 years based in Wellington.

We've had two meetings so far this year - YGP presentations in March and a site visit to Project West Wind in April. Details of the meeting are:

31st March. 2008 Young Geotechnical Professionals Conference Local Paper Presentations.

Andrew Kennedy and Beverley Curley gave a brief account of the very successful 2008 Australia NZ Young Geotechnical Professionals Conference held in Wellington late last year. Bev and Andrew were part of the conference organising committee.

The main event of the evening was presentations from two 'young geotechnical professionals' from the Wellington area, who presented the papers they gave at the conference. The talks covered different aspects of geotechnical projects in the Wellington area. The talks were:

Silverstream "Class A" Landfill: Lining Solutions for Leachate Control

By Carys Everett (Tonkin & Taylor)

Carys' very interesting talk included presentation of innovative ("no fines" concrete) lining methods for steep slopes, as well as geotechnical issues encountered during construction of the landfill lining. Cary was the recipient of one of the conference awards and as such the abstract of her talk was published in the last issue of Geomechanics News.

Bored Belled Piles within Wellington Alluvium

By Tim Croad (Tonkin & Taylor)

Tim provided a very useful coverage of piling experience in Central Wellington, mainly in alluvial materials - consisting of gravels and sands interbedded with layers of silt. He discussed the use of bored belled piles and also design and construction monitoring methodologies developed to address issues with weak silt beds that are found within the alluvial sequence.

The full papers are available in the proceedings of the YGP Conference.

Left: Meridian Energy's Project West Wind – site visit party Wellington Branch

14 April 2008. Site visit to Project West Wind;

West Wind is Meridian Energy's 62 turbine windfarm on the western outskirts of Wellington, which is currently under construction. The visit started with a brief presentation by David Stewart on the project geotechnical issues and the geotechnical assessment carried out prior to construction. We then jumped in a bus and travelled to site, with Jayne Hodgkinson (Opus' project geologist during the construction phase) as our tour guide. We saw turbine sites at various stages of construction - including turbine tower components and foundations; and were blown away (excuse the pun!) by the power and scale of a completed turbine when standing directly under the huge spinning blades. We visited the central part of the site, past the newly completed substation and roadcuts up to 20 m high, and down to the temporary wharf site at Oteranga Bay, on the way passing roadcuts exposing the K-Surface (ancient peneplain) gravels and a major fault. Many thanks to Jayne for organising the trip and Abseil Access Ltd for the refreshments to keep us going. An article by Jayne Hodgkinson on the project is presented elsewhere in this issue.

Upcoming Activities:

The provisional programme of the rest of the year is outlined below. The timing of some events may change, so the aim is to provide updates of the programme on our branch webpage.

Feedback from members on ideas for activities or offers to help with organising activities is much appreciated.

2009 Provisional Programme

MONTH	Wellington Branch Activities	Other Activities
MAY	**Ian McPherson** - Screw Piles at Daly Street	China Earthquake reconnaissance – NZSEE Roadshow
JUNE	**Opus** - Transmission Gully geotechnical investigations and assessment	
9 JULY	**Graham Hancox** – Abbotsford Landslide Revisited	
AUG	**Local Forum – on technical** issue (Structural / Geotech interaction?)	
SEPT	Talk from Tonkin & Taylor	
OCT	**Evan Giles** - Pike River Coal Tunnel	
26 NOV	**Prof Wong Kai Sin** – Singapore Highway collapse due to tunnel construction activities	NZGS - Deep Excavations Short Course 27 November - **Prof Wong Kai Sin**

Nelson Branch Activity Report

Jeff Bryant presented his Rockslide Dams talk on Tuesday 21 April at the Nelson Club. The talk was well attended by 25 local engineers and geologists who greatly appreciated Jeff's fascinating presentation relating to the recent North Young River, Shotover River and Beichuan rockslide events. Participants were enlightened on the mechanics of rockslide events, the potential modes of landslide dam failure and a semi-quantitative assessment of overflow channel erodibility and dam breaching.

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Tim Coote is an Engineering Geologist for Tonkin & Taylor in Nelson.

Canterbury Branch Activity Report

Earlier in the year I called a planning meeting with the geotechnical group leaders from the member organizations in the Canterbury Branch. There was a great turnout with plenty of offers and ideas for presentations that will comfortably see us through into 2010.

On 20th April Jeff Bryant (Geoconsulting Ltd, Queenstown) gave a thought-provoking presentation on rockslide dams including the North Young River rockslide event of September 2007, a rockslide in the lower Shotover Valley and devastating examples from China triggered by the May 2008 Beichuan earthquake. Jeff discussed the challenges encountered and overcome during his hazard assessment of the two NZ examples. There is great uncertainty in assessing the dam-break risk from the 75m high North Young River rockslide dam, which presented a "non-textbook" problem. With pressures from the public to reopen access up the valley Jeff, staff from the client organizations (DOC, ORC and QLDC) and members of the public kept vigil over the reservoir level at it rose to overtop the dam. Fines have been scoured from the veneer of rock debris forming the overspill channel, which is subject to period flood flows but the bed remains relatively stable. The public access tracks are open with the dam subject to on-going monitoring as part of a hazard management plan. The talk attracted a good turnout and there were plenty of questions and discussion. Thank you, Jeff.

Up coming evening meetings confirmed for the Canterbury Branch include:

19 May – Vic Park Tunnel, Auckland – presentation by Leah Bateman, SKM.

20 July – Sinkhole hazard assessment on the Kaikoura Peninsula – presentation by Ian McCahon, Geotech Consulting.

We look to hosting Prof Wong Kai Sin later in the year as part of his lecture tour on the recent collapse at the Singapore Nicoll Highway.

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Nick is a consulting Geotechnical Engineer who leads the geotechnical group of the Opus Christchurch office, and also oversees the Opus' Christchurch materials testing laboratory.

Otago Branch Activity Report

The Otago Branch had their first meeting under the new coordinators on 25th February. Simon Cox gave an informative talk on rock avalanches in the Mt. Cook region. Some nibbles beforehand, and a couple of friendly beers help lubricate discussion after Simon had finished. It was a good chance to get the ball rolling for Otago again and me and Markus got to meet some of the other members in the Dunedin area. We are hoping to have another talk by May or June with a couple of potential topics including, Gravity Surveys in Preliminary and Detailed Site Investigation, as well as Alluvial Fans in Otago.

Please let us know if you wish to give a talk or have ideas for site visits.

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Markus is a consulting Geotechnical Engineer with Opus International Consultants in Dunedin. Markus came to New Zealand from Germany in December 2006 and has extensive experience as a consulting engineer in a wide range of geotechnical projects. Markus is the Geotechnical Workgroup Leader in Dunedin and has been kept busy designing retaining walls and pile foundations, assessing rock slope instability and holding the reins for the Opus Dunedin Geotechnical Team since October 2007.

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Shane is an Engineering Geologist with Opus International Consultants in Dunedin. Shane came to New Zealand from Canada in January 2006 and has been working with the Opus Geotechnical Team since that time. Shane has specialisations in Hydrogeology and Contaminated Land Assessment however since coming to New Zealand has turned his hand to everything from foundations to slope stability investigations.

MEMBER'S PAST CONTRIBUTIONS

Reprinted from 6th International Symposium on Landslides 1992

Engineering Geology of Schist Landslides, Cromwell, New Zealand

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DSIR Geology & Geophysics, Dunedin, New Zealand

B.W. Riddolls
Riddolls Consultants Ltd, Queenstown, New Zealand

N.A. Crampton
Crampton Engineering Geological Ltd, Cromwell, New Zealand

M.R. Foley
Mark Foley and Associates, Nelson, New Zealand

ABSTRACT: Engineering geological work for the completion of the Clyde Power Project has mainly involved detailed assessment of the characteristics of landslide-affected schist terrain bordering the reservoir area in the Cromwell Gorge, immediately upstream from the Clyde Dam. Landslide development has been strongly influenced by various geological characteristics of the schist bedrock, with initial failure having been controlled to a large extent by foliation shears and low-angle faults, which commonly form persistent rock mass defects. Investigation of such defects in both bedrock and where they form slide surfaces was hampered by a high degree of geological variability. Geological modelling for assessment of the stability of reservoir slopes and landslide hazards has been aided by recognition of different categories of slope movement materials, distinguished on the basis of characteristics that reflect varying degrees of disruption and downslope displacement.

1 INTRODUCTION

The Clyde Power Project has involved the construction of a 102 m high concrete gravity dam near Clyde for Electricity Corporation of NZ Ltd. Upon full commissioning, it will impound a reservoir (Lake Dunstan) up to 62 m deep, extending through the Cromwell Gorge and upstream into the Upper Clutha Valley, and along the lower reaches of the Kawarau River. Through the Cromwell Gorge, nearly 40% of the shoreline will be bordered by large landslide areas (Figure 1), some of which currently exhibit creep movements (Gillon & Hancox, 1992).

This paper describes the principal engineering geological characteristics of schist terrain in the Cromwell Gorge, with particular reference to the nature and variability of geological materials and defects in relation to assessment of landslide extent and development. The results of this work were used in geological modelling for evaluation of the effects of lake-filling on stability, in hazard and risk assessments, and in determining the need for and design of remedial works.

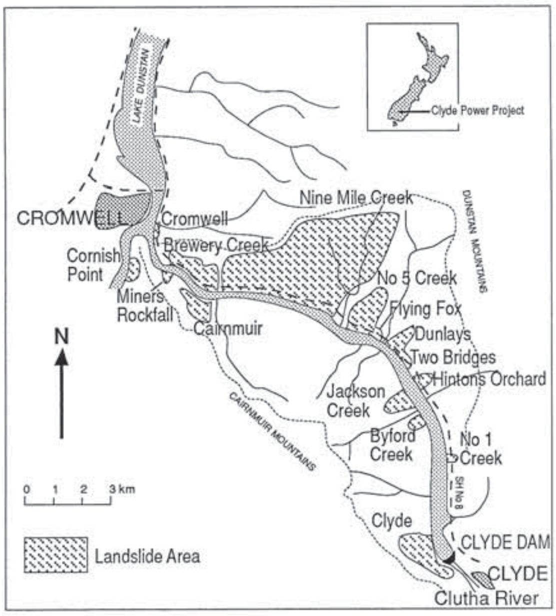

Figure 1. Location plan, Cromwell Gorge

Proc. 6th International Symposium on Landslides 1992, pp 2137-44

2 GEOLOGICAL SETTING

The complex nature of the schist terrain may be attributed to the influence of several geological factors, both regional and local in extent. Those relevant in the understanding of many engineering geological characteristics are described as follows.

2.1 *Geomorphology*

The Cromwell Gorge is a broadly V-shaped antecedent valley, incised up to 1400 m below the adjacent mountain crests. The valley is more-or-less symmetrical in cross-profile, with average slopes of 22 to 24 degrees. However, a marked asymmetry of the lower slopes is evident above a steeper inner gorge incised into bedrock.

2.2 *Stratigraphy*

The main geological features of the project area are outlined by Gillon & Hancox (1992) and more detailed descriptions are provided by Turnbull (1981, 1987). The regional stratigraphy is dominated by lithologically variable schist which forms the basement rock. Metamorphism occurred under high strain, low thermal gradient conditions during the Jurassic to early Cretaceous periods. Regional uplift and erosion during the Cretaceous-early Tertiary period resulted in the development of an extensive low relief surface (peneplain) over the entire region. During the mid Tertiary, terrestrial sediments were widely deposited but were subsequently stripped from rising mountain ranges.

Late Quaternary age (last 500,000 years) glacial moraines, interglacial lake sediments, outwash gravels and alluvial fans largely obscure the remaining Tertiary sediments and are well preserved throughout Central Otago, mainly in intermontane basins. Discontinuous remnants of fluvioglacial outwash terraces are preserved at several levels through the Cromwell Gorge.

2.3 *Schist lithology*

In the Cromwell Gorge area the dominant rock type is textural zone IV greyschist, with subordinate greenschist and metachert. The greyschist material grades lithologically between two main types (Turnbull 1981, 1987):

- mica-rich: pale greenish-grey to silvery grey, thinly and regularly laminated schist.
- quartz-rich: pale to mid-grey schist characterised by well developed quartz-albite segregation laminae.

The quartz-rich lithology predominates in the Gorge, except in the central part where, on the north side, mica-rich is more common. Planar mineral segregation laminae of variable thickness ($\leq$ 5 mm) are typical of both lithotypes. Locally, quartz-rich schist exhibits either contorted, frequently disrupted, segregation laminae, or has quartz occurring as rods or irregular masses.

2.4 *Structure*

Regionally, the structure of the gently-dipping Otago schists is dominated by successive generations of large recumbent folds. The Jurassic-early Cretaceous Rangitata Orogeny resulted in three recognisable phases of such folding in the schists, as well as considerable fault movement (Turnbull 1981, 1987). The subsequent Miocene-Recent Kaikoura Orogeny (which locally has occurred over the last 3-5 million years, and is still active) has further deformed the schists, by both folding and faulting, accompanied by regional uplift and down-warping to form extensive 'basin-and-range' topography (McSaveney et al 1992).

In the Cromwell Gorge schist foliation attitudes are very variable. Folding has warped the schist into a broad, open, westward-plunging fold (the Leaning Rock Antiform of Turnbull, 1981). Small (parasitic) drag folds occur adjacent to both low-angle faults and steeply dipping faults. A broad zone of steeply dipping to overturned schist has been recognised beneath the lower parts of several landslides along the left bank (eg. No. 5 Creek Slide, Nine Mile Downstream Slide), and is attributed to bedrock flow (Beetham, Smith et al 1992) rather than tectonic deformation.

Significant moderately steeply dipping faults include features along and parallel to the Cromwell Gorge (eg. in Brewery and Firewood Creeks), the Fish Creek Fault, the Mystery Fault Zone and the Cairnmuir Fault Zone (Gillon & Hancox, 1992). The latter, which crosses the valley about 1 km upstream from the dam, is known to have undergone late Quaternary displacement along parts of its length.

Two major low angle south-dipping faults have been recognised along the right bank of the gorge - a higher level fault (itself displaced) separating schists of distinctly different texture and a lower fault separating quartz-rich and mica-rich schist. Low angle faults have also been identified on the left bank during detailed investigation of the landslides (Gillon & Hancox, 1992).

3 ENGINEERING GEOLOGICAL DESCRIPTION OF SCHIST TERRAIN

3.1 *Bedrock*

In situ rock, widely affected by varying degrees of stress relief, has been extensively investigated throughout the Cromwell Gorge. This work has provided much useful information for assessment of the main engineering geological controls on slope evolution and landslide formation, and has formed an important part of the evaluation of reservoir slope stability.

3.1.1 *Rock Material Characteristics*

Unweathered greyschist is typically light to dark grey where fresh, variation in colour normally being related to relative proportions of quartz/albite (light) and mica (dark) minerals. The rock is generally only slightly to moderately weathered in outcrop and within a few metres of the surface or, at depth localised adjacent to defects .

Uniaxial compressive strength varies markedly in relation to lithology, mica-rich lithologies tending to be of lower strength than quartz-rich. The more mica-rich schists are typically in the weak to moderately strong category (q_u 10 to 50 MPa) while schists dominated by quartz are strong (q_u 30 to 90 MPa). Massive and poorly laminated schists, and greenschists are very strong (q_u 100 to >250 MPa).

All lithologies display marked strength anisotropy, with strength normal to foliation 3 to 8 times greater than the strength parallel or moderately oblique to foliation. Strength anisotropy appears to be greatest in quartz-rich schist with well developed quartz/albite and mica segregation laminae.

The following peak effective strength parameters parallel to foliation have been determined for quartz-rich schists: $c' = 2$ MPa; $\phi' = 23$-32 degrees.

3.1.2 *Rock Mass Defects*

Rock mass defects are well exposed in outcrop and excavations throughout the project area, especially in Clyde Dam foundations, (Paterson et al 1983, Hatton et al 1987, 1991). They include weak seams (crushed and sheared zones and gouge), shattered zones, and joints.

Crushed and sheared zones typically have low shear strengths, commonly controlled by thin gouge seams consisting of moderately to highly plastic clay with persistent slickensided surfaces. Gouge seams include a clay-size fraction of at least 30%, mixed layer mica-smectite being the dominant clay mineral. They range from very thin slivers (0.5mm) to seams several tens of millimetres thick.

Both tectonic and stress relief mechanisms of defect formation are recognised. Tectonic defects include faults, consisting of crushed zones up to several tens of metres thick with large offsets; typically with distinctive footwall, hanging wall and internal gouge seams. Crushed, shattered, and incipiently fractured rock may persist for many metres away from the principal zone of deformation.

Crushed and sheared zones with smaller apparent fault offsets, and joints of similar attitude, typically form defect sets sub-parallel to the mapped major faults. Detailed mapping of the Clyde Dam foundations (and other exposures throughout the gorge) has revealed a number of defect sets, generally arranged orthogonally, but locally displaying a high degree of variability and structural complexity which constrains accurate modelling and projection of persistent defects. For example, even after surface exposure, core drilling, and a network of sub foundation drives, the absolute geometry of one of the most extensively investigated crushed zones at the Clyde Dam site remained uncertain beyond the immediate vicinity where it was physically defined (R. Thomson, pers. comm.)

Defects sub-parallel to foliation are common in the schist. These features, called "foliation shears", include crushed and sheared zones, resulting from both flexure in response to folding and direct shear displacement in response to faulting. Generally they are between 50 mm and 2 m thick, with very thin gouge seams present as either thin slivers located immediately adjacent to the wall rock, or as narrow seams within crushed material. Foliation shears are typically spaced 5 to 50 m apart and locally splinter, warp, terminate against other defects, or die out.

Stress relief features, formed in response to removal of overlying materials and valley downcutting, are recognised mainly by opening of defects, resulting in deterioration of rock mass quality. Suitably oriented pre-existing defects have accommodated often large differential movements.

In addition, extensional joints, infill zones and conduits have developed as a result of gross rockmass dilation. Significant flexural deformation, attributed essentially to stress relief, is recognised in the weaker, mica-rich greyschists (Beetham, Smith et al, 1992). Crushed zones and joints along foliation are more closely spaced than in the undeformed rock, imparting a secondary fissility.

The field strength of typical defects is derived principally from two components: the residual friction angle and waviness effects. Residual strength is dependent on the presence of gouge seams, which are identified on surfaces with as little as 50 mm displacement, and their clay mineralogy. Typical values range from 8 to 14 degrees for samples of highly plastic clay. Waviness is almost always present and, depending on whether previous displacements have produced thick or thin crushed zones (containing the gouge), and on the nature of stress distributions during displacement, may contribute between 10 and 50% of the field strength (G. Salt, pers. comm.). Shear strengths of joints have not been evaluated in detail because most sets are steeply dipping and act as release surfaces rather than surfaces of sliding.

3.2 *Landslide masses*

3.2.1 *Slide materials*

Extensive deposits of schist-derived bouldery debris, and displaced schist masses have been mapped within the delimited slide areas shown in Figure 1. Their surface extent is often poorly defined (Figure 2) and well-developed scarps are rare, mainly because of the age of the features (Section 4.3). They also tend to be obscured by erosion and the accumulation of surficial materials, such as loess or colluvium. However the surface morphology of the schist debris is mostly hummocky, indicating that these deposits have undergone significant movement.

Schist debris is typically greyish-brown, and up to 80 m thick consisting of sub-angular blocks up to 20 m across, mostly randomly oriented in an unconsolidated, well graded, finer-grained matrix. Beneath the debris, defects commonly exhibit varying degrees of dilation or mechanical disruption resulting from slope movements along low-angle weak seams (eg. foliation shears, faults). These dilated rock mass conditions have been recorded both from direct investigations, and indicated from seismic refraction and reflection surveys (Bryant et al 1992).

Figure 2 Typical subdued landslide topography, Cromwell Gorge

Both within and at the base of the landslide masses, crushed zones have been proved in many surface excavations, tunnels, shafts and cored drillholes. These zones clearly comprise defects along which sliding has taken place, and contain either moderate to highly plastic clay gouge seams, derived from original tectonic crushed zones, or lower plasticity gouges thought to be due solely to gravitation processes.

The effective field strength of these defects has been determined using the resistance envelope method. Resultant frictional strengths have been estimated to lie in the range 21 - 29 degrees, with most slides of the order of 26 ± 2 degrees.

3.2.2 *Geological controls on development*

Slope movements in the Cromwell Gorge (and the nearby Kawarau Gorge; Bell, 1976, 1987) have been strongly influenced by the nature of the schist bedrock. Failure is favoured along pre-existing defects and is most common on slopes formed parallel or subparallel to foliation, persistent low angle faults, and weaker rock types such as mica-rich schist. Landslides are extensively developed where foliation generally dips into the valley, ie. along the left (north) side downstream of Brewery Creek Slide area to near Dunlays Slide. Defects are also recognised as controlling lateral margins, internal zonation, and headscarps to landslides.

Most of the initial movement appears likely to have been a result of stress relief along pre-existing defects within the rock mass. Where adversely oriented and critically located, ongoing movement is readily reactivated by other disturbing forces.

Low-angle defects along which stress relief displacement has occurred are considered to have little or no potential for on-going movement because the driving force causing stress relief displacement is necessarily dissipated during that displacement. However, such low angle defects at valley floor level may allow toe breakout of a slide across the rock fabric, and steeper features may be reactivated as slide failure surfaces if they are subsequently under-cut.

As well as forming failure zones, many defects also act as groundwater barriers (aquicludes or aquitards) below and within landslides. This commonly results in complex groundwater systems which include multiple perched aquifers within slides, and/or confined conditions both within and beneath slides (Macfarlane et al, 1992).

Confined groundwater systems locally exert significant uplift pressures which can have a major influence on landslide stability (Gillon & Hancox, 1992).

4 INTERPRETATION OF SLOPE MOVEMENTS

4.1 *Investigation difficulties*

Old, large landslides can be difficult to recognise (Patton & Hendron, 1974), and this is commonly the case in the Cromwell Gorge where most of the landslides are ancient features exhibiting subtle, subdued morphology. Because surface evidence of landslide boundaries is often poorly defined, mapping of foliation or lineation attitudes was important in helping to distinguish in situ outcrops from those affected by slope movements.*

While small diameter (HQ) core drilling has been used extensively to investigate sub-surface conditions (Gillon, et al 1992), the results obtained have rarely been unequivocal in terms of definition of slope movements. Particular difficulties have been experienced in the recognition and correlation of weak seams forming basal failure surfaces, even where exposed in drives or surface excavations. Not only are such seams often not recovered in drillcore but their projection and interpolation between investigation points is commonly constrained by the same high degree of geological variability, complexity and lack of continuity and marker horizons that has been demonstrated in the in situ schist. Similarly, while mechanically disrupted rock masses were readily distinguishable in excavations (eg. by open joints), recognition in drill core was more difficult, being based mainly on trends in fracture and weathering intensities. It follows that geological models developed for use in assessment of lake filling effects were inevitably a simplification of actual conditions, containing many uncertainties which had to be allowed for in numerical analyses.

4.2 *Modes of failure*

The complex structural controls on slide geometry have often made it difficult to identify failure mechanisms.

While several of the commonly described types of slide movements (Varnes, 1978) appear to have operated, translational failures controlled principally by pre-existing foliation shears or low-angle faults seem to predominate. Debris slides can also be assumed to approximate such surfaces in many places. It is likely that the debris has undergone significant movement (10's to 100's of metres), such that any asperities could have been reduced, and hollows infilled. Curvilinear failure surfaces may therefore be inferred locally within or at the base of the debris, especially in toe areas, implying an element of rotation within otherwise generally translational movement. Internal failure surfaces are also likely to be curvilinear in both upslope and cross slope directions.

For many of the slides some combination of structural control and adverse groundwater conditions has resulted in complex failure mechanisms, which may have varied through time to cause the development of the slides in their present form.

4.3 *Landslide evolution*

The slide areas delimited along the reservoir are considered to have developed as part of an ongoing process of landform evolution that has resulted in the formation of the Cromwell Gorge as the river has down-cut through the actively rising mountain blocks over at least the last 3-5 million years (McSaveney, et al 1992). Landslides have probably been present in the gorge since about 300

**Slope movement* is used here in the sense of Varnes (1978), i.e. comprising falls, slides, spreads, and flows, (also widely known as *mass movement).*

m of downcutting was reached. Incision of the rivers appears to have kept pace with uplift, and development of the very large landslides has probably been gradual.

During the long history of landsliding in the Cromwell Gorge, most of the larger slides would have gradually propagated to greater depths by ongoing erosion at their toes, and exploiting successively lower favourably inclined weaknesses in the rockmass. Deepening and/or enlargement will have occurred at irregular intervals with periods of greater activity as the river has oversteepened slopes or down-cut in response to episodic uplift, climatic changes, variations in river sediment load and/or lowering of base level during glaciations. In contrast, the aggradation of gravels during warmer stadials during glaciations will have acted to reduce (or even halt) slide movements so that deep under-cutting and large-scale activity have been episodic over hundreds of thousands of years. Although there is no conclusive evidence, it is possible that large major earthquakes may have also influenced the early development of the landslides. However, geological evidence shows that there have been no significant large-scale rapid movements in the last 15-30 000 years, even though many large earthquakes are inferred to have occurred during this time (Gillon & Hancox, 1992).

There is no evidence to suggest that any of the landslides have moved as single masses. Most can be zoned into areas with distinctly different surface morphology, degrees of slope deflation, and mate-

Term	Mass Description	Surface Characteristics	Type of Movement	Degree of Displacement
Chaotic Debris	Gradation from large competent rock blocks to fine grained material (commonly intensely sheared and crushed schist) with gouge seams. Foliation attitudes of blocks normally highly variable. Slickensided downslope-dipping internal or basal failure zones.	Laterally impersistent breaks in slope (i.e. hummocky). Well developed slide scarps where active. Blocks on surface.	Rotational, translational, or complex slide.	10's to 100's of m.
Displaced Schist	Large competent rock blocks either in contact or separated by open and/or infilled joints or by zones of sheared/crushed material. Foliation either parallel or oblique to undisturbed rock, and adjacent blocks may be slightly rotated relative to one another. Slickensided near downslope-dipping internal or basal failure zones, typically sub-parallel to foliation or pre-existing rock defects (Also termed "Blocky Debris", where disruption is greatest).	Laterally persistent breaks in slope (i.e. broadly irregular). Forms outcrops locally.	Translational slide.	m's to 10's of m.
Basal Failure Zone	Crushed zones and gouge seams with some sheared and shattered schist of variable thickness. Fabric may be sub-parallel to boundaries, contorted, or totally disrupted. Gouge seams typically thin but persistent with slickensides, oriented downslope.	Outcrops rare.	Slide.	mm's to 100's of m.
Disturbed Schist	Sub-slide rock mass with partly open defects often infilled. Typically quartz-rich massive and laminated schist. (Termed "Relaxed Schist" in near-surface situations).	Relaxed schist forms prominent outcrops similar to undisturbed schist.	Stress relief processes.	mm's to m's.
Deformed Schist	Sub-slide rock mass, fissile with discrete sheared and crushed zones sub-parallel to flexurally deformed ("buckled") foliation, steepened to overturned locally. Typically mica-rich laminated schist.	Prominent outcrops with foliation dipping at moderate to high angles to undisturbed schist.	Bedrock flow, by stress relief and/or gravitational processes.	mm's to 10's of m.
Undisturbed Schist	*In situ* rock mass with closed defects.	Forms prominent outcrops.	None.	None.

Table 1 Terminology used in slope movement interpretation, Cromwell Gorge

rial types. Variation in the depth to slide base at several of the slides is indicative of steps across defects trending normal and parallel to slope direction, often approximately coincident with the boundaries of zones mapped at the surface. These characteristics strongly suggest that different parts of the slides have moved by differing amounts, at different depths and probably at different times. Slow creep movement has probably always been the dominant rate of movement.

4.4 *Interpretive Categorisation of Materials*

4.4.1 *Basis of Terminology*

Interpretation of the results of the extensive programme of subsurface investigations in the landslide-affected Cromwell Gorge was directed principally towards the development of the most likely geological models for use in numerical analysis assessment of the effects of lake filling. Due to frequent difficulties in recognising and correlating weak seams forming either internal or the deepest (basal) failure zones, these zones were commonly identified approximately by categorising and mapping various soil and rock mass characteristics attributable to varying degrees of disturbance inferred to have resulted from differing amounts of movement. Table 1 gives the terminology and characteristics for each category.

In summary, slide movement is inferred from the presence of "chaotic debris" and "displaced

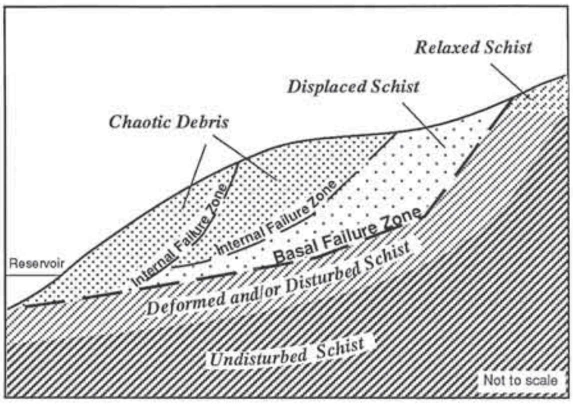

Figure 3 Schematic representation of general distribution of slope movement material categories, Cromwell Gorge

schist", bounded at depth by a "basal failure zone". "Disturbed" and "deformed schist" are categories of sub-slide slope movement, originating through rock creep and/or stress relief. Figure 3 illustrates general spatial relationships.

4.4.2 *Recognition and distribution of materials*

An example of interpretation of material distribution (part of Clyde Slide Area) is given in a cross-section, Figure 4, with emphasis on the typical evidence used for the definition of internal and basal failure zones. Additional examples (in Nine Mile Creek Slide Area) are described by Beetham et al (1992). In most slide areas, recognition of the

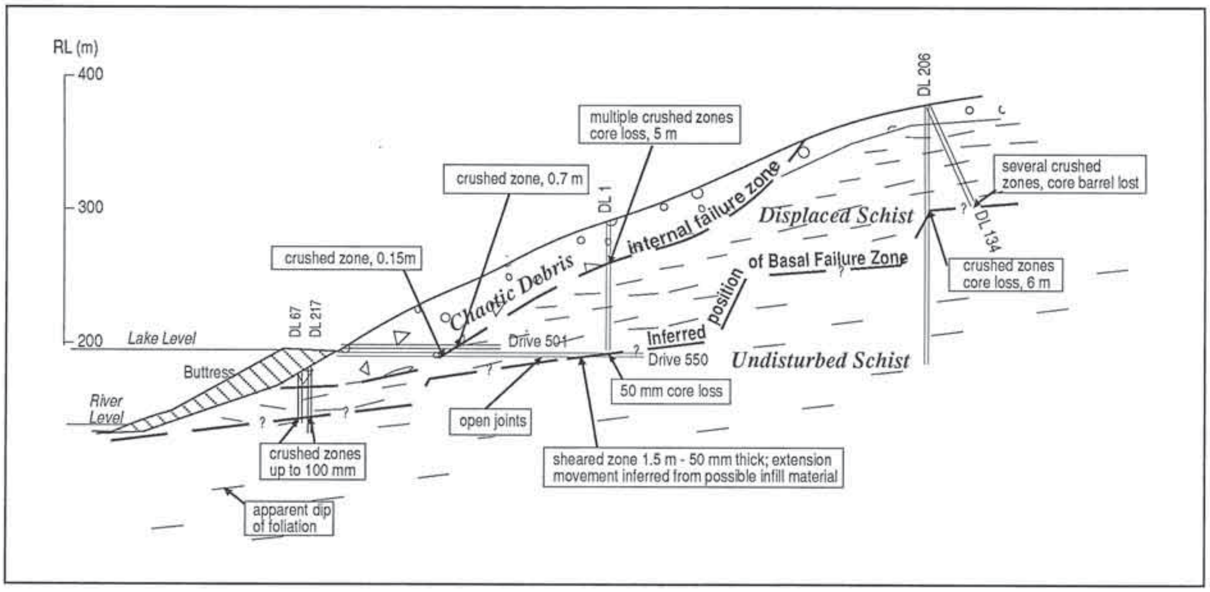

Figure 4 Typical data and interpretation of slope movement, Cromwell Gorge

different slope movement material types was relatively straightforward. In some cases, however, if available data did not conclusively allow identification of a basal failure zone, it proved difficult to determine whether rock mass dilation was the result of landsliding or stress relief movement processes. Distinguishing between displaced and disturbed material categories was therefore somewhat arbitrary, requiring a conservative approach in selecting basal failure zones.

The presence and distribution of material categories both within and between landslide areas is typically variable. In the simpler landslides either chaotic debris or displaced schist are the only materials overlying the basal failure zone, whereas elsewhere, both are present.

The recognition of basal failure zones did not remove the need for recognition of defects in the undisturbed bedrock which could have the potential for "first time" landslide development.

5 CONCLUSIONS

Complex schist geology in the Cromwell Gorge has contributed to a range of controls on slope movement. As a result, subsurface definition of slide extent and geometry by conventional investigation methods has been constrained mainly by difficulties in correlating major defects, notably those forming basal failure zones. Geological models developed for use in hazard assessment and determination of lake filling effects were therefore necessarily based on a simplification of real conditions. Recognition of the inherent uncertainties in geological modelling was an important factor in engineering decisions on the need for and scope of remedial works carried out to offset possible adverse effects of lake-filling on reservoir slopes on Clyde Power Project.

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STANDARDS, LAW AND INDUSTRY NEWS

NZGS Young Geotechnical Professionals

Firstly I would like to thank Liz Bowman for all her efforts to promote and encourage the NZGS Young Geotechnical Professionals (YGPs) during her role as YGP Representative. There are some big shoes to fill and I hope to make them fit.

This new role for me encompasses my ongoing developments with working on the IAEG 2010 Congress organising committee as the Young Professionals coordinator.

Who are YGPs?

YGPs are a special interest group of the NZ Geotechnical Society that focuses on the needs of young professionals and students under the age of 35 and with less than 10 years of experience.

We represent a lively, increasingly influential and rapidly growing section of Geotechnical Engineers and Engineering Geologists nationwide. The Young Geotechnical Professionals group has been encouraged to be formed to represent, support and provide a voice for the young professionals. Through a social culture of innovation, integrity, networking and the pursuit of excellence, we anticipate facilitating in the professional and personal development of the young professionals.

Aims are to:

1. Motivate young geotechnical professionals to actively engage in the engineering geological/geotechnical profession
2. Generate excitement and inspiration amongst the young members of the profession
3. Increase the participation in NZGS activities by the young members of the profession
4. Empower all young geotechnical professionals and students to build their careers and promote their profession through networking, educational and social events

What's in it for you?

- Learning opportunities for your professional development
- Advice from experienced geotechnical professionals and your peers
- Networking with all the right people
- Awards and fellowships
- Cultivation of life-long learning
- Growing professional and personal relationships
- Celebrate excellence in Geotechnical Engineering and Engineering Geology

The YGP network relies on the combined efforts of active NZGS members – young and old – who volunteer their expertise, time and enthusiasm to support us and our initiatives to maintain our future geotechnical significance. All young professionals, students and graduates are encouraged to become involved in NZGS activities, as it's never too early to start thinking about your future career.

We welcome support and inspiration from the young geotechnical professional's community. If you have any ideas or activities you would like to see happen please contact me.

Reported by:
Kate Williams
YGP Representative
Email: kwilliams@tonkin.co.nz

Fellows of Institute of Professional Engineers New Zealand

At the annual IPENZ convention held on Friday 27 March 2009 two members of the geotechnical society were elected to the class of Fellow of IPENZ for their contributions to the advancement of geotechnical engineering practice and leadership in the profession of engineering.

Fellows of IPENZ are members who have made a significant contribution in one or more of the following areas:

- Advancement of engineering knowledge or practice
- Application of engineering or technology in the community
- Advancement of technological education
- Innovation in creation of engineering works or technological products
- Leadership in the engineering profession
- Development of the Institution.

Congratulations go to the following members of the Geotechnical Society.

Michael Davies

Michael Davies is elected a Fellow of IPENZ for his contribution to the advancement of engineering knowledge and advancement of technological education, particularly recognising his contribution to geotechnical engineering and engineering education. He has an international reputation for published research in fields such as ground improvement, soil-structure interaction, geo-materials and geo-environmental issues. In parallel he has held leadership roles in engineering education in the United Kingdom and now in New Zealand where he is seeking to address engineering skill issues.

Malcolm Stapleton

Malcolm Stapleton is elected a Fellow of IPENZ for his contribution to the advancement of engineering practice, particularly recognising his contribution to geotechnical engineering. He led a major initiative to improve the methodologies used by regulatory authorities for assessing contaminated land, demonstrating where there were flaws requiring improvement. He has also contributed to the development of liner systems for landfills, including modelling of the settlement and tensile strain. He has contributed extensively to activities of the New Zealand Geotechnical Society.

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International Society for Rock Mechanics

ISRM

ROCHA MEDAL 2011

Since 1982 a bronze medal and a cash prize have been awarded annually by the ISRM for an outstanding doctoral thesis in rock mechanics or rock engineering, to honour the memory of Past President Manuel Rocha while stimulating young researchers.

Starting with the Rocha Medal 2010, one or 2 runner-up certificates may also be awarded.

An invitation is now extended to the rock mechanics community, and especially to Faculty members, for nominations for the Rocha Medal 2011.

Full details on the Rocha Medal are provided in ISRM By-law No. 7.

Application

To be considered for an award the candidate must be nominated within two years of the date of the official doctorate degree certification.

Nominations shall be by the nominee, or by the nominee's National Group, or by some other person or organization acquainted with the nominee's work.

Nominations shall be addressed to the Secretary General and shall contain:

- a one page curriculum vitae;
- a written confirmation by the candidate’s National Group that he/she is a member of the ISRM;
- a thesis summary in paper and digital formats, written in English, with between 5,000 and 10,000 words, detailed enough to convey the full impact of the thesis and accompanied by selected tables and figures;
- one copy of the complete thesis and one copy of the doctorate degree certificate;
- a letter of copyright release, allowing the ISRM to copy the thesis for purposes of review and selection only;
- an undertaking by the nominee to submit an article describing the work, for publication in the ISRM News Journal.

Application Deadline

The nomination must reach the ISRM Secretary General by 31 December 2009.

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2009	G. Li	CHINA

All relevant information can be obtained from the ISRM website, at http://www.isrm.net.

IAEG Congress Report

Report on Planning for IAEG2010 Geologically Active, Auckland, Aotearoa NZ

Planning is well underway for the 11th Congress of the International Association for Engineering Geology and the Environment (IAEG), to be held in Auckland in 2010. IAEG2010 is hosted and underwritten by the New Zealand Geotechnical Society Inc. (NZGS) and will be a five-day congress including a day of field trips mid-week, preceded by meetings of the IAEG Executive, the Council and competition for the Richard Wolters Prize.

While Geologically Active is the principal Congress theme, the five sub-themes that will recur throughout the week are:

- geohazards at the leading edge, focussing on identification and evaluation of natural hazards;
- managing geological risk, focussing on approaches to hazard mitigation around the world;
- advances in engineering geology, addressing state of the practice;
- applied engineering geology, examining the interaction of geology, water and structures; and
- evolving engineering geology, bridging the gap between scientists, engineers and non-practitioners into the future.

The themes of IAEG2010 extend beyond engineering geology and the congress aims, at least in part, to bridge the gap between science and practise. Abstracts are now open on www.iaeg2010.com.

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Above: Congress planning James Arthurs, Mandy Mills, Doug Johnson, Kate Williams, Sally Hargraves. **Standing:** Debbie Fellows, Ann Williams, Tim McMorran and Gregory Pinches.

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Ann Williams
Co-convenor IAEG2010

Local Registers for Producer Statements

Department of Building and Housing (DBH) recently issued a note reiterating their advice to Building Control Authorities (BCA's) that they should utilise statutory registers (Chartered Professional Engineer) rather than stand alone local registers. DBH has convened an industry representative group involving local Government, engineers and builders, chaired by Alan Bickers, to consider issues relating to producer statements and to develop a best practice guide for producer statement authors and BCAs. As at February 2009 this working group is finalising a draft guide which will be circulated to the sector for comment shortly.

Department of Building and Housing
Te Tari Kaupapa Whare

Producer statement issues

The Department of Building and Housing has received enquiries from building professionals registered on statutory registers asking about whether they need to be re-registered by building consent authorities (BCAs) and carry high levels of professional indemnity insurance when providing BCAs with producer statements.

This letter explains the Department's response to these enquiries.

BCA producer statement policies and procedures
Building consent authorities are required to develop and implement policies and procedures that document how they undertake building control work. Because producer statements have been widely used by BCAs to help establish reasonable grounds for making a decision about compliance with the provisions of the Building Code, BCAs are required to develop policies and procedures about how they use and accept producer statements.

The Department's previous advice to the sector is the recommendation that BCAs' policies and procedures include:

- the form and content of producer statements
- acceptance criteria
- how compliance with the Building Code will be demonstrated and any assumptions or conditions
- how the competency of each producer statement's author will be established.

A number of professions have statutory, competency-based registration systems including engineers and architects.

The Department has advised BCAs that they should make use of statutory registers to assist in determining competence while noting that in some situations, such as high risk or highly complex work, supporting information about specific areas of competence may also be required by a BCA. The Department acknowledges that it is inefficient to require professionals who are already registered on a competency-based register to also have to apply to one or more BCAs for registration on their stand-alone registers.

Professional indemnity insurance requirements

The Department has been informed that some BCAs, in addition to duplicating registration procedures, also require blanket professional indemnity insurance cover well in excess of that required by professional bodies. The Department has advised the sector that BCAs should not require producer statement authors to hold unreasonably high levels of insurance cover. Instead, the level of cover should be matched with the risk or complexity of the project. In most cases the standard levels of cover should be sufficient.

Department activity

Under the Building Act, the Department does not have the authority to direct a BCA to follow its guidance. However, the Department is concerned with the apparent level of duplication of effort and inefficiencies which currently exist. To develop a consistent national approach, the Department facilitated a group involving local Government, engineers and builders, chaired by Alan Bickers, to consider issues relating to producer statements and to develop a best practice guide for producer statement authors and BCAs. This working group is finalising a draft guide which will be circulated to the sector for comment shortly. Once finalised, it is anticipated the guidance will be adopted by the sector which will lead to a nationally consistent approach.

Groundwater Specialists Delegation to China

In October 2008, I took part in a groundwater delegation to China organised by the People to People Citizen Ambassador Programme (USA) on behalf of the NGWA – National Groundwater Association (USA). The delegation was lead by Mr Kevin McCray, the Executive Director of the NGWA and consisted of 19 members (plus partners) from seven countries. The delegates represented a wide range of experience within the industry and included practicing and retired hydrogeologists, academics, regulators and contractors working in the areas of water resources, geotechnical engineering, mining and contamination.

The purpose of the delegation was to learn about current groundwater issues facing China and to meet with our Chinese counterparts to discuss current technical challenges in the profession. The delegation visited and held meetings with the following organisations:

i. Beijing
- Centre for Groundwater Monitoring of Ministry of Water Resources (CGM).
- Gaobeidian Sewage Treatment Plant – Beijing (1 million m³/d capacity)
- China University of Geosciences.

ii. Guilin
- Institute of Karst Geology.

iii. Shanghai
- Shanghai Institute of Geological Survey

Over the past 30 years and specifically the last ten years, China has experienced huge growth which has placed significant demands on groundwater resources, particularly in Northern China where rainfall is limited (North China Plain Rainfall).

From information provided by Beijing University, China's groundwater use is estimated to have grown from 60 million m³/yr in 1980 to 112 million m³/yr in 2000. Groundwater over-abstraction has led to the following effects:

i. Falling groundwater levels – for particular areas since the 1960's, shallow unconfined levels have fallen 10 m to 50 m and deep confined aquifers 30 m to 90 m (Evans, 2002).
ii. Drying up of wells and river channels. Increasing areas of dry land.
iii. Land subsidence.
iv. Saline intrusion.

To address the above problems the Chinese have either adopted or are carrying out the following:

i. Reducing rural groundwater use – through improved irrigation efficiencies and requiring permits for new bores.
ii. Reducing industrial and urban groundwater use. For example, water conservation measures observed at the Beijing Sewage Treatment Plant included the use of "reclaimed water" (470,000 m³/d of treated effluent) for industrial cooling at a thermal power station, grassland irrigation and landscape features (e.g. Olympic Park rowing venue).
iii. Surface water treatment.
iv. Assessment of sustainable levels of groundwater extraction.
v. Groundwater injection.

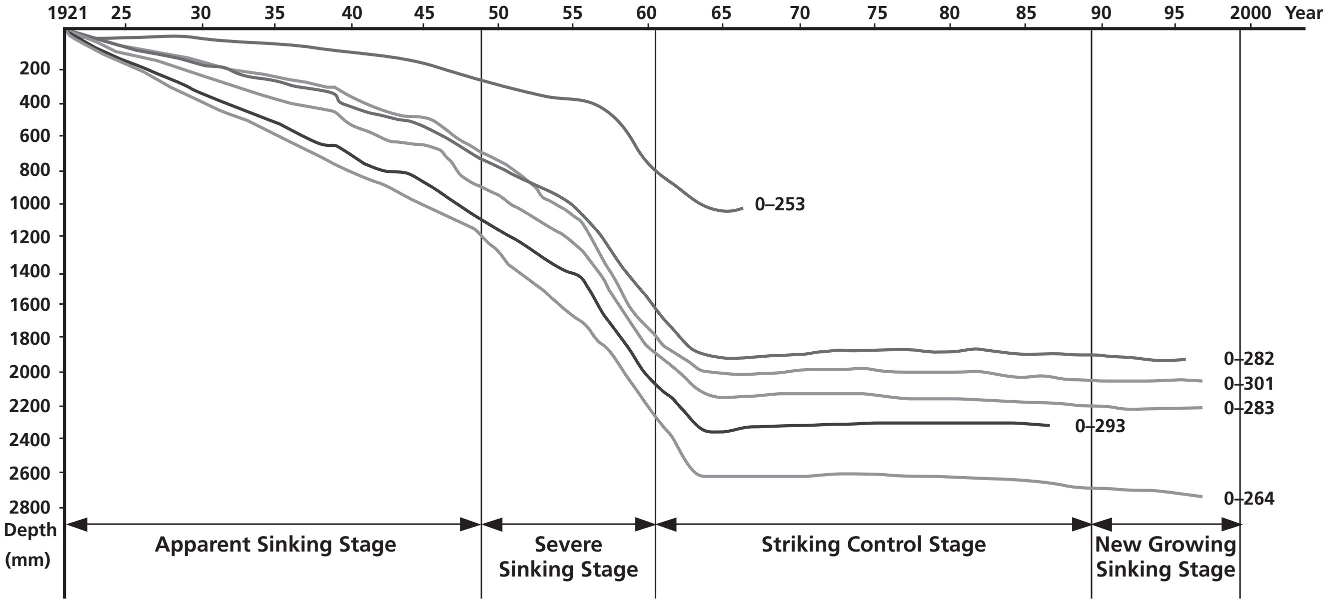
One of the highlights of the visit was the presentation by the Shanghai Institute of Geological Survey regarding land subsidence and mitigation works.

Shanghai City is located on the extensive Quaternary Yangzi River Delta Sequence (200 m to 400 m depth of delta sediments). Groundwater is present in a near surface unconfined aquifer and at deep confined systems in a series of silts.

Shanghai has used groundwater over the last 150 years. Prior to the 1960's groundwater abstraction was uncontrolled with pumping rates reaching a maximum of 200 million m³/yr. Widespread ground subsidence developed with measured settlements up to 2.6 m (Figure 1). With existing ground levels generally between 3 m and 5 m asl flooding became a major problem for the city. Buildings, bridges and other structures were also damaged.

Between 1963 and 2006, Shanghai City has adopted five sets of regulations aimed at reducing groundwater abstraction, monitoring subsidence and injecting water into the deep aquifer systems. Presently well pumping is estimated to be at 50 million m³/yr (Figure 2) and it is proposed by 2010 that groundwater abstraction is reduced to 25 million m³/yr.

Existing areas of subsidence are protected by a bund. Future subsidence is primarily controlled through the use of injection bores into the deep confined aquifers. Treated surface water is used for injection and by 2010 it is planned that a balance is achieved between 25 million m³/yr of abstraction and 25 million m³/yr of injection. Survey monitoring has also shown that the water injection has been successful in halting ground subsidence. Ground heave in the range of 0 to 10 mm has been measured in the



Land Subsidence Curves of some typical benchmarks in Shanghai

Above: Figure 1 Shanghai Ground Subsidence
Source: Shanghai Institute of Geological Survey – Slide 13 of "The Groundwater Resource of Shanghai" presented by The People to People Ambassador Programme, 29 October 2008.

Above: Figure 2 Shanghai Injection Well
Source: Shanghai Institute of Geological Survey – Slide 33 of "The Groundwater Resource of Shanghai" presented by The People to People Ambassador Programme, 29 October 2008.

vicinity of the injection bores.

The Shanghai Institute of Geological Survey had carried out extensive investigations and 3D groundwater modelling to plan and execute the mitigation works. Monitoring was carried out with 330 monitoring wells, 25 extensometers to bedrock and 1,400 survey pins.

We were very impressed how the Survey together with the City were able to work together to implement mitigation works on such a large scale.

The trip to China was successful and I appreciated the professional discussions both with the other delegates and our Chinese counterparts. I would also recommend the People to People Ambassador Programme to others.

Reported by:
Philip Kelsey
Senior Hydrogeologist
Earthtech Consulting Limited

AWARDS

New Zealand Geotechnical Society Geomechanics Award 2008

Background

The New Zealand Geomechanics Award is presented every three years to the Society member or members producing the best published paper, over that three year period. All Society members who are authors of any paper published within the previous three years are eligible.

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

- To advance the study and application of soil mechanics, rock mechanics and engineering geology among engineers and scientists
- To advance the practice and application of these disciplines in engineering

2008 Award

A sub committee was formed to review nominated papers and to scan publications over the three-year period.

From a long list of over thirty potential papers the sub committee identified three papers that were all worthy contenders for the award. Following detailed discussion and assessment against the award criteria the sub committee recommended that the 2008 Geomechanics Award be awarded to **Misko Cubrinovski, Kenji Ishihara and Harry Poulos,** for their paper **"Psuedo-static analysis of piles subjected to lateral spreading".**

The objectives of the paper are highly commendable – how to design piles to withstand lateral spreading. The paper draws from actual experience of soils that liquefied in the Kobe earthquake and observations of damage that occurred to those piles, linking this to laboratory research on large scale shake tables. The output is a clear analysis of the loads/actions acting on the piles - resulting from both the initial cyclic ground displacement and the subsequent lateral spreading displacement of the liquefiable soils. The paper assesses the uncertainties involved and finishes with a very useful step by step practical procedure for preliminary assessment and design of such piles.

The NZGS wishes to commend the authors on a very accessible and useful paper that will assist the understanding and practice of foundation design in liquefiable soils in New Zealand.

The NZGS also acknowledges the many other excellent papers considered and commends the various authors on their study and reporting. The wider NZ Geotechnical community is encouraged to take the opportunity to read the award paper and other recent New Zealand papers, which indicate the healthy state of Geotechnics in New Zealand.

Reported by
Richard Young
Beca Infrastructure
NZGS Treasurer

GEOMECHANICS AWARD PAPER REFERENCE:

Cubrinovski, M., Ishihara, K. & Poulos, H. (2006) Pseudo-static analysis of piles subjected to lateral spreading. Christchurch: New Zealand Workshop on Geotechnical Earthquake Engineering 2006, 20-21 November 2006.

Pseudo-static Analysis of Piles Subjected to Lateral Spreading

Misko CUBRINOVSKI, Kenji ISHIHARA and Harry POULOS

ABSTRACT

Soil liquefaction during strong ground shaking results in almost a complete loss of strength and stiffness in the liquefied soil, and consequent large lateral ground movements. Both cyclic displacements during the intense ground shaking and development of liquefaction, and especially post-liquefaction displacements due to spreading of liquefied soils are damaging for piles. Characteristics of the liquefied soils and loads on piles are significantly different during the cyclic phase and in the subsequent lateral spreading phase, and therefore, it is necessary to separately consider these two phases in the simplified analysis of piles. This paper describes a practical procedure for preliminary assessment and design of piles subjected to lateral spreading, and addresses key parameters and uncertainties involved.

INTRODUCTION

There are several methods available for analysis of piles in liquefied soils including sophisticated finite element analysis based on the effective stress principle and simplified methods based on the pseudo-static approach. A rigorous effective stress analysis permits evaluation of seismic soil-pile interaction while considering the effects of excess pore pressure and eventual soil liquefaction on the pile response. Whereas the predictive capacity of such analysis has been verified in many studies, its application in engineering practice is constrained by two requirements, namely, the required high-quality and specific data on the in-situ conditions, physical properties and mechanical behaviour of soils, and quite high demands on the user regarding the knowledge and understanding both of the phenomena considered and particular features in the adopted numerical procedure. Provided that the above requirements are met, however, the effective stress analysis provides an excellent tool for assessment of the seismic performance of pile foundations in liquefiable soils.

For preliminary assessment and design of piles, however, a simplified analysis may be more appropriate provided that such analysis can satisfy the following requirements: i) the adopted model must capture the kinematic mechanism associated with the spreading of liquefied soils; ii) The analysis should allow us to estimate the inelastic response and damage to piles, and iii) the method should allow for variations in key parameters and assessment of the uncertainties associated with lateral spreading. Based on these premises, this paper addresses the use of the pseudo-static analysis of piles in liquefying soils and focuses in particular on its application to the analysis of piles subjected to lateral spreading.

GROUND DISPLACEMENTS IN LIQUEFIED SOILS

When analyzing the behaviour of piles in liquefied soils, it is useful to distinguish between two different phases in the soil-pile interaction: a cyclic phase in the course of the intense ground shaking and consequent development of liquefaction, and a lateral spreading phase following the liquefaction. During the cyclic phase, the piles are subjected to cyclic horizontal loads due to ground displacements (kinematic loads) and inertial loads from the superstructure, and the combination of these oscillatory kinematic and inertial loads determines the critical load for the integrity of the pile during the shaking. Lateral spreading, on the other hand, is primarily a post-liquefaction phenomenon that is characterized by very large unilateral ground displacements and relatively small inertial effects. Thus, the liquefaction characteristics and lateral loads on piles can be quite different between the cyclic phase and the subsequent lateral spreading phase.

Cyclic Ground Displacements

In order to illustrate some important features of ground displacements in liquefied soils, observations from well documented case histories in the 1995 Kobe earthquake are discussed in the following. Figures 1a and 1b show computed horizontal ground displacements and excess pore water pressures that developed in an 18 m thick fill deposit during the intense part of the ground shaking in this quake. This response is representative of the cyclic phase of the free field response of the fill deposits in areas that were not affected by lateral spreading.

Several features of the ground response shown in Figure 1 are relevant to the behaviour and analysis of piles in liquefied soils. First, the cyclic horizontal ground displacements in the course of the strong shaking are very large with peak values of about 35-40 cm. These displacements correspond to an average peak shear strain of about 3-4 % throughout the 10-12 m depth of the liquefied layer. Next, it is important to note that at the time when the ground displacement reached a large value of about 30 cm for the first time since the start of the shaking, i.e. at approximately 5.3 sec, the excess pore water pressure was well below the effective overburden stress thus indicating that the soil has not fully liquefied, at this stage. These large displacements were accompanied with high

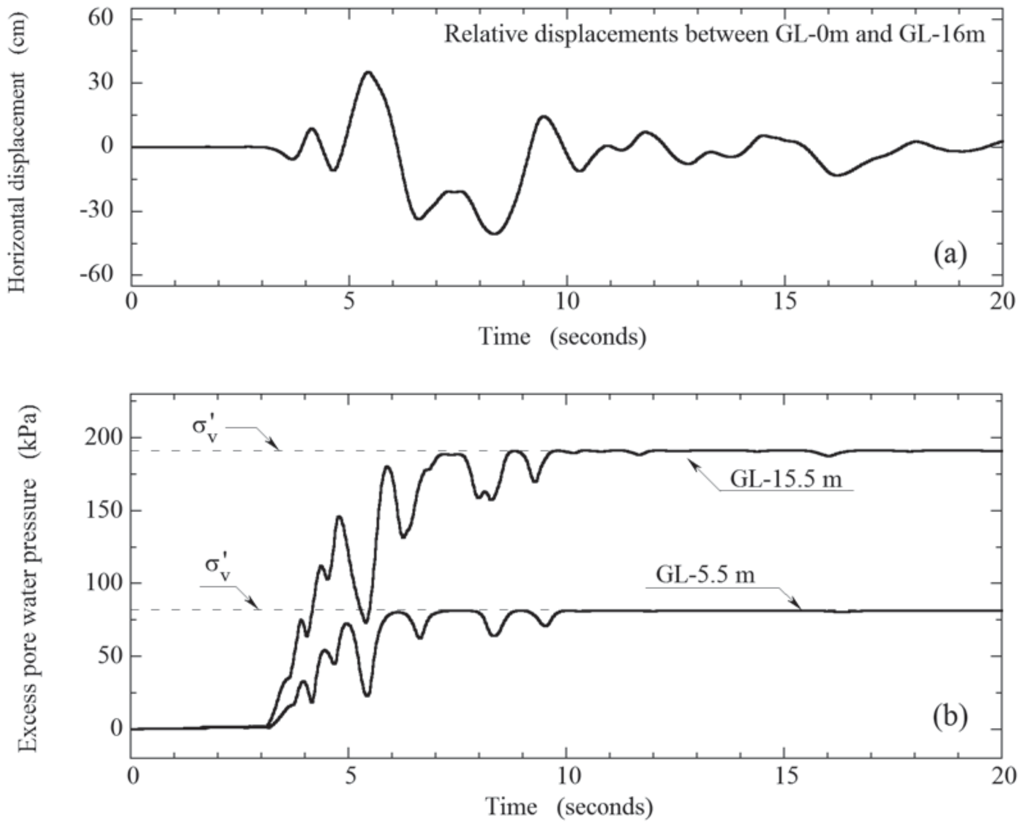


Figure 1: Ground response of liquefied deposit in the 1995 Kobe earthquake: (a) Cyclic ground displacement; (b) Excess pore water pressure

ground accelerations of about 0.4 g at the ground surface. This type of behaviour, where large ground displacements and high accelerations concurrently occur just before or at the time of development of full liquefaction, has been also observed in shake table experiments, thus highlighting the need to carefully consider the combination of inertial loads from the superstructure and kinematic loads due to ground displacements when analyzing the behaviour of piles during the cyclic phase. The magnitude of these loads depends on a number of factors including the excess pore water pressure build-up, relative displacements between the soil and the pile, and relative predominant periods of the ground and superstructure, among others. Clear and simple rules for combining the ground displacements (kinematic loads) and inertial loads from the superstructure in the simplified pseudo-static analysis have not been established yet, though some suggestions may be found in Tamura and Tokimatsu (2005) and Liyanapathirana and Poulos (2005).

Lateral Spreading Displacements

In the 1995 Kobe earthquake, the ground distortion was particularly excessive in the waterfront area where many quay walls moved several meters towards the sea and lateral spreading occurred in the backfills that progressed inland as far as 200 m from the revetment line. Ishihara et al. (1997) investigated the features of movements of the quay walls and ground distortion in the backfills by the method of ground surveying and summarized the measured displacements in plots depicting the permanent ground displacement as a function of the distance inland from the waterfront, as shown in Figure 2. Here, the shaded area shows the range of measured displacements along N-S sections of Port Island, and the solid line is an approximation for the average displacement. Superimposed in Figure 2 are the cyclic ground displacements in the free field showing that in the zone within a distance of approximately 50 m from the quay walls, the permanent ground displacements due to lateral spreading were significantly greater than the cyclic ground displacements. The permanent ground displacements reached about 1 - 4 m at the quay walls. Since the lateral spreading is basically a post-liquefaction phenomenon, it is associated with higher excess pore pressures and hence lower stiffness of the liquefied soils, as compared to its preceding cyclic phase. This feature, together with the unilateral down-slope or seaward ground movement, results in very large permanent spreading displacements. Clearly the magnitude and spatial distribution of ground displacements, as well as the stiffness of the soils undergoing large lateral movements, are quite different between the cyclic phase and lateral spreading phase, and these differences have to be accounted for in the simplified analysis of piles.

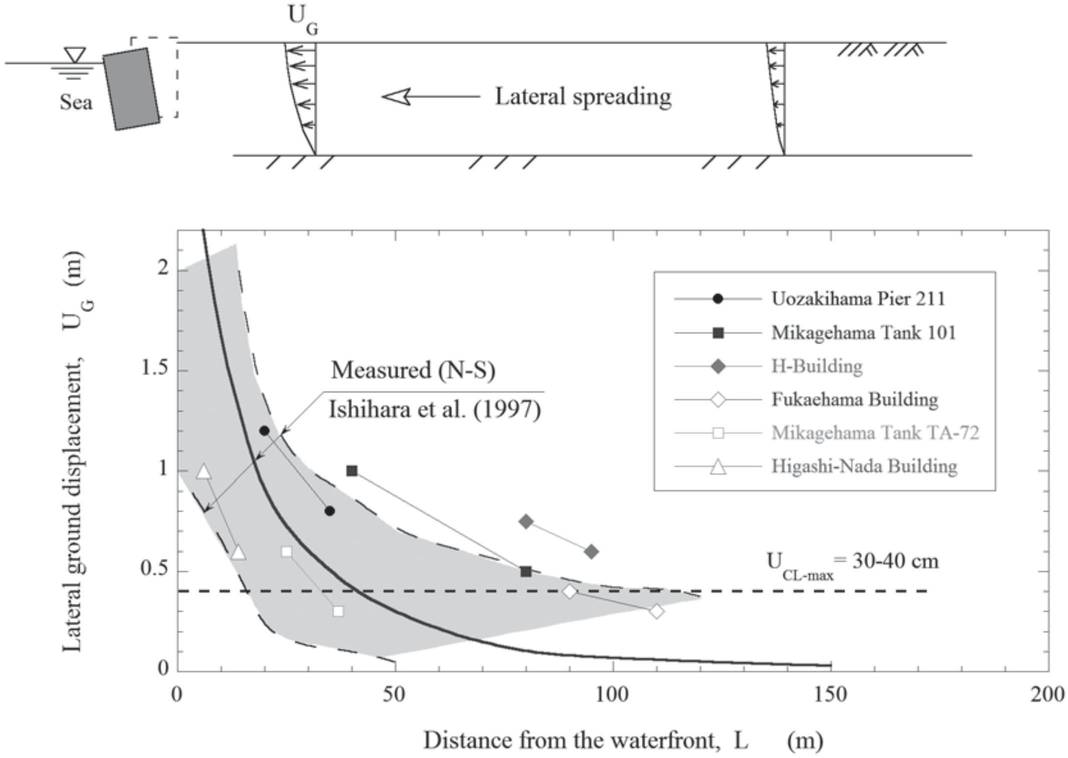


Figure 2: Permanent lateral ground displacements due to spreading of liquefied soils in the 1995 Kobe earthquake

TYPICAL DAMAGE TO PILES

A large number of pile foundations of buildings, storage tanks and bridge piers located in the waterfront area of Kobe were damaged in the 1995 Kobe earthquake (Ishihara and Cubrinovski, 1998; 2004; JGS, 1998; Tokimatsu and Asaka, 1998). Detailed field investigations were conducted on selected piles using a borehole video camera and inclinometers for inspecting the crack distribution and deformation of the pile respectively throughout the depth of the deposit, as well as by a visual inspection of the damage to the pile head. By and large, the damage to the piles can be summarized as follows:

1. Most of the piles suffered largest damage at the pile top and in the zone of the interface between the liquefied layer and the underlying non-liquefied layer (Figure 3).
2. Piles in the zone of large lateral spreading displacements were consistently damaged at depths corresponding to the interface between the liquefied layer and the underlying non-liquefied layer. Since this interface was at large depths where inertial effects from the superstructure are known to be less significant, this damage can be attributed to the lateral loads arising from the excessive ground movement due to spreading.
3. Damage at the pile head was encountered both for piles in the free field and piles located within the lateral spreading zone, near the quay walls. Both

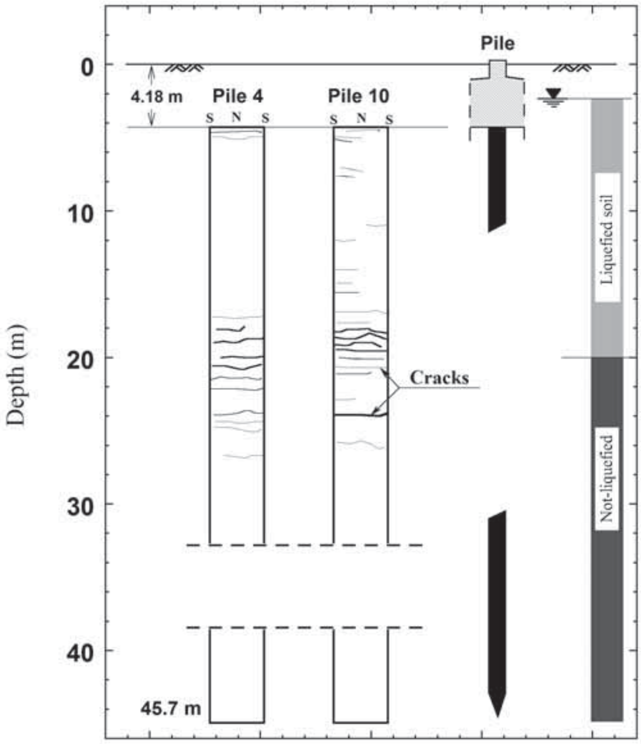


Figure 3: Typical damage to piles observed in the 1995 Kobe earthquake (Uozakihama bridge pier P211)

inertial loads from the superstructure and kinematic loads due to lateral ground displacements contributed to the damage at the pile head.

4. The variation of lateral spreading displacements with the distance from the waterfront shown in Figure 2 may result in different lateral loads being applied to individual piles, depending on their position within the pile-group. This in turn may lead to significant cross-interaction effects and consequent bending deformation and damage to piles in accordance with these interaction loads from the pile-cap-pile system. In some cases where these pile-group effects were significant, the piles failed within the liquefied layer or at least several meters below the pile head.

In addition to the typical damage patterns described above, other less significant damage was consistently found at various depths for many of the inspected piles thus reflecting the complex dynamic nature of loads and behaviour of piles in liquefying soils.

Pseudo-Static Approach for Simplified Analysis

The most frequently encountered soil profile for piles in liquefied deposits consists of three distinct layers, as illustrated in Figure 4, where the liquefied layer is sandwiched between a non-liquefied crust layer at the ground surface and non-liquefied base layer. Liquefaction during strong ground shaking results in almost a complete loss of strength and stiffness of the liquefied soil, and consequent large lateral ground displacements. As demonstrated in the previous section, particularly large and damaging for piles are post-liquefaction displacements due to lateral spreading. During spreading, the non-liquefied surface layer is carried along with the underlying spreading soil, and when driven against embedded piles, the crust layer is envisioned to exert large lateral loads on the piles. Thus, the excessive lateral movement of the liquefied soil, lateral loads from the surface layer and significant stiffness reduction in the liquefied layer, are key features that need to be considered when evaluating the pile response to lateral spreading.

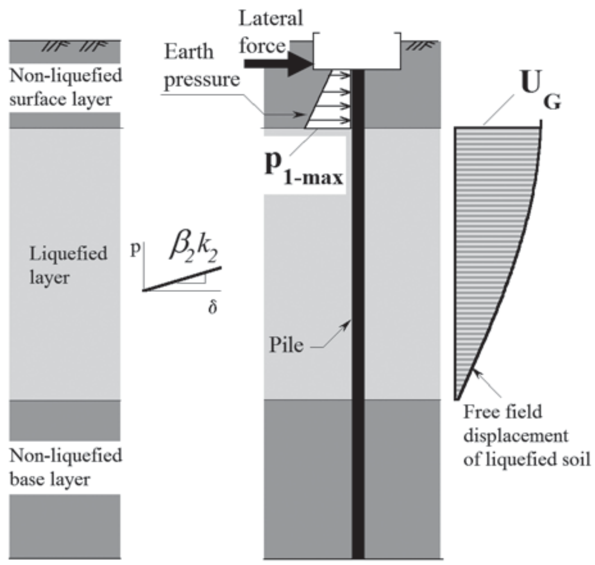


Figure 4: Simplified kinematic mechanism of lateral spreading

In light of the liquefaction characteristics and kinematic mechanism as described above, a three-layer soil model was adopted for a simplified analysis of piles based on the pseudo-static approach, in a previous study (Cubrinovski and Ishihara, 2004). As indicated in Figure 4, in this model the pile is represented by a continuous beam while the interaction between the liquefied soil and the pile (p-δ relationship) is specified by an equivalent linear spring ($\beta_2 k_2$). Here, k_2 is the subgrade reaction coefficient while β_2 is a scaling factor representing the degradation of stiffness due to liquefaction and nonlinear behaviour. In the analysis, the spreading is represented by a horizontal free-field displacement of the liquefied soil while effects of the surface layer are modelled by an earth pressure and lateral force at the pile head, as illustrated in Figure 4. Using an iterative procedure based on the equivalent linear approach, a closed-form solution was developed for evaluating the pile response to lateral spreading. The analysis permits estimation of the inelastic response and damage to piles, yet it is based on a simple model that requires a small number of conventional engineering parameters as input (Figure 5). Needless to say, one may use an FEM beam-spring model instead of the above closed-form solution and conduct even more rigorous analysis, as compared to the three-layer model, because it will permit consideration of a multi-layer deposit with different load-deformation properties. In principle, however, the following discussion applies to the pseudo-static analysis of piles, in general.

Input parameters of the computational model and adopted load-deformation relationships for the soil and the pile are shown in Figure 5. The equivalent linear p-δ relationship for the liquefied layer was adopted in order to simplify the modelling of the highly nonlinear behaviour of liquefied soils undergoing spreading and to allow parametric evaluation of the effects of this parameter. In the analysis of a given pile, it is envisioned that β_2 will serve as a parameter that will be varied over a relevant range of values, thus permitting evaluation of the pile response by assuming different properties of the liquefied soil. On the other hand, bilinear p-δ relationships and a tri-linear moment-curvature relationship (M-ϕ) were adopted for modelling the nonlinear behaviour of the non-liquefied soil layers and the pile respectively. Note that $p_{1\text{-}max}$ defines the ultimate lateral pressure that can be applied by the crust layer to the pile. In cases when the relative displacements between the soil and the pile are very large, it would be necessary to limit the maximum pressure that the liquefied soil can apply to the pile. The residual strength of liquefied soils would be one obvious choice in

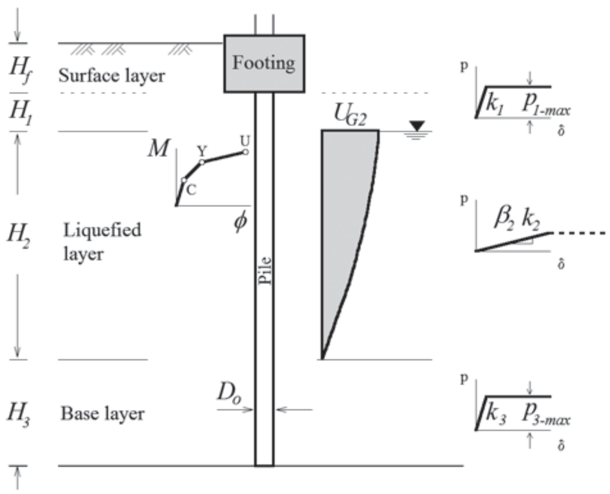


Figure 5: Characterization of nonlinear behaviour and input parameters of the model

the definition of the ultimate pressure from the liquefied soil on the pile. This modification of the original model is indicated with a dashed line in the p-δ relationship for the liquefied soil in Figure 5.

Key Parameters and Uncertainties Involved

The analysis of piles in liquefying soils is burdened by unknowns and uncertainties associated with liquefaction and lateral spreading in particular. Thus, it is very difficult to estimate the strength and stiffness properties of liquefied soils or predict the magnitude and spatial distribution of lateral spreading displacements. One of the key aspects of the simplified analysis is therefore to properly address these uncertainties.

Lateral Ground Displacements

The lateral displacement of the spreading soil (U_{G2}) can be evaluated using empirical correlations for ground displacements of lateral spreads (Ishihara et al., 1997; Tokimatsu and Asaka, 1998; Hamada et al., 2001; Youd et al., 2002). It is important to recognize, however, that in most cases it would be very difficult to make a reliable prediction for the spreading displacements. This difficulty is well illustrated in Figure 2 where a large scatter in the ground displacements is seen, even for a single earthquake event and generally similar ground conditions. In this context, Youd et al. (2002) suggested the use of a factor of 2 for the displacements predicted with their empirical model, in order to cover the expected range of variation in the spreading displacements.

Cyclic ground displacements can be estimated more accurately by means of an effective stress analysis, but this combination of an advanced analysis being used for the definition of the input in a simplified analysis is not consistent or practical. For this reason, it seems more appropriate to estimate the peak cyclic displacements for the simplified analysis by using simplified charts correlating the maximum cyclic shear strain that will develop in the liquefied layer with the cyclic stress ration and SPT blow count, as suggested by Tokimatsu and Asaka (1998), for example. The horizontal cyclic displacement profile can be then easily obtained by integrating the shear strains throughout the depth of the liquefied layer. In both cases of cyclic liquefaction and lateral spreading, the lateral ground displacement that is used as an input in the simplified analysis of piles is a free field ground displacement which is unaffected by the pile foundation.

Crust Layer

The lateral load from the non-liquefied crust layer may often be the critical load for the integrity of the pile because of its large magnitude and unfavourable position as a "top-heavy" load acting above a laterally unsupported portion of the pile in the liquefied soil. For the adopted bilinear p-δ relationship for the crust layer shown in Figure 5, the key input parameter is the ultimate lateral pressure, p_{1-max}.

The ultimate soil pressure from the surface layer per unit width of the pile can be estimated using a simplified expression such as, $p_{1-max} = \alpha_u\, p_p$, where $p_p(z_1)$ is the Rankine passive pressure while α_u is a scaling factor to account for the difference in the lateral pressure between a single pile and an equivalent wall. Figure 6 shows the variation of α_u with the relative displacement observed in a benchmark lateral spreading experiment on full-size piles (Cubrinovski et al., 2006) with the maximum lateral pressure on the single pile being about 4.5 times the Rankine passive pressure. Data from other experimental studies, shown in Figure 7, also indicate quite large values for the parameter α_u, clearly indicating that very large lateral loads can be applied by the crust layer to the pile. Here, it is important to distinguish between two types

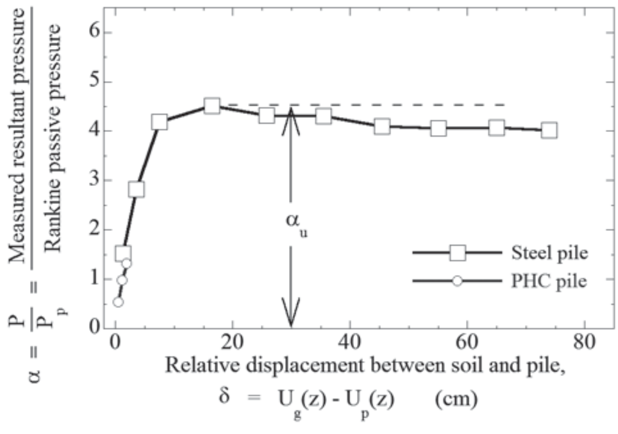


Figure 6: Ratio of lateral pressure from the crust layer on a single pile and Rankine passive pressure, measured in full-size test using large-scale shake table

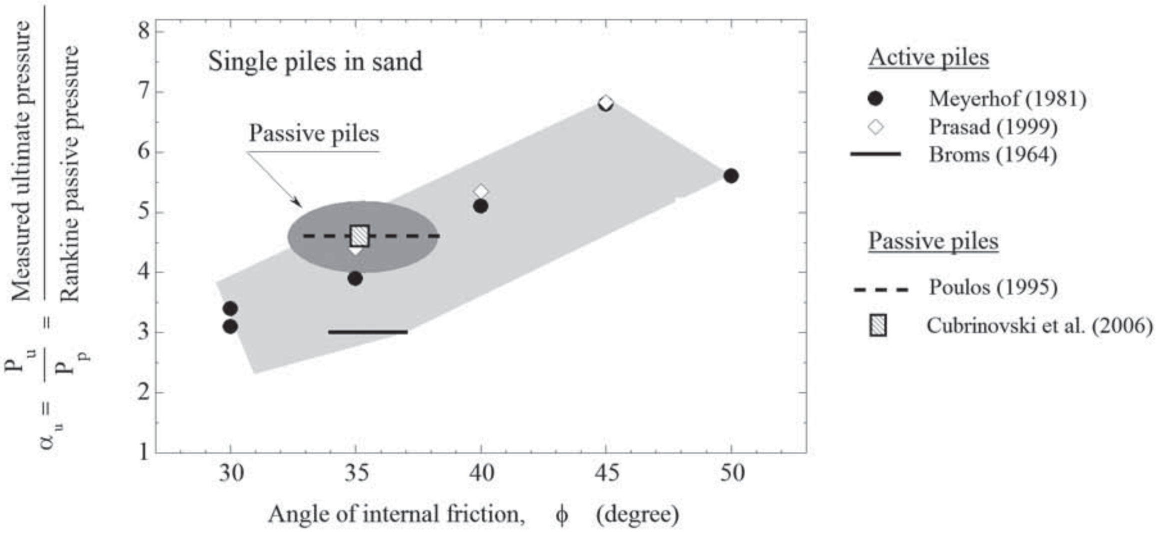


Figure 7: Ratio of ultimate pressure from the crust layer on a single pile and Rankine passive pressure, obtained in experimental studies

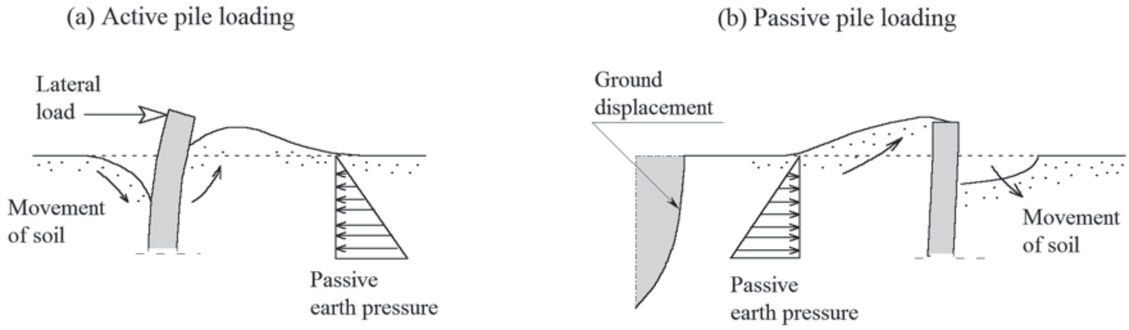


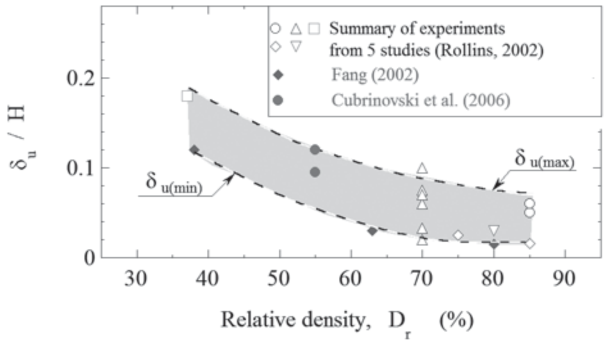
Figure 8: Schematic illustration of lateral loading of piles: (a) Active-pile-loading; (b) Passive- pile-loading

of loading conditions, namely, active pile loading and passive pile loading. In the case of active pile loading, the horizontal force at the pile is the causative load for the pile deformation, as shown in Figure 8a; in this case, the mobilized earth pressure provides the resisting force. In the case of passive pile loading, on the other hand, the mobilized pressure from the crust layer provides the driving force for the pile deformation, as illustrated in Figure 8b. Note that in Figure 7 the two sets of experimental data on passive piles yield a value of α_u = 4.5. The test data used by Broms (1964) yielded mostly values of α_u = 3 – 6, and Broms adopted the lower-bound value of α_u = 3 as a conservative estimate for active piles. This value has been adopted in many design codes for active loading on piles, but may be unconservative for passive piles.

It is important to note in Figure 6 that a large relative displacement of nearly 20 cm was needed to mobilize the ultimate lateral pressure from the crust layer. This relative displacement α_u at which p_u is mobilized depends on the relative density of the sand, as illustrated by the experimental data summarized in Figure 9. Here H denotes the height of the model wall or pile cap used in the test. It is evident that for dense sands with D_r = 70 % to 80 %, the ultimate pressure was mobilized at a relative displacement of about δ_u = 0.02H to 0.08H and that larger movement was needed to mobilize the passive pressure in loose sand. Rollins (2002) suggested that the presence of a low strength layer below the surface layer may increase the required deflection to mobilize the passive pressure, and this appears to be a relevant observation for a crust layer overlying liquefied soils.

Liquefied Layer

The factor β_2, which specifies the reduction of stiffness due to liquefaction and nonlinear behaviour ($\beta_2 k_2$), is affected by a number of factors including the density of sand, excess pore pressures, magnitude and rate of ground displacements, and drainage conditions. Typically, β_2 takes values in the range between 1/50 and 1/10 for cyclic liquefaction and between 1/1000 and 1/50 in the case of lateral spreading. The values of β_2 back-calculated from full-size tests on piles (Cubrinovski et al., 2006) are shown in Figure 10 as a function of lateral ground displacement, illustrating that β_2 is not a constant, but rather it varies in



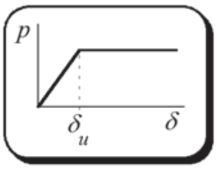


Figure 9: Relative displacement required to fully mobilize the passive pressure as a function of the relative density of sand: summary of data from experimental studies

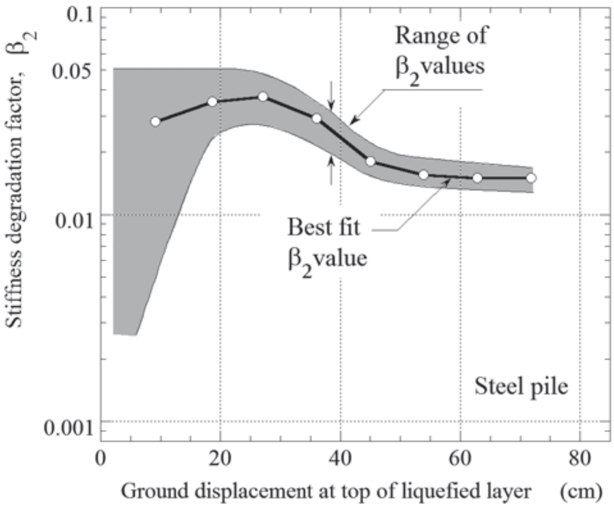


Figure 10: Degradation of stiffness in the liquefied layer as a function of lateral ground displacement observed in full-size test on piles (Cubrinovski et al., 2006)

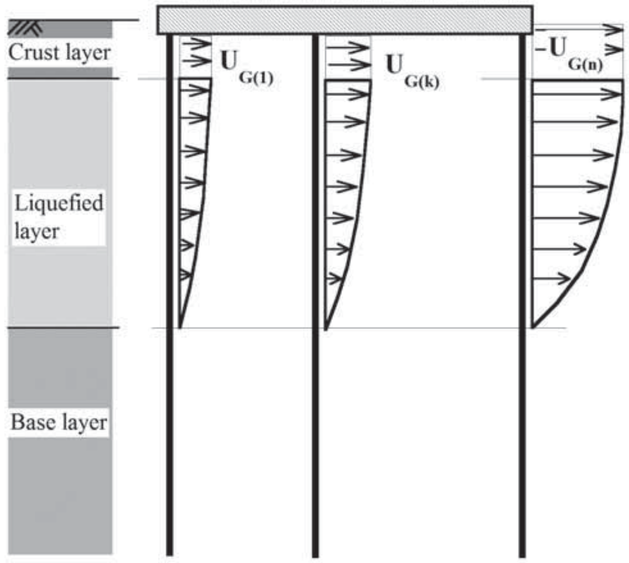


Figure 11: Piles in a group subjected to lateral ground displacements due to spreading

the course of lateral spreading.

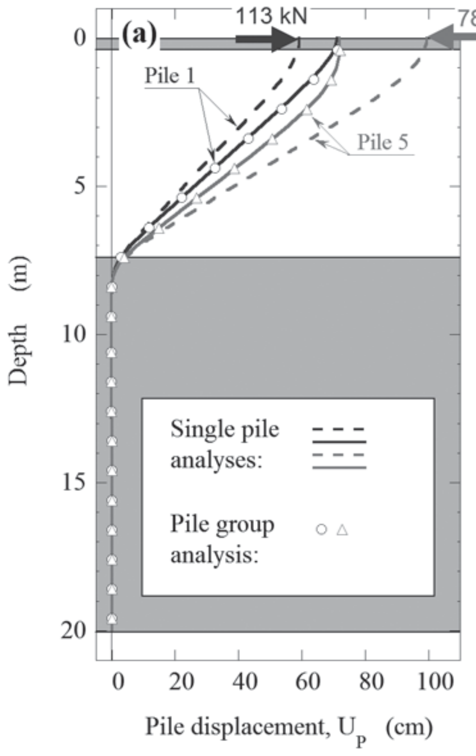
The equivalent linear p-δ relationship for the pile, defined by the degraded stiffness $\beta_2 k_2$, can be easily extended to a bilinear p-δ relationship by using the residual strength of liquefied soils in the definition of the ultimate lateral pressure from the liquefied soil on the pile. The empirical correlation between the undrained strength and SPT blow count proposed by Seed and Harder (1991) can be used for approximating the undrained strength in these calculations.

Pile-Group Effects

Pile groups may generally affect the behaviour of piles in liquefying soils in two ways, first through the cross interaction among the piles within the group, and second, by influencing the key parameters controlling the pile response such as the stiffness of the liquefied soils, and the magnitude and spatial distribution of spreading displacements. Both effects are briefly discussed in the following.

Piles in a group are almost invariably rigidly connected at the pile head, and therefore, when subjected to lateral loads, all piles will share nearly identical horizontal displacements at the pile head. During lateral spreading of liquefied soils in a waterfront area, each of the piles will be subjected to a different lateral load from the surrounding soils, depending upon its particular location within the group and the spatial distribution of the spreading displacements (Figure 11). Consequently, both the interaction force at the pile head and the lateral soil pressure along the length of the pile will be different for each pile, thus leading to a development of distinct patterns of deformation and stresses along the length of individual piles in the group (Figure 12). This response feature, in which the piles share identical displacements at the pile head but have different deformations throughout the depth, is considered to be a significant feature of the deformational behavior of pile groups subjected to lateral spreading. These pile-group effects can be easily captured by a simplified method of analysis using a single pile model (Cubrinovski and Ishihara, 2005).

The second influence of the pile-group regarding its effects on the magnitude and distribution of ground displacements, stiffness characteristics of spreading soils and ultimate soil pressure, is more difficult to quantify.



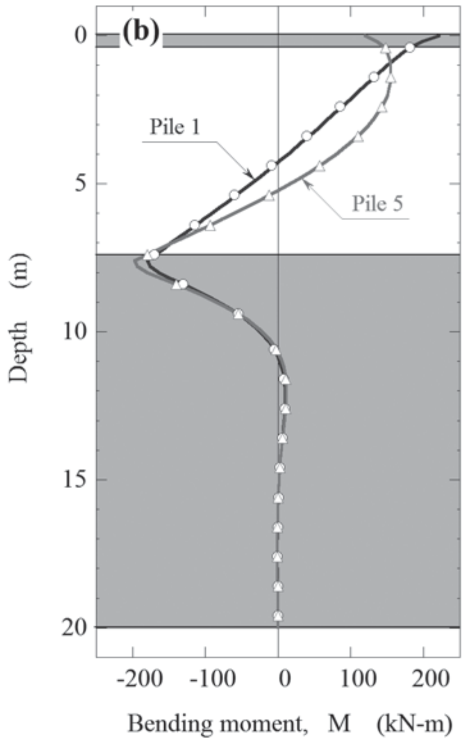


Figure 12: Illustration of cross-interaction effects on end piles subjected to different ground displacements (small ground displacement acting on Pile 1; large ground displacement acting on Pile 5); dashed lines indicate response of individual piles without cross –interaction effects; solid lines indicate response of individual piles including pile-group effects; (a) Pile displacements, and (b) Bending moments

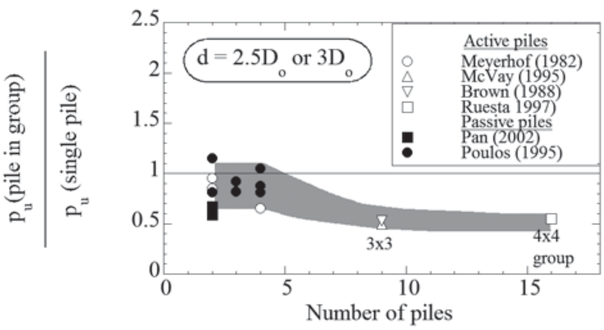


Figure 13: Reduction of lateral soil pressure due to pile-group effects

Experimental data on these effects for piles in liquefiable soils is scarce and not conclusive. Figure 13 illustrates a clear tendency for reduction in the ultimate lateral soil pressure with increasing number of piles within the group, as compared to that of an individual pile. These data are for pile spacing of 2.5-3 diameters, and include both active and passive piles, though the trend is basically derived from active piles. Further evidence for the pile-group effects on key parameters controlling the pile response in liquefying soils such as U_{G2}, β_2 and $p_{1\text{-}max}$ discussed herein is urgently needed.

Summarized Procedure for Pseudo-Static Analysis

For analysis of the cyclic phase of the pile response, key requirement is to concurrently consider the effects of ground displacements and inertial loads from the superstructure, and to properly consider the characteristics of liquefaction and subsequent ground displacements during the cyclic phase of loading. Cyclic ground displacements can be evaluated either by means of an effective stress analysis or by estimating the maximum cyclic shear strain in the liquefied soil based on empirical correlation. The maximum inertial load from the superstructure can be simply approximated as peak ground acceleration times supported vertical load by the pile.

The proposed practical procedure for preliminary assessment and design of piles subjected to lateral spreading can be summarized in the following steps:

1. A simplified three-layer model is developed for the soil deposit, where the liquefied layer is sandwiched between a non-liquefied crust layer at the ground surface and a non-liquefied base layer. The water table

may be used in defining the thickness of the surface layer. Properties of the base layer within 4-6 pile diameters below the interface with the liquefied layer generally control the p-δ relationship of the base layer. A single pile with a nonlinear moment-curvature relationship is adopted in this model.

2. The magnitude of lateral spreading displacement can be estimated using empirical correlations for ground surface displacements of lateral spreads. In view of the uncertainties involved in the assessment of these displacements, a range of values needs to be considered. It is practical to assume a cosine distribution for the ground displacement within the liquefied soil and that the surface layer will move together with the top of the liquefied soil.
3. Initial stiffness in all p-δ relationships can be defined based on empirical correlations between the subgrade reaction coefficient and SPT blow count or elastic moduli. This stiffness should then be degraded in order to account for the effects of nonlinearity and large relative displacements that are required to fully mobilize the lateral soil pressure.
4. Stiffness degradation of liquefied soils is generally in the range between 1/50 and 1/10 for cyclic liquefaction and 1/1000 to 1/50 for lateral spreading.
5. Ultimate lateral pressure from the crust layer can be approximated as being 4.5 times the Rankine passive pressure. Empirical charts for the residual strength of liquefied soils can be used for the ultimate lateral pressure from the liquefied soil on the pile.
6. A static analysis in which the pile is subjected to ground displacements defined in step 2, and adopted stiffness degradation in step 4, is performed and pile displacements and bending moments are obtained. The analysis should be repeated while parametrically varying the magnitude of applied ground displacement and stiffness degradation in the liquefied soil.
7. Pile group effects should be eventually considered including cross interaction among the piles within the group through the pile-cap-pile system, and effects on key parameters controlling the pile response such as the stiffness and strength of liquefied soils, and the magnitude and spatial distribution of spreading displacements. The latter effects may potentially reduce the severity of the ground movement influence.

CONCLUSIONS

Lateral ground displacements of liquefied soils can be quite large during the intense shaking or cyclic phase of loading and especially during the post-liquefaction lateral spreading phase. Since the properties of liquefied soils and loads on piles can be remarkably different during the cyclic phase and subsequent spreading phase, it is necessary to separately consider these two phases in the simplified analysis of piles. When evaluating the pile response during the cyclic phase it is important to consider a relevant combination of kinematic loads due to cyclic ground displacements and inertial loads from the superstructure. In the case of lateral spreading, the uncertainties associated with the spreading of liquefied soils, and in particular, the magnitude and the spatial distribution of spreading displacements, as well as stiffness and strength degradation of liquefied soils need to be carefully considered. The lateral load from a non-liquefiable crust layer at the ground surface may often be the critical load for the integrity of piles subjected to lateral spreading, and therefore, special attention needs to be given to the modelling of the surface layer and its effects on the pile response. Cross-interaction effects may be significant for pile foundations near the waterfront area, where individual piles within the group are subjected to variable ground displacements. Effects of group interaction on key parameters controlling the pile response need to be considered in perhaps reducing the severity of the ground movement effects. Based on these premises, a practical procedure for preliminary assessment and design of piles subjected to lateral spreading has been proposed.

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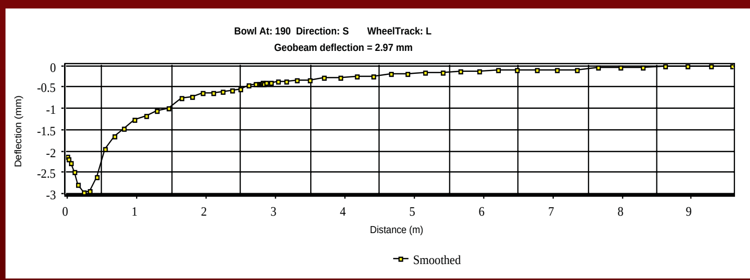

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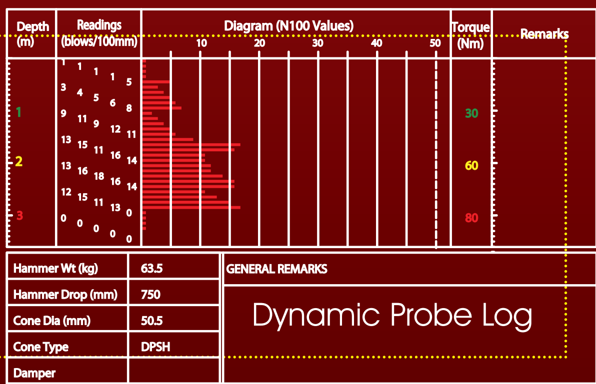

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PROJECT NEWS

Landslides in 2008 –

G.D. Dellow[1], S. Allen[2], S. Cox[1], B. Ferris[1], G.T.Hancox[1], S. McColl[1], S. Nelis[3], N. Palmer[1], N.D. Perrin[1].

Summary

GNS Science recorded over 300 significant landslides in 2008. These occurred throughout New Zealand and were most commonly triggered by individual rainstorms, although prolonged rainfall over several months during the winter in Auckland resulted in widespread sporadic landsliding.

At the start of the year between January and April at least five large landslides occurred in the Southern Alps at Vampire Peak (2), Douglas Peak, Mt Spencer and Mt Halcombe. Two separate rock avalanches were recorded at Vampire Peak on the GeoNet seismograph network in the Southern Alps on 7 and 13 January. These rock avalanches disturbed and/or buried older rock avalanche deposits dating from 2003.

The most significant rainstorm occurred in late July – early August when a storm passed over New Zealand, travelling from Northland to Otago. Landslides were reported in Northland, Auckland, Waikato, Gisborne, Wanganui, Manawatu, Wairarapa, Wellington, Marlborough, Canterbury and Otago regions. The greatest disruptions occurred in the Auckland area, where several roads were blocked, disrupting traffic flows; and in North Canterbury where both the main trunk railway line and State Highway 1 were closed by rock-falls and debris flows at several sites.

Other significant landslides during 2008 included:

- Three debris flows on Mt Taranaki with one damaging the water reservoir intake for Opunake (July);
- A rock slide on the banks of the Shotover River (July –August); and
- A rock topple killing a tramper in the Raukumara Ranges (December).

Vampire Peak Rock Avalanches

Two rock avalanches fell from Vampire Peak (2645 m) onto Mueller Glacier in Aoraki/Mount Cook National Park in January 2008. The avalanches fell from the south face of Vampire. There were no direct witnesses to these avalanches but recordings made by the national seismograph network and field observations allow inference of a more precise event sequence. Avalanche-type shaking of 60 seconds duration, equivalent in amplitude to a magnitude (ML) 2.4 tectonic earthquake, was recorded on Monday 7 January at 2349h (NZDT) and was followed by 45 seconds of shaking at ML 2.5 on Sunday 13 January at 0923h (NZDT). About 150,000 m^3 (±50,000) of rock fell from a 73º slope between 2380-2520 m, in a retrogressive collapse initiating as a joint controlled wedge failure in a prominent sandstone layer. Debris initially travelled down Vampire's south face and across Bannie Glacier, rising 80 m up the southeast spur of Bannie before it was deflected. It then continued down the Bannie Glacier icefall and 1.7 km out across Mueller Glacier (Figure 1).

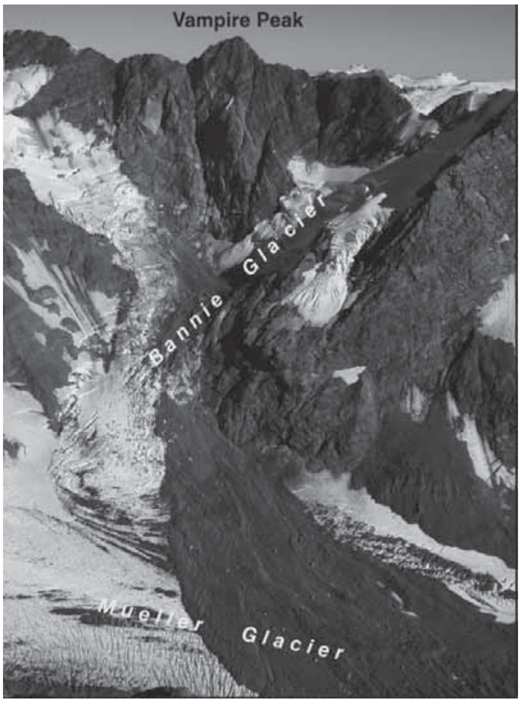


Figure 1: Two rock avalanches fell from the area below and to the left of Vampire Peak (the 2645 m peak top centre) on 7 and 13 January 2008. The rock avalanches travelled down the face of the peak and across the Bannie Glacier before being deflected out across the Mueller Glacier by a spur on the southeast side of Bannie Glacier. (Photo: S. Cox, GNS Science)

The 2008 avalanches produced three distinct debris lobes in the 300,000 m^2 deposit (Figure 1). The 2008 debris deposit overlies most of an earlier (2003) avalanche debris deposit that was subsequently disturbed by glacier movement between 2003 and 2008. Some erosion and remobilisation of the 2003 debris occurred during the 2008 events. Calculated debris deposit volumes can be reconciled with rock failure volumes, given a 20-30% bulking factor, although the amount of erosion of ice and debris along the flow path is poorly defined.

The Vampire rock avalanches of 2003 and 2008 are part of the natural process of erosion in the central Southern

Alps. Other recent rock falls of note are Mount Beatrice (23 November 2004), Douglas Peak (18 February 2008), Mt Spencer (6-7 April 2008) and Mt Halcombe (24 April 2008). The main factors contributing to gravitational failure are steep slope angles, low rock mass strength, accentuated by rock mass dilation (joint opening) and stress relief in response to glacial and permafrost recession in high alpine areas. A period of prolonged warmer weather was recorded prior to the January 2008 Vampire failures which may have lead to snow melt and ingress of water. However, no direct trigger has been identified.

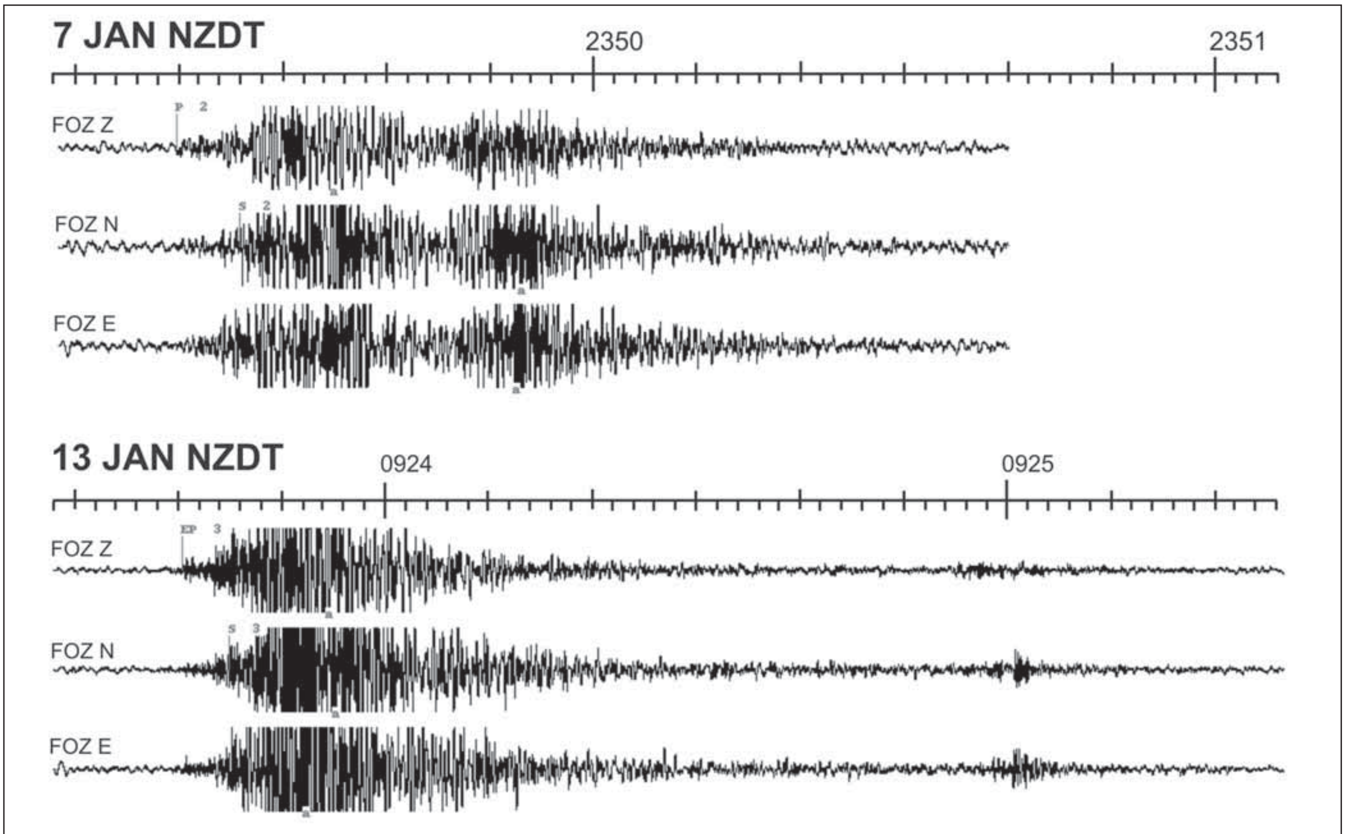


Figure 2: Seismic signals recorded by the GeoNet seismograph FOZ at Karangarua, 27 km to the northwest of Vampire. Times are shown in New Zealand Daylight Time (NZDT) with 2 s time scale divisions. In the 7 January record two distinct pulses can be seen. These two pulses are believed to represent two distinct events producing two of the three debris lobes subsequently observed on the Mueller Glacier (the origin of the third debris lobe is the 13 January rock avalanche).

SHOTOVER LANDSLIDE

On 8 July 2008, a large landslide reactivated near Arthur's Point, on the Shotover River, Queenstown. Initially the landslide experienced down-slope movements of 1700 mm/day, with subsequent movement rates of 100-300 mm/day. As a consequence, the Otago Regional Council and the Queenstown Lakes District Council closed the Shotover River to commercial rafting activities, while the Department of Conservation took the precaution of closing the Moonlight Track (mt on Figure 3) which traverses around the head-scarp of the landslide. Geomorphological evidence indicates that there has been a long history of failures from the side slopes of the Shotover Gorge, with numerous relict scars visible. In July 1893, approximately 12 km north at Maori Point, a landslide blocked the Shotover

Figure 3: The 2008 Shotover River landslide with the main geological and geomorphological features annotated. The head-scarp has formed along the Moonlight Track, (mt) with two main foliation surfaces: fs1, which forms the back-scarp of the slide dipping at 87/074; and fs2, dipping at 40/021 and day-lighting in the main gully (g).

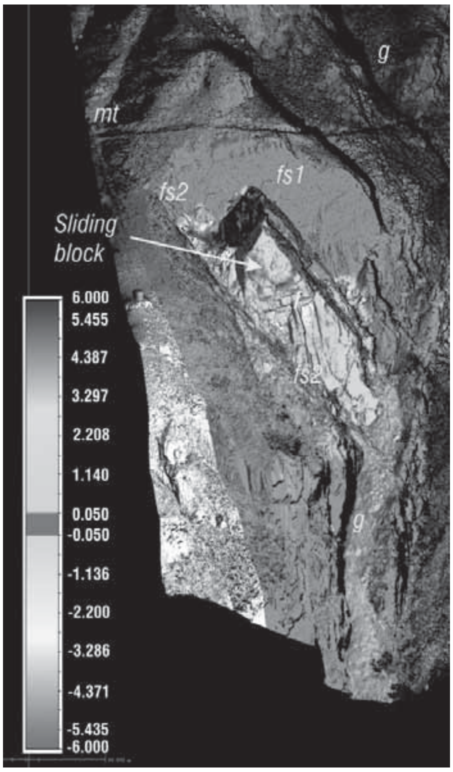


Figure 4: A comparison of the differences between scan images taken on the 24 July 2008 at 1530h and 20 August 2008 at 1300h has highlighted areas where movement has occurred during this time period. Blues show movement towards the observer, i.e. down-slope movement and reds indicate movement away from the observer, i.e. erosion. Maximum down-slope movements over the period were of the order of 6 m. **> Refer to colour images on page 96.**

River temporarily and the subsequent dam breach resulted in the death of 13 miners.

The 2008 reactivation of this landslide involved a combination of block sliding and rock-fall within schist. The block slide is a wedge-type failure, with the failure surface developed at the intersection of two persistent joints within the rock mass (fs1 and fs2 on Figure 3). The head-scarp has formed along a defect dipping steeply towards the Shotover River. Movement of the larger rock blocks mass has led to a number of rock-falls occurring from the top and around the edges of the sliding block.

The landowner first reported tension cracks in the slope below the Moonlight Track approximately 20 years ago in 1988. The long term history at this site is attributed to gradual stress relief and dilation of the slope in response to continued down-cutting of the Shotover River and in the deep gully to the north of the slide.

Monitoring of the landslide was undertaken using the GNS Science owned Riegl 420i Terrestrial Laser Scanner (TLS). Between 23 and 24 July, five laser scans were taken during daylight hours. Comparison between the surveys was used to detect and quantify movement. Local consultants proposed water jacking as a method of removing the sliding blocks and therefore temporarily stabilising the area. A sixth scan was undertaken on 20 August 2008, prior to water jacking operations which began on 22 August at 0630h (NZST). After approximately seven days of water jacking and a period of heavy rain between 21 and 22 August the sliding block failed into the gully (g on Figure 3) with some debris reaching the Shotover River.

In the TLS image (Figure 4), blues show movement towards the observation position, i.e. down-slope movement, while reds show movement away from the observation position, i.e. erosion. Down-slope movements are the result of continuing movement of the failed block, while erosion is caused by the detachment and ravelling of blocks from the highly dilated and broken rock mass. The accumulation and transport of debris within the gully to the right of the slip is clearly shown, with very little material having reached the debris fan. Areas with less than 50 mm of movement are shown in grey and are considered essentially stable, with no evidence of clearly detectable movement. The mottled greens and yellows to the left of the image above the Moonlight Track are caused by vegetation introducing 'noise' between successive scans.

Rainfall-induced landslides in Auckland

The New Zealand MetService reported that the winter of 2008 was very stormy and wet, with high rainfall and floods in many areas. From June to August, rainfall in Auckland was 150% above average. Not only were the monthly rainfall totals far greater than average, but the number of days with rain was also much higher. The increased rainfall caused numerous landslides, closing roads, destroying several houses, and causing damage worth several millions of dollars. Under the GeoNet Project, GNS Science undertook data-gathering aerial and ground inspections of some of these landslides.

On 29-30 July 2008, 65 mm rainfall in Auckland caused widespread flooding and numerous damaging landslides. A house in Torbay on Auckland's North Shore was evacuated and later demolished because of continuing damage by slow ground movements. Several other houses in the area were also affected and evacuated temporarily.

The steep cliffs on Auckland's east coast and the houses situated on them were significantly affected by landslides, both first-time failures and reactivation of older landslides, caused by prolonged rainfall. Although most of the cliff failures were relatively small, a slow-moving landslide at Buckland's Beach (Figure 5) resulted in serious damage to one house, which had to be evacuated along with several others. The period of reactivation was attributed to 116 mm rainfall over two weeks from mid-late August. Excavation and building too close to the unstable cliff edge, however, seems to have provided the basic preconditions for slope failure at this site, as it has at a number of other locations on Auckland's east coast cliffs.

Figure 5: This innocuous landslide (lower left) severely damaged the house (top centre) and two houses on either side. These three houses remain evacuated (as at April 2009) as slope movements continued after more rain fell in December 2008. Although rainfall seems to have triggered the landslide movements, excavation and building too close to the unstable cliff edge, especially the yellow house (star indicates) and the large house to the north (right), appear to be important underlying reasons for the failure.

On Sunday 24 August, the prolonged rainfall triggered a 500 m^3 landslide on Turei Hill, closing the only access road to the coastal settlement of Kawakawa Bay, 40 km southeast of Auckland. Three properties were found to be at risk because of a developing, slow moving large landslide above the road near the coast. The road was closed for about a month while drainage works and ineffective sluicing was carried out to dislodge partially failed material poised precariously above the road (Figure 6). Slope movements slowed and stopped once the August rain had eased, allowing the road to be reopened on 25 September and easing the burden on Kawakawa Bay residents of having to make a 100 km detour, or a long walk over the hill, to go to work.

Figure 6: A landslide blocked the road at Kawakawa Bay for several weeks during August and September 2008. Although this failure was triggered by prolonged rainfall during August, it is actually a toe reactivation of a much larger old landslide, the head-scarp of which is behind the house at the top of the slope. The small pond (brown semi-circular feature, upper left) is a landslide pond formed at the back of an old slumped block. This landslide illustrates the danger of constructing an unsupported road cut across the toe of an old landslide.

Although prolonged and higher than average rainfall in Auckland during the winter of 2008 triggered many landslides across a wide area, other factors appear to have provided the preconditions for the slope failures. Foremost amongst these is the desire to build houses in positions that have great views, but invariably are sited too close to the edge of unstable cliffs that are prone to slope failure and erosion.

More detailed reports on these and other landslides are available on the GeoNet website: www.geonet.org.nz/landslide/resources

1. GNS Science; 2. University of Canterbury;
3. Beca, Auckland

Project West Wind: Geotechnical Inputs to New Zealand's Latest Wind Farm Project

Photo 6: Turbine in operation, with turbine blade storage area at bottom of photograph

In September 2007 construction began on Meridian's new 62 turbine wind farm at Makara, west of Wellington city. The site is renowned for its strong and constant winds, due to the funnelling effect of Cook Strait. Once completed later this year, the wind farm will generate enough energy to power up to 70,000 average homes.

The turbines will each be 111 m tall, and are supplied by the Danish company Siemens Wind Power. Each turbine foundation is 15 metres in diameter and contains 48 tonnes of reinforcing steel within 370 cubic metres of concrete. The turbines are connected to a central substation by underground cable routes.

Turbine components are being delivered to site from Picton via a new temporary berthing on the south coast (at Oteranga Bay). Groups of turbines are being commissioned as they are constructed, with all 62 of the turbines expected to be turning by late 2009.

The construction phase has been successfully led by the Meridian site management team along with Higgins Contractors and their subcontractors. Opus International Consultants (Opus) provided geotechnical and structural advice during construction, including carrying out inspections on foundation and crane platform subgrades and road cuttings.

Photo 1 (above): Turbine foundation excavation ready for geotechnical inspection

Photo 2 (top): Turbine foundation prior to concrete pour

The large scale of the project (including 33 km of new road) meant the programme of geotechnical testing had to be undertaken in a timely manner, in line with the tight earthworks timeline. It was critical the construction team were supplied with geotechnical information as soon as it

Photo 3: Turbine blades being lifted into place

was available, to minimise any delays.

Opus' involvement with Project West Wind began in 2006, when a desk study, extensive engineering geological mapping and site investigations were carried out. The site investigations involved the drilling of boreholes, excavation of trial pits and geophysical surveys at selected turbine sites, road cuttings, the substation site and the Oteranga Bay wharf. A geotechnical assessment was then carried out to determine founding ground conditions for the turbines and the stability of turbines in very steep topography. Appropriate cut slope angles were also assessed and design considerations made for fill site construction.

As part of the geotechnical assessment, turbine sites and intervening access roads were characterised into rock and soil categories, allowing predictions of founding conditions, excavatibility and cut slope stability. Proposed cut slopes were characterised on the basis of factors such as geology, slope heights and angles, previous slope failures and groundwater. This enabled a GIS analysis of slope failure susceptibility across the site.

On-site geotechnical input during construction allowed confirmation of founding conditions for each turbine and crane platform (large cranes used for erecting the turbines weigh approximately 400 tonnes). Regular visual

Photo 4: Road cuttings on the site are up to 25 m high, in weathered greywacke

Photo 5: Borehole being drilled by Griffiths Drilling (NZ) Ltd during site investigations phase

monitoring of road cuttings was also carried out, to enable identification of unstable slopes. This allowed high risk areas to be identified and reported to Meridian in advance of failures, so pre-emptive measures could be taken to improve the stability of cuttings.

Post-construction geotechnical inputs will include an assessment of the long term risk to roads, cable routes and turbine sites.

As with any wind farm site, strong winds often contributed to some challenging working conditions for both the Meridian team and their contractors, however the construction timeline has remained on track for a successful final commissioning of Project West Wind, and another step in New Zealand's move towards greater sustainability in power generation.

Reported by:
Jayne Hodgkinson
Engineering Geologist
Opus International Consultants

Pike River Coal Mine defies the odds to strike coal

A major milestone was reached for the underground Pike River Coal Mine on 3rd November 2008, when the mine access tunnel reached Chainage 2,300 m – the point where underground coal mining commenced. Once in full production this mine will be the second largest coal export mine in the country and the largest underground coal mine, exporting about one million tons of coal per year over its planned 20-year life.

Owned by Pike River Coal Limited (Pike), the mine is located within the Paparoa Ranges, about 40 km northeast of Greymouth. Due to the rugged alpine topography, with beech-covered mountain slopes at 50 degrees, the mine is fully underground, reached only by a 5.5 m wide, 2.3 km long tunnel. The site was remote - no access roads and no human activity for miles. It is picture perfect country, perhaps due to the fact that this clean & green part of NZ receives around 6 metres of rainfall a year.

The tunnel, associated pit-bottom workings and a 108 m vertical, 4.15 m diameter ventilation shaft form a parcel of work awarded as a design-build contract to McConnell Dowell Constructors Limited, with URS New Zealand Limited (URS) as the design consultants.

Pike was severely restricted from any activity that could cause any land disturbance in the forest areas and no exploratory drilling could be carried out along the tunnel route due to the high relief terrain. In a separate contract, Pike commissioned the construction of a 14 km access road to the tunnel portal. The developed portal area (Photo 1) is only the size of a tennis court, yet through innovative planning, holds the vital infrastructure required for tunnelling.

Obtaining Geotechnical Information

Contractors tendering for the tunnel were provided with a geotechnical report based on walk-over field work. The forest at the site is dense and near impenetrable. To obtain geotechnical information for tender and construction purposes, rock exposures were mapped within river beds and in occasional fault-generated small cliff faces. Photo 2 shows a typical river bed scene and what has to be contended with in the field. Through this field mapping exercise, information was obtained on the structural geology – the location of some faults, the identification of joint sets in the rock and the condition of the joints, as well as some indication of rock strengths by sampling river boulders and having them tested in a laboratory.

Portal and Tunnel Design

The unique initial challenge to the URS team was designing the portal, tunnel and shaft with no subsurface geotechnical information. A geotechnical team was taken to the portal by helicopter and made an assessment of the portal area and of a 50 m zone beyond the portal crest to evaluate landslide risk. This team was led by a mountaineering guide and included abseiling gear for cliff face work.

Phot1: Mine portal aerial shot.

With the portal addressed, attention shifted to the tunnel design – how do you demarcate structural zones and associated conditions and provide suitable rock support for them without sub-surface data? How do you plan and provide for the associated mining cycle times so that your design-build team receive adequate compensation for different rock conditions and the associated slower progress that will result? How much reliability can be placed on the geological report that indicated that in excess of 90% of the tunnel would be in good quality rock that would either be self supporting or require nominal support only?

A system of ground support that would cope with a range of rock conditions was developed. Using in particular the NGI Q-Index rock mass classification system, different mining conditions and their associated work quantum and support requirements were evolved to cover a spectrum of conditions and generic designs were prepared to address them. During tunnelling these generic systems were employed, with each advance being based on face assessed conditions. In many instances support and mining methods had to be revised. A rapid response system was developed, so that a methodology was in hand to address changed conditions and to formulate a new form of rock support. These were made available within 3 to 12 hours throughout the 24 hour / 7 day construction period. In terms of rock mass classification systems, the rock encountered may be described as ranging between 'fair' to 'poor' and 'very poor'.

With the largest town on the West Coast boasting around 12,000 people, resources are limited. What is not available must be imported with long lead times. 'Kiwi ingenuity' was often required to overcome unplanned requirements. For example the design of 4 m long grout bar for use in poor ground conditions. By fixing a basal grout entry tube

Photo 2: Pike River boulders shot

to a bolthole end seal and a top-entry grout bleed tube to a 25 mm Reidbar, a full column cement grout dowel could be installed. The grout was pressure pumped from the base until it flowed out from a bleed tube fixed to the top of the bar. The grout take in the poor fractured ground was high at 4 to 7 times the hole volume. The cost implications can be surmised, but the net impact on the rock mass is a gratifying risk reduction through the cementing action achieved.

Fault Crossing and Pit Bottom

Tunnelling had to traverse the Hawera fault, which separates the Pecksniff meta-sedimentary gneiss from the coal measures. With more than 1000 m of displacement between these two rock bodies it was recognised that the rock close to the fault would be extremely damaged by tectonic activities. It was a great relief when penetration through the fault was achieved without incident. In parts of the tunnel (see cover photo) water inflows appeared like rainfall due to the many discontinuities and rock fractures. Fortunately ground conditions in the Hawera fault were found to be essentially dry and only minor gas from coal stringers was encountered.

Every mine requires work areas where plant can be serviced and where materials handing takes place. The selected methodology at Pike is to slurry the coal and pump out the product to a processing plant some 5 km from the workings.The work zone where these operations take place is called the Pit Bottom. It may loosely be described as a warren of interlinked adits and caverns. Drilling for coal sampling completed in parallel with tunnel construction led to a decision by the mine to construct part of the pit bottom in the better quality meta-sedimentary gneiss and not within the coal measures. This increased by 42% the contract volume of rock removed during tunnelling.

The largest cavern constructed was the Pump Hall at 8m wide by 40 m long and 12 m high. The numerous sumps and link drives required careful planning as multiple cuts were necessary and a sequencing error would have grave consequences for constructability. This additional work virtually doubled the design input, with many intersections and adits of different widths and heights being required. In Photo 3 a view is given across from the Pump Hall towards one of the coal slurry sumps, a distance of some 80 m. Some of the adjoining adits and the different levels across this length can be discerned from a careful examination of the different light sources.

Ventilation Shaft

To supply ventilation requirements to the mine, a 4 m

Photo 3: View of Pit Bottom across the pump hall

diameter vertical shaft is required. At the selected location, the shaft is about 108 m deep. The ventilation shaft daylights on the side of a 50° slope. A shaft top platform has been built to house ventilation equipment and for shaft construction needs, no small ask in the mountainous terrain, where all plant and materials reach the site by helicopter. The optimised low environmental impact structure is a combination of a mass block retaining wall cut-fill exercise combined with a HIGHBOND deck supported on steel stanchions. This structure boasts possibly the country's most expensive concrete with air freight resulting in a rate of around $3,000 per cubic meter! The shaft is currently under construction. Photo 4 provides a peek down the shaft to the working platform from which rock support is being installed.

Health and Safety

Despite very challenging rock conditions, the work has been completed without a single rock-related injury, a tribute to the site staff, in particular Seth Tiddy, the URS on-site Engineering Geologist who in addition to his main duties has also provided strict health and safety discipline related to strata support for the last 2 years.

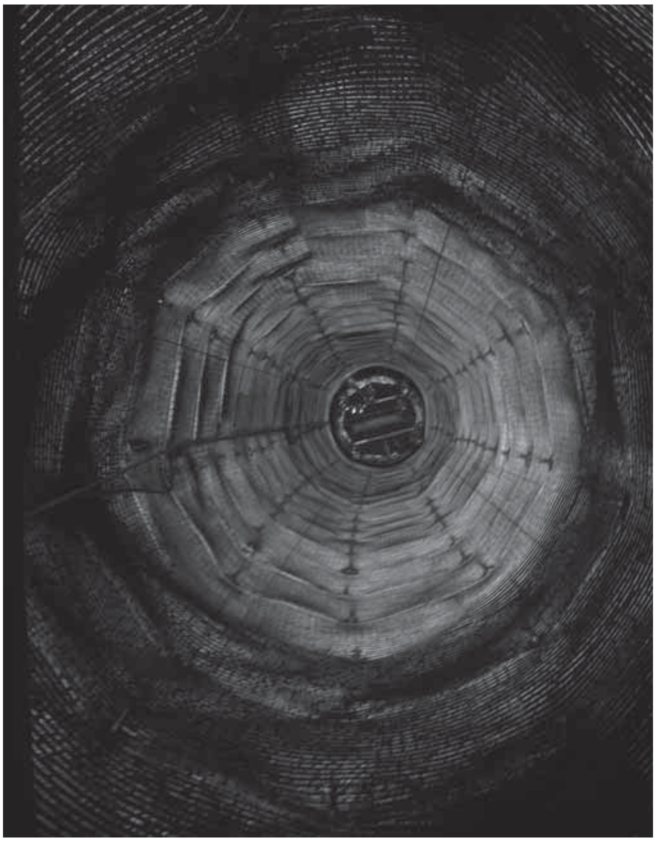


Photo 4: View down the ventilation shaft showing support being installed

Reported by:
Evan Giles
Principal Geotechnical/Tunnelling Engineer
URS New Zealand Ltd

Bridge 60A Rail Line Extension Swanson, Auckland: Reinforced Soil Slopes – Case History. Maccaferri NZ Ltd

Above: Site prior to construction Date: Oct 2007,
Below: Lower prefilled gabion section Date: Nov 2007

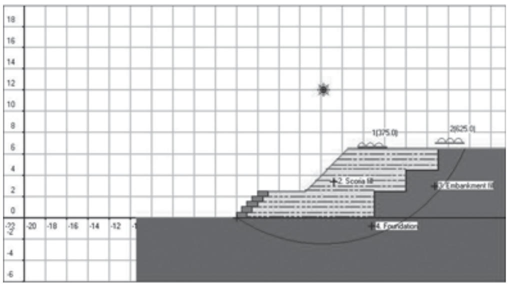
Problem

Part of the ongoing Ontrack New Zealand double rail track upgrading works for the Auckland Region included the requirement to widen the bridge across Candia Road to cater for an additional railway lane. The site had limited space available for the construction of a shallow slope between the existing railway embankment and stream adjacent to the bridge.

A steeper reinforced soil embankment was chosen to solve the problem of space as well as having the ability to support the heavy loads expected during placement of the bridge elements.

The choice of facing also presented some problems as there were not only the technical and durability issues to be considered but also speed of construction when working adjacent to a working rail network.

Left: Placement of geogrid and fill Date: Nov 2007



Above: Section showing a typical circular analysis for crane loads Date: October 2007

Left: 300 t crane operation Date: Nov 2007

Client name: **ONTRACK NZ**
Main contractor name: **ROGERS EARTHWORKS**
Construction date: **NOVEMBER 2007**
Consultant: **FRASER GEOLOGICS**
Product used: **GABIONS, TENSAR RE GEOGRID**

Solution

The design of the 7 m high reinforced soil embankment considered the short term load case for a 300 t crane on the level platform, the critical case being 625 kPa on one leg, as well as the long terms loads that include a 4 m high vertical faced anchored earth wall to be constructed at a later date on the platform for the purpose of the rail line extension.

A number of stability checks had to be considered to take into account the various potential failure modes and external load cases including:

- Internal & Compound Stability - the critical circles from the wall toe are forced back into the retained soil zone due to layers of geogrid reinforcement
- Global Stability - this is to investigate deeper seated potential failure circles behind the reinforced soil block and anchored earth block
- Wedge Stability - potential sliding at the base

Construction

Pre-filled gabions with ballast stones were selected for the facing of the structure. These could be mechanically placed to form the steeper face for the lower 2.5 m within the design flood zone and a 45 degree slope constructed on top to provide a platform sufficiently wide for a 300 t crane to be positioned in place and operate from.

The placement of the pre-filled gabions provided a rapid method of front face construction. A highly frictional and free draining scoria fill along with Tensar geogrid reinforcement ensured that construction could continue in a range of weather conditions with minimal interruption to the construction program.

Gabions were pre-filled so that they were not within the critical path of the project installation schedule.

No deformations or distress to the structure was observed during the critical phase of placement of the bridge elements. These observations supported the design principles adopted for this site.

TECHNICAL ARTICLES

Innovative Thermal Consolidation Technique for Soft Soils – Hossam M. Abuel-Naga[1]

Abstract

Several research works have demonstrated that subjecting normally consolidated clays to temperature less than the boiling point of water (100˚C) will have positive effects on its hydro-mechanical behavior. Such effects can be exploited in improving the performance of the well-known preloading ground improvement technique that utilizes prefabricated vertical drains (PVD). In this paper, the applicability of a novel thermal consolidation technique will be discussed using results of large oedometer test and full-scale embankment test on soft Bangkok clay. The large oedometer test results gave a promising outcome since the temperature accelerates the rate of consolidation and increases the amount of total settlement. The viability of the proposed technique was also confirmed by the full-scale embankment test results. The success of the proposed technique can be attributed to the thermally induced volume change and the increase in the hydraulic conductivity as the soil temperature increases.

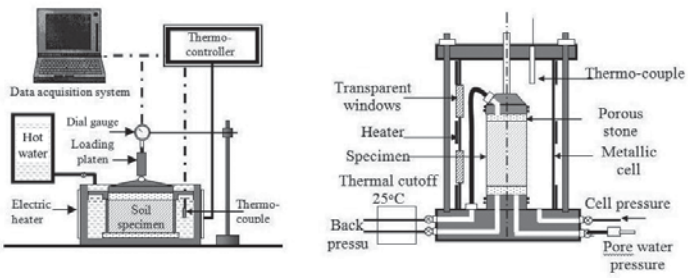
Keyword: Thermal consolidation, temperature effects, Bangkok clay, prefabricated vertical drain (PVD), ground improvement

Introduction

Construction of road embankments on top of soft normally consolidated clay deposits requires pre-consolidation and strengthening of the weak compressible soils. Prefabricated Vertical Drains (PVD) are a time tested, very effective and economical ground modification technique in such deposits. However, the installation of prefabricated vertical drains using a mandrel causes disturbance of clay surrounding the drain, resulting in a smear zone of much lower horizontal permeability of this clay. The presence of a smear zone significantly influences the horizontal consolidation, resulting in retardation of the overall consolidation rate. The long duration required to accomplish the ground improvement using PVD is the main disadvantage of this technique.

The aim of this paper is to investigate the effect of soil temperature on the performance of preloading with PVD ground improvement methods using a large oedometer apparatus and full-scale embankment test. The experimental program of the large oedometer test was designed to understand the thermo-mechanical consolidation behavior of soft Bangkok clay using PVD, a line heat source, and prefabricated vertical thermal drains (PVTD) with different arrangements. To look into the field feasibility of this innovative thermal technique, two identical 6.0 m high full-scale test embankments for preloading were constructed over the soft Bangkok clay. A conventional PVD system was installed underneath one embankment and the novel PVTD system was utilized for the other.

In the following sections, a brief background pertaining to the thermo-mechanical behaviour of soft Bangkok clay is presented based on the extensive experimental studies by Abuel-Naga et al. (2006b; 2007a,b). After this, large oedometer tests conducted by Abuel-Naga et al. (2006a) are described and discussed. Then, the consolidation behavior of the full-scale embankment test on soft Bangkok clay using PVTD is presented and discussed. Finally, conclusions are drawn.



a) Modified oedometer cell b) Modified triaxial cell

Figure 1: Schematic diagram of test apparatus.

Thermo-mechanical Behavior of Soft Bangkok Clay

An intensive experimental study has been conducted by Abuel-Naga et al. (2006b; 2007a,b) to investigate the thermo-hydro-mechanical behaviour of soft Bangkok clay using a modified oedometer and triaxial apparatus as shown in Fig. 1. Moreover, a robust constitutive model for predicting the temperature effects on saturated clays was also proposed by Abuel-Naga et al. (2007c; 2009b). Soft Bangkok clay samples obtained from 3.0 to 4.0 m depth have been used in these studies. Table 1 shows the physical properties of soft Bangkok clay. XRD analysis shows the mineralogical composition of soft Bangkok clay consists of Smectites (Montmorillonites and Illites) ranging from 54 to 71% with Kaolinites (28 to 36%) and micas.

The test results from a modified oedometer test where the specimen temperature was raised to 90ºC under fully drained constant stress condition, have shown that the thermally induced volume change is stress history dependent as illustrated in Fig. 2. The normally

Table 1

Physical properties of soft Bangkok Clay	
Liquid limit (%)	103
Plasticity index	60
Water content (%)	90-95
Liquidity index	0.62
Grain Size Distribution	
Clay (%)	69
Silt (%)	28
Sand (%)	3
Total unit weight (kN/m^3)	14.3
Dry unit weight (kN/m^3)	7.73
Specific gravity	2.68
Specific surface area (m^2/g)	237

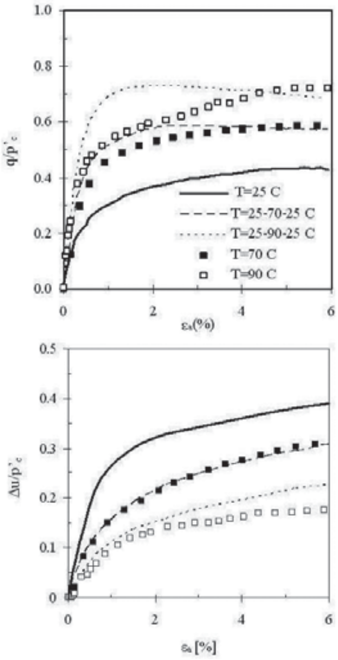


Figure 5: Undrained triaxial compression test results of normally consolidated soil tested at different temperatures or after subjecting to different heating/cooling cycles.

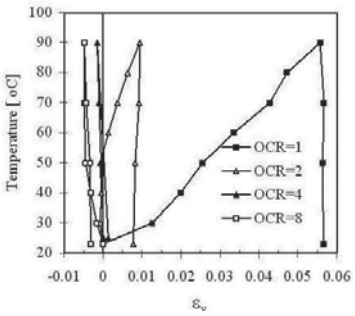


Figure 2: Soft Bangkok clay temperature volumetric strain under drained heating/cooling cycle at different OCR values (preconsolidation pressure = 200 kPa).

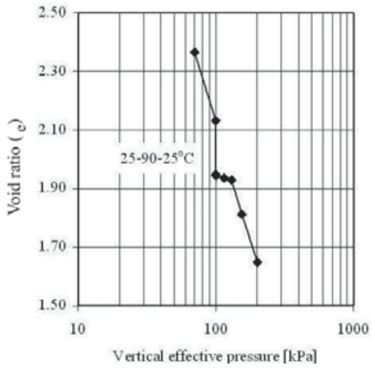


Figure 3: Temperature induced overconsolidation state of normally consolidated soft Bangkok clay after drained heating/cooling cycle.

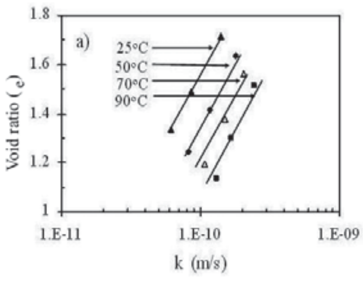


Figure 4: Effect of temperature on hydraulic conductivity of soft Bangkok clay.

consolidated clays contracted irreversibly and non-linearly upon heating whereas the highly overconsolidated clays exhibited reversible expansion. Moreover, an apparent overconsolidation state was observed after subjecting the normally consolidated specimen to a heating/cooling cycle as shown in Fig. 3. The effect of temperature on hydraulic permeability of soft Bangkok clay was also investigated by Abuel-Naga (2006b). Flexible wall permeameter tests were conducted at different temperatures up to 90°C. The results indicated that as the soil temperature increased, the hydraulic conductivity also increased as shown in Fig. 4. This behaviour was attributed to the thermal evolution of the pore soil liquid viscosity.

Abuel-Naga et al. (2006b; 2007a) investigated experimentally the effect of temperature on the undrained triaxial compression shear strength behaviour of normally consolidated soft Bangkok clay specimens at different temperature levels and histories. Temperature histories relate to specimens being subjected to heating/cooling cycles before conducting shear testing. The test results indicated that the undrained shear strength and secant modulus of the normally consolidated clay increases as the soil temperature increases or after subjecting to a temperature history as shown in Fig. 5.

Large Oedometer Test

Abuel-Naga et al. (2006a) investigated the effect of soil temperature on the performance of PVD using a large oedometer apparatus as shown in Fig. 6. The height and inner diameter of the odemoeter cell was 200 mm. Dead load was used to apply the vertical stress. The soil temperature was raised using a line heat source either attached to the

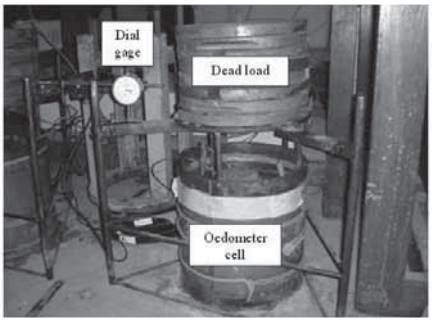


Figure 6: Large oedometer apparatus

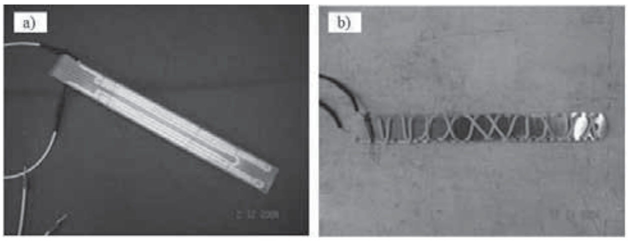


Figure 7: a) Thermo-PVD configuration; b) Line heat source configuration

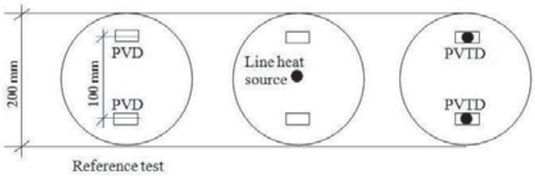


Figure 8: Thermo-mechanical consolidation test configurations

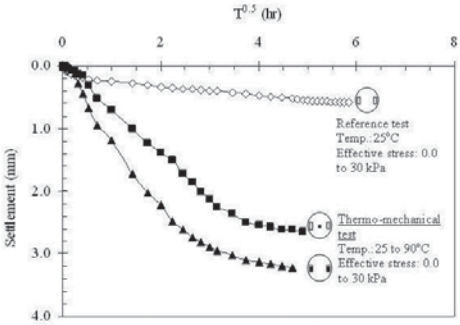


Figure 9: Thermo-mechanical consolidation test results of undisturbed specimen

PVD point (PVTD) or installed independently between the PVD points. The scaled-down PVDs were created by disassembling, cutting, and reassembling the full-size drains. The core was cut to 20 mm in width and about 200 mm in length. The PVTD was created by using two scaled-down PVD cores fitted back to back with a flexible wire heater (2 mm in diameter) placed in the grooves as shown in Fig. 7a. The separate line heat source was created by wrapping a flexible wire heater around a metal plate 20 mm wide and 200 mm long as shown in Fig. 7b. For both types of line heat source, a thermocouple (K-type) was placed at the mid-height of the line heat source with direct contact with the surrounding soil. This thermocouple was used for both temperature measurements and the feedback signal for the thermo-controller unit. Figure 8 shows different arrangements of PVD, PVTD, and line heat source that were investigated under thermo-mechanical consolidation, with simultaneous increases of soil temperature (from 25 to 90ºC) and effective stress (from 0.0 to 30 kPa). Reference testing was also conducted where only the vertical effective stress was increased from 0.0 to 30 kPa while the soil temperature was not changed. The settlement induced by mechanical (reference test) and thermo-mechanical path was measured with time.

The results in Fig. 9 show that raising the soil temperature increases significantly the consolidation rate. Therefore, the thermal consolidation method can be considered a promising approach since it enhanced the performance of preloading with PVD by reducing the consolidation time. The results in Fig. 9 also indicate that using PVTD is preferable than the separate line heat source since the thermo-PVD induced higher consolidation rate. The advantage of the PVTD can be attributed to the coincidence of the drainage point and the smear zone at the center of the maximum temperature zone. Consequently, significant reduction of the smear effect can be achieved in this case since raising the temperature increases the hydraulic conductivity of soils.

Full-scale Embankment Test

Embankment Site

The site of the embankment tests is located inside the campus of the Asian Institute of Technology (AIT), which is 42 km north of Bangkok, Thailand. AIT is located within the Central Plain of Thailand which contain the deltaic-marine deposit of soft clay widely known as "soft Bangkok clay". The typical stratigraphy at the location of AIT consists of an upper weathered crust of dark brown clay from the ground surface to about 2.0 m depth. This layer is underlain by soft, highly compressible, gray clay with fissures, silt seams and fine sand lenses down to about 8.0 to 9.0 m depth. Below the soft clay layer lies about 6.0 m of stiff clay. Then, dense to very dense sand and gravel layers alternate with stiff to hard clay layers starting at 14.0 m to about 400 m depth. The ground water table fluctuates with the season but is close to an average value of 2.0 m below the ground level.

Embankment Test Construction

Two identical 6.0 m high full-scale embankments were constructed at the AIT site where the distance between them is 60 m. Figure 10 shows the general layout of the constructed embankment. The dimensions of the

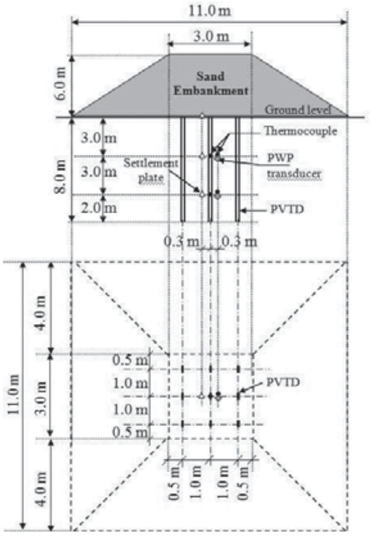


Figure 10: Layout of full-scale PVTD embankment

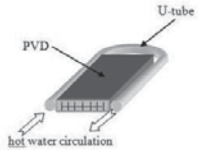


Figure 12: PVTD configuration

Figure 11: Use of geogrid, geotextile, and sand gabion blocks for increasing the slope of embankment

embankments are 11 m x 11 m at the bottom and 3 m x 3 m at the top. Fill material consisted of compacted Ayuttaya sand. The embankment fill material was compacted by a vibratory hand compactor. Geogrid and geotextile were used for increasing the slope of embankment. Sand gabion blocks were also used along the upper part of embankments as shown in Fig. 11. A conventional PVD system was installed underneath the first embankment whereas a novel prefabricated vertical thermo-drain (PVTD) system was utilized for the second one.

Nine PVD/PVTD's 8.0 m deep were installed beneath the embankments on a square grid of 1.0 m spacing. A commercial PVD with 100 mm × 4.3 mm cross-section was utilized in this study. The PVTD unit consists of a U-tube made of cross-linked polyethylene plastic (PEX) attached to a conventional PVD unit as shown in Fig. 12. Preheated water at about 70 to 90ºC is circulated through the attached U-tube to raise the soil temperature underneath the embankment. A solar panel system was used to heat the circulated water from ambient temperature (25ºC) to 72ºC then an electrical heater was utilized to raise its temperature from 72ºC to 90ºC. A special water pump able to work at elevated temperatures was used to circulate the hot water through the PVTD system.

The monitoring system of PVTD embankment consisted of settlement plates installed 0.3 m away from the central PVTD point at three different depths (0.0, 3.0, 6.0 m) as shown in Fig. 10. On the other side of the central PVTD two pairs of thermo-couples were installed at 3.0 and 6.0 m depth. Each thermocouple pair consisted of one thermocouple attached to the outer side of the U-tube whereas the second was located 0.3 m apart from the PVTD unit as shown in Fig. 10. Furthermore, two pore water pressure (pwp) transducers were installed 0.3 m away from the central PVTD point at two different depths (3.0, 6.0 m) as shown in Fig. 10. The monitoring system of the PVD embankment was similar to the PVTD embankment except for the thermo-couples, as one thermo-couple was installed only 0.5 m away from the central PVD point at 3.0 m depth. Moreover, two additional pore water pressure transducers were installed 50.0 m away from both embankments at two different depths (3.0, 6.0 m) to record reference pore water pressure at the site. The testing program of the PVD and PVTD embankments included; the embankment building stage, the mechanical consolidation stage for the PVD embankment and thermo-mechanical consolidation stage for the PVTD embankment. During these stages, temperature, pore pressures, and settlement readings were collected at different time intervals. The thermo-mechanical consolidation stage involves circulation of hot water (70 to 90ºC) through the PVTD system. The heating stage lasted for 110 days until the primary consolidation was completed. Then, the whole system was left to cool down for about 90 days.

Heat Transfer

The temperature at the PVD embankment test was approximately 25±1ºC during the test period. This observation confirms that the heat radius of influence of the PVTD embankment is less than the distance between the two embankments (60.0 m). Figure 13 shows the PVTD embankment temperature history at 3.0 m depth during the testing period. The first curve (solid line) shows the interface temperature between the PVTD point and the surrounding soil whereas the second curve (dashed line) shows the soil temperature 0.3 m away from the central PVTD as shown in Fig. 10. The observed temperature fluctuation was due to a technical problem that occurred in the electrical heating system during the test period.

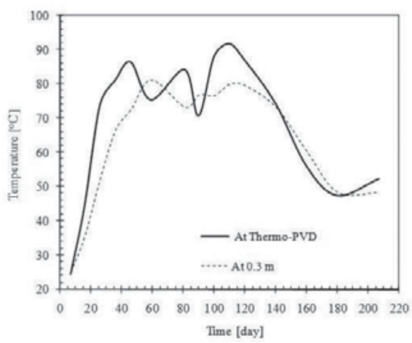


Figure 13: Typical temperature history of PVTD embankment at 3.0 m depth.

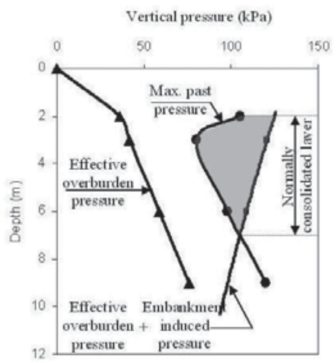


Figure 14: Stress condition under the center line of the test embankment.

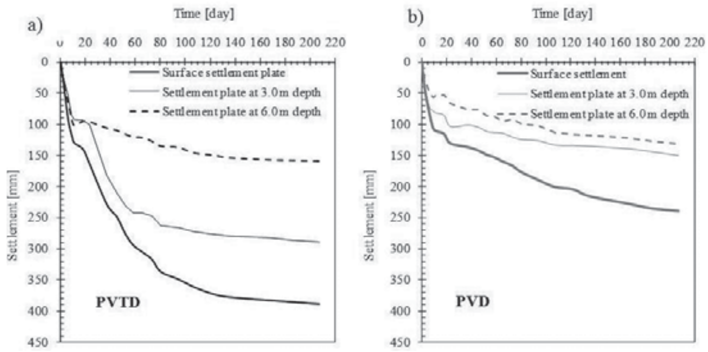


Figure 15: Settlement results of PVD and PVTD embankment.

Abuel-Naga et al. (2009a) used these results to numerically back-calculate the thermal conductivity of soft Bangkok clay. The numerically obtained thermal conductivity value (1.3 W/m°C) was very close to the laboratory determined value (Abuel-Naga et al. 2008, 2009a).

Consolidation Results

Figure 14 shows the initial and final stresses under the center line of the test embankment and the clay maximum past pressure as well. The stress condition indicated that the soft clay layer located between 2.0 to 7.0 m depth is normally consolidated. The settlement results of both embankments are plotted in Fig. 15. In general, the PVTD embankment yields more settlement.

The analysis of settlement data of the layer from 0.0 to 6.0 m depth is illustrated in Fig. 16. The change in the thickness of this layer (ΔH) was calculated using the measurements of the settlement plate at surface, S_o, and 6.0 m depth, S_6, ($\Delta H = S_o - S_6$) as shown in Fig. 16a. The excess pore water pressure measurements at 3.0 m depth were used to determine the end of the primary consolidation stage as shown in Fig. 16b. The PVTD embankment shows higher excess pore water pressure than the PVD embankment due to thermally induced pore water pressure and volume

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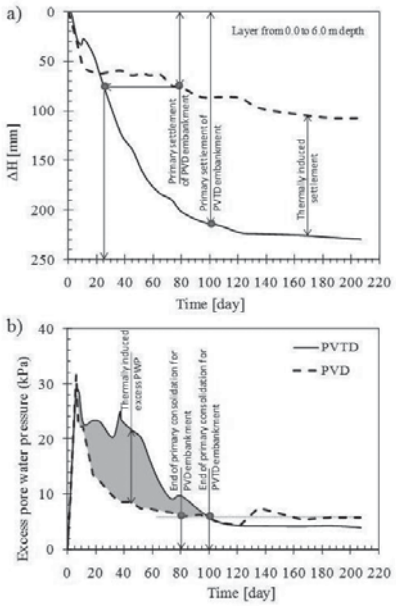

Figure 16: PVD and PVTD embankment results, a) consolidation of the layer from 0.0 to 6.0 m depth, b) excess pore water pressure at 3.0 m depth.

change (Abuel-Naga et al. 2007a). The thermally induced pore water pressure is generated due to the difference in the thermal expansion coefficient of water and the soil solids. The thermal expansion coefficient of water (α_w= 1.7 x 10^{-4} $^oC^{-1}$) is approximately 15 times larger than thermal expansion coefficient of solids (α_s= 10^{-5} $^oC^{-1}$).

The difference in ΔH between the two embankments is attributed to the thermal consolidation at the PVTD embankment, which shows irreversible behavior upon soil cooling. As the heating/cooling process of normally consolidated clay changes it to lightly overconsolidated clay (Abuel-Naga et al. 2006b), the clay underneath the PVTD embankment can carry extra load with less settlement. The results in Fig. 16a also show that the consolidation rate of the PVTD embankment is higher than the PVD embankment. The amount of consolidation generated by the PVD embankment at the end of its primary consolidation stage (after 80 days) can be achieved after 25 days for the PVTD embankment. This behavior can be explained in light of the increase in hydraulic conductivity at elevated temperatures as the result of the thermal evolution of water viscosity (Abuel-Naga et al. 2006b).

Conclusions

Based on the laboratory and field results, the PVTD system shows the following advantages over the conventional PVD system:

- The PVTD system yields more settlement due to the thermal consolidation of normally consolidated clays.
- The thermal consolidation is irreversible upon cooling of the clays.
- The PVTD system shows a higher rate of consolidation. This behavior can be attributed to the increase of the soil hydraulic conductivity at elevated temperatures which can reduce the detrimental effects of the smear zone.
- The PVTD system changes the normally consolidated clays to lightly overconsolidated clays. Subsequently, the soil can carry extra load with less settlement.
- Using a solar heating technique with the PVTD system is a cost effective method to heat the ground.

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Geotechnical Design for the Nakheel Tall Tower – Case Study

1 Introduction

The recently announced Nakheel Tower in Dubai, UAE (Figure 1) will extend in excess of 1 km in height and at about 2,000,000 tonnes dead load it will be one of the heaviest buildings on earth. The bearing pressures applied to the ground coupled with the soft rock ground conditions present at the site provided a significant challenge to the design of the footing system.

The following presents a brief summary of the ground investigation undertaken and of the development of the footing system which is currently being installed.

2 Geology

An arid climate prevailed in the area during Holocene times, facilitating the formation of coral reefs and shallow marine carbonate deposits. In addition, evaporite or Sabkha deposits, containing mainly gypsum are common and are associated with intertidal conditions on flat topography.

The carbonate rich sedimentary sequence underlying the site comprises mainly carbonate cemented siltstone (calcisiltite). Gypsum layers of up to 2.5 m thick are interbedded with the carbonate materials at levels lower than 75 m below ground level.

Recent aeolean deposits (sand dunes) form a capping over vast areas of the United Arab Emirates, including the Nakheel Tower site. Owing to different rainfall and groundwater regimes associated with past climates, the dune sands have become partially cemented forming calcarenite beds. At the Nakheel Tower site, the sand dune capping extends about 20 m below ground level.

3 Footing Concept

Based on preliminary information, the proposed footing concept for the tower comprised a raft supported by large diameter piles or barrettes. The base of the raft would found below the sand dunes at about 20 m depth within the carbonate cemented siltstone with piles or barrettes extending perhaps to depths of 60 m to 70 m below this level. As piles/barrettes were to be installed from the surface, and given the expected ground conditions, it was considered that installing bored circular piles to these depths may prove to be problematic, if not impractical, and hence the decision was made relatively early in the design process to adopt barrettes for the main deep foundation elements.

The temporary basement retention system, constructed through the sand dune deposits, would comprise a circular diaphragm wall which was to be installed prior to the foundation barrettes. The barrettes were to be installed from the surface, with excavation to pile cut-off level to proceed once barrette installation was complete.

Figure 1: Nakheel Tower, Dubai, UAE

4 Ground Investigation

On the basis of our previous experience at other sites in Dubai we were aware that when sampled and brought to the surface the carbonate cemented siltstone undergoes significant stress relief. This results in samples tested in the laboratory displaying significantly lower strength and modulus properties than measured by insitu testing. Significant emphasis was therefore placed on insitu testing, which comprised pressuremeter testing, cross hole sonic testing, water pressure testing and the testing of three full scale test barrettes.

Laboratory testing was also undertaken to better understand the constitutive behaviour of the cemented carbonate materials. Laboratory testing comprised characterisation tests and specialist testing. Classification testing included unconfined compressive strength (UCS) testing with modulus measurement (end platten measurement) and tests for carbonate content, unit weight, specific gravity, moisture content and dry density. Specialist laboratory testing comprised cyclic and monotonic constant normal stiffness direct shear testing, resonant column testing, drained triaxial testing, cyclic triaxial testing and high pressure oedometer testing.

The ground investigation was undertaken by Fugro Middle East in accordance with specifications provided by Golder Associates Pty Ltd, Melbourne, Australia office. Golder Associates' staff were on site during the ground investigation and independently logged the rock core.

Preliminary analysis of the footing design concept was undertaken using PLAXIS 2D and assuming axisymmetric conditions. These analyses indicated that more than 50% of the calculated footing settlement would occur below the toe of the barrettes. For this reason, significant attention was paid to estimating stiffness parameters of the ground below the toe of the barrettes (from about 80 m depth to 200 m depth).

Nine geotechnical boreholes were drilled to between 150 m and 200 m depth using PQ triple tube drilling techniques. Immediately upon being recovered from the borehole, core was logged, photographed and samples were extracted. Moisture content testing was undertaken on site and samples scheduled for off site testing were wrapped in plastic film, placed in snug-fitting cardboard tubes and sealed in wax.

As the rock materials were essentially unweathered, the application of a weathering classification system would be of little if any benefit. A relatively crude and simple hardness test was therefore developed to provide a continuous assessment of the core. The hardness test comprised inserting a knife into the core using a relatively constant pressure and measuring the penetration. The hardness values obtained through this process allowed assessment of the variation in ground conditions across the site and an estimate of potential tilt of the building under gravity loading.

Pressuremeter testing was undertaken at 5 m intervals in three boreholes. Pressuremeter tests were taken to the working limits of the equipment and incorporated "hold" stages of up to an hour to measure the creep characteristics of the ground. Due to the significant depths at which testing was to be undertaken, pressure measurements were taken within the probe. The pressuremeter test results provided data on rock stiffness, strength and creep characteristics.

Crosshole seismic testing to 200 m depth was undertaken in a further two boreholes. Two receiver boreholes placed 3 m and 6 m from the source boreholes were utilised in this testing. The cross hole seismic testing was analysed to provide continuous profiles of small strain shear modulus with depth.

5 Constitutive Behaviour and Properties

The founding conditions comprise predominately carbonate or gypsum cemented materials with a relatively high void ratio (0.4 to 0.7). Laboratory and insitu testing indicated the material has a relatively high stiffness below a "bond yield strength" after which the compressibility of the material increases significantly and exhibits properties similar to an uncemented, normally consolidated material at the same void ratio. Prior to reaching the bond yield strength the behaviour of the rock is dominated by the intergranular cementation and displays approximately linear elastic behaviour with deformations occurring essentially instantaneously. As the bond yield strength is approached, deformations become time dependent and consolidation and creep displacements dominate.

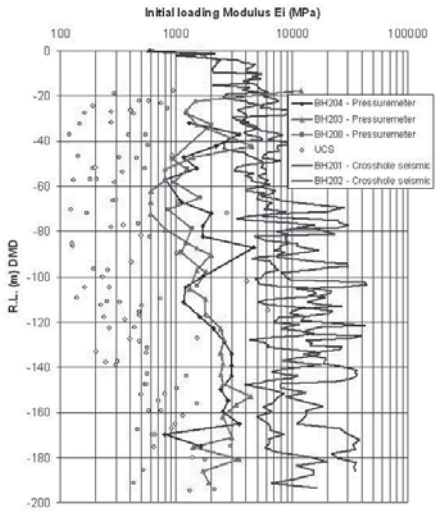

Figure 2: Young's modulus variation with elevation (surface level RL+2.5 m DMD)

Satisfactory performance of the footing system for the tower therefore required that the stress increase in the ground due to the loads from the tower were kept below the bond yield stress. A primary aim of the ground investigation was therefore to obtain good estimates of the variation of rock modulus and bond yield strength with depth.

Figure 2 compares the Young's modulus values estimated from the pressuremeter, cross hole seismic and laboratory UCS tests. The pressuremeter test results display similar initial loading and unload reload moduli values which is consistent with the absence of jointing in the rock and the domination of the cementation. The Young's modulus values obtained from the pressuremeter and cross hole seismic tests show reasonable agreement (see Figure 2) if the small strain modulus values obtained in the cross hole seismic tests are reduced by a factor of five.

An elastic, perfectly plastic (purely cohesive) constitutive model was found to provide an excellent fit to the pressuremeter expansion curves. This is consistent with the dominance of the intergranular cementation below the bond yield strength and should not be confused with "undrained" yield strength behaviour. We have interpreted the shear strength so obtained as an estimate of the bond yield strength.

Figure 3 compares the shear strengths measured in the UCS tests (taken as UCS/2) and those estimated from the pressuremeter tests assuming a purely cohesive strength criterion.

Figures 2 and 3 show that stiffness and strength properties measured in the laboratory were significantly less than obtained from insitu tests, and supported our hypothesis that

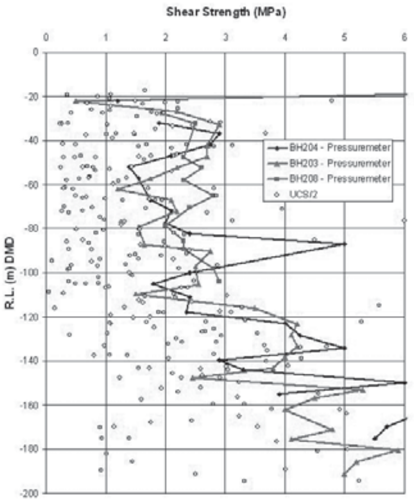

Figure 3: Shear strength variation with elevation (surface level RL+2.5 m DMD)

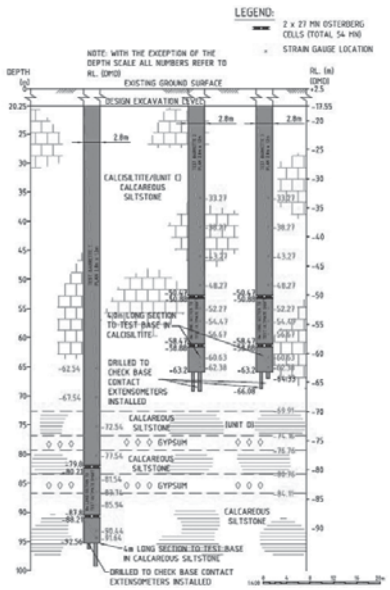

Figure 4: Test barrette configuration

the core samples were undergoing significant stress relief even with the care that was undertaken during the drilling, retrieval, storage, transportation and testing processes.

The full scale barrette load tests (see below) confirmed that the properties obtained from the insitu testing were reasonable and that the laboratory test results significantly under-estimated the properties of the insitu rock.

6 Barrette Load Tests

As part of the ground investigation, three full scale test barrettes were installed and tested in accordance with a specification provided by Golder Associates. The test barrettes were installed by a Soletanche-Bachy/Intrafor Joint Venture and load testing of the barrettes was carried out by Loadtest International Inc. The load tests comprised two levels of Osterberg cells in each test barrette as shown in Figure 4. Each level of cells was capable of providing a working bi-directional load of 54 MN. However, during testing loads were increased to the capacity of the equipment resulting in bi-directional loads of up to 83 MN.

On the basis of a preliminary concept for the footing design, barrettes were located under the main load bearing elements of the structure. This resulted in barrettes at relatively close centres and, as a consequence, most of the applied load would be transferred towards the toe of the barrettes. For this reason the Osterberg cells were positioned to measure performance of the lower 20 m or so of the barrettes.

The test barrettes were instrumented with displacement telltales and strain gauges. In addition, instrumentation was also located in the rock below the toe of the barrette to directly measure the displacement of the rock at this location.

The barrette load tests were used to investigate load deformation behaviour of the shaft and base of the barrette under static, cyclic and long term conditions. The measured load versus displacement performance of the two shorter test barrettes (TB02 and TB03) for loading at the upper and lower levels of Osterberg cells are shown in Figures 5 and 6 respectively. Also shown are the true Class A predictions of the performance. The Class A predictions were obtained on the basis of the adopted design properties for the ground and on the as-constructed barrette geometry. The predictions of performance were completed prior to testing of the barrettes.

For the Class A prediction, the rock-socket software ROCKET97 (Seidel, 2000) was used to calculate the shaft resistance performance of the test barrettes. The calculated shaft resistance performance was then used in an axisymmetric PLAXIS V8 model to obtain the calculated load versus displacement response shown in Figures 5 and 6. The comparison between the measured and predicted response is excellent, which provided further confidence that the design properties adopted on the basis of the insitu testing were appropriate.

PLAXIS V8 was also used to calculate the design top-of-barrette load versus displacement performance shown in Figure 7.

Figures 5, 6 and 7 clearly demonstrate the relatively stiff and strong response of barrettes in these ground conditions. Similar results were obtained from the other two test barrettes.

7 Footing Design and Analysis

The results of the above investigations were used to assess design profiles of strength and Young's modulus with depth to a depth of 200 m. Profiles of credible upper and lower bound properties were also assessed.

Preliminary analyses of the footing system were undertaken using PLAXIS V8 (axisymmetric). Barrettes

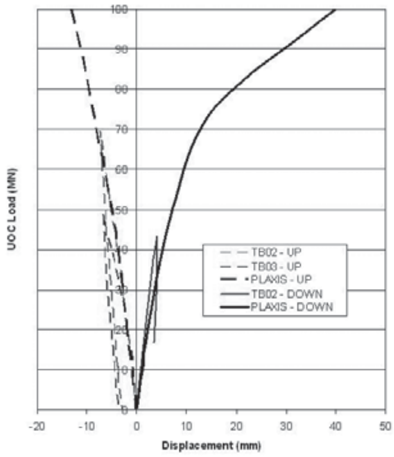

Figure 5: Measured vs predicted performance for loading at upper cells

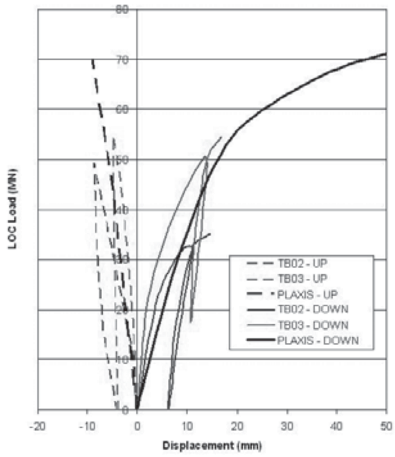

Figure 6: Measured vs predicted performance for loading at lower cells

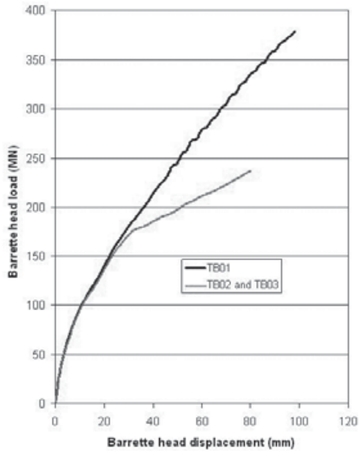

Figure 7: Calculated barrette head load versus displacement performance

were modelled either as rings of equivalent structural plate elements or equivalent concrete/rock blocks. Interface elements were used to model the shaft resistance performance of the barrettes, with shaft resistance values adjusted to account for the difference in shaft area between the two-dimensional model and the actual three-dimensional conditions. Similar results were obtained using both plate elements and concrete/rock blocks.

Significant consideration was given to the practicality of obtaining clean bases (free from debris) to the barrettes, and hence analyses were undertaken assuming both full and no base resistance. Another important consideration: was to stagger the length of the barrettes such that the concentration of load towards the toe of the barrettes was spread over a greater volume of rock and the risk of exceeding the bond yield strength of the ground was reduced.

The PLAXIS V8 results were further analysed to provide estimates of individual top-of-barrette stiffness values and stiffness values for the rock supporting the raft. Due to the axisymmetric assumption, stiffness values varied with radius from the centre of the tower. On the basis of additional analyses, barrette stiffness values were adjusted according to their location within a group of barrettes (eg at the corner and centre of a group). The stiffness values were provided to the structural engineers for the project for use in their structural models of the tower. This allowed column loads to be refined and the barrette layout and raft thickness to be modified accordingly. The above process was repeated until there was convergence between the structural and geotechnical models for the footing system refined by the above process.

Detailed three dimensional analyses of this footing system were then undertaken using the finite element software PLAXIS 3D. In general, the three dimensional analyses gave settlement profiles and barrette structural actions (loads, shear forces, bending moments) that were consistent with those obtained from the axisymmetric (PLAXIS V8) analyses. PLAXIS 3D analyses were undertaken for several serviceability and ultimate limit state load cases; design, credible upper and lower bound properties, and assuming full and no base resistance to the barrettes. The analyses indicated acceptable performance under all conditions analysed.

Probablistic analyses were also carried out to assess the potential tilt of the tower due to variations in ground conditions across the site and to provide a probabilistic estimate of settlement. Measured field hardness values were used as the basis of the assessing the variability.

8 Closing Comments

Construction of the foundation system is currently underway. Golder Associates has personnel on site to assist in maintaining the quality of construction of the barrettes. This is being facilitated through base drilling of select barrettes, cross hole sonic testing, maintenance and quality control of drilling fluids and checking of positioning and measurements. It is intended that instrumentation will be installed to monitor surface displacements and barrette loads during construction.

9 Acknowledgements

The authors gratefully acknowledges FoundationQA Pty Ltd for the use of ROCKET97.

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Editors note: This paper was orginally published in the ISSMGE Bulletin.

OBITUARY

William Robert Dearman

William (Bill) Dearman, Emeritus Professor of the University of Newcastle passed away on January 6th, 2009 at the age of 87.

Bill's contribution to both engineering geology and IAEG was very significant. He was a founding member of the Engineering Group of the Geological Society in 1964 and attended the first meeting of IAEG in Paris in 1970. He led the proposal to create a British National Group of IAEG at the Council Meeting in Krefeld in 1976.

Bill was a key member during the early days of IAEG. He was member of the Executive Committee and a very active member of the Commission 1: Engineering Geological Mapping of our Association. The Commission had been established in 1968 at the General Assembly of IAEG during the International Geological Congress in Prague.

He was always ready to help and did much during the pioneering days of engineering geology to assist young graduates in their careers. He also played a major part in disseminating information by his editing and editorial committee work.

The well known text book "Engineering Geological Maps, A guide to their preparation" was published by UNESCO in 1976 as the first accomplishment of the Commission in response to its stated aims. Members of IAEG commission who took part in the preparation of the guide were: Professor Milan Matula (Chairman), Czechoslovakia; Professor G. A. Golodkovskaja, USSR; A. Peter, France; A. Pahl, West Germany; Mrs Dorothy H. Radbruch – Hall (Secretary), USA. The editor was William Dearman. Later publications by the commission in 1981 presented recommended symbols and rock and soil description and classification methods for engineering geological mapping. Bill Dearman was the Chairman on the very successful IAEG Symposiums at New Castle upon Tyne, on this specific subject in 1979. William Dearman by a perceived need to combine and expand the work on Engineering Geological Mapping, already Emeritus Professor, prepared the book "Engineering Geological Mapping" published by Butterworth and Heinemann in 1991.

In recognition of his contribution to engineering geology and for his contribution to IAEG, Bill was awarded the Hans Cloos Medal in 1990. He was also honoured by the award of the Geological Society's William Smith Medal for his longstanding involvement in engineering geology, both nationally and internationally.

Sitting in the middle: William Dearman, and then from left to right: Ricardo Oliveira, Victor Osipov, Niek Rengers, Louis Primel, David Price, Paul Marinos. Standing Milan Matula, Pal Kertez, Michael Langer, George Kiersch, Jacques Locat, Valentina Samalikova, Wang Sijing, Asher Shadmon. Sorry, I can not identify the remaining colleagues of this photograph.

Bill will be sadly missed by many of his past students, colleagues and those who had the privilege to work with him.

For our younger colleagues: a photograph from 1984, taken at Moscow with members of the IAEG Council (President of IAEG at that time was E. Sergeyev). Bill Dearman is surrounded by a good number of people that shaped IAEG.

Reported by:
Paul Marinos

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FOREIGN CORRESPONDENT

Above: Cardo outcamp

Dennis Reeve

Perth, Western Australia

Dennis Reeve
Engineering Geologist
JFK and Associates

Foreign Correspondent, it used to be a word I associated with investigative journalism in far away places like Kathmandu, West Africa or one of the ex-Russian states. Why would it now be associated with Sandgropper country – Perth, Western Australia. To be honest, I don't know the answer to that question.

After working for Tonkin & Taylor for five or so years I decided it was time to move on, challenge myself and broaden my experiences. I had lived in Perth for 3 years when I first left university working in a gold mine so I had some good memories and connections from that time so decided to come back and see how things had changed. I moved to Perth in September 2008 just in time for the "mining boom" to come to an end so spent a couple of months job hunting. Over this time I became an avid watcher of daytime TV – no, not such well known shows as "Dr Phil" or "Oprah" but mainly cooking shows. A typical day for me would start at 9 am. Get up, shower, breakfast, check in with a range of recruitment agents and other contacts followed by a trip to the shop to buy a copy of "The West Australian" newspaper. This would take until about 11 am when "Alive and Kicking" would start with a series of three or four different dishes. After this Jamie Oliver would try to enthuse me with his cooking expertise while trying to cook rural Italian dishes using his style but would almost always fall short of the local feedback. I can't remember the number of times I helped him through these rough times. I am still to this day unsure if he heard me yelling at the TV, but this I do know – he has never once called me up for a beer. After Jamie Oliver, I would concentrate on "Heweys Cooking Adventures", "Judge Judy" and round out the day with "The Cook and The Chef".

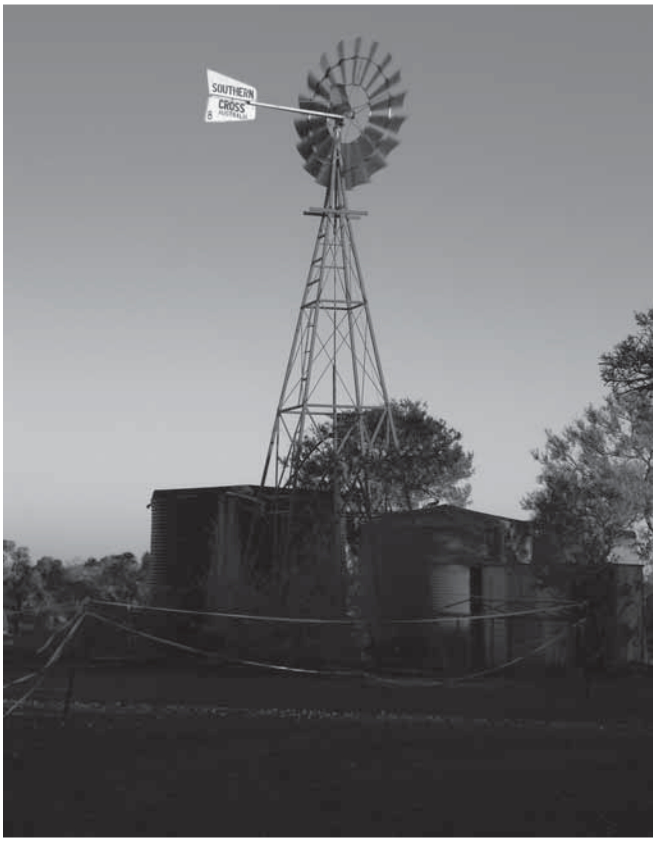
Thankfully, I got a job at JFK and Associates which is a small geotechnical engineering consultancy with approximately 20 employees. During my time at JFK I have worked on a variety of projects including investigations for a new water dam, desk studies for new railway lines, investigations for the duplication of railway embankments over soft marine sediments, geotechnical aspects of tender bids for oil and gas construction projects and construction supervision for a new mine access road, railway lines and mining infrastructure. For most of these projects, we (JFK) are part of an EPCM (Engineering, Procurement and Construction Management) team and are involved from feasibility studies right through to detailed design and construction supervision. Many of the JFK team are based in different EPCM project offices which helps build excellent working relationships with effective and timely interactions with the designers. This model works really well from what I have seen.

The end clients for our projects range from the major iron ore miners through to emerging and mid level iron ore miners. We also work for other organisations including earthworks contractors, oil and gas producers, government organisations and other consultants.

Based on my experience here so far, the projects we are involved with are much larger than the average project in NZ. For example, the Northern Gateway Alliance motorway extension north of Orewa is reported to have a budgeted construction cost in the region of $350 to $370 million. It was one of the biggest (in terms of construction cost) projects undertaken in recent times within NZ. The smaller jobs over here have similar construction costs and some are measured in the billions. Some of the mining companies are looking 20 to 50 years ahead when planning their expansion needs. All the same, it is common to need that report or design 'yesterday'.

Most of the work we do at JFK is related to mining related civil infrastructure based in the Pibara which is a region approximately 1200 km north of Perth. It is hot, can be humid and has regular cyclones between early December and late April each year. During cyclones 200 km/hr winds and 250 mm of rainfall are common. This makes doing fieldwork and construction activities a tad difficult and downright dangerous. The geology of the Pilbara is mainly Late Archaean/Early Proterozoic with such rock types like Banded Iron Formation (BIF), dolomite, shale, channel iron deposits and chert with intrusive dolerite dykes. There are a couple of phases of deformation (folding, faulting and intrusions) so the structure can be quite complex. There have also been a couple of phases of weathering, burial metamorphism, duricrust formation and erosion to further complicate matters.

Safety is the number one concern in the mining related industries in Australia. I recently had to attend seven days worth of safety inductions to enable me to carry out five days worth of test pitting. Admittedly, it was working next to active railway lines so I was more than happy to get all the facts before starting. On many sites, you have to do a compulsory alcohol breath test each morning and the limit is 0.00, i.e. no alcohol at all. The legal driving limit here is 0.05. The safety process seems phenomenal to begin with but once you understand all the systems and requirements most of it makes sense and you can understand why or how each piece has come into play. It still takes a fair while to obtain all the clearances and permits to undertake any field based work.



Above: Cardo outcamp

Perth is a fantastic place to live. The sun seems to always shine, it is warm, the people are friendly, there are lots of sports to get into here and its fun winding up the Aussies. I have spent a lot of my spare time wakeboarding on the Swan River, crabbing, fishing and heading out into the neighbouring towns. Of course there are also the beaches which run all along the western suburbs. Sharks, yes, don't mention the sharks though. There have been a few sightings earlier in the summer. "Rotto" – Rottnest Island – is only an hours ferry ride from Fremantle and is a fantastic place to go for a day trip or a weeks summer holiday. There are plenty of wineries within a couple hours drive of Perth and if all else fails, one of the greatest Australian inventions – the Sunday Session. The Sunday Session involves a few, or more than a few, beers with some mates at one of the suburban pubs. The most spectacular is arguably either the OBH or Cott. Both of these pubs are perched above Cottesloe Beach with a fantastic view out towards Rotto and down the beach towards Fremantle. One small problem I have found with the Sunday Session is that you have to go to work the next day. This is one thing I am keen to do a little more research on.

What do I miss most about New Zealand? I would say without a doubt the excellent colleagues I became friends with. Such illustrious people as "JT the Smokey Tohster", "The Bear", "Curley", "Goosio", "Taffy", "Dr Evil", "Frosty", "Slinky" and of course all those other people who have a nickname which may not be appropriate in this article or who haven't been given a nickname yet.

COMPANY PROFILES

RDCL Ltd

If you want a specialist group providing geological, engineering geological and geotechnical and ground investigation services to the New Zealand and international civil and mining industries then you need RDCL.

Resource Development Consultants Ltd (RDCL) was established in October 2006 to deliver geo-consulting services to the mining and civil engineering sectors. The following year a 'sister' company was formed and so Resource Development Contractors Ltd now provides Cone Penetrometer (CPT), Seismic Cone Penetrometer (SCPT) and geotechnical investigation services.

Both businesses can work independently or as a team to assess ground testing needs. RDCL's services are tailored to meet the specific requirements of each project and specialist staff on the RDCL team include geotechnical engineers, engineering geologists and mining geologists.

Collective expertise

Collectively RDCL offers extensive expertise and experience developing complex projects in remote locations, and pride ourselves on our ability to operate effectively in challenging environments explains founder and director Cam Wylie.

"Our technical, administration, and communication systems have been modified to meet the specific needs of such testing settings," says Cam.

We are experienced in a range of projects including underground and open pit coal and hard rock mines, dams, roads, foundations, land development, and geological exploration for coal and coal seam gas. The company has also undertaken technical feasibility studies, mine operations, civil construction and large scale drilling investigations.

Two years on from first setting up shop in Hawke's Bay, RDCL has completed a variety of projects throughout New Zealand and around the world in places such as Mozambique, Armenia, Indonesia and the Philippines. This global experience reflects RDCL's vision to be a global provider of specialist geotechnical and geological solutions.

Ultimately RDCL's goal is to enable its customers to improve their ability to manage ground risk and project costs.

Projects at home and abroad

RDCL has been involved in a range of interesting projects recently, from providing geotechnical support for subdivisions in Hawke's Bay through to the development of a new mine overseas.

Above: "Joy the Driller" operating the cordless CPT. All geologists secretly want to be one!

Philippines

Geotechnical investigation and construction support for a new mine project. Last year RDCL completed an extremely challenging project in a remote, mountainous area of the Philippines, with a wide range of surface cover materials including volcaniclastics and tropical laterites, and wide spread slope instability. Our role involved geotechnical investigation and assessment including deep drilling of an underground mine and open pit, extensive hydro-geological testing and drilling for tailings dam, metallurgical plant site and 500 person accommodation camp. We provided construction support for the project which faced some interesting challenges such as the upgrading of the new mine road which traversed 20-plus kilometres of extremely variable and unstable materials. A very high rainfall during the construction phase and the remote working environment added to the challenge of the job. It was an extremely interesting and exciting project to work on.

North Otago

Ongoing geotechnical support for an underground mine. For over two years we have been mentoring the client's geotechnical staff, building the capability of the underground technical team and providing technical input to the development of the mine. The operation is a true success story. The underground gold mine is situated in a large scale tectonic shear zone dipping at 20° east. Mining is undertaken by open stopping with yielding pillars to provide temporary support. Access is by drives and cross-cuts up to 6 m in height and 5.5 m wide, with ground support including rockbolts and mesh, fibre-reinforced shotcrete and cablebolts. Observational methods have been used to rationalise ground support designs and mining methods. A continuous improvement philosophy has improved efficiencies across the project to see gold production output improving. This has all been achieved with a safety record of Zero LTI's for 10 km of tunnel driven and two years operation.

Waikato

Geological services for coal and coal gas exploration drilling. This project drew on our expertise in field management of large scale and extensive drilling campaigns with associated testing and downhole work. Our most recent projects have run over five months, with RDCL staff residing on site and support from the Havelock North based 'home office'. Feedback from our clients indicates we have been proactive and strong in this role and look forward to more of it. It has been a great opportunity for our team to gain valuable field experience and understand the drilling and resource evaluation process in detail.

Hawke's Bay, Hamilton and Huntly

Cone Penetrometer (CPT) and Seismic Cone Penetrometer testing (SCPT).

"RDContractors bought our CPT and SCPT in September 2008 and since then we have used the machines throughout Hawke's Bay, in Hamilton and in the Huntly region. We have kept the machine 'close to our chests' as we have learnt its intricacies and built our operational support and processing capabilities."

Hawke's Bay

Geotechnical support for subdivisions and light industrial buildings.

"Being relatively new to the region we have landed a good proportion of the more difficult jobs, having missed out on the larger subdivisions on "the flats". We relish the challenge in a province on the edge of an active plate boundary and with some of the highest earthquake hazard factors in the country."

Moving forward

Currently RDCL is concentrating on growth so the company can fulfill its vision of being a world class professional services provider. The company recently won work on a major new gold mine in the Philippines and are building strong contacts in Indonesia. There is ongoing involvement in two new mining evaluation projects in New Zealand, and RDCL continues to strengthen ties with organsiations around New Zealand.

Back at the office, RDCL are building on its "team" culture and professional development of staff continues to take place. Staff are members of relevant professional organisations including ACENZ, NZ Drillers Federation and NZ Contractors Federation as well as numerous professional bodies. RDCL is ISO9001:2004 certified, has well established safety systems and training procedures in place and have invested heavily in technical software and hardware.

As we continue to develop our systems and infrastructure we are keeping in close contact with our clients. We believe our investment in people, tools and systems will continue to see us delivering high quality and sound advice so our clients can effectively manage their technical risk.

Current market conditions present both a significant opportunity and a challenge to relatively young companies such as RDCL. Like many businesses, they faced 2009 with some trepidation but in true RDCL style their approach this year has involved grit and characteristic determination. Smaller companies like RDCL are perfectly placed to respond quickly to changing needs locally, nationally and globally so we are rising to the challenge of tougher economic conditions and the demand for quality service.

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RDCL

Tonkin & Taylor Ltd – celebrating 50 years in 2009!

Above: Te Papa Tongarewa Museum of New Zealand

Tonkin & Taylor brings together the largest team of geotechnical consultants in New Zealand and this year celebrates its 50th birthday. This strong position in terms of size and reputation began from very modest beginnings, initiated and shaped by three outstanding practitioners. The first, Professor Peter Taylor, will be well known by all, now older, engineers who graduated from The University of Auckland. Peter was a brilliant teacher and geotechnical specialist who influenced the lives and careers of hundreds of New Zealand engineers. The other two were Ralph Tonkin and Don Taylor.

Ralph and Don met while working on the Auckland Harbour Bridge approaches. Peter Taylor describes in his recollections (NZ Geomechanics News, Issue 76, December 2008) how he suggested to Ralph that they set up a Laboratory, to which Ralph agreed, provided he could set up "a consulting practice alongside". Thus began Geotechnics and a consulting practice which became Tonkin & Taylor when Don Taylor bought into both the Laboratory and Consultancy a year later. Don describes some of the evolution of the investigative methods and projects of geotechnical significance in his recollections (NZ Geomechanics News, Issue 76, December 2008).

Setting up practice as a consulting engineer in 1959 in New Zealand was a bold move. At the time the vast majority of civil/infrastructure engineering was orchestrated by central Government through the Ministry of Works and only a very small private consulting fraternity existed.

Ralph and Don used the networks of their peers and contemporaries. Many of their early commissions were from colleagues in the Ministry of Works who were able to get an efficient, responsive service outside the bureaucracy of The Ministry. They quickly developed, probably, the widest client base of any private consultancy at the time, providing geotechnical services throughout New Zealand. In the early days they could not just rely on specialist work so many of the early jobs were small and low tech. Don describes a modest ambition from the outset of simply "putting bread on the table." A critical success factor from the outset was accessibility by and responsiveness to clients, which underpinned the growth of the business.

Peter Taylor, in his recollections, describes Ralph Tonkin: "He had a remarkable combination of talents. He could size up an engineering project in a flash and come up with a workable scheme. He could play golf with a prospective client and convince him of the soundness of the scheme and

the expertise of the firm. Truly a man to be remembered."

Ralph was indeed remarkable and very much a pioneer at his peak. He was driven by challenges and opportunities to pursue and conquer and could be labelled a "workaholic". Tonkin & Taylor still has reflections of the character of Ralph evident in its culture. Sadly Ralph died, only in his 50's, of Alzheimer's Disease in 1985.

Don Taylor was the perfect match with Ralph, Don being the consummate professional consultant. Don role modelled the ethic of high standards and accuracy and was a leading figure in both the geotechnical and consulting engineering communities in New Zealand and beyond. He was President of the Association of Consulting Engineers at the historic (only time to date) New Zealand hosted FIDIC (International Federation of Consulting Engineers) Conference. As well as having a specialist qualification in soil mechanics and a real feel for the art, Don was a geologist. An enduring legacy within Tonkin & Taylor was his insistence on working from the "big picture geological context" before planning and executing more detailed investigations. Don has a sharp intellect and wit and his penetrating summations of situations are still quoted e.g. "An ACENZ Conference is a gathering of sharks who have agreed not to bite for three days."

The International dimension of Tonkin & Taylor had its origin in 1969 with Ralph being part of the New Zealand Government Colombo Plan Mission to Malaysia and Indonesia. The large scale of overseas project opportunities led to the necessity to team up with other like-minded consultants and to the formation of entities such as ENEX, ANZDEC and GENZL, selling engineering, agricultural and geothermal expertise internationally.

An extreme example of the scale of projects requiring collaboration was the Nahar Saad project in Iraq, in 1975, with an estimated construction cost of $1 billion (in dollars of the day). In addition to projects throughout the South East Asia/Pacific region, Tonkin & Taylor has maintained a presence in Malaysia since 1972 and more recently has established offices in the Philippines and Australia.

Above: The Bukit Tagar Landfill project in Malaysia

The areas of practice for Tonkin & Taylor have expanded over its 50 year history to encompass dam, solid waste, coastal engineering and environmental engineering and science. However, the Company has retained its specialist focus with geotechnical being the core area of expertise.

The Company has had interesting, challenging and hugely satisfying achievements, many of which have been recognised with awards. Some milestone projects have included the Whau Valley Dam (described by Peter Taylor), the Lower Waikato River Control Scheme (described by Don Taylor), Aniwhenua Hydro project in Bay of Plenty, Te Papa Tongarewa Museum of New Zealand, Redvale Landfill, Vector Tunnel (Penrose to Auckland CBD) and the Northern Gateway project (Orewa to Puhoi). Internationally, the Bukit Tagar Landfill project in Malaysia was last year awarded the supreme award for consulting engineering in South East Asia.

A key differentiating feature of Tonkin & Taylor, and a foundation for its success, is its culture and the outstanding people who are attracted by this and are its ongoing stewards. Tonkin & Taylor is one of the few larger New Zealand consultancies who has not been bought by a multinational. The broadly distributed staff ownership and a clear sense of identity all contribute to its success. The Company works hard to live up to its Goal to be "the best to work for and the best to work with".

Reported by
Terry Kayes and Philippa Hair
Tonkin & Taylor

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1959 - 2009 50 YEARS

Tonkin & Taylor

NZ Geotechnical Society
2009PHOTO COMPETITION

THE YEAR 2009 THEME IS:
What we do best

Show us the things that geotechnical engineers, engineering geologists, and technicians in the profession get up to!!

WIN $250

A perfect chance to win some drinks money for the office. The winning photo will be printed in the December 2009 issue of the *NZ Geomechanics News* and put onto the Society Web Page.

SEND YOUR ENTRY TO:

- The Geomechanics News Editor,
  P.O. Box 5271, Wellesley Street, Auckland
- email to: kwilliams@tonkin.co.nz (send as jpgs)
- Entries close 31 October 2009
- Clearly mark your entry with your name and provide a caption for your photo

CONDITIONS OF ENTRY:

1. Only amateur photographers may enter.
2. Photos must be taken by the entrant.
3. No computer generated pictures.
4. Any photographs received may be published in subsequent Society publications.
5. Winning entries will be final and no correspondence will be entered into.
6. NZ Geotechnical Society members only may enter.

MEMBER PROFILES

Aaron George

Occupation
Senior Engineering Geologist /
Team Leader Geotechnical
Opus International Consultants Ltd

As a child from the age of three I was interested in rocks. I was always picking up shiny and colourful stones and displaying them around my parents' house. As I got older the interest never seemed to wane and I was told at age seven that someone who worked with rocks was called a geologist. I proudly told my teacher I was going to be a geologist and her response was to simply look at me for a moment before eventually commenting – "Hmmm you'll have to work hard to become one of those"! I'm not sure if she was trying to be encouraging or insulting, but it did make me determined to one day become a geologist.

Enrolling at the University of Auckland in the late 1980's I started out doing a BSc majoring in Geology and finished with a Masters. I was interested in all aspects of geology, however it was when I started doing the engineering geology papers taught by Warwick Prebble that I began to see how I could actually have a career here in New Zealand as a geologist. I particularly clicked with the applied nature of engineering geology and the interaction professionally between geology and engineering.

Graduating in the early 1990's there were very few jobs for geology graduates. It took about 18 months to actually get a permanent role, at Works Consultancy Services in Auckland. I've been with them (now Opus) ever since and have had the enormous privilege of working on both small and large engineering projects in the Auckland and Northland regions over the last 15 years. These included numerous motorway projects in the Auckland region such as the South Eastern Arterial Route, the ALPURT Sector A Northern motorway from the North Shore to Silverdale, the North Shore Busway including the Esmonde Road Interchange and the Upper Harbour Corridor Motorway. An aspect of these projects I personally found most enjoyable was being involved with many of them at initial planning stages through scheme, design, construction and sometimes post construction. A great experience for me has been seeing the geotechnical contribution as large projects develop and how geotechnical risk is managed through all the stages.

Other project work over that time has included slip repairs in Northland, numerous Scheme Assessments for infrastructure projects later undertaken by others and more recently some rail infrastructure work. Many of the most challenging and enjoyable projects have been the smaller slip repairs, which are often complex geotechnically and are also constrained with cost and have political implications for the community. The solutions often require considerable lateral thinking as no two slip sites ever seem to be the same.

One of the most enjoyable aspects of working on larger projects has been the opportunity to work alongside other geotechnical consultants from other companies. Meeting and working with fellow geotechnical professionals who would normally be competitors in the market place, sharing ideas and working for common project goals has widened my own vision of the role I play in the industry as well as giving me the opportunity to make friends with people I share a lot in common with.

Thinking ahead to future challenges I can't help thinking about the global economic environment we now all work in. One of the obvious challenges we all share in this economic climate is that work has now become a lot harder to find. The types of work we get to do may be limited to simply what we can get. Likewise many of us are facing uncertainty over employment. In many ways this is not to dissimilar to what the work environment was like when I first started out in the early 1990's. Clearly these types of things are cyclic and having some insight into the past may well help when trying to gain insight into the future. Change when it comes tends to happen rapidly, and although we have to adjust to the current circumstances it would be prudent to keep at least one eye open on opportunities when they come. Using the time now to prepare for when things improve, broadening our skill sets with more study or taking on different types of work.

Reflecting on the last 15 years I can honestly say I have thoroughly enjoyed working as an engineering geologist in New Zealand - who knows what the next 15 will hold? If I could go back in time and talk to myself again as a seven year old, I would certainly say the hard work is worth the effort.

Simon Woodward

Occupation
Geotechnical Engineer
Geotek Services Ltd

So where does one start with one's profile as a Geotechnical Engineer? Perhaps at the beginning. Perhaps as the son and grandson of engineers, it was pre-ordained, but in reality, for me, it really started in the basement of the Murray North (no longer trading) office in Auckland city, when I was in my 5th form year (Year 11 these days), testing and compiling Proctor (now NZ Standard) Compaction curves on soil samples. I thought it was pretty cool following the whole process from soil sample to the answers of Optimum Water Content and Optimum Dry Density, so I decided I would do Soils Engineering when I went to university.

Unfortunately, I wasn't such a dedicated student during my early BE, due to motivational issues like playing pool, and participating in the Auckland University Engineers' Society and rugby team.

Probably my most memorable (& notorious) non-academic activity was when I took part in a capping stunt with three other Engineering Students (who shall remain unnamed to protect the guilty). It was in the very early days of the concept of Political Correctness, and there was concern about how TV programs such as Starsky & Hutch, and Minder, were becoming more graphic in their violence, and de-sensitising society with regard to such violence. So here was our big opportunity to make some significant social comment - well that was my excuse anyway.

To cut a long story short, I was kidnapped at gunpoint by three mates on the corner of Queen & Wellesley Streets. We duly went to court on charges of Disorderly Behaviour, Assault, and Presenting an Imitation Firearm without Just or Sufficient Cause, and were lucky enough to get off with slapped wrists, thanks to (I think it was) Section 42 of the Criminal Justice Act or some-such, where in the event of a misdemeanour, you can be discharged without conviction, if a conviction could affect your Professional career, including overseas travel.

So after that, I started to focus a bit more on what I needed to do to even have a Professional career. That focus included a part-time stint in the School of Engineering Geomechanics Lab, researching the degradation and permeability changes of typical roading basecourse materials.

After a five and half year protracted University degree, I was lucky enough to get a job with Tonkin & Taylor, as an earthworks supervising engineer on the Marsden Point Oil Refinery expansion - one of Rob Muldoon's Think Big projects. That was later complemented with two separate stints on the NZ Steel Expansion, and as that work drew to a close, I joined the newly formed geotechnical division at Fraser Thomas.

After 14 months there, working on site investigations for subdivisions and building developments, I joined Foundation Engineering (FEL) in 1984 initially, on secondment to Consolidated Plastics, who had just acquired the NZ licence to manufacture the Netlon Civil Engineering meshes (the early predecessor of the Tensar products). When we determined that developing the opportunities to use Netlon meshes in NZ needed some academic support, for which no students were interested, I put my hand up to start a part-time Masters, but part way through that, Tensar hit the scene, and the Netlon meshes became pretty much redundant in Civil Engineering. That saw me return to the FEL offices, where I undertook the first design of a Soil Nailed retaining wall in NZ, which I then incorporated into my Masters Project instead of Netlon. This was followed with the presentation of a Technical Paper at the 1990 International Reinforced Soil Conference in Glasgow. During my time at FEL, the company had also sponsored me into Toastmasters, which I enjoyed for some 10 years or so, and would recommend to anyone seeking communication and leadership skills.

After 9 years at FEL, I was head-hunted in 1993 to run Neiderer Drilling, but was made redundant after 7 months, following the death of its major shareholder. In preparing to return to consulting, I attended an interview in West Auckland, and after a 1.5 hour drive home to Howick, decided that a commute like that, was not for me.

So after chatting to some associates, I struck out on my own, and founded Geotek Services in 1994, with an 'office' consisting of a computer trolley in the passage-way at home, and to assist in managing my new venture, I completed a Diploma in Business Studies in 1995. But it was while drilling a 5m deep hand auger on a site one day in Remuera, that I figured there had to be an easier way to get deeper soils data. So I scoured the engineering magazines at the School of Engineering, where I found an advert for the Pennine Dynamic Probe. A couple of phone calls and some faxes (email was uncommon at this time), saw me hop on a plane to the UK to have a look at one in action. After 5 days with only one night in a bed, I was home, severely jet-lagged, but convinced the probe was worth pursuing, and several months later, in November 1996, collected the country's first unit from an airport transition warehouse. Aside from its ability to reach depths beyond the capabilities of the hand auger (we have been to 26m in “rotten-rock”) the probe has also proved its worth in detecting failure planes of slips, especially those road-side failures where you often see a scallop of slump encroaching onto the carriageway, and I presented a

Case Studies paper on its use to the Roading Geotechnics Symposium 1998. Pennine no longer produce the probe, but ours has recently been rebuilt, with refinements, and the investigation technique is now about to undergo some local academic evaluation.

In 2003, I sat on the NZGS working party preparing submissions to the Department of Internal Affairs for amendments to the Building Act 1991, to improve and enhance the way "at risk" sites are recognized and ultimately recorded and dealt with at Council, but sadly, much of the party's good work appears to have fallen on deaf ears.

The introduction of AS2870 into Section 17 of NZS3604:1999 prompted me to explore the issue of expansive soils in more depth, and saw me co-author an "Acceptable Solution for Foundations in Expansive Soils" for the Manukau City Council in 2001, and then in 2004, I chaired the NZGS Working Party on expansive soils for a while, before work commitments require I pass those reins on.

Over the years to the height of the recent building industry surge, Geotek Services has grown on average by about one staff member per year, overflowed several home and commercial offices, peaking at 13 staff spread over two offices over the last couple of years, but with the subsequent economic decline, attrition has seen our Papakura office close, and our numbers fall to a hard-core unit of 7 dedicated staff, who are happy to work and socialise together as a 'family', with matching values.

Above: Pennine drill rig being used to investigate a landslip below a road.

There is no doubt that there are now challenges ahead of all of us, but if we have our way, we will still be here (perhaps without me??) in another 15 years.

Outside of work, I am a (col-) lapsed runner, sporadic (as in both time and direction) golfer, and a born-again fisherman. My wife and I have three children, (one just started at Unitec and two at College). However, after three consecutive generations of engineers, I don't yet rate the chances of there being a fourth - c'est la vie.

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Because of the high tensile strength of the TECCO (≥ 1770Nmm2) mesh system compared to that of standard mesh (4-500Nmm2) you can achieve a much greater nail spacing, no matter what nailing system you chose. As can be seen from the Pie Chart comparisons at left, the bulk of stabilisation/remediation cost is in nailing - particularly when the access is difficult for production machinery.

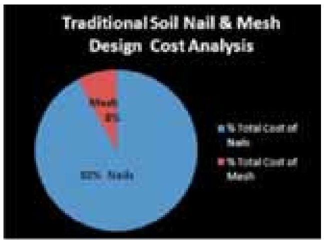
The increased nail spacing is a function of:

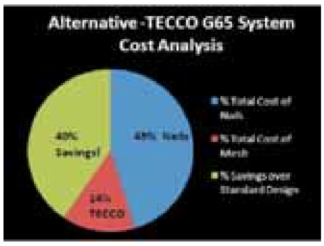
- the increased resistance of the high strength TECCO mesh to shearing off at the upper edge of the spike plate (standardised washer for the G65 system)
- the increased resistance of the high strength TECCO mesh to breaking from loading in shear against the nail

The resistance forces for these basic performance functions of the system as described above have been produced from test results in independent laboratories - this provides measurable data for **soundly engineered** solutions.

These specific resistance forces for the TECCO G65 system are fixed variables inside the RUVOLUM software.

The resistance force for mesh shearing against the washer and or the nail are not available for standard mesh because the testing has not be done by the manufacturers and because the washers for other systems have not been standardised in size and shape.





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> Refer to page 59

Figure 3: The 2008 Shotover River landslide with the main geological and geomorphological features annotated. The head-scarp has formed along the Moonlight Track, (mt) with two main foliation surfaces: fs1, which forms the back-scarp of the slide dipping at 87/074; and fs2, dipping at 40/021 and day-lighting in the main gully (g).

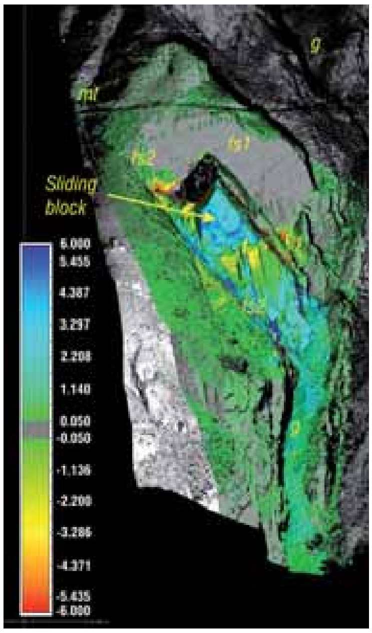

Figure 4: A comparison of the differences between scan images taken on the 24 July 2008 at 1530h and 20 August 2008 at 1300h has highlighted areas where movement has occurred during this time period. Blues show movement towards the observer, i.e. down-slope movement and reds indicate movement away from the observer, i.e. erosion. Maximum down-slope movements over the period were of the order of 6 m.

NEW ZEALAND GEOTECHNICAL SOCIETY INC.

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• Co-opted position
+ Appointed position
* Elected members of committee

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EVENTS DIARY

Links are available from the NZ Geotechnical Society website – www.nzgeotechsoc.org.nz

2009

25 JUNE 2009
London, UK
Overcoming geotechnical challenges: from flood defences to offshore construction
www.geocoasts.co.uk

JUNE 28 – JULY 1 2009
Ashville, North Carolina
The premier U.S. Rock Mechanics/ Geomechanics Conference http://www.armasymposium.org/

20-21 JULY 2009
Brisbane, Australia
Two-day Short Course on Rock Fracture Geometry Characterization and Network Modeling in 3-D including Validations.
Contact Management Secretary for Flyer if interested

22-23 JULY 2009
Brisbane, Australia
Two-day Short Course on Rock Slope Stability Analyses
Contact Management Secretary for Flyer if interested

22-25 JULY 2009
Harbin, China
3rd International Geotechnical Symposium (IGS 2009) on Geotechnical Engineering for Disaster Prevention and Reduction.

24-25 JULY 2009
Brisbane, Australia
Two-day Short Course on Block Theory & Applications for Surficial and Underground Rock Excavations Contact Management Secretary for Flyer if interested

26 JULY 2009
Brisbane, Australia
One-day Short Course on Measurement and Quantification of Joint Roughness and Aperture
Contact Management Secretary for Flyer if interested

9-11 SEPTEMBER 2009
Chengdu, China
The 7th Asia Regional Conference of IAEG "Geological engineering problems in major construction projects"
http://www.iaeg2009.com/

9-11 SEPTEMBER 2009
Perth
4th International Conference on Mine Closure
http://www.mineclosure2009.com/

20-24 SEPTEMBER 2009
Halifax, Nova Scotia, Canada
62nd Canadian Geotechnical Conference and 10th Joint CGS/ IAH-CNC Groundwater Specialty Conference
http://www.geohalifax09.ca/

20-23 SEPTEMBER 2009
Sun Moon Lake, Nantou, Taiwan
Fourth International Conference
On Geohazards
http://www.engconfintl.org/9ad.html

5-9 OCTOBER 2009
Alexandria, Egypt
XVII International conference on soil mechanics and geotechnical engineering.
www.2009icsmge-egypt.org

29-31 OCTOBER 2009
Dubrovnik-Cavtat, Croatia
ISRM International Symposium EUROCK'2009 - Rock Engineering in Difficult Ground Conditions – Soft Rocks and Karst
http://www.eurock2009.hr

9-11 NOVEMBER 2009
Santiago, Chile
Rock Slope Stability 2009 will provide a forum for open pit mining and civil engineering practitioners, consultants, researchers and suppliers worldwide to exchange views on best practice and state-of-the art of rock slope technologies.
www.slopestability.cl

2010

9-11 MAY 2010
California, United States
2nd International Symposium on CPT, CPT'10
www.cpt10.com/

MAY 10-13, 2010
Taipei, Taiwan
17th South East Asian Geotechnical Conference
http://www.17seagc.tw/index.htm

14-20 MAY 2010
Vancouver, Canada
The World Tunnel Congress 2010 and 36th ITA general Assembly
http://www.wtc2010.org/

23-27 MAY 2010
Brazil
9th International Conference on Geosynthetics
http://www.9icg-brazil2010.info

24-29 MAY 2010
San Diego, California, USA
5th International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics
http://conference.mst.edu/5geoeqconf2010/

7-10 JUNE 2010
Moscow
ISSMGE - International Geotechnical Conference - Geotechnical Challenges in Megacities www.geomos2010.ru

28 JUNE – 1 JULY 2010
Hönggerberg Campus, Zurich, Switzerland
7th International Conference on Physical Modelling in Geotechnics
www.icpmg2010.ch/

5-10 SEPTEMBER, 2010
Auckland,
Aotearoa New Zealand
11th IAEG Congress - Geologically Active
http://www.iaeg2010.com/

8-12 NOVEMBER 2010
New Delhi, India
6th International Congress on Environmental Geotechnics 6icegdelhi@gmail.com

2011

21-26 MAY 2011
Helsinki, Finland
The World Tunnel Congress 2011 and 37th ITA general Assembly
http://www.ril.fi/web/index.php?id=641

23-28 MAY 2011
Hong Kong, China
XIV Asian Regional Conference Soil Mechanics and Geotechnical Engineering

13-16 JUNE 2011
Maputo, Mozambique
XV African Regional Conference on Soil Mechanics and Geotechnical Engineering
See ISSMGE website

13-19 SEPTEMBER 2011
Athens, Greece
XV European Conference on Soil Mechanics and Geotechnical Engineering
See ISSMGE website

2-6 OCTOBER 2011
Toronto, Ontario, Canada
XIV Panamerican Conference on Soil Mechanics and Geotechnical Engineering & V PanAmerican Conference on Learning and Teaching of Geotechnical Engineering, & 64th Canadian Geotechnical Conference
See ISSMGE website

7-11 NOVEMBER 2011
Melbourne, Australia
11th Australia - New Zealand Conference on Geomechanics

NEW ZEALAND GEOTECHNICAL SOCIETY INC.

Objects

a) To advance the education 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.
d) To ensure that the learning achieved through the above objectives is passed on to the public as is appropriate.

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	**$75.00**
Students	**Free**
Annual IPENZ service centre fee applies to all NZGS members who are not members of IPENZ	**$42.75 (incl GST)**

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 Management Secretary. The postal address is:

NZ Geotechnical Society Inc, P O Box 12 241, WELLINGTON

The Secretary
NZ Geotechnical Society Inc.
The Institution of Professional Engineers New Zealand (Inc)
P.O. Box 12-241, WELLINGTON

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: (   )** .... **E-MAIL:** ....

DATE OF BIRTH ....

ACADEMIC QUALIFICATIONS: ....

PROFESSIONAL MEMBERSHIPS: .... **Year Elected**..

PRESENT EMPLOYER: ....

OCCUPATION: ....

EXPERIENCE IN GEOMECHANICS: ....

STUDENT MEMBERS: ....

TERTIARY INSTITUTION: .... **SUPERVISOR:** ....

SUPERVISORS 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 (IAEG)	Yes/No
& the Environment (with Bulletin)	Yes/No

DECLARATION: If admitted to membership, I agree to abide by the rules of the New Zealand Geotechnical Society

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 2009

Publication Name	List Price Members	List Price Non-Members
New Zealand Geomechanics Society Conferences:		
Proceedings of Technical Groups, Vol 22, Issue 1G **(1 left)** *Geotechnical Issues in Land Development* Hamilton 1996	$20	$35
Proceedings of the New Zealand Geotechnical Society Symposium – *Roading Geotechnics 98* Auckland 1998	$40	$70
Proceedings of the New Zealand Geotechnical Society Symposium – *Engineering and Development in Hazardous Terrain* Christchurch 2001	$50	$70
Proceedings of the New Zealand Geotechnical Society Symposium – *Geotechnics on the Volcanic Edge* Tauranga 2003	$50	$70
Proceedings of the New Zealand Geotechnical Society Symposium – *Earthquakes and Urban Development* Nelson 2006	$50	$70
Proceedings of the 18th New Zealand Geotechnical Society Symposium –	$50	$70
Soil-Structure Interaction, Auckland 2008. (CD)	$20	$25
Australia – New Zealand Conferences on Geomechanics:		
Proceedings of the 2nd Australia – NZ Young Geotechnical Professionals Conference, Auckland, December 1995	$25	$40
Proceedings of the 5th Australia – NZ Young Geotechnical Professionals Conference, Rotorua, March 2002 (spiral bound reprint)	$75	$85
Proceedings of the 6th Australia – NZ Conference on Geomechanics Christchurch, February 1992	$50	$100
Proceedings of the 9th Australia – NZ Conference February 2004 – 'To the enz of the Earth' **(Vol 2 only)**	$150	$200
Other Publications:		
NZ Geomechanics News Collection 1970–2003 Volumes 1–66 (CDRom)	$25	$40
2005 Soil & Rock Guideline	$25	$50
Shear Vane Guidelines	$15	$20
Back Issues of *NZ Geomechanics News* **(selected issues)**	$20	$20

Prices do not include GST or postage & handling

Orders to: Amanda Blakey, Management Secretary. Email: nzgeotechnicalsociety@xtra.co.nz

ADVERTISING INFORMATION

NZ Geomechanics News is published twice a year and distributed to the Society's 700 plus 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.

Advertisement Location	Single Issue	Advert. Size (mm)
Black & White		
Full Page Internal	$270	185 wide x 265 high
Half Page Internal	$210	90 wide x 265 high
		185 wide x 130 high
Quarter Page Internal	$180	90 wide x 130 high
Colour		
Back Cover	$720	210 wide x 297 high
Inside Cover (Front or Back)	$600	210 wide x 297 high
Full Page Internal	$480	210 wide x 297 high
Half Page	$240	175 wide x 130 high
A3 Centrefold	$900	420 wide x 297 high

Inserts

Insert to be posted with magazine – $240/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. A 3mm bleed is required on all ads that bleed off the page. Bleed must be set up on all files that are supplied.
4. Advertiser to provide all flyers.

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

Management Secretary
Amanda Blakey
Email: nzgeotechnicalsociety@xtra.co.nz