NEW ZEALAND GEOTECHNICAL SOCIETY INC

DECEMBER 2013 issue 86

NZ GEOMECHANICS NEWS

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SPECIAL FEATURE

Waterview Connection

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19TH NZGS SYMPOSIUM REVIEW

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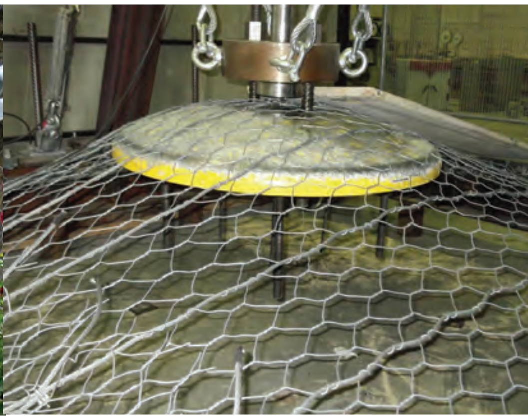

NEW ZEALAND GEOMECHANICS NEWS

CONTENTS

DECEMBER 2013, ISSUE 86

Cover image: TBM cutterhead being lowered into position (photograph supplied by the Well-Connected Alliance).

CHAIRMAN'S CORNER

I WAS FORTUNATE to be able to travel to Paris to collect our award as the ISSMGE outstanding member society for 2013. It is a great honour for us all, and I thank Beca and the Society for covering the cost of my travel to receive the award in person. I attended the ISSMGE Conference and the Council Meeting as the official NZGS representative. Mick Pender has prepared an insightful summary of the Conference, which is contained elsewhere in this edition. My observations from the Council meeting are also reported.

There have been a number of significant developments over the last six months, and considerable effort continues to be put into the running of our Society and into contributions to the wider profession. I have set out the highlights below, and would like to thank Amanda, Charlie, our Geomechanics News Editors, and our two conference organising committees for their work.

New ISSMGE Representative

It is great to have Mick Pender as our new ISSMGE Representative. He brings a wealth of wisdom and connections to the role and I'm sure he will add useful insights to our committee meetings. We have been very fortunate to have has Professor Michael Davies as our VP for the last four years and I wish him well in his new venture. Our new Regional VP, Professor Mark Jaksa, was at the Paris Conference, and I was pleased to be able to spend some time with him.

Conferences

Our 19th Geotechnical Symposium will be behind us by the time you read this. Tony Fairclough and his committee have been working hard to make this a success. This promises to be a great event, and I am looking forward to Queenstown in November.

Planning for the next joint ANZ conference to be held in Wellington in early 2015 is picking up momentum, with the appointment of The Conference Company as organiser. I'm pleased that a broad theme has been settled on: Changing the face of the earth – geomechanics and human influence. Large events such as this require a big effort to make them a success, and it can take a while to get the team properly firing. A concerted effort is required from all who volunteer for roles such as these, and the Society is very grateful to the individuals and their respective employers for making themselves available. We now have little more than a year to get this conference fully organised, so there is no time to waste!

Later in 2015, the 6th International Conference in Earthquake Geotechnical Engineering will also require our support. This is also beginning to feel very close, and you can be sure we will be looking for volunteers to assist Misko in the near future.

Gavin has specialized in geotechnical engineering since graduating from Auckland University in the mid 1980's. Following graduation, he spent seven years with Arup Geotechnics in the UK and Australia working on large building and infrastructure projects. In that period Gavin spent a year at Imperial College, London and was awarded an MSc and DIC in Soil Mechanics and Engineering Seismology.

He joined Beca on his return to NZ in 1993 and has led its Geotechnical group in Auckland, and geotechnical and multi-disciplinary teams on projects throughout New Zealand and in Australia and through South East Asia. Gavin's current focus is on the technical direction and review of projects, and his current challenges include highway embankments on peat, mine infrastructure in Indonesia, and deep basements in Singapore. Variety is the spice of life, and is what attracted Gavin to geotechnical engineering.

Registration of Engineering Geologists

A joint IPENZ/NZGS letter was sent to the Chief Executive of every TLA in the country in August, advising them of the PEngGeol quality mark and registration requirements. The aim of this letter is to increase awareness of the equivalence, for some areas of geotechnical work, with CPEng as the message hasn't travelled very widely to date. I'd like to thank Ann Williams and Geoff Farquhar for their wordsmithing of this letter, and Jeff Wastney and Nicki Crauford of IPENZ for adding the IPENZ weight to it and getting it out the door. A copy of the letter is available for downloading from our website for any members who need it.

Registration of Geotechnical Engineers

In a first for some time, the Society sought the views of members on aspects of an IPENZ consultation document on registration of engineers. The response was underwhelming, with around 20 respondents (including me!) from some 600 ISSMGE affiliates who presumably count themselves as engineers, but was used to inform our formal submission to IPENZ. Most of those who did respond took the time to thoughtfully set out the basis of their views.

Seismic Guidelines

Work on the first three modules of the Geotechnical Earthquake Engineering Practice continues but at a frustratingly slow pace. While a somewhat inevitable consequence of relying on well intentioned specialist practitioners in the most in-demand sector of our membership, we really do need to give this the upmost focus.

Industry Engagement

I continue to represent the NZGS on the Engineering Reference Group established by MBIE to overview building and construction policy and operational developments. The fourth meeting of the ERG occurred in October, and we are beginning to get some traction. The current focus is on regulation of the engineering profession, a topic near to the heart of many of us.

We have been asked to provide representatives on two committees reviewing standards, and I am heartened by the willingness of our members to contribute. My thanks go to Tony Fairclough and Kevin Anderson for offering to be our representatives on these committees, Site Investigations and Concrete Structures, respectively.

Our Regions

Following a fairly frank reminder to our branch coordinators and a call to arms to our wider membership, we are beginning to see some flickers of life in regions that have been dormant for quite a long time. I encourage all our members to support your local branch activities, and to attend any event that you might stumble across in your travels. Don't be shy about taking a few minutes to introduce yourself and describe what you're up to in their region as well as your own. It all adds to the vibrancy and success of our society.

Awards

The committee has been considering a range of awards and opportunities for industry recognition – the Geomechanics Lecture, Life Membership, FIPENZ nominations. We have prepared a short list of candidates and look forward to making formal announcements in due course.

With my very best wishes for the holiday season

Gavin Alexander
Chair, Management Committee

EDITORIAL POLICY

NZ Geomechanics News is a biannual bulletin 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 are 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 bulletin 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.

EDITORIAL

I WOULD LIKE to welcome Ross onto the Editorial team of NZ Geomechanics News. He has been a great help with this Issue and has prepared a member profile which provides a full introduction for those who are interested.

I grew up in a family of five kids on a farm in the Hawkes Bay. When I was ten years old the family moved to Auckland and one of the most challenging aspects for me was moving from a school of fifty pupils to one with over five hundred. While I miss aspects of Hawkes Bay I can't deny the great opportunities Auckland has presented me. The latest has been the opportunity to work on the Waterview Connection Project, which is the subject of our special feature. This project adds an alternative route for Auckland through-traffic and will therefore fundamentally change how people do business, visit, live and play in Auckland. It is exciting to be involved in a diverse project which will enhance and increase the beneficial opportunities I have enjoyed while living here.

Enough about Auckland! I hope you enjoy an interesting Issue with articles on projects from around the country. The recent 19th NZGS Symposium is reviewed and this very successful event is testament to the interesting and quality work which has filled a busy year.

Have a great holiday, a merry Christmas and we'll see you next year.

Hamish Maclean, NZ Geomechanics News Co-editor
HMaclean@tonkin.co.nz

Hamish is a Geotechnical Engineer with Tonkin & Taylor Ltd in Auckland. He completed his Civil Engineering degree at The University of Auckland. Following valuable construction experience working for Fletcher Construction on the later stages of the second Manapouri tailrace tunnel, he has spent the past seven years working as a geotechnical engineer in the Tonkin & Taylor Auckland office. This has included a wide variety of projects with a focus on retaining wall design and landslip assessment and remediation.

Ross is an Engineering Geologist with SKM in Auckland. He trained in the UK at Edinburgh and Newcastle, and has since worked on construction prjects ranging from motorways and railways to geothermal power stations and wharf structures. He has a particular interest in geohazard assessment, investigation and remediation. He has worked in the UK, Ireland, Australia, Java, Sumartra and New Zealand.

RECENT EVENTS SUGGEST that the geotechnical industry in New Zealand is in fine shape. We have great practitioners, leading academics, and an energetic society. It will be of little surprise to members that the NZGS won the ISSMGE 'Outstanding Member Society' award for 2013. We have one of the highest per-capita memberships across the ISSMGE, an impressive and varied range of visiting speakers helping to keep us up to speed with developments around the world, and a great publication in NZ Geomechanics News to enable us to share our knowledge.

NZ Geomechanics News is constantly evolving. It's important that the stated aim – to keep members in touch with matters of interest within the geo-professions both locally and internationally – is met. The Editors want to know what you would like to see changed, and just as importantly what should stay the same. We will be issuing an online survey in late December to elicit your opinions. The results from this will steer the future development of the publication, and I urge you to take this opportunity to be involved. We will publish a summary of the results in the next issue. If any members feel uncomfortable with the online survey format, we also welcome emails, letters or pigeon post to the editors.

Perhaps most importantly of all, the contribution of articles to Geomechanics News from all across the membership is essential to the future. When the first NZ Geomechanics News was published the chairman at the time, J.H.H. Galloway, said, "If you make good use of the newsletter by writing to the Editor expressing your point of view, contributing articles on your experiences or problems, or discussing the experiences of others it can grow into a lively and useful thing. But if you ignore it, it will droop and die." That sentiment is as true today as in 1970.

So please stay involved. Join the committee, respond to the survey, or write a letter to the Editor. The Society can only continue to be a success if you make it so.

Ross Roberts, NZ Geomechanics News Co-editor
ross.c.roberts@gmail.com

LETTER TO EDITOR

25 November 2013

Hamish Maclean
Editor
NZ Geomechanics News
By Email: HMaclean@tonkin.co.nz

Dear Hamish

Recognition of Commitment

I write as I fly home to Auckland having attended the 2013 NZGS Symposium, Hanging by a Thread. Congratulations to the organisers for an excellent conference with outstanding contribution from all of the authors, keynote speakers and invited speakers.The calibre of the work being undertaken in New Zealand or by New Zealand based practitioners is encouraging for an industry that is often second cousin to other engineering disciplines.

But the main reason for writing is to pay particular respect to two of the invited speakers who presented after lunch on the Friday afternoon. I refer of course to Don McFarlane and Mark Yatton, and their presentation on the response and recovery to earthquake related rock fall in the Port Hills, Christchurch, through their work and involvement with the Port Hills geotech Group.

It was clear from their demeanour and delivery just how much they were, and continue to be, affected by the events in Christchurch during February to June 2011. But I can assure you, they are not alone. As a former member of the now dissolved PHGG, listening and reliving the work of this collective in the Hills through Mark and then Don's heart felt commentary and photographs, the emotions started to run high, despite my part time involvement. So I can only imagine how the others from the group present in the room were feeling.

Whilst there has been a significant amount written on the causes, effects, responses, investigation and analyses of the earthquake sequence we endured in Canterbury, the impact to those engineers and scientists actively working, recording and reporting in what was an uncertain, difficult and on occasion downright dangerous environment is often overlooked. The engineering and geological aspects were, to an extent, exciting and what we trained for, but the humanistic side of the work was unchartered waters. The daily and relentless interaction with Civil Defence, Council, CERA, insurers, individual house owners and communities left many of the PHGG exhausted. The upside was the creation of a team with such a strong bond between the group, that collaboration and support became the norm, regardless of their respective parent organisation.

It was unmistakably evident from their presentation just how involved and dedicated Mark and Don were in their work, not just technically and politically, but emotionally. They presented on behalf of the Port Hills Group, but their personal contributions and commitment should be earmarked and celebrated. I wanted to stand up at the end of the talk and express the fact that these individuals deserve a level of appreciation that perhaps they have not yet enjoyed, but such was Don's passion for telling the story, the session overran and the my opportunity was quickly lost.

This letter is to a certain extent my only way of putting that right, but ideally, the geotech community needs to applaud them for carrying not only themselves but the reputation of geologists and geotechnical engineers with such dignity and professionalism.

Yours sincerely,

Peter Forrest
Aurecon

THE SECRETARY'S NEWS

IT WAS FANTASTIC to see so many members at the Symposium in Queenstown. Congratulations to the Organising Committee and everyone that contributed to such creating a successful event. If you are interesting in purchasing a copy of the proceedings please see below.

The NZGS currently has just shy of 1050 members. It is a pleasure to welcome all new members over the past year.

Where relevant, we 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. Our Branch co-ordinators work hard to organise presentations and events around the country throughout the year. Please remember to say thanks to these people who volunteer to find guest speakers, venues and sponsors – and perhaps lend a hand if the opportunity arises in your region.

A call for nominations for next year's NZGS Management Committee will go out in December (after this issue of NZ Geomechanics News is released). Please take some time over the Christmas break to consider standing for a role on the Committee. We have had good interest in recent years, culminating in a number of online 'elections'; and we anticipate the same again.

Please do contact me for any assistance you might require or any queries you might have. It is also a good idea to keep me informed of any change of contact details, emails and addresses.

It was fantastic to see so many members at the Symposium in Queenstown. I also met many people from the companies that support the Society through this Bulletin's advertising, Branch events and of course as exhibitors at our Symposia. Here are a few more awards, firmly tongue in cheek, in addition to the Student and Young Geotechnical Professional awards announced at the Symposium. Congratulations to all!

19th NZGS Symposium Exhibitor "Awards"

Best Pen – Aurecon (nice green grippy bit)
Best Cap – Avalon (breathable material – nice for hot-headed engineers); second Pro-Drill (always on-trend black)
Best Stand Gimmick – Geotechnics, Strength Gauge Strong Person (we could smell the testosterone); second Coffey core-sample (relevant and earthy)
Best 'Nice Guys' – Abseil Access (most chatty – or was that verbose?)
Most Expansive Exhibitor – Ground Investigation (utilising their space and then some – good on ya mate)

Amanda has been the NZGS Secretary since 2008. She works from home in Glendowie, Auckland, whilst juggling family (two children and husband) and an international ice skating career. OK, perhaps just the family. She enjoys the Game of Thrones books, cooking and sailing. In the distant past she worked as a planner at the now deceased Waitakere City Council, and even further back for URS.

Best Lolly Jar – Brian Perry Civil (pineapple lumps and mini Choc Fish – need I say more?); closely followed by OPUS' chocolate mud (we sense a chocolate theme…)
Best Bottle Opener – Hiway Geotechnical (just looked cool)
Best Cheeky Exhibitor – Perry Geotech (see photo)
Best 'Hanging by a Thread' – Queenstown Gondola after Symposium dinner

Thank you to Simon Woodward (previous NZGS Committee Member) for his continuing contribution to monitoring the LinkedIn group page for NZGS - I know this requires diligent effort we are grateful for his dedication to this role.

And, finally, season's greetings to you all for a happy holiday and summer break.

Amanda Blakey
Management Secretary
secretary@nzgs.org

Please contact Amanda Blakey if you wish to purchase a hard copy of the 19th NZGS Symposium Proceedings for $100 including GST, plus Post and Packaging.

INTERNATIONAL SOCIETY REPORTS

International Association for Engineering Geology and the Environment

Australasia VP Report: May 2013

SINCE MY LAST report I have participated in the Executive and Council meetings of the IAEG held in conjunction with the Asia Regional Conference of the IAEG in Beijing in September and the committee meeting of the Australian Geotechnical Society held in Sydney in October. I also gave a lecture to the staff of the Chinese Academy of Sciences Institute of Geology and Geophysics on ground hazards and damage from the Canterbury Earthquake Sequence. I outline some of the discussions that directly affect NZ members of IAEG below.

Membership and Representation as Australasian Vice President IAEG:

New Zealand membership of IAEG now exceeds that of Australia. Representation of NZ on the IAEG Executive has traditionally been divided between Australia and New Zealand on a 2:1 basis (2 terms Australia: 1 term New Zealand). This was because the number of Australian members was significantly greater than the number of New Zealand members. The AGS management committee agreed to change the proposal to adjust representation on the IAEG Executive to 1:1, term for term. This means that New Zealand Engineering Geologists have the (very great) opportunity to take on the role of Australasian VP every 4 years.

Hans Cloos Medal and Marcel Arnould Medal

At the Council meeting of 2011, held in Moscow, Carlos Delgado proposed the creation of a Marcel Arnould Medal, named after the former IAEG President and Honorary IAEG President; it was unanimously approved. It was proposed that the medal be awarded to an IAEG member for his/her outstanding service to the Association.

Up until 2012, the Hans Cloos Medal has been awarded to "an engineering geologist of outstanding merit… a person of international repute who has made a major contribution to engineering geology in his/her written papers or to the development of engineering geology *and/or the IAEG in their own area*."

With the introduction of the Marcel Arnould Medal, the intent is to separate technical merit (which would remain with the Hans Cloos Medal) from services to engineering geology (which would be the focus of the Marcel Arnould Medal). This required some adjustments to part E of the Bylaws which were agreed by Council in Beijing 2013.

In addition, it was agreed that the recipient of the Hans Cloos Medal should be invited to present a Hans Cloos Lecture addressing the nominated work at the event at which he/she receives the Medal. It is hoped that the recipient will also be invited to give the lecture at other venues during the following two year period. I note that past recipients of the award include David Varnes, Bill Dearman and other distinguished Engineering Geologists.

Council agreed unanimously that the first recipient of the Marcel Arnould Medal (for service to IAEG and the profession) should be Dr Brian Hawkins of the UK in recognition of his service to IAEG and in particular his role as Editor of the Bulletin of Engineering Geology.

Richard Wolters Prize

A new procedure for the Richard Wolters prize that was trialled at the IAEG Congress in Auckland 2010 and at the International Landslide Symposium in Banff 2012, has been accepted and changes to the Bylaws of the Association made to formalise these. The new procedure better spreads the opportunity to contest the prize to non-academics, reduces the maximum age of applicants to 35 and includes a presentation to be given at the conference at which the award is made and an opportunity for discussion of the applicant's work. The next Richard Wolters Prize will be awarded in Torino in 2014. The age cut-off will remain at 40 for the Torino event to allow older candidates to participate who could not have anticipated the change in age cut-off.

Commission-Led Awards

In order to encourage increased activity and productivity among the Commissions, two types of awards are being proposed. The first is an International Research programme (IRP-IAEG) and the second, Science and Technology awards (STA-IAEG). Both award types are aimed at encouraging research, innovation and international collaboration among our members. No decision was made on the awards at Beijing, but detailed proposals will be put to Council at Torino in September next year.

Newsletter

The new secretariat has reinstituted a regular (6 monthly) newsletter. Newsletters can be viewed on the IAEG (www.iaeg.info) and NZGS websites. Please submit contributions to Amanda Blakey at secretary@nzgs.org.

Bulletin of the IAEG

One of the issues that we discussed was how to get more of our members to subscribe to the Bulletin of Engineering Geology and the Environment. Currently, members have a choice to either be a member of the IAEG with the Bulletin, or without it. Currently, those who pay the additional sum for the Bulletin receive a paper copy *and* have access to the on-line version. However, at present, only around half of the membership actually subscribes to the Bulletin. The situation is exacerbated by members accessing the Bulletin via their company or university subscription rather than their own. The Executive wants to see more IAEG members receive the Bulletin in one form or the other. Consequently, we discussed whether there could be three classes of IAEG membership: without the Bulletin, with the on-line Bulletin only, and with the on-line and paper copy of the Bulletin. The Editor in Chief is exploring cost implications of these alternatives.

ANZ Conference

It had been brought to my attention that the ANZ Conference is in fact the Australasian Regional Conference of the ISSMGE. However because in Australasia we bring ISSMGE, ISRM and IAEG together under our respective Geotechnical Societies and the ANZ Conference addresses the range of geotechnical topics and affiliations, this seems inappropriate for our Region. The AGS Management Committee agreed that the next ANZ Conference should be the Australasian Regional conference of all three of the sister societies (on the understanding that ISRM and IAEG seek no funds contribution).

Upcoming International Meetings

There continues to be widespread interest in hosting upcoming IAEG sponsored events:

- Bidders for the next Asia Regional Conference (2015) are Japan and India
- Bulgaria has also invited the IAEG Executive and Council to meet in Sophia in 2015
- It was agreed that the IAEG will revert to meeting at IGC Conferences in accordance with our statutes. This means that the Executive and Council meetings for 2016 will be in South Africa
- San Francisco advised of its bid to host IAEG2018.

PEngGeol

I also took the opportunity to discuss PEngGeol with the AGS committee. IPENZ has hosted sessions in Christchurch and Auckland on Advancing your Professional Recognition for Engineering Geologists. Both were well attended. IPENZ has received a large number of applicants for assessment of competence under PEngGeol. Well done for your support of this register!

IAEG Sponsored Conferences

The next IAEG Council meeting will be held in conjunction with the Asia Regional meeting in Beijing "Global View of Engineering Geology and the Environment" September 24 – 25, 2013. See www.iaegasia2013.com.

The submission of abstracts to the next IAEG Congress to be held on the 50th Anniversary of IAEG in Torino, Italy, closed on 15 May 2013. Register on www.iaeg2014.com to receive updates.

New Zealand

As you will be aware, New Zealand has been working towards professional recognition of engineering geologists through IPENZ (PEngGeol). Guidelines and competency standards have been established and approved by IPENZ following consultation and the Register of Professional Engineering Geologists is now live. This is an outstanding achievement for the profession and I would like to acknowledge in particular the roles of Philip Robins (past Chair of NZGS, who was determined that this would be achieved), Geoff Farqhuar (many time committee member of NZGS and staff assessor for CPEng who has provided guidance throughout the process and facilitated the interface IPENZ), to Jeff Wastney of IPENZ who represented us to the IPENZ Board, responded to submissions and facilitated training as well as contributing to our many sub-committee meetings, and to David Burns (immediate past Chair of NZGS) and Warwick Prebble (formerly University of Auckland) who energetically participated in the sub-committee meetings and provided robust debate in the drafting of the guidelines and competency standards. IPENZ is now waiting to receive your PEngGeol application! http://www.ipenz.org.nz/IPENZ/finding/PEngGeol/

Ann Williams
IAEG Vice President, Australasia

International Society of Soil Mechanics and Geotechnical Engineering

Australiasia VP Report: October 2013

I'D LIKE TO begin this report by thanking the Committee Members of the Australian Geomechanics Society and the New Zealand Geotechnical Society for electing me to the role of Vice-President for Australasia for the next 4 years. It is with great pleasure and humility that I take on this role and I will do all I can to enhance the society for the benefit of members of the AGS and NZGS. I'd like to thank my predecessor, Prof. Michael Davies, for his excellent work in the role of VP over the last 4 years and for the support and encouragement that he has given to me during that period. He, the Immediate Past President, Prof. Jean-Louis Briaud and the Board have left the society in an incredibly strong state. It is a credit to their efforts and enthusiasm.

ISSMGE Council Meeting, 1 September 2013

The Council of the ISSMGE, which is constituted by representatives of each of the Member Societies together with the Society's officers, meets every two years. A meeting of Council always coincides with the quadrennial international conference of the ISSMGE (ICSMGE); whilst the one between these is held in association with one of the Society's regional conferences. At the Council meeting which coincides with ICSMGE the president is selected and a new ISSMGE Board constituted, therefore meetings that take place at a regional conference are frequently referred to as the "mid-term" meetings of Council. The timing of these mid-term meetings allows the society to reflect on the progress of initiatives brought in by the President and the Board and discuss the current and future activities of the Society.

The key discussions and decisions made at the Council Meeting included the following:

- **Item 5 – Membership:** The membership has grown to 88 member societies with nearly 20,000 individual members. Belarus and Bosnia & Herzegovina joined as Member Societies in July 2012 and May 2013, respectively, and the applications from Malaysia and Guatemala were approved at the Board Meeting on 31 August 2013. The Corporate Associates had grown to 59.
- **Item 6:** Proposed changes to make the Statutes and Bylaws gender neutral were approved.
- **Item 8:** The incoming Regional Vice-Presidents were noted as:
  - Dr Fatma Baligh (Vice-President Africa)
  - Professor Ikuo Towhata (Vice-President Asia)
  - Professor Mark Jaksa (Vice-President Australasia)
  - Professor Antonio Gens (Vice-President Europe)
  - Professor Paul Mayne (Vice-President North America)
  - Professor Jarbas Milititsky (Vice-President South America)

- **Item 9:** Professor Roger Frank (pictured) was elected as ISSMGE President 2013 – 2017.
- **Item 10:** The next Council Meeting will be held on Sunday, 13 September 2015 in association with the 16th European Conference on Soil Mechanics and Geotechnical Engineering in Edinburgh, UK.
- **Item 11:** The 19 ICSMGE will be held in Seoul, Korea. This was an extremely disappointing decision from the region's perspective, as this represents the 6th time that the AGS (and region) has unsuccessfully bid to host the ICSMGE. I'd like to take this opportunity to thank the following individuals for their very hard work over the last two years in preparing the Sydney 2017 bid: Graham Scholey, Sam Mackenzie, Rebekah Hayne, Mark Jaksa, John Carter, Michael Davies, David Airey, Harry Poulos, Max Ervin, Mark Randolph and Henk Buys.
- **Item 15:** New guidelines for Technical Committees were noted.
- **Item 20:** Progress with establishing the ISSMGE Foundation as a charitable organisation was noted.
- **Item 21:** The new conference manual was noted.
- **Item 27:** The independently reviewed accounts for 2011 and 2012 were approved.
- **Item 28:** The proposed budget for 2013 – 2015 was approved.

Subsequent to the Council Meeting an ad-hoc Board meeting was held on Thursday, 5 September 2013 with the new Board. At this meeting the First Vice-President, Prof. Antonio Gens, was selected by the President, and I was selected also to fulfil the role of Treasurer. The next Board meeting is to be held in London on March 18 or 19, 2014 and will coincide with the Rankine Lecture.

18th ICSMGE, Paris

The 18th International Conference on Soil Mechanics and Geotechnical Engineering, which was held in Paris from 2 – 6 September, 2013 and was well organised and the largest to date with over 2,100 delegates.

Technical Committees

As noted by the Immediate Past President in his 1370 day report "The Technical Committees will continue under the current chair without any interruption through the election of the new President. The Chairs will rotate off if they have been in that position for 8 years or more. If they have been Chair for 4 years they can reapply and be selected for another 4 years at which time they must rotate off."

Professor Mark B Jaksa
Vice-President for Australasia and Treasurer

International Society for Rock Mechanics

Australasia VP Report: September 2011

Submission of abstracts to the 2014 ISRM International Symposium – ARMS8 in Sapporo, Japan, is now open

The online submission of abstracts to the 2014 ISRM International Symposium and 8th Asian Rock Mechanics Symposium opened on 1st November, 2013. The Symposium will take place in Sapporo, Japan, 14-16 October 2014. The theme of the Symposium is "Rock Mechanics for Global Issues Natural Disasters, Environment and Energy.

For more information on the 2014 ISRM International Symposium visit the symposium website.

Important deadlines:

Abstract submission: 31 December, 2013

Author Notification: 31 January, 2014

Paper Submission: 30 June, 2014

ISRM COUNCIL MEETING

The ISRM held its Council meeting in Wrocław, Poland, in conjunction with EUROCK 2013, organised by the Polish ISRM National Group, the Polish Society for Rock Mechanics. 40 of the 53 National Groups were either present or represented. The Council was also attended by two Past-Presidents, the Chairmen of the ISRM Commissions and representatives of the IAEG the ISSMGE and the ITA.

Dr. Eda Quadros elected as ISRM President for the term 2015-2019

One nomination for President of the ISRM for the term 2015-2019 was received: Dr Eda Quadros, from Brazil. Dr Quadros was elected by acclamation as the next ISRM President. She will start her term of office after the 14th International Congress of the ISRM, in Montréal, Canada, in 2015.

Membership

The ISRM has now 7063 individual members and 145 corporate members, belonging to 53 National Groups. Albania, Hungary, Tunisia and Vietnam joined the ISRM in the last year. This is the highest number of individual members ever and represents an increase of 4% since 2012. 42% of the members come from Europe, and Asia has been the fastest growing region in the last years.

Commissions

A report on the activities of the Commissions was presented by Prof. Yuzo Onishi, chairman of the Technical Oversight Committee.

The following Commissions are now active:

- Application of Geophysics to Rock Engineering – Preservation of Ancient Sites – Testing Methods – Hard Rock Excavation
- Rock Engineering Design Methodology – Radioactive Waste Disposal – Rock Dynamics – Spall prediction
- Underground Research Laboratory Networking – Crustal Stress and Earthquake – DDA – Education
- Underground Nuclear Power Plants – Petroleum Geomechanics – Soft Rocks – Grouting
- Coupled THMC processes in Geological Materials and Systems

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- Coupled THMC processes in Geological Materials and Systems

For more details on any of the commissions, or to participate, contact me at dbeck@beckengineering.com.au

ISRM Online Lectures – 50th anniversary of Vajont Dam Catastrophe

The ISRM started a series of online lectures in 2013. They are broadcast at fixed dates and you can ask questions to the lecturers during a few days. Afterwards, they stay online on the website, where they can be watched. Three were already given. The 4th lecture is planned for December and will be given by Prof. Eduardo Alonso, from Spain, on Catastrophic landslides; the legacy of Vajont, to mark the 50th anniversary of this large landslide.

Rocha Medal 2014

The Board awarded the Rocha Medal 2014 to Dr Mandadige Samintha Anne Perera, from Australia, for the thesis Investigation of the effect of carbon dioxide sequestration on coal seams: A coupled hydro-mechanical behaviour. She will receive the award at the 2014 ISRM International Symposium in Sapporo, Japan.

The Board also awarded 2 runner-up certificates to Dr Ricardo Resende, from Portugal, for the thesis An investigation of stress wave propagation through rock joints and rock masses and to Dr Sevda Dehkhoda, from Australia, for the thesis Experimental and numerical study of rock breakage by pulsed water jets.

Submissions for the Rocha Medal 2015 shall be sent to the Secretariat by 31 December 2013.

Upcoming meetings

26-28 May 2014, Vigo, Spain. EUROCK 2014. An ISRM Regional Symposium

10-13 September 2014, Goiânia, Brazil. VI Brazilian Rock Mechanics Symposium. An ISRM Specialised Conference.

14-16 October 2014, Sapporo, Japan. 8th Asian Rock Mechanics Symposium. The 2014 ISRM International Symposium.

4-5 November 2014, Sydney. The 3rd Australasian Ground Control in Mining Conference. An ISRM Specialised Conference.

10-13 May 2015, Montréal, Canada. ISRM 13th International Congress on Rock Mechanics.

24-25 September 2015, Island of Ischia, Italy. 4th Workshop on Volcanic Rocks and Soils. An ISRM Specialised Conference.

7-9 October 2015, Salzburg, Austria. EUROCK 2015 – Geomechanics Colloquy. An ISRM Regional Symposium.

May 2016, Cape Town, South Africa. African Rock Engineering Symposium. An ISRM Regional Symposium.

29-31 August 2016, Cappadocia. Turkey. EUROCK 2016. An ISRM Regional Symposium.

October 2016, Bali, Indonesia. ARMS 2016 – 9th Asian Rock Mechanics Symposium. An ISRM Regional Symposium.

Dr David Beck
Vice President Australasia

NZGS BRANCH ACTIVITIES

Auckland Branch Activity Report

IN THE LAST 6 months a wide range of interesting presentations have been offered at our usual venue at Auckland University, including a first joint presentation with the Auckland Structural Group. Attendances have been excellent, with numbers consistently exceeding 50 people at the events. Live streaming has not been offered through this period, although we will look to provide this as soon as it is made available by the University. The following presentation have been held:

11 June 2013 – Daniel Coats presented on Helical Piles: True Pitch Helixes and Steel Specification for Screw Piles and Anchors. This was the first joint presentation with AGS and we thank them for their support and attendance. The presentation included some fascinating (and terrifying) stories about appropriate selection of steel standards, and the importance of experience in the design and construction of helical piles.

2 July 2013 Prof Vaughan Griffiths presented on the probabilities of failure and factors of safety in geotechnical engineering. He considered the representation of variability in materials and compared conventional factors of safety with probabilities of failure.

9 July 2013 Ken Stokoe discussed seismic measurements and geotechnical engineering. Professor Stokoe is well known in the seismic measurement field and has been in NZ carrying out site testing in Christchurch. His entertaining and engaging presentation contained a good introduction to seismic testing and the challenges and opportunities in that field.

23 July 2013 Huesker Graham Thompson discussed a case history using Geotextile Encased Columns (GEC) and their application in extremely soft sediments in Germany.

13 August 2013 Russell Green described his experiences in identifying paleoliquefaction and the implications for determining design ground motions for a nuclear power plant in the central USA. This included having to canoe down rivers after storms to inspect new cuttings for evidence of old sand boils. An insightful and interesting presentation.

17 September 2013 Jim Benson from SKM presented on the lessons learnt from the Kowloon Southern Link rail project in Hong Kong. The presentation highlighted the challenges involved in constructing a cut and cover and conventional tunnels with an urban environment.

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Pierre is a Geotechnical Engineer with Tonkin & Taylor Auckland. Pierre graduated from the University of Canterbury with a M.Eng and has subsequently worked around Auckland and throughout the United Kingdom and Ireland. He has worked on major infrastructure work, design and build contracts as well as a range of small to medium projects.

Luke Storie
PhD Candidate
Faculty of Engineering
The University of Auckland
Email: luke.storie@gmail.com

Luke is currently undertaking a PhD at the University of Auckland on the earthquake resistant design of foundations. He is investigating the response of a number of buildings in the Christchurch CBD following the 2010/2011 earthquakes and is following on from research that has been undertaken under the supervision of Professor Michael Pender. Previously, following his graduation from the University of Auckland with a BE(hons) and BA conjoint degree in 2009, Luke was a Geotechnical Engineer at Coffey Geotechnics (NZ) Limited where he worked on a range of small to large scale projects in New Zealand and Australia.

Aidan Thorp
Auckland Branch Coordinator
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Aidan is a Geotechnical Engineer with Beca Infrastructure Ltd, based in Auckland. He graduated from the University of Auckland in 2009 with a BE (Hons) and has a passion for slope stability and river engineering. Aidan joined Beca in 2010 and has worked in Auckland, Tauranga and Wellington on large infrastructure projects, as well as a variety of other projects throughout the country.

1 October 2013 Prof Malcolm Bolton (in conjunction with ICE) presented the 52nd Rankine Lecture on performance based design in geotechnical engineer. The lecture was a masterclass in understanding mechanisms of failure and was an engrossing and challenging presentation on how we as a profession carry out our designs and modelling.

The Auckland Branch is in good health, with strong attendances and a number of world experts presenting. We thank all of our sponsors and presenters for their support and look forward to a busy and interesting 2014.

Hawke's Bay Branch

FOLLOWING UP ON the launch of the new Hawke's Bay Branch, we held our inaugural Branch meeting on 18 September. It was a grand success! RDCL generously hosted the inaugural meeting at the Havelock North office, and two presentations by RDCL staff were well-received. The programme included presentations on downhole geophysical testing and Multichannel Analysis of Surface Wave (MASW) seismic testing. Refreshments and good conversations followed.

Several Branch members commented that they would like to engage other professional disciplines with discussions relating to geotechnical engineering and geology. As this can be achieved in conjunction with the Branch goals of connecting local professionals, promoting vibrant discussion on issues affecting our profession, providing a platform for members to enhance their knowledge and understanding of the industry and geotechnical design issues, and to present opportunities for networking and fun, the Branch is seeking opportunities to engage with other professional engineers. Further to this, planning is underway for the second Branch meeting to be held in late November 2013, and invitations will be sent out through the IPENZ network to other professional engineers. However, the Branch will maintain its focus on issues relevant to geotechnical engineering and geology. The November Branch meeting will be kindly hosted by Opus International Consultants at their Ahuriri office location, with refreshments also generously provided. Thanks Opus!

As the Branch co-ordinator, Riley Gerbrandt would like to remind members that the young Branch has many opportunities for local firms or to volunteer a venue for Branch meetings or to sponsor event refreshments. Ideas for presentations about interesting local projects, technical news or advancements in the industry are also sought. Riley would love to hear good ideas for Branch field trips or presentations, so please feel free to contact him if you would like to offer up an idea, suggestion or advice.

Riley Gerbrandt
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Riley is a Chartered Professional Engineer (CPEng) and Geotechnical Engineer with Opus International Consultants Ltd in Napier. He has been serving his clients in the geotechnical engineering industry in both New Zealand and California. He strives to provide practical, timely, and cost-effective engineering solutions. Riley earned BSc and MSc in Civil and Environmental Engineering in California, where he practiced geotechnical engineering and gained his PE (Civil) license. In 2011 Riley and his family moved to New Zealand with both an eagerness to further his geotechnical career and a desire for a better lifestyle for his young family.

Riley's experience incorporates geotechnical investigations and design for land development, structures, roading and infrastructure projects; geotechnical construction observation and quality assurance testing; and design for on-site stormwater and wastewater disposal. He particularly enjoys sub-surface interpretation, earthquake hazard assessments, seismic design and slope stability assessments.

Wellington Branch Activity Report

Meetings Held:

1. Geotechnical practice in Christchurch – a post-earthquake reflection, presented by Ian McCahon. This presentation was previously presented in Christchurch and very well received by the engineering community. It was an interesting and honest account of the performance of structures and the ground in the Canterbury earthquakes, with comments on interesting case studies and thoughts about future geotechnical design practice and associated risk from a highly experienced engineer who has practiced in Canterbury for many years.

2. Performance based design in geotechnical engineering (Rankine Lecture), presented by Prof. Malcolm Bolton. We were fortunate to have Prof. Bolton deliver this lecture to the Wellington community, and it was an interesting and entertaining discussion on the limitations of traditional safety factor-based design approaches and the improvements to be made through the adoption of Mobiliseable Strength Design (MSD) principles in which the designer explicitly considers the stress-strain behaviour of the ground.

Upcoming Activities:

13 November: Lessons learned from the collapse of deep excavations, presented by Prof. John Endicott
Nov/Dec: Memorial Park Alliance – Tunnel site visit.
Early 2014: Manawatu Gorge slip stabilisation/remediation

Andy Hope
Wellington Branch Coordinator
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He joined the Wellington office following completion of a bachelor degree in civil engineering at the University of Canterbury and has since been involved in a wide range of projects in both the North and South Island. Andy's particular areas of interest include the analysis and design for complex engineering problems, with a particular interest in numerical modelling.

Doug Mason
Wellington Branch Coordinator
Opus
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Doug is an engineering geologist and team leader with Opus in Wellington. Doug completed bachelor degrees in geology and history and an MSc (Hons) in geology at Victoria University, carrying out an EQC-sponsored research project into active faulting in Marlborough. He worked for GNS prior to joining Opus in 2004, and has been involved in geotechnical investigations and assessment of hazards and risks for infrastructure and land development projects around central New Zealand. He moved to the UK in 2007 and spent 3 years working on geotechnical and geoenvironmental projects around Wales and southwest England, before returning to Wellington a month before the 2011 Christchurch Earthquake. Doug's particular interests include geomorphology, rock slope stability, and earthquake and landslide hazards.

David Molnar
Wellington Branch Coordinator
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David is an engineering geologist at Aurecon Wellington. He has 5 years of geotechnical experience following graduating at Victoria University in Wellington.

During his professional career he has been involved in a wide range of projects throughout New Zealand, notably including the NZTA SH16 Causeway Upgrade Project and SH2 Muldoon's Corner Improvements, also KiwiRail's North to South Junction which won the 2012 Railway Technical Society of Australia (RTSA) Biennial Railway Project Award.

His areas of specialisation include carrying out geological hazard assessments and site investigations, retaining wall design, construction observation and contract management (NZS 3910).

Canterbury Branch Activity Report

THE CANTERBURY BRANCH has enjoyed a couple of evening presentation recently, including;

- Rapid-fire presentations by members on 1 September. We hope to run a similar format evening early in the new year, so please contact either of the coordinators to register your interest to present.

- 52nd Rankine Lecture by Prof. Malcolm Bolton on 3 October.

These meetings have been well attended and we thank the members for their support.

We are looking forward to the presentation by Prof. John Endicott on the 14 November (note the 'new' venue), and catching up with you at the Queenstown symposium.

As always, if members have any ideas for meeting please let us know (e.g. visiting expert from your company or a construction site visit etc.)

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Edwyn is an engineering geologist with 11 years geotechnical experience in New Zealand, United Kingdom, Caribbean, Algeria and Bulgaria, with skills in the following areas:

Geotechnical investigations for civil engineering works (dams, roads, land development, buildings, landfills etc); Geological hazard assessments for major projects; Engineering geological mapping and aerial photo interpretation; Assessment of risks associated with natural hazards; Groundwater investigations; Peer review and expert witness.

Edwyn's has developed expertise in feasibility studies and geotechnical investigations for infrastructure projects, ranging from dams and reservoirs to roads, wind power developments, buildings, and land stability assessments.

Shamus Wallace
Canterbury Branch Coordinator
Tonkin & Taylor Ltd
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Shamus is an Engineering Geologist who works for Tonkin & Taylor in Christchurch. Passionate about maps and landforms from an early age, Shamus graduated from Canterbury with a BSc Honours in Eng Geol in 2002 and has worked on a variety of geotechnical projects throughout New Zealand, as well as working in London, and travelling around the world, before repatriating to Christchurch. Faced with the aftermath of the 2010/11 Canterbury earthquakes, Shamus has been intricately involved with the land damage assessment team, working for EQC, and looks forward to helping Christchurch emerge from the rubble.

Nelson Branch Activity Report

IN SEPTEMBER WE kicked off our first branch meeting for the year – better late than never! We had a good turnout for the small region of Nelson with most of the local companies represented. We enjoyed a very good presentation on the overview of a dam construction project in Fiji, presented by Paul Wopereis (MWH) and then spent some time planning for future events that we wanted to hold.

In November there is a planned event where as a community we will discuss the outcomes from the December 2011 storm event that caused widespread damage across the region and called on geotechnical specialists to support the Civil Defence emergency. There is a lot to learn from such an event and we are hoping (this is written prior to the Nov meeting) that we can develop some collective agreement/guidelines on how to refine our inputs should we be called to assist in the future.

Its great to see some more connection in our community and start to share the wealth of knowledge we have in our little centre of the universe…

Grant Maxwell
Nelson Branch Coordinator
Asia Pacific Geotechnical Discipline Leader, MWH Global
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David's current role involves setting strategy and managing technical development for the geotechnical team across the Asia Pacific region. He is also acting as the manager for the New Zealand Structural team (a fair amount of 'acting' from a geotech!). He has 15 years experience working across NZ, Australia, Pacific nations and the UK on a variety of projects.

Otago Branch Activity Report

NO ACTIVITY TO report from the Otago Branch. An email invitation sent out to members in mid-February, inviting proposals for talks to provide impetus for branch meetings, elicited no replies. As it seems highly unlikely that nothing of interest is happening in Otago, presumably everyone is thoroughly snowed under with interesting work. As soon as they get a chance, members are most welcome to send in offers to present short talks, there is currently an infinite number of slots available.

David Barrell
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David is a geologist and geomorphologist at GNS Science in Dunedin. South Island born and bred, David's early professional experience included work as a coal geologist in Buller, and as an engineering geologist on the Clyde Power Project. Since joining GNS Science in 1993, he has specialised in Quaternary geology, landform evolution and landscape processes. David very much enjoys the mix of scientific research and applied geoscience that his work entails. He contributes extensively to regional geological and geomorphological mapping, as well as to a range of other disciplines including earthquake geology, groundwater geology, and engineering geology.

STANDARDS, LAW AND INDUSTRY NEWS

NZGS Young Geotechnical Professionals

THE YOUNG GEOTECHNICAL PROFESSIONALS (YGP) group has been formed to represent, support and provide a voice for the young professionals in the NZGS. We represent a lively, increasingly influential and rapidly growing section of Geotechnical Engineers and Engineering Geologists nationwide. 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.

All members of the NZGS who are 35 years old or younger automatically become part of the YGP group. Some members may not realise that they are part of this group and that they have a representative on the committee supporting their needs. As of October 2013 there are 397 YGP members in the NZGS, which is not an insignificant group within the Society. The majority of these members are from the Auckland (37%) and Canterbury (35%) regions, with the next highest regions being Wellington (8%) and Waikato-Bay of Plenty (8%). As members of the YGP group you are encouraged to have a say in NZGS matters through your YGP Representative on the committee and to get involved with your peers around the country.

At the end of 2013, I will be standing down as the YGP Representative as I have overseas work commitments for the first half of next year and personal commitments towards the end of the year. I have truly enjoyed my time on the committee representing the young professionals of the NZGS and am very thankful for all the support that has been given to me and the YGP group. Frances Neeson, from Opus in Christchurch, has volunteered to take over the role next year. She has recently become part of the YGP Liaison Group with Kelly Walker and has been organising YGP events for the NZGS Symposium as well as a recent Canterbury YGP event (see below). Her enthusiasm for the interests of the YGP group and her willingness to jump into organising events will be invaluable in this role.

Latest Activities:

Student Awards

The New Zealand Geotechnical Society Student Awards are presented to recognise and encourage student participation in the fields of geotechnical engineering and engineering geology. This year the awards are being run as a poster competition following on from the success of the new framework introduced last year. This year 7 students have registered for the awards, down significantly from the 25 students that registered in 2012. The due date for posters has been set as Friday 15 November and the posters will be displayed and prizes awarded at the NZGS Symposium between 21 and 23 November.

Three prizes are available for the Student Awards – 1st, 2nd and 3rd place with monetary values of $1000, $500 and $300 respectively. A panel of judges will be formed to decide the award winners, and these will be the same group used by the Symposium organising committee to judge the best presentation by a YGP member. It is also intended that attendees of the Symposium will have the opportunity to vote for their top posters for the judging panel to consider. The winning posters will be published in the next issue of the Geomechanics News.

A question that was brought up during the registration process was the number of authors allowed for each submission. Last year the rule was set that only 1 co-author could be included with each submission. Since we want to encourage as many students as possible at both undergraduate and postgraduate levels, there is a larger importance placed on poster layout and appeal, since postgraduate students have an advantage with academic content. Therefore students doing the same or similar work can still submit their own separate posters on the same or similar topic and will not be disadvantaged. In fact, last year we had a number of students from the University of Waikato submit on the same topic.

YGP Events at the NZGS Symposium

A YGP special presentation has been organised for the NZGS Symposium in November. Professor Harry Poulos and Don Macfarlane will make a joint 1 hour and 45 minute presentation entitled "Poulos and Macfarlane unplugged". It will focus on technical and career tips for young professionals. Frances Neeson and Kelly Walker in Christchurch have done a fantastic job organising this presentation with the Symposium committee and I am very grateful for their help and ideas. As discussed above, the NZGS Student Awards will be presented and awarded at the Symposium. The committee has indicated there will also be an award for the best presentation (and runner up) by a YGP member.

10th ANZ YGP Conference

The 10th Australia New Zealand YGP Conference will be held on the Sunshine Coast, Queensland, Australia from 3rd to 5th of September 2014. The organisers in Australia have just released a first call for nominations, which is included in this issue of the Geomechanics News. Nominations are due on 31 January 2014 and application forms will be sent out soon. A reminder that the Earthquake

Above: Ferrymead YGP and engenerate committee member David Rowland stands infront of 2.4m diameter pier pile casings

Above: Ferrymead Central pier piles

Commission (EQC) and NZGS have awards available for New Zealanders attending the YGP Conference – see http://nzgs.org/awards/young-geotechnical-professionals-conference-awards.htm

YGP Liaison Group and Regional Events

In 2013 a YGP Liaison Group has started to be formed. The idea of the Liaison Group is to gather a group of young professionals from different centres around the country and from different companies to share ideas about what the YGP group and NZGS can do for members aged 35 years and younger, and to organise events in different locations. Frances Neeson, who will take over as the YGP Representative next year, and Kelly Walker from Opus in Christchurch form the start of this group and as discussed above, have already organised YGP involvement in the NZGS Symposium. The next step is to find enthusiastic representatives from the other main centres. In particular we would like to find someone to replace me in Auckland, as well as additional people in the Waikato-Bay of Plenty and Wellington regions. If you are interested or know someone who would be perfect for this please get in touch.

Frances has been in contact with another YGP member, David Rowland, who is on the Canterbury Engenerate committee. Engenerate is a New Zealand wide IPENZ group set up for young professionals with less than eight years' experience and they organise events that range from technical presentations and site visits to social activities. David mentioned to Frances that the Canterbury branch of Engenerate's most successful event this year was a site visit to Bridge Street Bridge to see Jet Grouting ground improvement, and most of the people attending were NZGS members.

Following on from this, Frances and David organised the first combined Canterbury YGP and Canterbury Engenerate site visit to Ferrymead Bridge on 23 October 2013. The site visit was led by HEB Structures (Adrian Blok – Project Engineer) and Opus (YGP's Gemma Hayes – Geotechnical Engineer, and Frances Neeson – Engineering Geologist). Frances reported back that despite a miserable cold wind, 23 people attended the site visit to learn about the bridge replacement project to date and the design and construction challenges of this complex project. Particular geotechnical design challenges include: liquefiable sands at both abutments leading to large lateral spreading loads, highly variable volcanic rock, and a steeply dipping soil/rock interface. Attendees walked over the staging to view the construction progress and equipment used to date. A couple of photos from the site visit, courtesy of David Rowland, are shown below.

The site visit received great feedback and we would like to arrange other site visits to interesting sites in various New Zealand locations in the future. The YGP group would like to thank Engenerate Canterbury for their collaboration for this event.

Upcoming Activities and Ideas:

Watch this space for new ideas and initiatives from your new YGP Representative – Frances Neeson;

- Expansion of the YGP liaison group of interested young professionals throughout the country;
- Further liaison and combined events with other young professional groups such as Engenerate;
- Promotion of the NZGS at Universities;
- Part time work opportunities for students on the NZGS website;
- A YGP forum on the NZGS website with involvement from senior members;
- Social media groups;
- Social events - quiz night, rock climbing.

Reported by: Luke Storie
YGP Representative
Email: luke.storie@gmail.com

The Canterbury Earthquake Royal Commission Report – Some Aspects for Geotechnical Engineers to Consider –

Bishal Subedi, Senior Geotechnical Engineer, Aurecon NZ Ltd (Wellington), Alan Wightman, Associate Geotechnical Engineer, Geoscience Consulting (NZ) Ltd (Wellington)

Introduction

In response to the September 2010 Canterbury earthquake and numerous aftershocks, the New Zealand government formed a Royal Commission to inquire into these earthquakes. The Royal Commission's report, published in 2012, includes recommendations on the future design and construction practice of earthquake resistant buildings. The full report is presented in three parts and seven volumes and is available on the Commission's website http://canterbury.royalcommission.govt.nz/. Volume I (Part One) contains in-depth discussions and recommendations on geotechnical earthquake engineering aspects of building design and construction. The authors expect that these recommendations will shape the future research and practice of geotechnical engineering in New Zealand.

We recommend that at least Volume 1 should be read by all practicing geotechnical engineers and engineering geologists in New Zealand. The two authors both work as consulting engineers and read Volume 1 shortly after it was released. With a year now having passed, we have distilled the Royal Commission's many recommendations down to the parts we find most interesting, most contentious and most relevant to our own practices. Our discussions are related to:

- Extent of geotechnical investigation
- Revision of capacity reduction factors
- Consideration of the effects of excess pore pressure
- Effect of vertical acceleration
- Foundation settlements for ULS and earthquake over-strength cases
- Use of driven piles to resist seismic loads
- Use of belled piles and uplift capacity of belled and screw piles
- Thrust on piles caused by lateral movement of surface crust

Quotes from the Royal Commission report are in italics; our comments are in normal font.

Extent of geotechnical investigation

While it is difficult and dangerous to set exact norms, a geotechnical investigation (which can be up to two per cent of the whole construction cost) would seem a modest investment compared to the risk of unsatisfactory performance. Given the extent of unsatisfactory seismic performance of foundations in Christchurch, a greater expenditure on geotechnical site investigations in future is warranted.

The necessary depth of … exploration … requires careful judgement … (and) should extend through all soil strata considered able to affect the behaviour of the site and the building foundations, and then continue to a sufficient additional depth to ensure all potential problem soils have been identified. Where deep pile foundations are being considered, the exploration should continue well into the bearing stratum and at least 10 diameters below the intended found depth (Section 4.10.1).

We agree that careful judgement is required, but question the need to extend 10 pile diameters (B) below the bearing surface. For instance, Meyerhof (1976) considers a depth of influence of up to 3B below the bearing surface, the authors also recall 5B being used in some methods. The engineer will have to consider for each site if soft or liquefiable soils are likely to be present at greater than 3B-5B, whether liquefaction is actually possible at such a great depth (and hence confining stress), and what effect these soils would actually have on the pile.

Based on our experience, clients or project managers frequently comment on the required geotechnical investigations for their projects as being excessive. While depth, frequency and type of investigations are site-specific and needs assessment on a project-by-project basis, the above recommendations from the Royal Commission may be used as a basis to discuss and convince clients of the need for adequate geotechnical investigations.

Strength Reduction Factors

The concessional strength-reduction factors in B1/VM4 for load cases involving earthquake load combinations and overstrength actions ($\phi g = 0.8 - 0.9$) should be re-assessed (Recommendation 14).

The strength reduction factors in B1/VM4 should be revised to reflect international best practice including considerations of risk and reliability (Recommendation 15).

The Report recommends that a risk based approach, such as taken in the Australian Piling Code AS2159, should be put into place. We approve of this philosophy – factors such as the certainty of the soil parameters used, the extensiveness of the ground investigation, and the consequences of failure, should be explicitly or implicitly considered in every foundation design. This is fairly well set out in Table 4 of B1/VM4 for non-overstrength loading – we suggest that designers take a similar approach for overstrength loading also.

Pore Pressure Effects

Liquefied soils' impacts on foundations are well recognised. During earthquakes, founding soil layers may not have reached the state of liquefaction. The foundation however could still have been impacted due to the soils' reduced strength and stiffness from excess pore pressure development. The Commission's comments on this issue are as follows:
Section 4.10.6, e of the report states:
Where the bearing stratum overlies liquefiable soils the ground floor slab (or basement slab or raft) should be able to resist the very high pore-water pressures resulting from soil liquefaction at depth. Such high pressures have been found in Christchurch to penetrate even dense overlying gravels and cause heaving failure of floor slabs in contact with the ground surface.

Later in the same Section, the report continues:
…Allowance should be made for loss of soil strength during earthquake shaking (from increased pore-water pressure and other forms of cyclic softening), for inertial effects (so-called seismic bearing factors – e.g., Ghahramani and Berrill), and friction acting along the base of the footing from lateral loading (inclined loading).

The above analyses are important for assessing foundations' behaviour during earthquake events. In practice, such analyses are not done routinely. There is no guideline that could be used for the assessment, and the use of sophisticated analyses might be required. As an example, it is complex to assess excess pore water pressure within an unliquefied layer underneath a slab due to a liquefied layer at depth. It is important that New Zealand engineering firms consider developing resources and capability for such analysis in routine projects.

Vertical Ground Motion

The implications of vertical ground motion for seismic design actions should be considered and locations identified where high vertical accelerations may be expected in earthquakes (Recommendation 34).

Although the above recommendation comes mostly from a structural perspective, it is also an important consideration in geotechnical practice. At present, it is not clear how vertical accelerations should be taken into account for geotechnical design.

For liquefaction, Method 1 of NZGS (2010) specifies how to calculate the horizontal ground acceleration, but not the vertical acceleration. The authors assume that vertical acceleration is just as likely to cause liquefaction as horizontal acceleration, so this gap in the method seems significant. In the Christchurch earthquakes vertical peak ground accelerations exceed horizontal peak ground accelerations in some instances.

For slope stability, Kramer (Section 10.6.1.1) points out that vertical acceleration has limited importance because it reduces (or increases) both the driving force and resisting force, and hence has an approximately neutral effect.

Foundation settlements

The report recommends estimating foundation settlements not only for Serviceability Limit State (SLS) but also for Ultimate Limit State (ULS) including earthquake over-strength. We view this to be a major shift in settlement assessments. It is a common practice among geotechnical engineers to estimate the foundation settlement only for the SLS case. One of the reasons for the Commission's recommendation is to limit excessive ductility demand on the structure from excessive foundation deformations. We consider that the estimation of ULS and overstrength settlements will be a challenge.

Driven Piles

The Commission recommends:
Driven piles have a significant advantage over other pile types for seismic design because the driving process pre-loads the base of the pile in the targeted bearing-stratum while simultaneously mobilising negative side-resistance along the shaft in the overlying soils. This effect may significantly reduce pile settlement if liquefaction or cyclic softening occurs in the overlying soils…….., building consent authorities should allow driven piles to be used in urban settings where practical. (Section 4.10.9)

Driven piles are generally popular for relatively low loads and in green field sites. However geotechnical engineers can play a role to increase their use by careful consideration and consultation with structural engineers and territorial authorities in early stages of the building development.

Belled Piles

In some locations (notably Wellington), belled piles have frequently been used to improve the uplift resistance of bored piles. The upper surface of the bell is considered to act as an upside down footing and treated as such for the calculation of capacity. However, the mobilisation of end-bearing in soil, upwards or downwards, may require significant movement of the piles (5-10% of diameter in each direction) and is likely to result in a very soft load-displacement response, especially if gapping develops. The resulting structural response may be more like foundation rocking, which can be quite different to that intended by the designer. Foundation movements, both upwards and downwards, are likely to govern design and need to be considered. In overseas practice, belled piles are used infrequently because with modern drilling equipment it is preferable to use deeper-drilled, larger-diameter piles because side-resistance increases rapidly with depth. This should also provide a stiffer response under seismic loading.
Belled piles should only be used in firm, cohesive soils or weak rock in dry-hole conditions where the bell can be excavated

without risk of collapse, and carefully cleaned out and confirmed before concreting. Drilling belled piles in granular soil under fluid should not be permitted where the integrity of the bell cannot be assured (Section 4.10.13.)

The authors, although they have designed belled piles below the water table in the past, agree with the above comments. It is more sensible to simply go deeper and resist any uplift loading in friction. In some cases this may also be the cheaper option, as constructing a bell in alluvium can be difficult and hence attract a high price from the piling contractor.

Going deeper also has a benefit for the compressive stiffness of the pile. For a shorter belled pile, static building loads may be resisted mainly in friction, even though the pile is designed to be (predominantly or entirely) end-bearing. If a strong earthquake occurs, the soils providing friction may liquefy or soften, causing the load to transfer to the pile end. End bearing capacity is usually fairly soft, and hence significant deformation may occur (albeit probably not enough to cause building collapse). This load transfer reportedly occurred in Christchurch in the recent earthquakes and may have the potential to occur in Wellington (and other places) in future.

Uplift Capacity

An upside-down punching shear failure is also possible where weak or liquefied soils overlies the founding stratum. Penetration of about five diameters into the founding stratum is necessary to develop maximum uplift capacity. The "punch through" failure mechanism should be considered for lesser embedment depths. (Section 4.10.13.)

We agree with this comment.

Thrust on piles caused by lateral movement of surface crust

Where there is a risk of significant liquefaction, deep piles should be designed to accommodate an appropriate level of lateral movement of the surface crust even when they are far from any watercourse (Recommendation 27.)

Even where soil liquefaction is not considered an issue, kinematic effects can still arise through deformations of other weak soils, especially adjacent to steep slopes such as waterfronts and bridge abutments (Section 4.10.14).

One approach would be to design the piles with enough flexural capacity that they can resist passive pressure over the depth of the non-liquefiable crust. This may be uneconomical, so a second approach is to ensure the piles can sustain whatever permanent ground movement the engineer thinks likely. If this too is impractical, then an assessment can be made as to what effect shear / flexural failure of the piles would have on the structure.

Methods for calculating permanent lateral displacements (e.g. Zhang et al (2004), Bartlett & Youd (1995)) are fairly well known. Methods for calculating cyclic (that is, temporary) lateral displacements are less well known. We are aware of a method by Tokimatsu & Asaoka (1998), although we have not put this method into practice.

CALL FOR ABSTRACTS & NOMINATIONS

The Australian Geomechanics Society and the New Zealand Geotechnical Society invite you to attend the 10th ANZ Young Geotechnical Professionals Conference (10YGPC).

The 10YGPC is for geotechnical professionals from Australia and New Zealand 35 years old and younger with a maximum of 10 years industry experience. It is designed for all attendees to present a technical paper on any topic of interest / experience relating to the field of geomechanics or geotechnical engineering.

Nominations

Please complete the nomination form attached to this call for abstracts. Nominations of delegates must also be supported by a senior mentor and include an abstract of 200 words on a topic that is related to geotechnical practice or research. Successful nominations will be selected based on the quality and relevance of the abstract.

Positions are limited to approximately 50 attendees and all successful nominations will be expected to present their technical paper at the conference.

Cost

The cost of this three day event is anticipated to be approximately AUD$1000 (incl. GST). The exact cost will be confirmed at the time of nomination acceptance.

This cost includes conference venue and dinner, accommodation for the duration of the conference, an arrival drinks reception and field trip to discover the regional geology of the Sunshine Coast region.

Important Dates

31 January 2014 - Nominations including abstracts due

28 February 2014 - Nominations accepted

30 April 2014 – Full Papers due

AUSTRALIAN GEOMECHANICS SOCIETY

NEW ZEALAND GEOTECHNICAL SOCIETY INC

ENGINEERS AUSTRALIA

Further Information

Further information will be available shortly on the Australian Geomechanics Society’s website (www.australiangeomechanics.org).

For any urgent queries or return of nomination forms / abstracts, please contact David Lacey (*Organising Committee Chair*) – dlacey@globalskm.com

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AWARDS

ISSMGE Awards Ceremony and Council Meeting September 2013

I WAS FORTUNATE in being able to represent the NZGS in Paris in September to collect our award for Outstanding Member Society and to attend the Council meeting and the Conference. Mick has shared his experiences at the conference elsewhere in this issue. It was a humbling experience to stand on a stage in front of around 1800 people and hear nice things being said about the Society I have been part of for over 20 years by a President who had visited the great majority of the 80 or so member societies in the course of his four year term. One observation he made that struck a chord with me related to our per capita membership. If the US had the same per capita membership of the ISSMGE as New Zealand, then it would have around 40,000 members. To put this in context, the ISSMGE only has around 20,000 members in total! We are clearly punching well above our weight.

The Conference was preceded by a full Council meeting, attended by representatives from nearly all the 80 member societies. The meeting was extremely well run, with sufficient levity from John-Louis and the Secretary General, Neil Taylor that the day passed relatively quickly.

Above: Gavin Alexander receiving our award on behalf of the NZGS


INTERNATIONAL SOCIETY FOR SOIL MECHANICS
AND GEOTECHNICAL ENGINEERING

ISSMGE SIMSG

SOCIÉTÉ INTERNATIONALE DE MÉCANIQUE DES SOLS
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ISSMGE OUTSTANDING
MEMBER SOCIETY AWARD

The New Zealand
Geotechnical Society

In recognition of the support, promotion and contribution to ISSMGE related activities over the period 2009-2013

Chair, Award Committee

President

2 Sept 2013
Date


All of the award winners together at the completion of the ceremony

John-Loius Briaud introducing the awards

It was disappointing to see Sydney come in second in the race to host the 2017 Conference, which was decided by the Council following three very polished presentations. I understand that the Australian Geomechanics Society is likely to pursue the 2021 Conference. It would be great to get this event down under. Roger Frank was elected as the new ISSMGE President, effective from the end of this year's conference, and it will be interesting to see how John-Louis' many initiatives evolve under Roger's leadership. There were two recent initiatives that were summarised at the council meeting and which I think are particularly worth a look. The first of these is a push to raise the profile of the online International Journal of Geoengineering Case Histories. This is potentially a valuable source of case history data and is well worth keeping an eye on. Secondly, a sub-committee has developed a short video explaining what geotechnical engineering is all about. This is aimed at the general public and potential engineering students, rather than at engineers and related professionals, and again is worth a look. It's available on Youtube, titled What is Geotechnical Engineering? Other useful initiatives that can be accessed through the ISSMGE website are recorded webinars and honour lectures, a multi-lingual lexicon of geotechnical terms, and a link to the GeoWorld professional network. President Briaud and his Executive and numerous committees have made huge strides in refreshing the ISSMGE and setting a great platform for future development.

Reported by: Gavin Alexander
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gavin.alexander@beca.com

19th Symposium

NEW ZEALAND GEOTECHNICAL SOCIETY INC
www.nzgs.org

HANGING BY A THREAD?

Lifelines, infrastructure and natural disasters


BRIAN PERRY CIVIL
For a

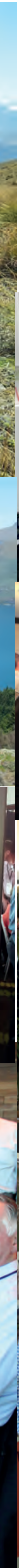








MILLENNIUM



19th NZGS Symposium, 20-23 November 2013, Hanging by a Thread – Lifelines, Infrastructure and Natural Disasters

THE ORGANISING COMMITTEE of the 19th NZGS Symposium got lucky. This was an event with the potential to go badly wrong. The problem was clear from the moment the delegates arrived – the setting. Who would sit in a darkened lecture theatre in Queenstown listening to discussion about soil mechanics? The Remarkables and Lake Wakatipu were beckoning the delegates to partake in the splendours of hill walking, mountain biking, kayaking and more. Even the recent 18th International Conference on Soil Mechanics and the 5th International Young Geotechnical Engineers' Conference, both in Paris, surely cannot have been so blessed.

So why were the organising committee lucky? Fortunately, the 100 presentations were of a most impressive standard, and kept the 300 delegates firmly glued to their seats. We were fortunate to have international experts including Dr Lelio Mejia and Prof Harry Poulos, as well as a very strong contingent of local talent from academia, government and business.

The highlights of the Symposium are hard to choose simply because there were so many. There is no doubt that the Gala Dinner, held at the top of the Queenstown Gondola, was a favourite for many. The food was superb, the wine delightful, and the view truly unbeatable. This must surely be the best venue for such an event in the world. The pre-symposium workshops were fascinating, and the keynote and invited speakers put on a fantastic performance. We learned a huge amount about the geological and engineering background to the earthquake events in Canterbury, the strengths and weaknesses of the engineering response, and of particular interest the political and social factors which underpin all our work.

For me, however, the highlight was more diverse. The individual presentations given by nearly a third of the delegates reminded me of the strength we have in our Society. Consultants, contractors, academics and government came together and shared their knowledge and experiences openly, freely and frankly. It is a sign of the health of our industry that these individuals put the development of the industry as a whole ahead of their personal needs, and for that they are all to be commended.

After days of full time concentration and networking, the post-symposium field trips were a perfect end to the event. Although some delegates were feeling slightly worse for wear after celebrating the final evening, the trips were so well organised and to such fascinating locations that everyone soon overcame their tiredness. I was fortunate to visit the Clyde Dam and the surrounding landslides. The tunnels draining the hills around the lake defied belief. The dam, and particularly the slip joint allowing fault movement through the dam, was astounding.

The stated aim of the Symposium was, "to reduce human suffering due to a future natural disaster". This bold goal may not have been achieved in the four day event alone, but the steps taken to spread best practice and openly learn from mistakes made have surely set us in the right direction.

Congratulations to the organising committee (Tony Fairclough, Kirsti Murahidy, Nick Harwood, Paul Salter, Aaron George and CY Chin) for pulling off the highlight of the year with panache, and to everyone who helped with the event and field trips. Thank you.

Prepared by: Ross Roberts
NZ Geomechanics News Co-Editor

Pioneering Innovative Ground Improvement Techniques

Modified DitchWitch Horizontal Drill fitted with customised mixing tools


Beams set out in a grid in various formations for testing


STA grout pump imported from Italy


The final result – beams exhumed after blasting trials

A trip to Italy by CLL managing director Terry Donnelly brought him in to contact with SIREG S.P.A. who have developed the Durvinal grouting system. This is a ground improvement technique that involves injecting high and low pressure grout through custom designed pipes to improve soil around the installed pipe (see www.sireg.it). The system is used extensively in Europe for underpinning historic buildings (including the Louvre). The advantages of the durvinal system are many ie: the grout ports can be reused with repeated low pressure injections, the grout permeates and strengthens the soils without creating heave. The durvinal pipes may also be used to inject low mobility, low or high strength, faster setting grout to densify and displace ground in targeted locations to deliberately lift structures as necessary. The pipes may be installed using directional drilling technology and also by standard drilling techniques.
It was apparent that this technology was potentially useful for remediating earthquake damaged land in Christchurch, particularly under existing houses. A trial was arranged with Tonkin and Taylor and EQC as part of a wider trial of potentially useful ground improvement systems in open ground. A consignment of Durvinal pipes was sent from Italy, along with a very experienced Italian geotechnical engineer, and trials commenced. The objective was to permeate the ground under existing houses with grout to strengthen the crust of ground above and at the water table level as the crust having been thinned during the earthquakes, needed re-strengthening. The very fine Christchurch sands proved too impermeable for the permeation technique to be economic. However as the CLL team became familiar, with the CHCH sands and through a process of inginuity and perserverance they came up with a system of constructing horizontal soil mixed beams that has proven to be efficiently and economical. The system, involves using a modified horizontal drill with a custom-designed mixing tool to mix a specially designed low viscosity, high mobility grout through the soil. These horizontal beams, can be installed under houses with no effect on the existing structure and services. The recent blasting trials carried out at Avonside dr in CHCH, which simulated earthquakes, were conducted to test the performance of the HSMB`s (Horizontal soil mix beams).

In the process of trialling the Durvinal system a good relationship has been established between SIREG of Milan and CLL, and we are now their New Zealand agents for this system. CLL have also taken up the distribution rights for STA grout pumps and mixing systems which have proven extremely effective and reliable during the trials. The knowledge and skills developed during this operation are already proving useful for numerous other ground remediation solutions in other parts of NZ. The Italians are looking to learn what we have developed and use it elsewhere in the world.

Sireg Durvinal Grouting Pipes

For more information visit www.sireg.it, or contact CLL who are the New Zealand agents for this system

Q&A with the Convenor of the 19th NZGS Symposium, 20-23 November 2013, Queenstown – Tony Fairclough

Q Have you organised a conference before and have you enjoyed the role of convenor?

This is the first conference I have been involved in organising. It was probably a brave move to be the convenor before being a helper but I have really enjoyed it. I treated it as I would any other project. I sat down and mapped out paths and key milestones, identified key tasks and people to complete them. I was really a project manager making sure tasks were complete within promised timeframes and looked for gaps which were not being addressed.

Q What were some of the key challenges in making this conference a success?

My biggest worry at the start was the financials as cash flow is a key issue with these sorts of events. You need to begin to pay for deposits for hotels and take out insurance at an early stage in the process. We engaged The Conference Company Ltd to help us with organisation which was a big help. We were very appreciative of early support from our sponsors, many of whom we found had been very loyal in supporting these types of NZGS events for a number of years. Engaging sponsors early gave us confidence that the symposium would be a success.

Getting a good team of people together to help with the organisation was also key to the success of the symposium. Kirsti, Nick, Paul, Aaron and CY Chin volunteered very quickly and I was lucky to have such a good team behind me.

Q Ok be honest, what little things have gone wrong?

There are always little things that need sorting out but the most frustrating aspect was late changes required to the proceedings. We were very lucky to have CY Chin editing our proceedings and he allowed space for all abstracts and papers. Unfortunately, a few people had to pull out shortly before the conference requiring time consuming reorganisation and renumbering of the proceedings. I am told that this is the norm for all conferences but prior to this I did not realise just how much disruption it causes. I will personally be more conscious of delivering material if I commit to presenting at future conferences.

Q Is there a key paper which has stood out to you?

I was particularly pleased with the presentations given by the keynote and invited speakers. Prof Harry Poulos and Dr Lelio Mejia both gave very interesting presentations which were concise, insightful and communicated a lot of practical information. This was obviously the intent when the keynote and invited speakers were identified at an early stage so it was nice to be proved correct! In saying that I think that the standard of all the papers delivered was very high.

Tony is a Christchurch based University of Auckland educated civil and geotechnical engineer with 20 years experience. During his career Tony has worked on numerous private development and public infrastructure projects throughout New Zealand and the Asia-Pacific region including Australia, Malaysia (4.5 years), Vietnam, Hong Kong (1.5 years), Fiji, The Solomon Islands, Vanuatu, the United Arab Emirates, and, the United States of America.

Tony is currently employed by Tonkin & Taylor Ltd, who he joined in October 2000, and previously worked for Worley Consultants Ltd (1986 - 1990), Soil and Rock Consultants Ltd (1991 - 1993), and, Woodward Clyde (NZ) Ltd / URS (NZ) Ltd (1993 - 2000).

Q What are the key themes or conversations that you have noticed emerge during the symposium?

There has of course been significant and constructive discussion around the symposium theme of natural disasters with a focus on Christchurch and the recovery.

I was interested in a comment from Prof Harry Poulos about the slenderness ratio for piles. I have always designed to a maximum slenderness ratio of 1:80 to 1:100. However, despite the numbers checking out ok, when you draw the piles up to scale they often look very slender. It was interesting to hear Prof Poulos recommended using 1:50 which helps justify detailing more robust piles.

There have been a couple of papers discussing emerging methods of ground improvement such as mixing with nano copper and microbial-induced calcite precipitation. I think as engineers we often have a tendency to jump to a negative conclusion about these new approaches. It is worth acknowledging that this is an area under development, keeping an open mind and encouraging those perusing something which has the potential to yield exciting results.

NZGS 19th Symposium – YGP Special Presentation Review

ONE OF THE highlights for Young Geotechnical Professionals at the NZGS conference was the YGP workshop where 40 YGP's took the opportunity to attend an informal and interactive session with Professor Harry Poulos and Don McFarlane.

Harry was one of the key note speakers at the conference and is recognised as an international authority in geotechnics. The presentation Harry gave to the YGP focused on the importance of developing sound geotechnical models and using simple analytical methods to predict the likely outcome. Harry advocated this 'back to basics' approach before embarking on complex modelling and therefore the designer will know if the complex modelling to ensure the design identifies the correct parameters and best solution before commencing modelling to obtain meaningful and realistic results. Harry impressed upon the audience that analytical and chart solutions still play a vital role in practical foundation design and the success of the design depends on the following:

- Understanding the mechanics of the problem;
- Intelligent simplification of the geotechnical conditions;
- Understanding the limitations and assumptions;
- Correct interpretation of the results;
- Appropriate quantification of the relevant parameters; and
- Presenting the results in a clear manner – preferably on one page…. All of these key points were also endorsed by Don!

Harry encouraged questions from YGP throughout his presentation, which made for a very interactive session in which he could pass on some of his wealth of knowledge gained working in a range of situations around the globe.

Following Harry's presentation, Don McFarlane, Senior Principal and Engineering Geologist at URS lead an informal discussion that was tailored to the young engineering geologists in the audience but relevant to all young practitioners in the geotechnical industry. Don recounted that the first lesson he learnt as a new graduate on the job was that "Greywacke hasn't read the text books!" Don also shared quotes from his time working on the Clyde Dam project "Geology is done through the boots" and "never saw a drain he didn't like (…unless it was blocked)". Don stressed that engineering geologists fill the gap between geologists and engineers and it is the role of the engineering geologist to translate the important information. Engineering geologists need to remember the bigger picture and that sometimes the gaps in the outcrops are just as important as mapping the outcrops. Don also took the opportunity talk about the new PEngGeol registration for engineering geologists. Don's words of wisdom, such as "know your driller" will no doubt resonate with the audience for a long time to come!

The relaxed nature of the workshop lead to an abundance of questions directed to both Harry and Don and resulted in an informative discussion of the topic's that matter to our YGP's. Our sincere thanks to Harry and Don for presenting at the workshop and providing words of wisdom that will help throughout our careers. The organisers were thrilled with the YGP turnout and feedback so far. And we look forward to building momentum for the YGP group following this conference.

I wish to thank the NZGS conference committee for their support and inclusion of the YGP at this event. Thanks also to fellow YGP's Luke Storie (YGP Rep) and Kelly Walker (Conference Committee YGP Liaison) for their assistance to make this workshop possible.

Frances Neeson
Conference Committee YGP Liaison.

Technical Article: Judged Best Young Geotechnical Professional Paper at the 19th NZGS Symposium

Characterisation of modern and paleo-liquefaction features in eastern Christchurch, NZ following the 2010-12 Canterbury earthquake sequence

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Keywords: Paleoliquefaction, liquefaction, seismic hazard, paleoseismology

ABSTRACT

Liquefaction during the 2010 Mw 7.1 Darfield earthquake and subsequent aftershocks (Canterbury earthquake sequence, CES) caused severe damage to land and infrastructure in Christchurch, New Zealand. As many as ten liquefaction episodes occurred in parts of eastern Christchurch, which manifested at the surface as sand blows, fissures and differential settlement. Trenching to water-table depths (~1-2 m below surface) across aligned sand blow vents and fissures revealed the subsurface geometry of the feeder dike system. In addition to the multiple CES dike generations, subsurface evidence for paleo liquefaction was found at two study sites in Avonside, eastern Christchurch. The tendency for reactivation of the liquefaction source conduit indicates that the locations of modern liquefaction are likely to contain geologic evidence for paleo liquefaction. We outline strategies that combine geomorphic, geological and geotechnical approaches for investigating paleoliquefaction elsewhere at sites that may or may not have historical evidence of liquefaction.

INTRODUCTION

Liquefaction occurs where earthquake-induced cyclic shearing of loose, saturated sediments results in the collapse of the soil skeleton and the commensurate transfer of the overburden stress to the pore fluid and transition to a liquefied state (Seed & Idriss, 1982; Idriss & Boulanger, 2008). Such soil deformation may cause severe land and infrastructure damage (e.g., Cubrinovski & Green, 2010). Understanding the liquefaction susceptibility of geologic deposits is therefore an important component of increasing resilience to earthquakes. The susceptibility of soils to liquefaction can be assessed by historical records, in situ geotechnical testing such as Cone penetrometer tests (CPT), Standard Penetration Tests (SPT), Swedish Weight Sounding (SWS), and Dynamic Cone Penetration Tests (DCPT), physical criteria such as grain-size distribution, particle shape, and plasticity characteristics, and the geologic characteristics such as depositional setting and groundwater depth (see Kramer, 1996 for review; Green et al., 2005; Green et al., 2011). Information on the location and magnitude of both historic and/or pre-historic earthquakes can be obtained by detailed studies of liquefaction-induced features such as sand blows and their 'feeder dikes' (Tuttle, 2001, Green et al., 2005).

The 2010-2012 Canterbury earthquake sequence (CES) caused at least ten distinct liquefaction episodes in parts of eastern Christchurch (Quigley et al., 2013), resulting in significant land and

infrastructure damage (Cubrinovski et al. 2012). The most severe liquefaction was associated with the September 2010 Mw7.1 Darfield event, and the February 2011 Mw 6.2, June 2011 Mw 6.0, and December Mw 5.9 events (Quigley et al., 2013). Surface manifestations of CES-induced liquefaction included sand blow (Figure 2) and blister formation, lateral spreading induced fissuring (Figure 1), and other surface deformations including differential settlement. The high liquefaction potential of the sediments underlying eastern Christchurch had long been recognized based on geotechnical, compositional, and geological characteristics (Elder, et al., 1991). There are no reports or evidence of historical or paleoliquefaction in eastern or central Christchurch prior to the CES, although liquefaction was reported in the township of Kaiapoi to the north of Christchurch and in the northern suburb of Belfast following the 1901 Cheviot earthquake (Berrill et.al., 1994; Downes & Yetton, 2012). Given that the sand blow vents were repeatedly reactivated during the CES, we sought to investigate these features in the subsurface to see whether any evidence for paleoliquefaction could be found. In this paper we document the subsurface expression of liquefaction caused by the CES at two study sites in eastern Christchurch. We provide evidence for paleoliquefaction at these sites and outline strategies for investigating liquefaction histories in areas that may or may not have evidence for modern liquefaction.

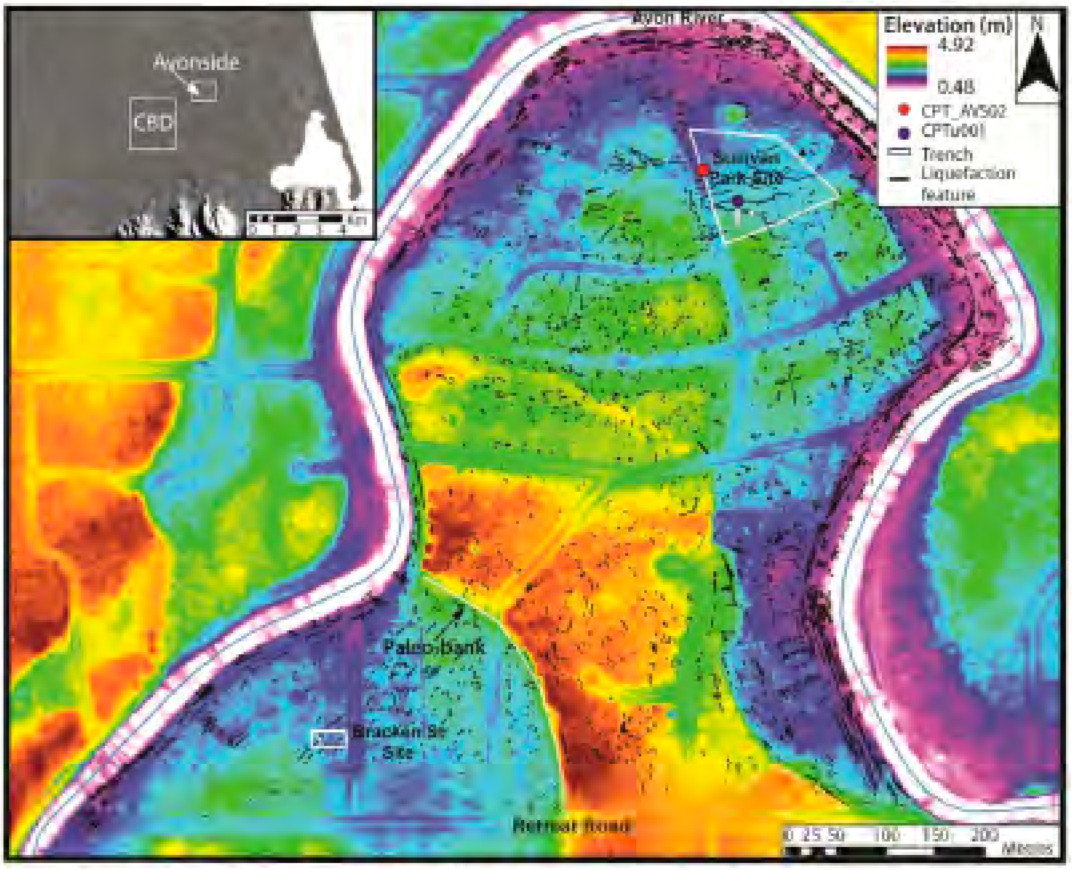

Figure 1: Study area showing distribution of surface liquefaction features and investigation sites. Liquefaction features aligned with the closest down-slope free face, being either the Avon River or the paleocut bank.

STUDY AREA

Figure 1 above shows the study area. Christchurch is primarily built on alluvial silt and sand deposits, drained peat swamps and estuaries, sand of fixed to semi-fixed dunes, and underlying marine sands (collectively referred to as the Christchurch Formation) that formed as sea-levels transgressed then regressed from a mid-Holocene highstand up to ~3km inland of the central city at 6.5 ka (Brown & Weeber, 1992). The combination of loose, fine-grained sands and silts, with high water tables (typically 1-2 m depth), and localized artesian water pressures that minimize soil cementation and 'ageing' effects, were known to pose a significant liquefaction hazard for much of Christchurch (Elder et al., 1991).

The study area of Avonside, eastern Christchurch is located within an inner meander bend of the Avon River (Figure 1), which undergoes tidal current reversals. Near-surface deposits beneath the top soil and anthropogenic material consist of alluvial silts and sands, with some gravel units in abandoned channels. Two study sites are investigated in this paper (Figure 1). The study site at 11 Bracken St experienced liquefaction and formation of sand blows (Figure 2) in at least ten CES earthquakes with local PGA ≥ 0.1g (Quigley et al., 2013). The Sullivan Park study site experienced significant liquefaction in the largest CES earthquakes and formed large lateral spreading cracks up to 0.5 m in width during the February 2011 Mw 6.2 event.

Figure 2: Photographs taken in the backyard of the Bracken St site following the September 2010 Mw7.1 event (a), and the February 2011 Mw 6.2 (b), June 2011 Mw 6.0 (c), and December Mw 5.9 (d) events, with the alignment and re-activation of the sand blow vents evident.

INVESTIGATION METHODS

Sand blows, vents and subsurface feeder dikes at 11 Bracken Street were mapped by Quigley et al. (2013). In this study we used Lidar and aerial photographs (data collected by NZ Aerial Mapping on 24 February 2011 [NZST] is available at http://koordinates.com/layer/3185-christchurch-post-earthquake-aerial-photos-24-feb-2011/) to map 4875 lateral spreading cracks (Figure 1) and other liquefaction features in the Avonside study area induced by the 22 February 2011 Mw 6.2 earthquake. Trenches were excavated perpendicular to the axis of the CES sand blow vents and lateral spreading cracks at both the Bracken St and Sullivan Park sites. A ~10 m long trench was excavated to a depth of 1.4 m at the Bracken St site, with the depth of the trench limited by the depth of the water table, which was at ~1.3-1.4 m during excavation. At the Sullivan Park site two trenches 6 and 18 m long were excavated perpendicular to two lateral spreading cracks to depths of ~1.5 m. The depth of the water table was at ~1.55 m during excavation. The trench walls were cleaned using hand scrapers, then logged and photographed to capture small scale changes in the geometry of the CES and paleoliquefaction features. At the Sullivan Park site, the trench floor was also cleaned and logged at several locations of interest. Small pits were dug vertically and laterally in areas with liquefaction dikes or other important features to investigate their three-dimensional geometries.

A truck mounted CPT system was used to perform a piezocone penetrometer test (CPTu) adjacent to the trench at the Sullivan Park site to a depth of 20 m. This enabled the depth of the liquefiable strata to be determined. The results of this sounding were correlated with a borehole augered within the trench, and the results of a CPT conducted post September 2010 at another location in the park that also exhibited surface liquefaction (Figure 1). The liquefaction potential was evaluated for each CPT site using the Idriss and Boulanger (2008) method.

RESULTS

The trench at the Bracken St site revealed alluvial sands and silts cross-cut by a sub-vertical planar dike consisting of grey, uniform, fine grained sand (Figure 3A). The dike was the source conduit for the surface sand blows that were successively deposited on top of each other, above the modern soil profile in liquefaction-inducing earthquakes. A lateral sill of uniform, grey, fine grained sand identical to the feeder dike material (Figure 3B) was cross-cut by the main feeder dike, as indicated by a dike-parallel silt lining at the boundary between these features. This feeder dike was the source conduit for at least 8 CES liquefaction events (Quigley et al., 2013), however only two clearly distinguishable dike generations were identified in subsurface trench mapping. The sill, dike, and sand blow material does not contain any of the 'mottling' that was observed in the sand and silt deposits beneath the soil profile (Figure 3A). This mottling develops over time as a consequence of fluctuating water tables and related oxidation. Both the CES feeder dike and sill cross-cut a bulbous-shaped lens of oxidised and mottled fine-grained sand, which had distinct upper and lower contacts. At the base of this deposit, a feeder dike analogous to the CES-induced dikes was identified by deeper excavation below the water table. Because this feature was cross-cut by the CES feeder dike system, contains significant mottling analogous to other alluvial material in the trench, and is sourced by a feeder dike, it was interpreted as a 'paleoliquefaction' feature. The irregular bulbous shape and lack of buried soil surface beneath the bottom contact of this unit suggests that it is a subsurface injection feature.

At the Sullivan Park site the trenches revealed sub-vertical planar feeder dikes of grey, uniform, fine to medium grained sand that aligned with the surface lateral spreading cracks and localized sand blows. The larger lateral spreading cracks contained clasts of the modern top soil which was most likely fragmented as the sediment was ejected through it and then settled through the sediment as the flow waned (Figure 3C). These dikes ranged in width from 5 cm to 50 cm on the trench floor, with the widest dikes corresponding to the lateral spreading cracks observed at the surface. These wider dikes on the trench floor were composed of medium grained sand with gravel, with the narrower dikes containing uniform, fine grained sand. On the floor of the trench, the CES feeder dikes were found to cross-cut a dike of oxidised, uniform, fine grained sand that had bioturbated contacts and well-developed oxidation 'mottling' (Figure 3D). This oxidation and bioturbation was more severe than that observed within the CES features, indicating that it formed prior to the CES (Figure 3D). Ongoing investigations at other sites within the wider Christchurch area have also uncovered evidence of 'paleoliquefaction'. For example, in Kaiapoi, highly-deformed and mottled sediment is overlain and truncated by undisturbed alluvial sediment (Figure 3E). Both the deformed unit and overlying deposit are cross-cut by CES feeder dikes, indicating that the deformed deposit is likely to have been deformed by pre-CES ground shaking.

The CPT sounding conducted at the Sullivan Park site reveals material at 1.6 – 2.15 m depth that is liquefiable under the PGA of the Mw 7.1 September 2010 and Mw 6.2 February 2011 earthquakes (Figure 4A). The minimum calculated PGAs required to liquefy this unit during the September and February events were 0.15 g and 0.19 g respectively, additionally the minimum PGAs for liquefaction in the June and December 2011 aftershocks was ~0.2 g. The results from the borehole augered within the trench at the Sullivan Park site correlate this liquefiable unit to a blue-grey fine to medium grained sand. This is underlain by dense, medium grained sand with granules to cobbles, which is interpreted as a -flood deposit and correlates to the dense unit that was not liquefiable under the PGAs generated by the CES (Figure 4C). The results from the other CPT sounding conducted within the park indicate that the thickness of the liquefiable strata varies across the park, with it being relatively thin at the trench site (Figure 4B).

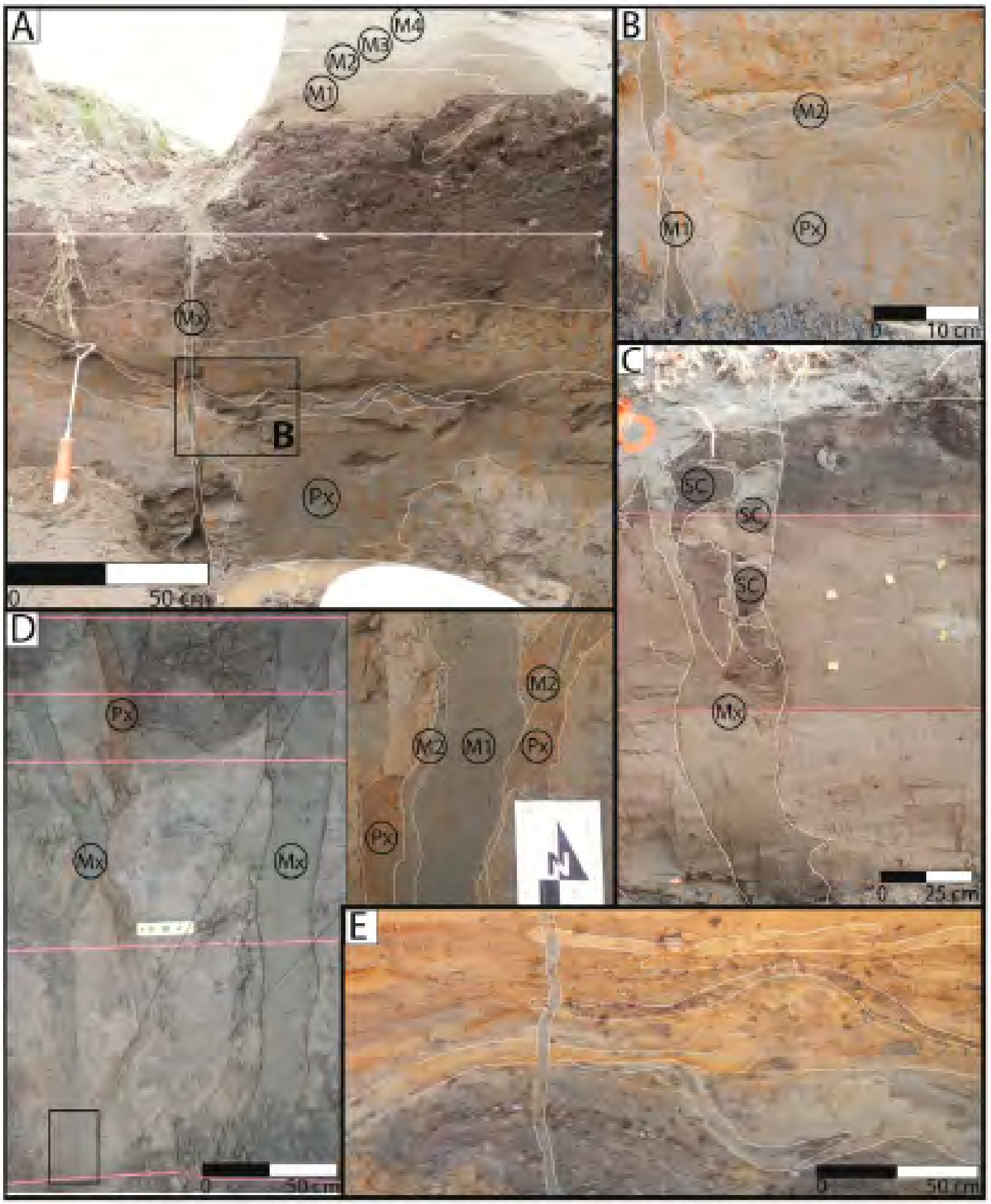

Figure 3: A) Interpreted photograph of the Bracken St trench, the modern feeder dike (Mx) aligns with and cross-cuts an oxidised, bulbous paleo-liquefaction injection feature (Px). The modern dike fed the surface sand blow with 4 events identified (M1-4). B) Close up of the lateral sill (M2) coming off the main dike (M1) with the silt layer separating the feature from the main dike indicated. This cross cuts the paleo-feature (Px). C) Photograph of a lateral spreading crack (Mx) on the Sullivan Park trench wall with down-dropped clasts of modern top soil (SC). D) Photograph of the Sullivan Park trench floor indicating the varied widths of the CES dikes (Mx), and their alignment with the oxidised paleo-liquefaction dike (Px), two modern events (M1 and M2) were identified within the dike, separated by a silt drape. E) Soft sediment deformation identified in Kaiapoi resulting in deformed stratigraphy. This sediment is also cross-cut by a modern CES dike.

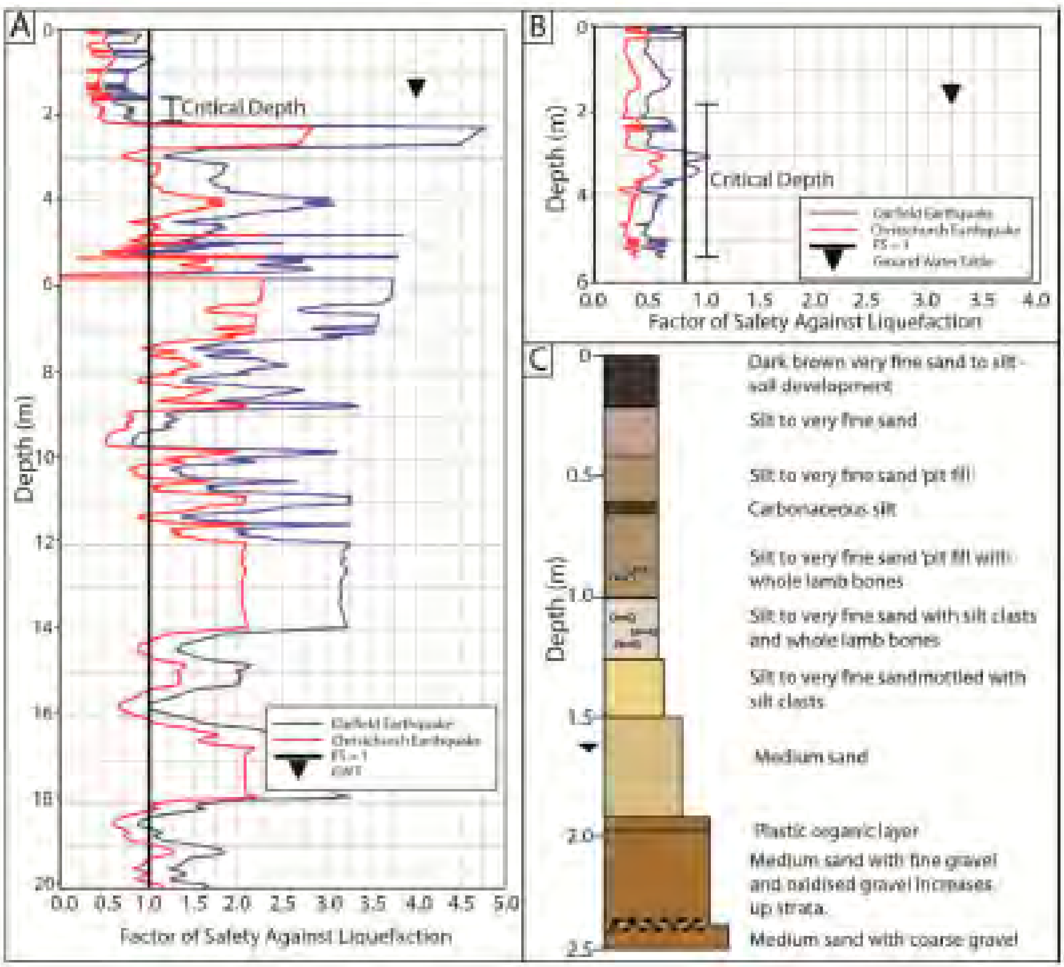

Figure 4: A) CPT log taken adjacent to the Sullivan Park trench (CPTu001), a potentially liquefiable unit is identified at 1.6-2 m depth underlain by a unit not liquefiable under the PGA's of the CES. B) CPT log (CPTAVS-02) conducted post September identifying a thicker unit of potentially liquefiable sediments that are also underlain by a dense unit that was not liquefiable. C) Borehole log from within the Sullivan Park trench. The saturated blue-grey sand at 1.6 m depth aligns with the liquefiable sediment identified within the CPT, while the un-liquefiable unit correlates with a gravel unit.

DISCUSSION

Subsurface investigations of liquefaction features induced by the CES revealed multiple generations of feeder dikes. Some dikes consist of similar, unoxidized grey sand and can be traced to surface sand blows, indicating that they developed in the CES. The observation of only two clearly distinguishable CES-induced feeder dike generations at the Bracken Street site despite 8-10 episodes of surface sand blow deposition indicates that feeder dike generations provide only a minimum estimate of the number of liquefaction-inducing earthquakes, and may significantly under represent the number of events (Quigley et al., 2013). Other feeder dikes and intrusions were cross-cut by, and have a much different appearance to, the CES features; these were composed of oxidised, mottled sand. We interpret these as paleoliquefaction features. Further investigations of these features and dating of their associated strata will provide better constraints on the timing and magnitudes of the causative earthquakes. CPT data confirm that the shallow source for both the CES and paleoliquefaction is at depths of 1.6 – 2.15 m at the Sullivan's Park site. The identification of paleoliquefaction within Avonside and in Kaiapoi (Figure 3E) confirms that most of the areas that experienced severe liquefaction during the CES were developed where subsurface investigations would have provided geologic evidence of paleo liquefaction.

CONCLUSIONS AND RECOMMENDATIONS

Many areas within New Zealand have a high liquefaction potential as indicated from geotechnical data. However the frequency and severity of paleoliquefaction in most of these areas is undocumented. Based on our findings in Christchurch, we recommend steps for conducting paleoliquefaction investigations as follows:

1) Regional geomorphic mapping should be conducted to identify settings where liquefaction susceptible sediments are likely to be present. Fluvial and estuarine settings are typically the most susceptible, although a variety of other deposits may also be under certain conditions. Smaller scale geomorphic mapping may be used to delineate free-faces likely to enable lateral spreading, and any other evidence for subtle landscape anomalies that may reflect liquefaction-induced surface deformation. Inner meanders of streams, due to the lower surface elevations, rapid sedimentation leading to loosely compacted material, shallow water tables, abundance of proximal 'free-faces', and relative sediment 'youth' with respect to the deposits formed on the outer meander bends, make these sites most likely targets for paleoliquefaction studies.
2) Areas with shallow water tables that contain fine-grained sands and silts should be investigated with geotechnical methods including CPTs, SWS, and grain-size analysis. Areas identified as highly susceptible should be targeted for trenching investigations.
3) Trenches should be oriented perpendicular to the closest down-slope free face. This will often be perpendicular to the orientation of the nearest section of river or internal paleo-banks. A trench length of at least 20 m and preferably upwards of 50 m is recommended, based on the density of mapped lateral spreading cracks in the Christchurch case. Trenches should be excavated to approximately water table depth, as trench collapse is likely if the trench is deeper. To identify liquefaction features, careful cleaning of the trench walls and observations on grain size changes is required. This may require digging deeper or back in areas of interest.
4) Mapping of trench walls and floors at cm-scale using detailed logging and photo-logging techniques is essential. Information about sediment grain-size and grain-size variability, presence or absence of intervening silt layers, three-dimensional geometries, stratigraphic relationships, and degree of weathering or 'mottling' of sedimentary deposits is essential.
5) Dating of the liquefaction features may be possible if they involve surface deposits (sand blows) that are likely to have been completely 'bleached', so that optically stimulated luminescence may be applied, or if they incorporate organic surface materials during formation which can be dated using radiocarbon dating. Alternatively, and more practically, sediments that are cross-cut by or post-date liquefaction deposits may be dated in order to 'bracket' the timing of paleoliquefaction. It is essential to understand the spatial and relative temporal relationship between the liquefiable and non-liquefiable deposits prior to undertaking dating if information about paleo-liquefaction is to be obtained. The number of feeder dike and/or sand blow generations, as indicated by cross-cutting and/or other stratigraphic relationships, differences in weathering state or grain size, and/or the presence of intervening silt layers can provide important information relevant to the relative timing of liquefaction-inducing earthquakes.
6) Information about the location (Obermeier et al., 2005), magnitude (Green et al., 2005 and Tuttle, 2001), and paleo-PGAs (e.g., Quigley et al., 2013) associated with liquefaction-inducing paleo-earthquakes may be deduced from combining geologic data with empirical relationships. Information on feeder dike widths, depth of liquefiable material, and sand blow thickness and extent are important to document. Seismologic information such as attenuation relationships, site amplification, shaking duration scenarios will be important for deriving information about the causative earthquake from the geologic record of liquefaction. The presence of paleoliquefaction features within eastern Christchurch and Kaiapoi confirms that earthquakes having sufficient shaking intensity to induce liquefaction have occurred prior to the 2010 Darfield earthquake, and that evidence of these features was obtainable from the geologic record prior to residential development at some sites.
7) The identification and interpretation of paleoliquefaction features in the geologic record provides information on the timing and shaking intensity associated with past earthquakes. This can be used to further inform probabilistic seismic hazard models,

improve risk modelling and land-use planning in urban environments, predict the impacts of future liquefaction events, and proactively remediate areas of high liquefaction-susceptibility.

ACKNOWLEDGEMENTS
We thank Martitia Tuttle, Pilar Villamor, and Peter Almond for their discussions and assistance that aided the interpretation of these paleoliquefaction features, and Matthew Hughes for assistance with ArcGIS. This work was funded by an EQC Capability Fund and the University of Canterbury Mason Trust Fund.

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CONFERENCE REPORTS

5th International Young Geotechnical Engineers' Conference, Paris, France – August 2013

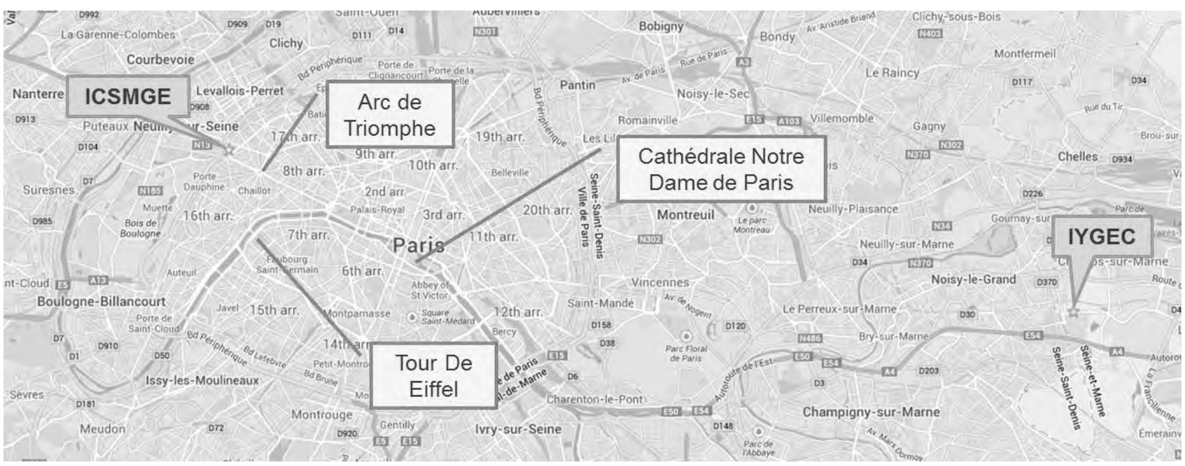
THANKS TO A scholarship from NZGS and support from our respective companies, Andrew Holland and I were able to attend the 5th International Young Geotechnical Engineer' Conference (IYGEC) held at Ecole des Ponts Paristech in Paris, France. The conference was organised by the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) and was held the weekend before the main quadrennial conference (International Conference on Soil Mechanics and Geotechnical Engineering – ICSMGE) which was also in Paris.

The aims of the conference were to provide 'an exchange of ideas and networking for young geotechnical engineers' and to allow for 'engagement of all attendees – both regional nominees and non-nominees'. These goals were certainly achieved with 160 attendees – 100 of which were nominees from their country's geotechnical engineering society – from over 60 different countries.

Every attendee submitted a paper and gave a brief presentation. A wide range of topics in geotechnical engineering were covered from investigations and ground modelling, to design and construction, as well as maintenance and monitoring. The main theme that came out of the conference was about geotechnics for the new generation, and examining what the challenges the future will bring. In particular, challenges regarding energy, innovation, difficult environments, preservation of existing structures, and design optimisation and modelling tools.

The most enjoyable part of the conference for me was gaining a much deeper appreciation of the similarities and differences between geotechnical engineering around the world. We all work within different parameters based on the resources available. But soil is the same throughout the world and there is so much knowledge and skills around the world that we can all learn from. Our cultures are different but the geotechnical challenges are the same.

Prepared by: Richard Heritage
Aurecon



18th ICSMGE Paris 2013 – An Appreciation

Paris en Musique

The advertising video for the conference had a sequence of wonderful shots of the city backed with a recording of Edith Piaf singing *La Vie en rose* – very enticing!

Part way through the opening session there was a musical interlude with a young woman violinist and another with a harp.

At the end the first day we were treated to a musical entertainment – a mini-opera entitled *Paris en Musique* - which tells the story of a group of street musicians. At the start we hear a subway singer, with a harsh voice, trying to scrape together a living but all the while dreaming of a much grander life. She is joined by other street musicians – a chap with an accordion, another with a double bass, and a violinist and harpist (the same two who featured in the conference opening session).The group eventually attracts the attention of an orchestral conductor who takes them in hand and develops a very impressive ensemble. The street singer emerges as an accomplished soprano and sings a Handel aria and another from Bizet. She finishes with an encore – you guess.

All present agreed that this hour-long presentation was quite charming and moved off to the welcome reception in high spirits. I resolved never again to walk past subway musicians without contributing!

Paris the city

According to Gershwin Paris is beautiful any time of the year (although he did have a La Vie en rose idea in mind). It was certainly beautiful under the late summer weather during the week of the conference; to the extent that some discipline was required to attend the conference sessions rather than enjoying strolling along the wide tree-lined boulevards or visiting some of the marvellous museums and art galleries the city has to offer.The conference registration pack included a free metro pass – lending additional weight to the temptation!

The fifteen minute walk from the hotel to the conference venue each morning was a true delight.

The only downside was that Paris is infested with some remarkably skilled pick-pockets.

The conference venue

The Palais des Congres de Paris is situated about half way between the Arc de Triomphe and the Grande Arche at La Defense (an ultra-modern commercial centre). One can stand at La Defense and look directly down the avenues and see the Arc de Triomphe in the distance. The conference venue is huge, only half the main auditorium was needed for our conference of about 2000 attendees. The third and fourth days of the conference were in parallel sessions so other rooms in the building were used.

The ISSMGE family

The immediate-past president, Jean-Louis Briaud, often speaks about the ISSMGE family. The getting together of ISSMGE members from around the world and the pleasure in renewing old contacts certainly has a family feeling about it.

I found particular delight in meeting up with two people I had not seen for 40 years, and yet the mutual recognition was immediate!

Then, of course, there are the new contacts made, and agreements to exchange information and engage in collaborative projects.

The only problem is that with so many delegates the meeting process is somewhat random. The first half of a conversation takes place, and then an interruption, so the remainder of the exchange will have to be via email.

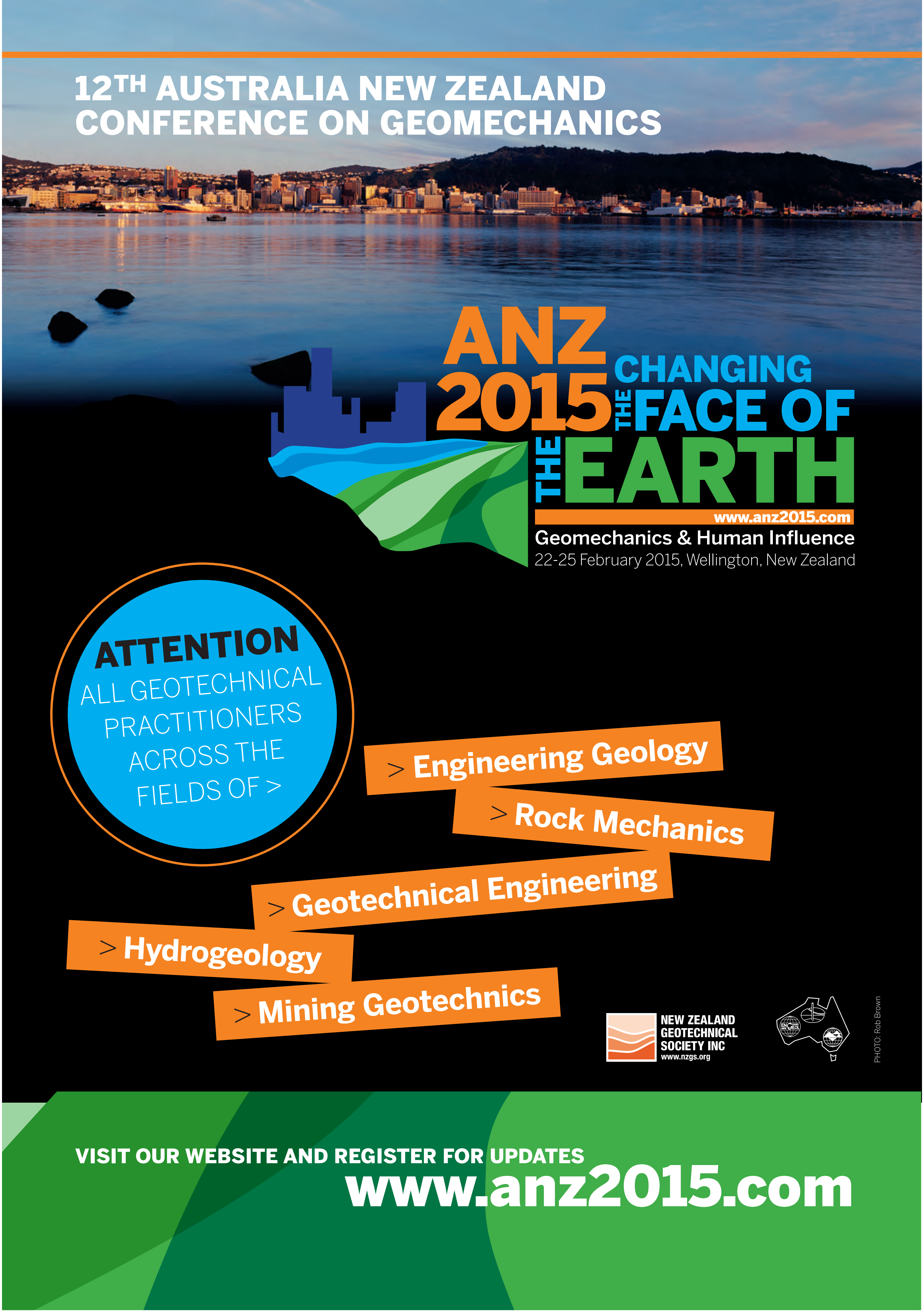
I talked with someone who had passed through Auckland a couple of months ago, whom I had not been able to meet because of other commitments, but there we were in Paris and happily able to talk. We reflected on the marvellous world we inhabit where it is possible to travel half-way around the globe in a day or so with little in the way of the jet-lag that used to plague air travellers. In comparison with this I often think of how tedious it was for my parent's generation to travel between NZ and Europe, not to mention, of course, the settlers who came this way in the nineteenth century.

Technical highlights of the conference

There was plenty of fascinating technical information to be picked at the conference sessions; the assessment of highlights will vary from conference participant to participant.

The first two days had a series of named lectures: Terzaghi, Ishihara, Menard, Bishop, Kerisel, McClelland, Kerry Rowe, and Schofield. Each of these gives a fascinating overview of a subject area backed up by an extended paper in the conference proceedings.

Days 3 and 4 of the programme were surrendered to the technical groups – a new initiative of the ISSMGE Board. These were in parallel sessions, so one had to make choices with some flitting between various rooms. Not all this material was published in the conference proceedings, one might need to search the Technical group web sites for further information. Two notable sessions I attended covered limit state design of foundations and how, in the European context, variability of soil properties is dealt with. Another on the cyclic loading of pile foundations presented the results of some excellent large scale cyclic load testing of piles. One of the drivers for this work is the ambitious European plan to develop extensive wind farms



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in the North Sea. This work illustrates just how effective large scale central funding of geotechnical research can be.

The French centralised organisational approach to getting big geotechnical research done was explained in an interesting lecture given by Francois Schlosser on *French Innovations in Geotechnics,* many of which have been verified by large scale monitoring exercises.

One of the paper presentations I attended, by flitting to the room next door, was by John Hughes showing how, even with indifferent drilling techniques, it is possible to estimate the in-situ lateral stress in the ground with a pressuremeter.

Trade display

There was a well-populated trade display. Items covered geotechnical software, geotechnical construction processes, field test equipment, laboratory equipment, displays from book publishers, and glossy presentations from construction and consulting organisations.

A device that took my eye could be described as a high-tech Scala penetrometer. This employs the wave equation PDA process that is used for assessing pile capacity. The penetrometer assembly is struck with hammer blows and the wave passage down and up the rod measured and from there calculations, via software, about the soil properties are made; additionally, the settlement per blow is obtained. It appeared to be a good deal more informative than the Scala test. There are a range of devices with various capacities, the "Scala" version was said to be capable of penetrating up to about 5 m in sand. The suppliers were promoting it as a site investigation and compaction control tool.

Conclusion

All-in-all a most successful conference from the cultural, social, and technical points of view; a memorable "family" event for the ISSMGE! Congratulations to the French Society for Soil Mechanics and Geotechnical Engineering.

Prepared by: Michael Pender
University of Auckland

6ICEGE, Christchurch, 7-11 November 2015

WE ARE DELIGHTED to announce that we will host the 6th International Conference on Earthquake Geotechnical Engineering (6ICEGE) in Christchurch, 7-11 November 2015.

ICEGE is the premier international conference on earthquake geotechnical engineering organized under the auspices of the Technical Committee TC203 (Technical Committee on Earthquake Geotechnical Engineering) of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The work of this technical committee (formerly known as the TC4) was initiated in the late 1980's and resulted in several well-known international guidelines on geotechnical microzonation and liquefaction remediation under the leadership of Profs. Kenji Ishihara and Liam Finn, in the early 1990's. To further stimulate the activities and involve worldwide participation in the discipline, TC4 initiated an international conference focussing on earthquake geotechnical engineering (the ICEGE series). The first conference was held in Tokyo 1995, and was followed by the 1999 Lisbon (Portugal), 2004 Berkeley, California (USA), 2007 Thessaloniki (Greece) and 2011 Santiago (Chile) conferences. In the line of these cities all residing in regions of very high seismicity, we will host the sixth edition of the quadrennial international conference in Christchurch, arguably the most appropriate venue for a conference on earthquake geotechnical engineering at present.

Christchurch undoubtedly offers an outstanding combination of venue and context for the 6ICEGE. The unprecedented series of strong earthquakes that repeatedly hammered the city in 2010-2011 inflicted extensive damage, 185 fatalities and a total economic loss on the order of $40 to $50 billion New Zealand dollars. While undoubtedly the 2010-2011 Canterbury earthquakes had many engineering, scientific and social facets, from an engineering viewpoint these earthquakes were largely 'geotechnical' in nature, and were dominated by unprecedented impacts in extent and severity from soil liquefaction and rock-falls that directly affected nearly half of the city area. Following these impacts to its native and built environment, Christchurch is now embarking on a large reconstruction project in which ground remediation, foundation engineering and restoration of lifelines will be the principal activities in rebuilding the city. We believe that the 6ICEGE will provide an excellent opportunity to showcase the state of geotechnical engineering of New Zealand, and share with the international community the lessons we have learned from the earthquakes and the reconstruction of Christchurch.

The conference will be diverse, including case histories and practice-oriented papers, recent research findings, novel technologies, and the emerging arts across various disciplines. Over twenty keynote and theme lectures will be presented including the 5th Ishihara Lecture. Ishihara, Finn, Idriss, Boulanger, Bray, Kramer, Stokoe, Kokusho, Tokimatsu, Towhata, Yasuda, Ansal, Gazetas, and many other internationally renowned leaders in the discipline will make key contributions to the 6ICEGE. Coming out from the very successful NZGS conference last week in Queenstown, we are particularly keen to have a very strong and active participation of New Zealand engineers and researchers, and will put particular emphasis to bring on board the young professionals and engineers whose excellent work and refreshing ideas were so eloquently presented in Queenstown.

This conference will be made possible by the hard work of an enthusiastic organizing committee and the great support of the New Zealand societies for geotechnical and earthquake engineering (NZGS and NZSEE), and the Earthquake Commission (EQC), New Zealand. Stay tuned for more information and continuous updates though the NZGS information system and the upcoming 6ICEGE Conference website.

On behalf of the Organizing Committee I look forward to welcoming you to the 6ICEGE, Christchurch 2015, and to providing an exciting and an above all successful conference.

Misko Cubrinovski
Chair, 6ICEGE

Programme overview:

Conference: 9-11 November 2015 (including over 20 keynote/theme lectures and 5th Ishihara Lecture)
Pre-conference workshop: 7-8 November 2015 (co-founded by NZ and NSF, National Science Foundation, USA)

PROJECT NEWS

Christchurch Gondola Rockfall Protection

THE CHRISTCHURCH GONDOLA in the Port Hills area is an iconic tourist attraction in the region. The gondola ride from the base to the top of Mt Cavendish Summit is a 1 km ride that goes through the crater rim of the Port Hills. The ride offers stunning views for the passengers overseeing Lyttelton Harbour.

After the 22nd February 2011, this favorite attraction was closed due to the rockfall concern and the risk posed to the tourists should another earthquake happen. Since the event the client had been working with the consulting engineer Golder Associates to investigate the rockfall risk and design protection measures to ensure the risk was controlled to a minimum and in full compliance with the council's requirement.

In addition to trajectory analysis using computer software to determine the kinetic energy and bounce height of the falling rock the engineer also carried out scaling of rocks above Summit Road, monitoring their size and trajectory. These field test results provided invaluable data to assist the engineer in the simulation work and planning the necessary protection measures.

A number of different design approaches have been taken by the engineer in this project. Firstly, the larger rocks were scaled off from the slope surface so that the immediate rock source threat was taken away. The residual risk of potential rock fall was then further controlled by installing an attenuator fence up slope from Tower 6.

The attenuator fence features include its long drape, giving it the ability to attenuate or dissipate the energy of the falling rocks bringing them down at rest or allowing the rock to pass through the tail with much reduced energy. This particular attenuator fence was 40 m long with 4.0 m high supports at the uphill entrance. The drape comprised Ring Nets up to 14 m long. Based on the experience Maccaferri has acquired in this specialist field internationally, this fence is designed to be capable of handling a maximum impact kinetic energy of up to 2,000 KJ. Another important feature of the attenuator

Figure 1: Helicopter positioning the drape

fence is its ability to resist multiple rock impacts under the most severe events of boulder flux. This is an important consideration when future maintenance maybe an issue. The fence is supplied as a 'kit set' including base plates and anchors; with a standard installation manual and drawing to assist an engineer in the preparation of their design.

The preparation, set out surveying, drilling of the anchors and casting of the concrete base for the fence posts took Solutions2Access Ltd, the specialist installer, approximately 14 days to complete. Installation of the fence posts and Ring Net drape panel took another 10 days for a four man operational team; all with the help of a helicopter.

A Green Terramesh® bund was also constructed at the toe of the hill near the car park. This protection bund serves as the last defence line should any rocks pass the fence to offer protection to the Base Station and car park areas where visitors congregate. The Christchurch Gondola was officially reopened to public by the New Zealand Prime Minister and Minister of Tourism Rt. Hon John Key on 19 April 2013.

Photograph 2: Completed attenuator fence

Client: Welcome Aboard
Main Contractor: Solutions2Access Ltd
Engineer: Golder Associates
Products Used: Maccaferri 2000 KJ Attenuator Fence and Green Terramesh®
Date of Construction: March 2013

Prepared by: Felicity Bartlett
Maccaferri NZ Ltd
Email: fbartlett@maccaferri.co.nz

Photographs courtesy of Solutions2Access Ltd

Auckland Rail Electrification Project – A Geotechnical Perspective

Figure 1: A typical view of overhead lines supported by twin and single track cantilever masts on the North Auckland Line.

MANY AUCKLANDERS, AND indeed many New Zealanders, wouldn't know it, but since 2010 one of New Zealand's largest current infrastructure projects has quietly taken shape across Auckland in the form of the Auckland rail electrification project. Earlier this year this $500M project reached a significant milestone when the last of some 3,350 foundations for electric traction masts was completed. The electrification project comprises design and construction of a 25 kV AC overhead electric traction system, a new track signal system, a new fleet of electric trains and various other civil and electrical projects to support the new electrification infrastructure. The total project spend on Auckland rail amounts to at least $1.6B since 2006. Design and construction of the traction system alone has presented an interesting range of ground engineering challenges.

In March 2008 KiwiRail appointed AECOM to design the electric traction system. With more than 3000 foundations planned for construction it was simply not possible to specifically design each one from the outset. The design challenge was to develop standard foundation designs that would be adequate in most ground conditions without unnecessary complexity, cost or customisation required. Work got underway to develop a range of foundations to support the ten standard mast types in the varying ground conditions encountered across the Auckland region. KiwiRail required the basic design to initially allocate foundations based on a desk study determination of ground conditions and across track services at each site, with confirmation following geotechnical investigations by the contractor during construction. Avoidance of conflicts with along-track services and other constraints such as retaining walls was deferred to the construction phase.

The principle design case for the mast/foundation system was essentially derived from wind loading on the contact and catenary wires. Tight deflection criteria for the mast and foundation (measured at contact wire height above ground) had to be maintained under the various wind loading conditions. The design team was able to draw on AECOM and Balfour Beatty experience with similar railway systems internationally to produce a series of piled foundations, shallow pad footings and rock anchors. Following ground investigations at each mast location, the foundation type was able to be allocated from design charts based on mast type and selection of one of a number of predetermined ground types. The range of foundation types included a selection of different diameter piles, and pilecap assemblies to offset masts from the pile, and allowed the project team to place foundations in, over and around the multiple buried services present in the rail corridor.

Construction of the traction system commenced mid-2010 following award of the contract to a joint venture between Hawkins Infrastructure and Laing-O'Rourke ("HILOR"). Major progress on construction of foundations has been made by utilising 'block-of-line' track closures, typically around public holiday periods. Across each of

Figure 2: This pile foundation for a twin-track cantilever mast has been constructed in the line of an existing retaining wall between buried services and private property.

the last three Christmas blocks-of line (about a two week closure period) the project has achieved construction of some 300 foundations. During these blocks-of-line AECOM maintained an on-call design team able to provide fast turn-around redesign of traction wiring, mast placement and foundation design. This has resulted in significant project cost savings from reducing the need for construction remobilisation to various widely distributed parts of the rail network.

Conflict with along-track services has proved to be the single most significant challenge for foundation construction. The location of signal cable, multiple fibre optic cables, and a suite of stormwater and sewer services have had to be accurately located on site at each mast location with pot-holing and an appropriate foundation selected. In many cases multiple clashes with services and other built infrastructure in the rail corridor has resulted in relocation of masts outside the lateral tolerance of the initial wiring design position, triggering a round of site-specific wiring and foundation redesign.

In parallel with construction, a quality assurance programme for foundations has also been underway. In addition to contractor HILOR's required project quality procedures, KiwiRail staff provided construction-phase monitoring and review, whilst AECOM has undertaken a comprehensive post-construction review of each foundation. Since most foundations were allocated from design charts, the post construction review has confirmed that performance of each as-built foundation/mast system conforms to the design serviceability requirements.

From start to finish, the foundation construction process has necessitated at least 10,600 individual site visits across 80 km of wired track from Papakura to Swanson! In reality this figure will be much higher and is a small way

Figure 3: Shallow pad foundations are installed in the railway embankment near Morningside during a block-of-line. Piling conditions at this location were difficult due to the presence of basalt boulders.

Figure 4: Foundation anchor bars are drilled into basalt rock close to the Onehunga Branch Line.

of demonstrating how large a logistical challenge this construction project has been. The Auckland Electrification Project is still underway, with a number of track segments in the process of being wired up. The first electric train services are planned to be operating in 2014.

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Re-construction of Infrastructure Following the Canterbury Earthquakes

THIS PAPER IS a brief overview illustrating earthquake damage and the points of interest in the repair and re-construction of two infrastructure assets damaged by lateral spreading in the Canterbury earthquakes:

- Fitzgerald Avenue Road and Retaining Wall
- The Heathcote-Opawa Bridge.

1. Introduction

Repairs to the Council owned earthquake damaged infrastructure are being carried out by the Stronger Christchurch Infrastructure Rebuild Team (SCIRT). This is an alliance involving the following major parties:

- Christchurch City Council
- New Zealand Government
- NZTA (New Zealand Transport Authority)
- Five Non Owner Participants (City Care, Downer, Fletcher, Fulton Hogan and McConnell Dowell).

An estimated $2bn of work is programmed to be done over a five-year period with completion anticipated to be in 2016.

2. Fitzgerald Avenue – Earthquake Damage

Fitzgerald Ave is one of four major arterial roads forming an orbital route around the city centre of Christchurch and carries 30,000 vehicles per day.

Fitzgerald Avenue suffered major damage due to lateral spreading of the River Avon's east bank in the February 2011 earthquake. The existing cast in-situ retaining wall experienced differential settlement along its length causing cracking and separation at close intervals. Damage to the adjoining section of carriageway featured 1.6m deep cracks up to 1.2m wide. Both lanes of the north bound carriageway were closed from February 2011 to September 2012.

The repairs carried out by SCIRT with design by Opus International Consultants and construction by Downer NZ Ltd and McMillan Drilling Limited as specialist sub-contractor.

3. Fitzgerald Avenue – Ground Remediation

The existing ground conditions under the North bound carriageway of Fitzgerald Avenue were poor with an average bearing strength of 3-4 MPa. In order to support the replacement retaining wall structure, traffic loading and to resist lateral spreading in future earthquakes a strength of 13 MPa was set as a guide pending more detailed assessment.

The ground stabilization of a 200m long stretch of road involved the installation of approximately 500 stone columns. The columns were 600mm diameter, 10m deep and set out in rows in a staggered pattern at a spacing of approximately 1.6m. The intended outcome of the stone columns was to densify the existing silt and sand ground and for the columns themselves to provide bearing capacity for the overlying structure.

The main considerations when selecting the method of stone column installation was:

- A method that would achieve densification of the existing ground between the columns through displacement and thereby reducing the volume of fill required and waste material.
- Due to the proximity of neighbouring domestic properties it was imperative to minimise the vibrations generated during construction.
- The site was located on the east bank of the River

Above and right: Major latteral spread damage to Fitzgerald Avenue

Above: Stone columns by McMillan Drilling Ltd

Above: The dry process resulted in a clean site and reduced environmental risk

Avon making it particularly environmentally sensitive. Alternative methods involving vibro-replacement or high pressure water jetting were not considered appropriate.

A process for installing the stone columns was developed by McMillan Drilling Limited specifically for these challenging conditions:

- A drill casing with a displacement auger was inserted 10m deep into the ground.
- 40-60 drainage metal was then fed through a reverse auger forcing the material outwards under high levels of torque as the outer casing was withdrawn. This method had the advantage of not inducing vibration as the new material was placed.
- This was a dry method greatly reducing the level of environmental risk on site.

As shown in adjacent photos the final diameter of the stone columns was greater than the 600mm diameter casing used to initially penetrate the ground. Columns measured up to 1.1m in diameter and often required over 2 times the theoretical volume to fill.

4. Fitzgerald Avenue – Retaining Wall

The retaining wall consisted of 127 pre-cast concrete panels, each 1.5m wide. The retaining wall was designed in individual segments to allow movement of the ground without stressing of the wall and would allow individual sections of the wall to be replaced if damaged in a future earthquake. Inter-panel joints were flexible and consisted of a groove cast into the vertical side of each panel which was filled with bentonite granules. Elastomeric sealant was applied to the joints front and back. The base of each panel was poured once the panel was in its final position and this allowed ease of casting and transport and also ensured that the base had full contact with the ground.

The new retaining wall was required to resist the significant lateral loads from not only the retained soil but also from the traffic guardrail mounted to the top of the wall. Beyond these structural requirements there was a focus to construct an aesthetically pleasing asset which not only provided a good view of the River Avon but harmoniously followed its alignment and the increasing height of the east bank. This was achieved by revising the original stepped design to a wall which continuously increased in height. Each of the 127 pre-cast panels had a sloping top surface and different heights on each side reflecting the change in level from one end of the wall to the other.

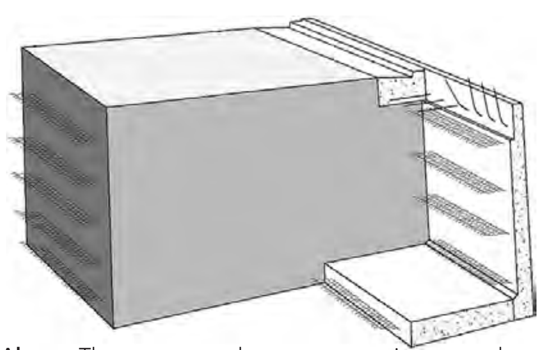
To construct the mechanically reinforced earth wall RE580 Tensar geogrid starters were cast in to each panel at 550mm lifts. Layers of Tensar were attached to the starters using a fibreglass bodkin joint, each layer of geogrid extended the full width of the North bound carriageway. CCC AP65 structural fill was then compacted in 200mm layers between the Tensar to 95% of MDD.

Above: Tensar geogrid tails cast into the units

Above: A low-height precast panel. All panels were cast face down

Left: Higher panles propped in preparation for in-situ bases. All bases had a separation joint to match the joint in the panels

Once the wall was in position and backfill completed a cast in-situ kerb and channel beam was poured along the back face of the wall. The kerb and channel beam had construction joints every six metres – again to allow for differential movement along the wall in the event of future earthquakes.



Above: The precast panles were set up in a smooth 3-dimensional flowing curve and the cast in-situ base ensured full contact with the ground

Left: View of the higher precast panles with cast in-situ bases complete and ready for AP65 structural fill

5. Fitzgerald Avenue – Road Re-construction and Wharf-Like Pedestrian Walkway

The 240m long elevated timber walkway was designed to provide a "wharf like" pedestrian/cycleway at the same level as the adjoining road. The structure, supported on the precast panels and on driven timber piles, extends over the Avon and provides an enhanced riverside experience for the pedestrian.

Key Statistics:

- 43 Timber piles driven 8m below ground level
- 8m long 250mm x 150mm Steel RHS beams each spanning two piles.
- 79 125mm PFC's mount to the retaining wall panel and cantilevering over the RHS beams.
- 546 timber stringers 3.6m long spanning between the PFC beams.
- 1560 timber decking planks nailed to the stringers with 32,600 80mm ring shanked stainless steel nails.
- Steel balustrades fixed to 79 timber posts to form the walkway handrail.

Above: The RHS stanchions were fitted with slotted timber posts to give a 'wharf like' appearance

Above: At the lowest point in the road, cement was used to stablize the basecourse

During the reconstruction of the North bound carriageway it was discovered that the ground water levels at the low point of the road were within 500mm of finished level. To ensure the road could withstand the significant traffic load the basecourse was stabilized with 1.5% Portland cement.

The cement was applied with a purpose made spreader mounted to the tailgate of a six wheeled truck which controlled the cement application. The precision of this process was tested by laying a 1m x 1m sheet on the ground and measuring the mass of cement deposited on to it. A mill with a wide tool spacing was then passed through the base course to mix the cement. The wide tool spacing helped to reduce particle breakdown which could have detrimentally altered the materials grading curve. The application of cement reduced the moisture sensitivity of the base course, added strength and maintained the flexible properties of the pavement.

Above and left: Completed pedatrian walkway

6. Summary – Fitzgerald Avenue

The stone column ground improvement involved recently patented innovations and low levels of vibration which proved successful in preventing detrimental effects to the river and the surrounding properties. Waste material was minimal and adequate ground strength was achieved – however this did require 300 additional stone columns.

The innovative precast concrete – MSE retaining wall was quick and simple to construct and its aesthetic qualities were acknowledged by the public.

"As odd as it sounds, this piece of road looks brilliant. Following the curve of the river, they could have easily put ugly big slabs of concrete up. With the smartly finished railing not blocking the view, you can see the river, and all the flax bushes on its banks. Good work by these guys."

Stakeholder Feedback "Fitzgerald Avenue to fully reopen" The Press 11/12/2012

Above: Fitzgerald Avenue project complete

Despite record rainfall, high river levels and relentless traffic loading the cement stabilised area of the north bound carriageway continues to perform well. The timber walkway is an attractive feature allowing pedestrians to enjoy a close encounter with the Avon River.

This project was a milestone for the Christchurch earthquake rebuild and set the standard for future earthquake repair work.

7. Heathcote Opawa Bridge – Background

The Heathcote Opawa Bridge which spans the Heathcote River carries approximately 20,000 vehicles per day of which 10% are heavy goods vehicles on SH73. SH73 is a primary lifeline route of strategic importance providing access to the Port of Lyttleton from SH1 west of Christchurch. Constructed in 1990, the bridge is a continuous three span superstructure with a total length of 58.0m and an overall width of 11.13m. It carries two 3.5m traffic lanes.

The bridge superstructure consists of four continuous, 1.6m deep precast, pre-stressed concrete I-beams with an in-situ concrete deck. The central span beams are also post-tensioned. Continuity is provided by the deck slab reinforcement and transverse in-situ diaphragms between the I-beams over the central piers. The diaphragms resist the compression forces between abutting I-beams at the piers. The superstructure is supported on concrete abutments and hammerhead piers.

Above river bed level, the piers consisted of a hammerhead cross beam and a 1.8m wide octagonal pier. Below bed level a 2.0m diameter cylindrical column extends 14.0m deep and is supported on seven 200x146UBP steel H-piles which were driven to a depth of approximately 45m into gravels.

Each abutment was originally detailed to have four steel H-piles driven to 25m and 40m depth at the eastern and western abutments respectively. But when additional H-piles were found to be needed in the pier construction there were concerns that bearing may not be achieved at the original depth. The abutment piles were thus

re-designed to ten closely spaced H-piles driven to found on a shallow layer of dense sand at approximately 6.0m and 12.0m depth at the eastern and western abutments respectively. Each end span is tied to the abutment with six 42mm diameter linkage rods providing longitudinal restraint and three 100mm diameter dowels providing transverse and vertical restraint.

Above: Rubber bearings support the I-beams and there are six RHS earthquake shear keys cast into the hammerhead pier and diaphragm

Above: Both abutments have settled approximately 200mm

Earthquake damage:

- Both abutments settled approximately 200mm causing the previously level bridge to hog
- Cracking has occurred in the deck
- Spalling has occurred to the beam flanges
- The abutments have rotated and displaced inwardly so that they bear against the beams effectively locking in the bridge
- The rubber bearings have distorted in shear
- The paved batters in front of the abutments have settled

The repairs are being carried out by SCIRT with design work by Opus International Consultants and construction by Downer NZ Ltd and Daniel Smith Industries as specialist sub-contractor.

8. Heathcote Opawa Bridge – New piled abutment beams

Four new reinforced concrete piles with permanent steel casings were installed outboard of the bridge girders to support the two new reinforced concrete abutment beams. The bridge will be supported with new bearings on these new beams reducing the end spans by 3m.

Due to the likelihood of soil liquefaction at existing H-pile founding depth, it was decided during the design process that a low vibration method of pile installation was essential due to the risk of further settlement of the already sensitive structure. To reduce this risk, the four piles consisting of 1200mm diameter x 24mm thick permanent steel casings, were installed to a depth of 32m with the use of a Hibrabayashi HS 360 TP-148 Caisson Oscillator with hydraulic power pack. A Hitachi KH 300-3 80 Tonne Crawler Crane utilised a hammer grab/clamshell bucket to excavate the spoil from the casing as it was installed.

The location of the new piles was tight against the bridge beams which required the demolition of parts of the cantilevered outstands of the bridge deck to allow headroom to pitch the 12m high casing. This demolition was carried out using high pressure (20,000 psi) water jets to minimise damage to the reinforcing steel and prevent micro cracking in the adjacent bridge deck.

Above: Daniel Smith Industries installed the piles by oscillator. Part of the bridge deck was demolished to make room for the pile casing

Providing enough kentledge as reaction mass for the oscillator, in the form of steel cased reinforced concrete blocks, proved challenging at times due to limited space as a result of the close proximity of the bridge.

Advancement of the casing varied from 500mm to 3m per day depending on soil conditions, and the rate at which the casing could be cleaned out. Casings were advanced through predominately dense sand layers with some intermittent clay/silt layers until they reached a 10m band of dense gravels and gravelly sands with N-values of 60+. The piles were all founded in the middle of this dense non-liquefiable gravel layer.

With the casings at target depth and all except the last 0.5m of material excavated out, a precast concrete plug was placed at the bottom and driven to a pre-determined set with a heavy-walled steel tube mandrel. A set of 8mm from the 9 tonne mandrel falling 3m was required to meet the design requirements. A satisfactory set of 3mm was achieved with all four piles.

After the precast plug had been driven, the bottom of the pile was sealed off with approximately 4 m^3 of concrete containing an expansive admixture.

The 24m long prefabricated reinforcing cages were delivered to site and suspended in the casings. The concrete was then placed using a tremie pipe.

One of the challenges to the piling operation was the presence of high ground water pressures. Artesian water pressures of the order of 3.5m above ground level were present at each pile location. This was overcome by maintaining a positive pressure head of water in the casing. Consequently, all stages of the piling were done in the wet – the excavation of the spoil, placing and driving the plug, placing the seal concrete and placing the reinforcing and concrete for the pile. This also meant that the casing had to be left high during the piling and was not cut to its final height until the pile was complete.

Another challenge was the assembly of the beam reinforcing cage around the pile reinforcing, both of which had multiple layers of 32mm diameter bars. In some cases, anchorage for the reinforcing was provided by the use of flanged nuts on Reidbar in lieu of standard hooks and this aided the task by reducing the congestion.

Above: and left: Reinforcing for foundation piles and the bridge abutment

9. Heathcote Opawa Bridge – Temporary bridges and jacking.

The project is still under construction at the time of writing with several tasks yet to be completed:

- Transfer of the superstructure load from the existing abutment to the new beam.
- Jacking of each end of the bridge to relieve the stresses in the superstructure.
- Demolition of the existing settlement slabs, abutments and wing walls.
- Construction of new head walls, abutment wing walls and settlement slabs.
- Reinstatement of the road surface.

All this work must be accomplished with minimum disruption to SH73 traffic. Weekend closures of the bridge have been planned and temporary bridge spans will be installed on each approach to allow traffic to use the bridge whilst abutment re-construction is completed.

The philosophy of the jacking is to remove all elastic deformation of the superstructure which may or may not remove all of the hog in the previously level deck. The in-situ diaphragm bridge beam connections will be monitored during jacking, to ensure no additional permanent deformation is caused. It is expected that the cracks at the top of the joint will close up, but jacking will be halted if cracks begin to open in the bottom.

10. Heathcote Opawa Bridge – Central pier strengthening.

The pier columns required strengthening as part of the remedial works. The main challenge was dewatering around the piers in order to carry out the works. Sheet piling would have been very costly and impractical with the low headroom under the bridge. Also traditional mass bunding of the river with metal would have taken up a large area and restricted the river flow. The delivery team used a temporary damming system frequently utilised for flood relief work.

The system consisted of modular lightweight composite frames joined with plates. A high strength polythene sheet was then lapped over the frames and out onto the adjacent river bed. Once dewatering commenced, the difference in water levels caused a greater hydro static force on the river side of the dam, sealing the sheet onto the riverbed. Two small submersible pumps were then used to maintain the dewatering, providing a dry work area to carry out the works.

The existing concrete cover was broken back to expose the pier spiral reinforcing and the top of the 2.0m diameter cylindrical column. 32mm bars were then grouted into 900mm deep holes drilled into the column. Two bars were pull-out tested to 80% yield in order to prove the grout hold was sufficient. The circular reinforcing hoops were then installed and the steel cylinder formwork erected. Self-compacting concrete containing a shrinkage compensating admixture was then introduced into the annulus and a good concrete finish was achieved.

Above: Central pier strengthening works and the temporary damming system used. (Diagram courtesy Hydro Response Ltd)

11. Summary – Heathcote Opawa Bridge

The piling has been successfully completed using the low vibration method and both abutment beams have been completed. Whilst the project is far from over, many of the project activities have been successfully completed or are currently underway with SH73 traffic continuing to use the bridge.

Above: and left: View of the bridge as ta May 2013

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Further reading:

J. Waldin, J. Jennings & P. Routledge *"Critically damaged bridges & concepts for earthquake recovery"* (Paper No 104, NZSEE Annual Technical Conference, Christchurch, 2012)

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Mat Avery
(Geovert)

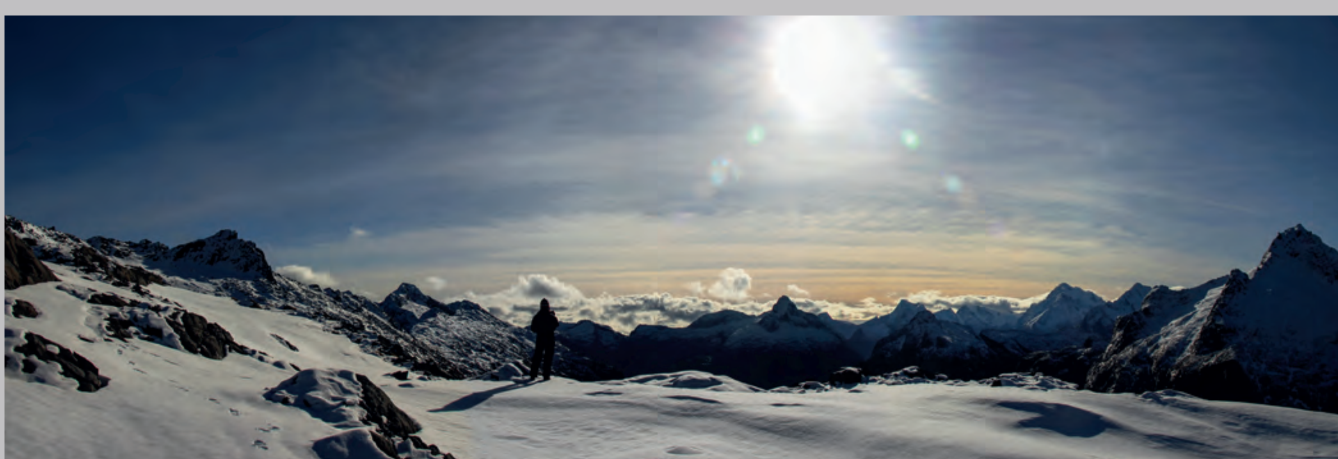
FIVE HIGHLY COMMENDED

Exposed in the Paparoas – ***Flo Buech*** (Golder Associates)

NZ's Next Top Geotech Model – ***Alistair Stuart*** (Fraser Thomas)

Sunrise over the Avon Heathcote Estuary with liquefaction sand ejecta and the Chilean volcanic ash cloud, June 2011 – ***Lee Buhagiar*** (Coffey)

Night-time drilling in the heart of Christchurch's rebuild – ***Paul Hoskin*** (Landtest)



Supervising the blasting work at the Homer Tunnel western portal – ***Rob Bond*** (Opus)

Waterview Connection

VISION FOR A WORLD CLASS CITY

Project Overview

IN JANUARY 2012, the NZ Transport Agency began construction on its largest, most challenging and most expensive project to date. The $1.4b Waterview Connection project will integrate an additional 5km of 6-lane motorway through and beneath Auckland's western suburbs, linking two existing State Highways to complete a long-awaited motorway ring route around the city. In cost terms alone the project is some four times bigger than anything the Agency has previously undertaken. Featuring twin 2.5km tunnels, multiple bridge structures and a panoply of urban design and landscaping components, it is also infrastructure of a scale without precedent in New Zealand.

Completing the Western Ring Route is of huge strategic importance to New Zealand's economic growth, and as such has been designated as one of 7 Roads of National Significance. Currently all Auckland through-traffic, which can reach daily vehicle counts of up to 200,000, relies on a single motorway that runs directly through the city, putting freight, commuters and tourists in direct competition with each other. By providing a viable motorway alternative for through-traffic, it is anticipated that a completed Western Ring Route will create a safer, more resilient motorway network that will improve journey time reliability throughout the region for all. It will also reduce the dependence on the Auckland Harbour Bridge – an iconic yet ageing piece of the city's infrastructure.

In addition to completing the city's Western Ring Route, the Waterview Connection will provide Auckland with a direct, time-saving motorway link between its International Airport and Central Business District. Beyond the economic benefits associated with the 15-minute reduction in travel time between the two points, this new direct link will create a more welcoming gateway to Auckland and New Zealand for visitors. Rather than being forced to navigate their way through a maze of unfamiliar back streets as they are today, visitors will be able to stay on the same piece of road until they reach the city, without encountering a single set of traffic lights in between. For a city striving for a world-class reputation, this streamlined first impression will be a crucial improvement.

In late 2011 the Transport Agency appointed an Alliance of leading domestic and international companies to meet the many challenges of this landmark project. Alliancing has served the Agency well in the delivery of its most complex projects to date, including the recently-completed Victoria Park Tunnel through Auckland city. This particular alliance, known as the Well-Connected Alliance, comprises the Agency as both client and participant, Fletcher Construction, McConnell Dowell Constructors, Parsons Brinkerhoff NZ, Beca Infrastructure, Tonkin and Taylor, and

Obayashi Corporation. This arrangement brings together the knowledge and strong, home-grown reputation of leading New Zealand engineering companies with the world-class tunnelling expertise of its international partners, to ensure delivery of a world class project with a distinctly kiwi accent.

Going underground

At the heart of the Waterview Connection project will be 2.5km of twin three-lane tunnels, meaning more than half of the new motorway link will be underground. The tunnels will dive to a depth of 45 metres, passing beneath a 15-metre thick shelf of basalt that remains as a hard-rock legacy of the region's volcanic past.

The project will create a continuous route past the five volcanic mountains that stand between Auckland's Waitemata and Manukau harbours. Maori legend celebrates a close relationship between New Zealand's people and its volcanic landscape, and the Transport Agency has been sure to recognise both the historical and future significance of the route. When the Tunnel Boring Machine that will build the tunnels was first built, its 14.5m face was draped in a specially commissioned artwork. Called Te Haerenga Hou ("A New Journey") the artwork illustrates the route of the new motorway past the five mountains, around the lovers Tama Ireiea and Hine Mairangi, whose own story is synonymous with the region's volcanic past.

The machine in question is an Earth Pressure Balance Machine that has been custom built for the local geology by German manufacturer Herrenknecht. Originally put together at its Guangzhou factory in south east China, it, or rather she (named Alice, after the Lewis Carroll character through a competition for local school children), has been reassembled hard up against the first tunnel to be bored. With a 14.5m diameter cutting face - featuring a 45% opening ratio to allow it to munch its way through the softer sand and siltstone of the local East Coast Bays formation that sits beneath the basalt – and a 90-metre long gantry, Alice the TBM is not just huge by New Zealand standards, but is the biggest ever to have been used in the southern hemisphere.

On arrival in New Zealand, the immediate challenge was not how to assemble the 97 loads of TBM paraphernalia, but how to get the largest elements from the port to the work site. The heaviest load, the 260-tonne main drive, required a kind of giant, mechanised push-me-pull-you (we do like our mythical creatures in New Zealand) to negotiate the 11km journey: two trucks in front of and one behind a 192-wheel reinforced trailer. The next leg of the journey for the main drive was to lower it the final 40 metres into place in the trench. The meticulously planned manoeuvre was carried out using a 600-tonne crane, positioned on top of a deep-piled, reinforced crane platform, capable of supporting a total weight of almost 1200 tonnes above the trench. Despite being only a temporary structure, the platform required deeper and longer piles than any of the permanent structures that will be built on the entire project. At the time of writing, Alice is fully assembled and ready to go – her return journey to Waterview expected to be completed by Christmas 2015.

Tunnelling is expensive, and accounts for over two-thirds of the Waterview Connection project's $1.4b budget. When the project was first mooted back in 2000, it was put forward as a uniquely over-ground link with an estimated cost of less than $100M. The leap to $1.4b reflects the value placed on retaining the communities, green space and public amenities that sit between the two existing motorways. However, not all the work will be carried out underground, and not all will be for the unique benefit of motorway users.

Alice the TBM is not just huge by New Zealand standards, but is the biggest ever to have been used in the southern hemisphere.

The Waterview Connection project will also integrate road bridges, cycleways and pedestrian bridges within a suite of urban design, landscaping and environmental enhancements. The inclusion of these elements is complemented by ongoing community involvement, as the Agency has committed to delivering its biggest ever project with maximum benefit and minimum disruption to the local community.

The project is scheduled for completion in early 2017, to coincide with the realisation of a number of other capacity and resilience improvements on the motorway network. The Alliance's relationship with the project will be a little longer though, as it remains responsible for the operation and maintenance of the tunnels for a further ten years.

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Well-Connected Alliance

'WONDERLAND' – Preparing To Send Alice Down The Rabbit Hole, The Southern Approach Trench to the Waterview Connection Tunnels –

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Introduction

The Southern Approach Trench (SAT), situated in Alan Wood Reserve, Owairaka, Auckland has provided unique geotechnical challenges during design and construction. Excavation of the SAT started the underground works of the Waterview Connection Project and the northern extent of the trench now forms the south portal for the 13.3m internal diameter twin bored tunnels. The tunnelling operation began in October and the SAT provided the space for the assembly of the 14.4m diameter Earth Pressure Balance (EPB) Tunnel Boring Machine (TBM), aptly named Alice. The location of the SAT in relation to the Waterview Connection Project's main construction areas is shown in Figure 1.

The SAT is approximately 400m in length and at the tunnel portals the temporary excavation reached 29m depth at the portal sump. Key features of the construction works are shown on Figures 2, 3 and 4. Through an intensive investigation programme and sound understanding of the complex geology and hydrogeology of the site, the geotechnical engineering team was able to deliver a safe design, allowing early site establishment and commencement of tunnelling.

Figure 1: Location of the SAT in relation to the other major parts of the Waterview Connection Project

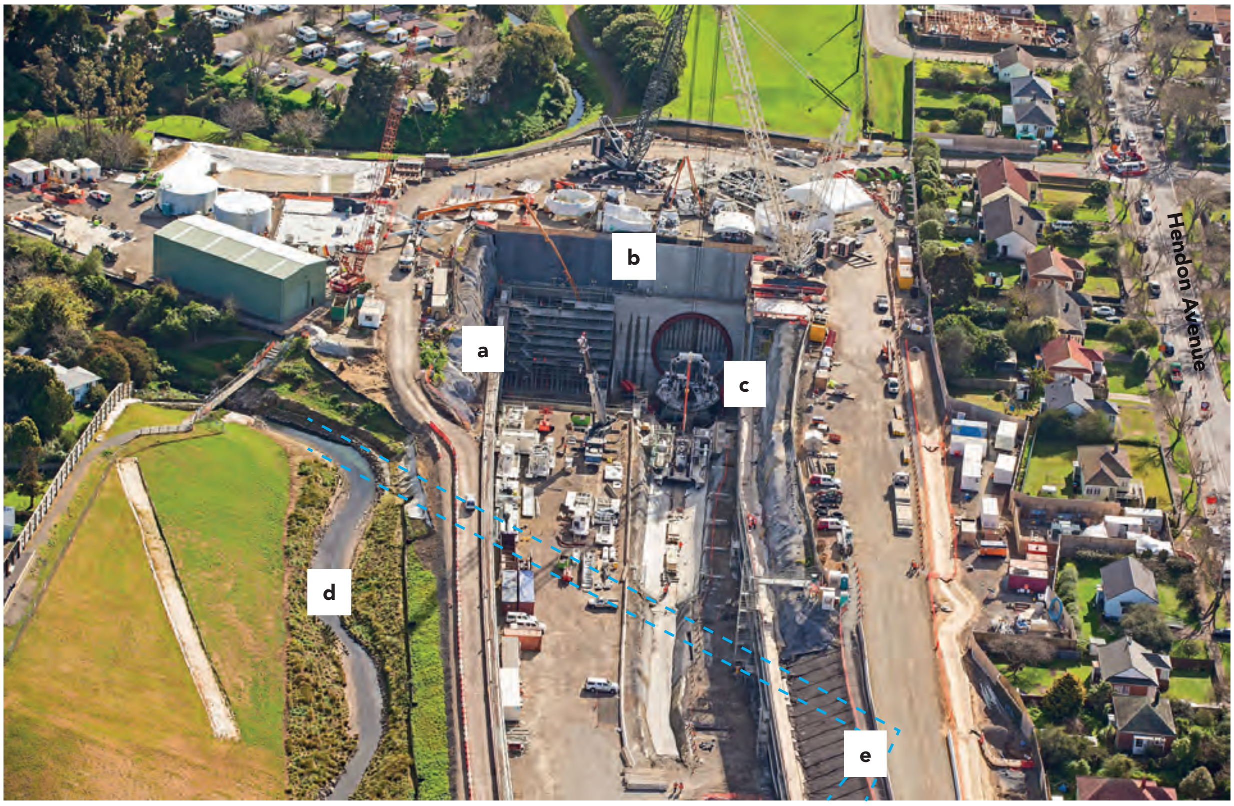

Figure 2: North aerial view of SAT on 7 August 2013. a) Western retaining wall RW909, b) Portal headwall RW910, c) Eastern retaining wall RW911, d) Diverted Oakley Creek, e) Position of Oakley Creek prior to diversion

Figure 3: East aerial view of SAT on 7 August 2013. a) Portal sump excavation b) extent of stabilised block and MSE wall c) temporary crane platform supported on capping beam, d) Shotcrete and rock bolt supported basalt excavation, e) Passively drained pile retaining wall (RW911) supporting excavation in Tauranga Group and weathered East Coast Bays Formation soils as shown in Figure 4. Note the capping beam of RW911 following the base of the basalt.

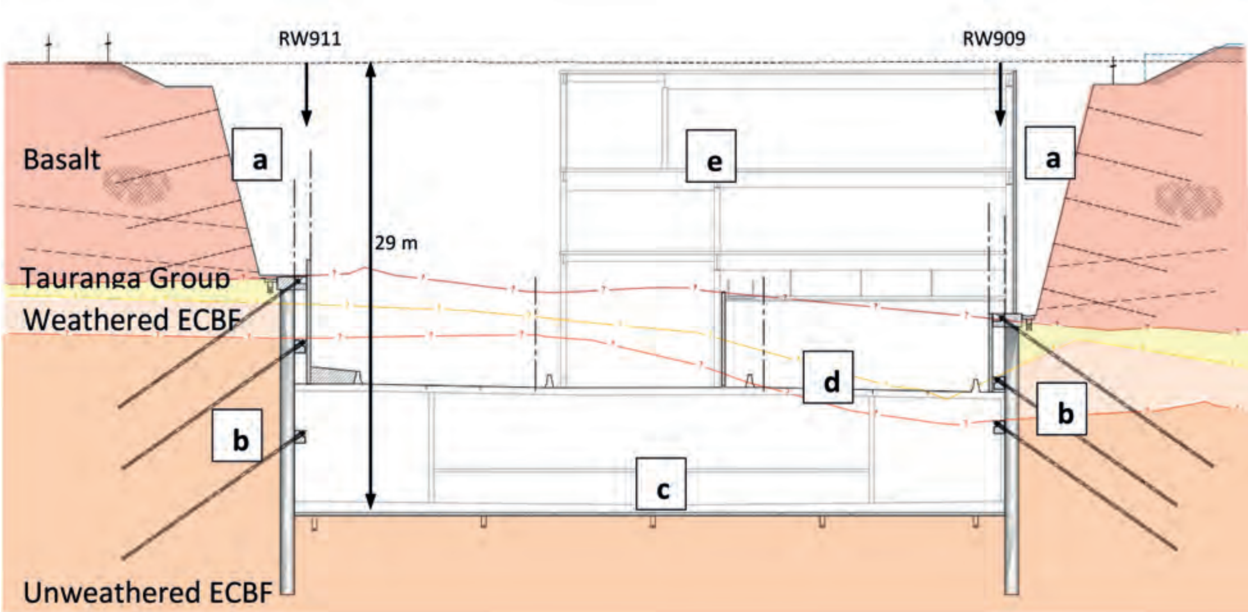

Figure 4: General arrangement of Southern Approach Trench 20 m south of tunnel portal. a) Basalt cut, b) Anchored bored pile walls, c) Portal sump, d) Road deck, e) Southern Vent Building

Geological Setting

Background geotechnical investigations of the site were carried out between 2000 and 2011 when route options, specimen design development and resource consent applications were underway. The award of the Contract to the Well-Connected Alliance in October 2012 signalled the start of the detailed investigation programme at the site.

The SAT is situated on a circa 100,000 year old (pers comms Dr B.W Hayward) basalt lava field sourced from Mount Albert volcano 1.2 km to the north east. The relatively flat nature of the Alan Wood Reserve is a function of the ponded basalt lava that has infilled a series of paleovalleys that drained from the hill slopes located to the south west. Oakley Creek has incised itself along the edge of the basalt and delineates the south western extent of the basalt lava field. A unique aspect of the SAT is it's

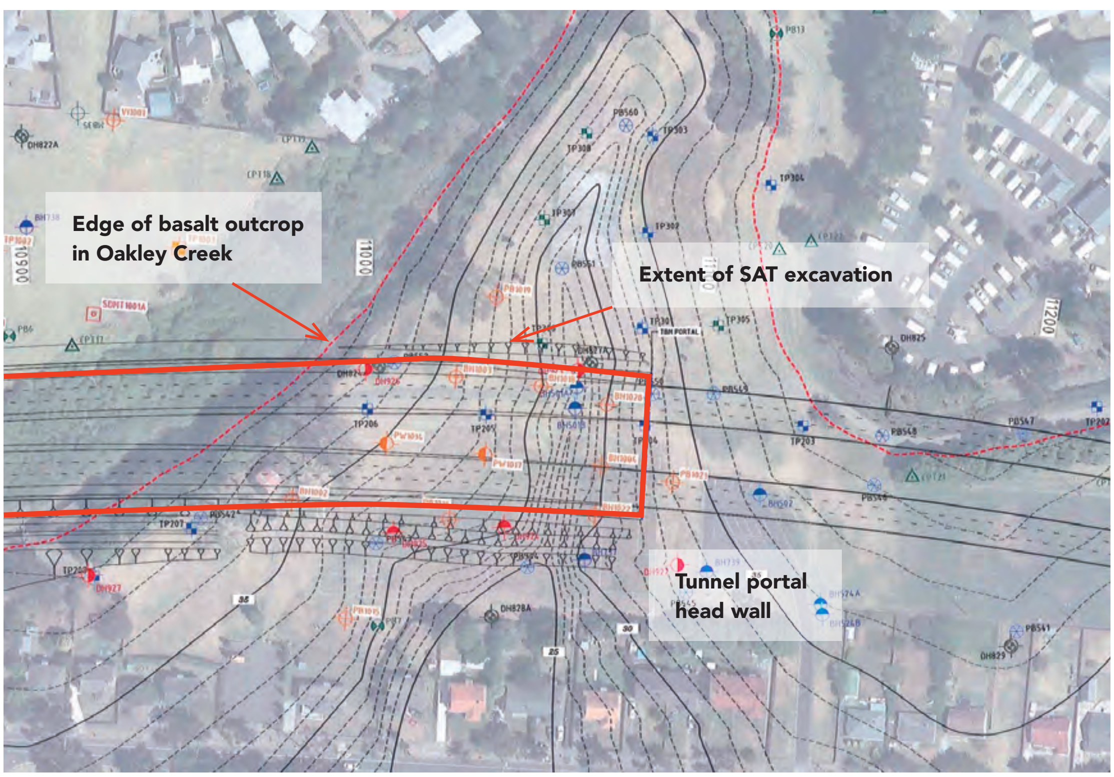

Figure 5: Aerial photograph of the SAT prior to construction starting overlaid with contours defining the relative level at the base of the Mt Albert basalt flow that mantles the paleo-topography of the Tauranga Group soils underneath. Note the relative closeness of the contours through the axis of the paleovalley.

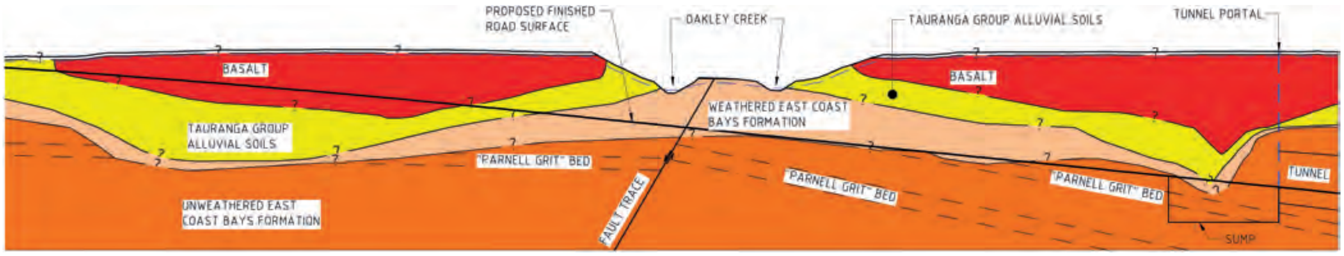

Figure 6: Geological long section along RW911

"toasted cheese sandwich" like stratigraphy, whereby the strong basalt rock (UCS of 25-120 MPa) is located at the surface with the comparably weaker ECBF rock (UCS of 1-5 MPa) located at depth. The cheese in the sandwich is represented by the fine grained soils of the Tauranga Group and the weathered East Coast Bays Formation.

An important aspect of the investigation programme was to establish enough data points to accurately map in three dimensions the basal contact of the basalt lava flow, and in particular, the position and shape of the paleovalleys crossing the SAT. Figure 5 shows an example of the three dimensional model produced for the basalt flow. Similar models were also developed for the geological contact between the Tauranga Group, the weathered East Coast Bays Formation and the surface of the unweathered East Coast Bays Formation. The 3D model was developed in CAD software 12D and imported into the Geometric Software MX. The ease in generating geological sections through cross sectional cuts in MX saved processing time and allowed for quick and accurate design development, such as presenting concepts for positioning the tunnel portal, accurately modelling groundwater flows and establishing the retaining wall capping beam levels (refer Figure 4). The RW911 geological long section is shown in Figure 6.

The strength of the paleosol directly below the basalt was considered an important feature to quantify for design development. Borehole logs from earlier geotechnical investigations showed the contact to be a black organic soft soil. Preliminary stability modelling using limit equilibrium software identified a risk of basalt batter instability due to the presence of such a soft layer. Using the 3D model to anticipate the base of basalt, the "soft" layer was targeted during drilling and carefully exposed without penetrating into the underlying soils. A seismic dilatometer (SDMT) was then attached to the end of the drill string and pushed into the contact to measure stiffness and, by empirical correlation undrained shear strength (Su) was obtained. Samples were also collected for organic, atterberg and triaxial testing. The investigation results concluded the contact to be much stiffer than previously logged and almost devoid of organics. The dark nature of the soil was attributed to 'baking' from the hot basalt flow during emplacement and manganese staining. This was subsequently verified during excavation. Figure 7 shows how the un-drained shear strength correlated from SMDTs compared to the strength obtained from hand held shear vanes when the contact was exposed during excavation. The range of Su measured in the field was typically between 40 kPa and 70 kPa. SDMTs penetrating into the Tauranga Group also suggested some overconsolidation of the soils as a result of the basalt overburden.

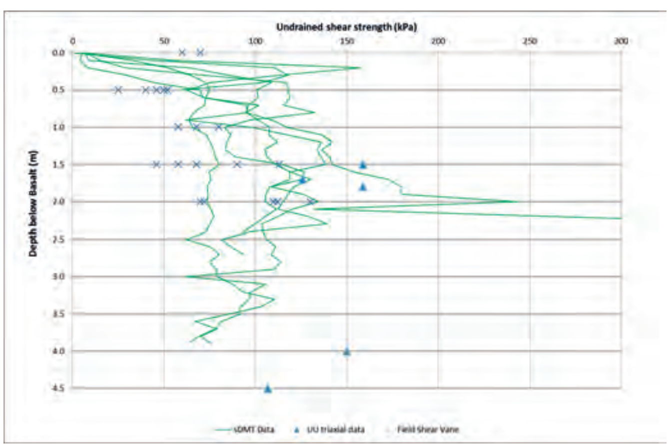

Figure 7: Comparison of the undrained shear strength determined from Seismic Dilatometers (sDMT) undertaken at the base of boreholes drilled through the basalt with field shear vane tests carried out when the Tauranga Group soil was exposed during excavation.

The fracture state of the basalt and ECBF was assessed by plotting Rock Mass Rating (RMR) from cored boreholes and using acoustic televiewer geophysical investigation methods. The results of the televiewer correlated well with the exposures of ECBF mapped during excavation. Borehole logs and the RMR assessment determined a change in the fracture state of the basalt at 6-8 m depth, later confirmed during excavation, which determined a vertical columnar jointed upper section and a platy sub-horizontal jointed lower section.

Given the depth of the basalt lobe to be excavated and dewatered, another hot topic for the SAT was the expected groundwater inflows (that could be large should there be sufficient connection to the main basalt flow and / or Oakley Creek). The hydrogeological properties of the basalt lobe crossing the SAT were investigated by in-situ permeability tests and a series of short constant rate and constant head pumping tests. These proved that the basalt lobe was of high permeability (i.e. 1 x 10^{-5} m/s or

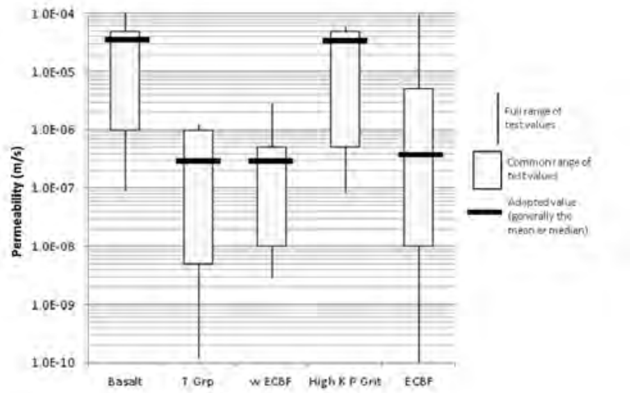

Figure 8: Range of permeability test results and values adopted for design

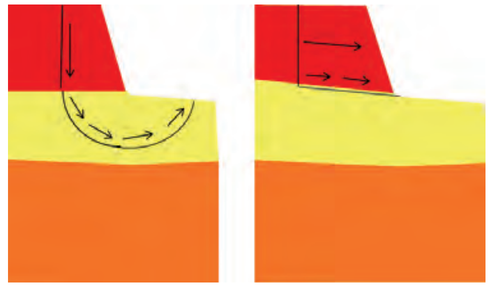

Figure 10: Postulated failure modes in basalt over alluvium. a) 'Bearing' type failure requiring dilation of cooling joints. b) Sliding failure on alluvium with tensile failure in basalt.

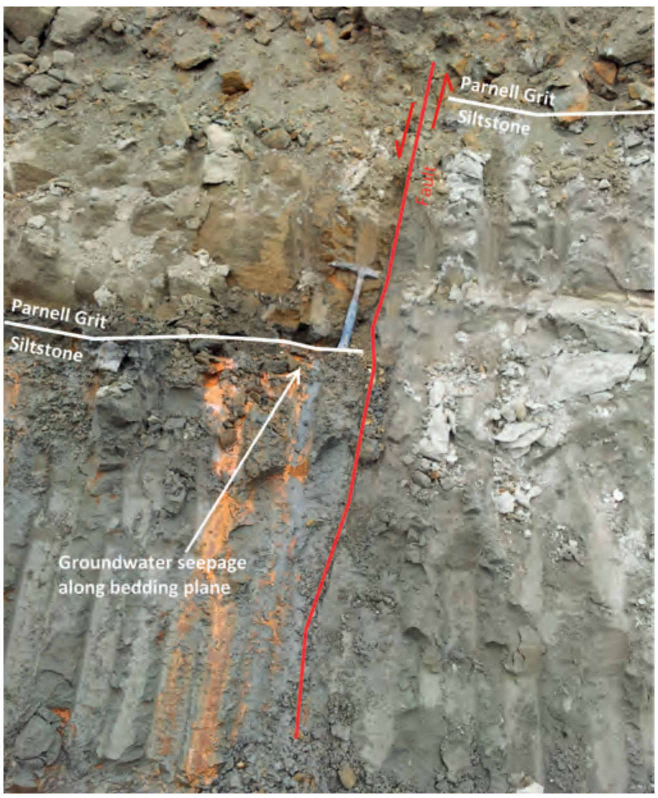

Figure 9: Shows the influence of the rock structure on groundwater flow in ECBF. Note that there is negligible groundwater discharge on the right hand side of the fault.

greater) but low storativity, and with limited connection to the main basalt lava flow or the creek; meaning that the basalt can be readily dewatered without the significant groundwater flows that have been a feature of other such excavations in Auckland. A substantial database of permeability tests results was built up from in-situ testing in the different geological units encountered across the Project. A summary of the test results and values adopted for design are presented in Figure 8.

Interpolating marker beds (i.e. coarse grained sandstone "Parnell Grit") between boreholes determined the existence of a fault in the East Coast Bays Formation (Figure 6) crossing the SAT. This was later confirmed by mapping during excavation of the SAT and had an influence on the groundwater regime (Figure 9). Piezometers located on the southern side of the fault recorded much higher groundwater levels in the ECBF relative to those located on the northern side.

Analysis of Basalt Cut Stability Above Daylighted Alluvium

One of the more unusual features of the design of the SAT is the open excavation to the base of the thick basalt deposit where the alluvium is daylighted, temporarily without the support of the piled wall. The likely behaviour of the excavation, in the short and long term was of interest. A conventional bearing capacity calculation, with the basalt considered as a uniform surcharge leads to low factors of safety (or an estimate of instability) depending on assumptions regarding the strength and state of drainage of the alluvium. However due to the nature of the basalt rock mass (specifically the persistence and waviness of cooling joints), significant dilation of the basalt would be required for such a mode of failure, as the basalt would need to displace vertically downwards. A more likely failure mode is that of sliding on the upper surface of the alluvium where a block of basalt would only need to fail in tension (which could then lead to a bearing failure once the basalt is "pulled apart"). Figure 10 shows a diagram of these postulated failure modes. The condition of the uppermost portion of the alluvium, as well as the geometry of the contact was therefore of primary importance. The effort undertaken both in characterising the properties at the top of the alluvium, and of modelling the paleo-topography as described above was of great use in assessing this potential mode of failure. A photograph of the geological contact is shown in Figure 11. The firm strength of the alluvial soils is reflected in the good stand-up time of the vertical side walls of the excavated slot.

Based on the result of this assessment the basalt excavation along the side walls of the SAT was able to

Figure 11: Contact between Basalt and Tauranga Group Alluvium exposed in short sections prior to bulk poor for concrete mass block stabilisation. Note the stiff nature of the soils evident by the stability of the vertical cut.

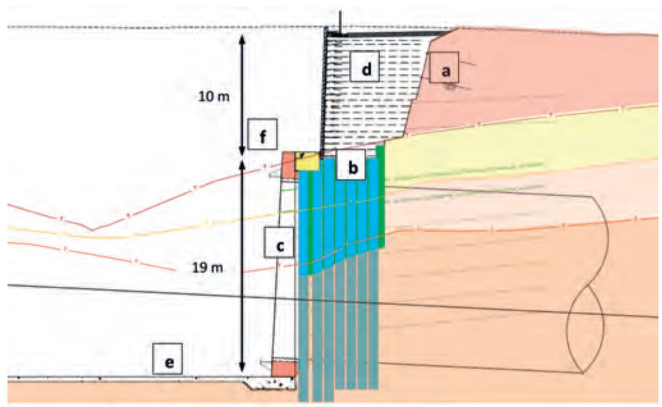

Figure 12: Section through portal headwall showing major features in order of construction sequence. a) Basalt excavation and temporary bolting, b) Weak concrete piled block c) Excavation to maximum depth in front of piled block undertaken concurrently with d) MSE Wall with full height pre-cast concrete facing panels, e) sump base slab constructed then, f) reinforced concrete headwall

be constructed without any engineered support elements which would have been expensive to implement.

Drained Sub-basalt Retaining Walls

The retaining walls below the basalt consist of bored piles with up to three rows of multi-strand ground anchors through reinforced concrete walers and capping beams. The piles were spaced at approximately two to three pile diameters, with drainage and shotcrete lagging installed between piles. The environmental effects of this 'drained' wall design were assessed with the aid of three-dimensional seepage and settlement analysis. This assessment indicated any ground settlement and groundwater inflow would remain within consented limits. The drained wall design significantly reduced the structural demands on the retaining walls as a result of less ground water pressure, particularly as a significant area of the retained ground was unweathered ECBF rock which otherwise required minimal retention.

Ground Anchor Design

Approximately 400 multi-strand ground anchors were installed in the SAT ranging in length from around 15m to 30 m. The use of ground anchors, instead of internal propping, allowed an open excavation free of obstructions. This is particularly important in enabling the efficient assembly of the TBM and backup gantries in configuration prior to launch. Similarly it enables the construction of the Southern Vent Building to be undertaken within the trench independently of the critical path excavation prior to assembly and launch. The ground anchors were all tremmie grouted straight shaft borehole anchors with double corrosion protection with bond lengths in the unweathered ECBF.

Design of Portal Retention

Several design, construction and site constraints had to be met in the design of the portal retention works. As shown in Figure 12 approximately 10-12m of basalt was present above the portal, with alluvial and residual soils extending into the driven portal itself. A primary challenge was the requirement to retain and/or improve the soil in the portal face. Ground improvement techniques from the ground surface, such as jet grouting, were considered but found to be uneconomic due to the presence of basalt and the depth of treatment required. Retention with glass fibre reinforced polymer (GFRP) reinforced piles, or GFRP soil nails were considered, but were not favoured due to the potential for complications with launching the TBM through such materials.

Another challenge was the inclination of the base of the basalt at the portal. The alignment portal face is approximately parallel with the paleo gulley that was infilled by the basalt. This meant that, unlike the sidewalls, the dip of the base of basalt/ alluvium interface was relatively steep.

The solution was to over-excavate the basalt behind the portal face, exposing the soils beneath the basalt. From this level at the base of the basalt, a series of partially interlocked weak concrete (5-15 MPa) piles were installed, mainly using a continuous flight auger piling rig to create a block of improved ground. The area of over excavated basalt was then backfilled with an MSE wall, buttressing the basalt cut. The piled block was designed to act in two ways, both as a gravity wall and to horizontally distribute load into the adjacent anchored RC piles. This enabled a relatively thin area of weak concrete piles to be installed, minimising piling and basalt over excavation.

Figure 14: Continuous Flight Auger (CFA) piling rigs forming the stabilised block behind the tunnel portal head wall. A fleet of trucks located above the SAT provided concrete to the CFA rig via the boom.

In order to act as a gravity wall, the improved ground needed to be able to perform as a block. One issue with a piled block with contiguous piles only is that vertical planes of weakness are created between adjacent rows. The block, however, needs to be able to resist shear forces on these vertical planes through the piles as this direction is orthogonal to the horizontal plane along which sliding must be resisted (this is on the basis that the piles have no reliable bending strength). For this reason the piles needed to be interlocked to tie each row together. Shear forces can then be transferred via the cold joints between adjacent interlocking piles. Another reason the piles are interlocked is to provide a stable excavation above the TBM during launch to minimise the loads on the TBM shield. A layout of the piled block is shown in Figure 13 and the CFA rigs in operation in Figure 14.

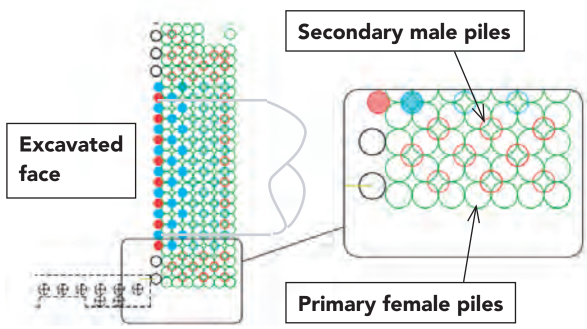

Figure 13: Plan showing layout of weak concrete piles at headwall

Support of Crane Platform

In order to minimise the lift radius for the major TBM components during assembly, a temporary crane platform has been constructed above the eastern SAT sidewall, together with a smaller auxiliary crane platform to support the cranes superlift counterweight. After assembly, the crane platform will be removed and rebuilt on the western side on the SAT for disassembly of TBM after the completion of tunneling. The crane platform is supported on one side by columns founded on plinths on the sidewall capping beam, and on the other by a footing on the basalt. Figure 15 shows a photo of the configuration. The loadings from

Figure 15: TBM cutterhead being lowered into position by crawler crane on platform

Figure 16: Highly weathered and extremely weak vesicular water carrying basalt adjacent to strong, unweathered low vesicularity basalt

Figure 17: View of two distinctive jointing patterns in the basalt flow. A) Vertical columnar jointed basalt in upper part of flow B) Low persistence, sub-horizontal jointing in platy basalt at the lower part of the flow. The Rock Mass Rating (RMR) determined from boreholes drilled during the detailed design anticipated a change in rock structure at approximately 6m depth. Blasting was typically required in Type A with ripping and free digging possible to an extent in Type B.

the crane platform did not significantly affect the side wall design, although the embedment lengths of the relevant piles were lengthened slightly for vertical capacity.

The basalt cut face under the crane platform footings was supported by a number of multi-strand anchors (pre-loaded to approximately 200 T) to mitigate against defect controlled failures in the rock mass due to the crane platform loading. The ground anchors were founded in the basalt itself, beyond the vicinity of the crane platform footing. Generally the anchors bearing plates were able to bear directly onto the basalt face, with some levelling concrete used to infill depressions in the face. For areas of weathered or highly fractured basalt (refer Figure 16), a reinforced concrete bearing pad was used to reduce bearing stresses.

To Grout or Not to Grout

During consenting an almost 600m long grout curtain had been proposed as a cut off to reduce the anticipated large groundwater flows from the basalt and creek. Given the uncertainty around the number of holes, grout take, effectiveness, cost, and hence value for money, omission of the grout curtain was identified as a potential opportunity.

Based on a review of groundwater levels, Oakley creek bed stage data and borehole records it was considered likely that the basalt lobe intersecting the SAT had limited connectivity to the main basalt lava flow (where significant groundwater flows had been encountered during site investigations) or the creek. This was confirmed by the aforementioned pumping tests which were unable to be maintained for any significant discharge rate or period of time.

3D groundwater modelling, using the 12D model described earlier, indicated that without the grout curtain peak inflows were likely to be typically less than 1,000 m^3/d (12 l/s). The modelling also indicated that groundwater drawdown and settlement would not be significantly greater than that calculated for a grout curtain. In fact analyses suggested that omission of the grout curtain would result in reduced differential settlements and therefore provide a better outcome in terms of the potential for damaging settlement.

Bolts and Anchors

Three main rock mass types were encountered during the excavation. The weakest and most irregular is pictured in Figure 16 with the two more common types pictured in Figure 17. The complexities of the different rock mass types required the use of variations of support type and the use of an observational approach during construction.

Initially, support for the upper basalt layer comprised rock bolts, steel mesh and shotcrete. This was suitable for poor quality rock such as that shown in Figure 16, but was very difficult and time consuming to install on the blocky columnar jointed basalt due to the stiffness of the mesh and difficulties moulding it to the face. It also consumed a large volume of shotcrete in order to reach the minimum cover over the mesh and bolt heads (refer Figure 18). Design development during construction allowed the removal of the steel mesh with a change to a two pass approach (Figure 19). An initial 75mm of fibre reinforced shotcrete was sprayed on the face followed by rock bolt installation and then a final 75mm to cover bolt heads and achieve a minimum thickness of 150mm. Poly fibre reinforced shotcrete was trialled however, this was prone to 'balling up' in the concrete mixer and frequently blocked shotcrete lines and thus steel fibre was determined to be the better product. The change in shotcrete application methodology allowed for significant production gains in excavation and support.

Figure 18: Initial trialled support installed in blocky basalt using mesh and single shotcrete application. Note large shotcrete volume required to fill between the mesh and the basalt face

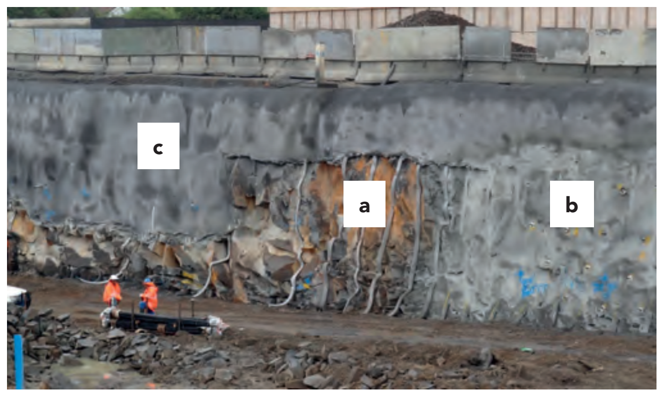

Figure 19: Installation of the two-pass steel fibre reinforced shotcrete and rock bolts at various stages of the process a) Strip drains attached to basalt b) first pass shotcrete sprayed and rock bolts installed c) second pass shotcrete sprayed and bored drains installed

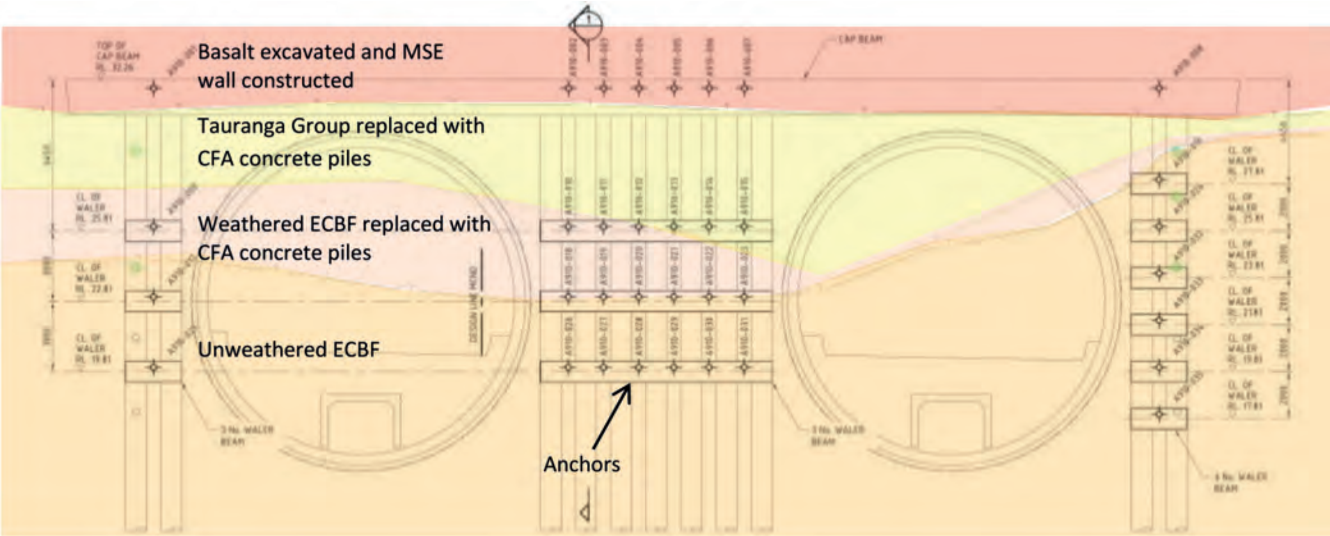

Figure 20: Headwall anchor arrangement.

One of the construction challenges involved assuring the alignment of the ground anchors in the headwall (refer Figure 20) where they passed close to the excavated profile of the mainline tunnels. The design line of the anchors passes approximately 1.5 m from the tunnels at the closest point. Because of the potentially significant consequences of misalignment resulting in a clash with the TBM cutter head, the highest risk anchor boreholes were surveyed to verify the as-built location prior to installation. Initially a magnetometer based instrument was tried, however the natural variations in magnetic field of the basalt caused unsuitable variations and instead a gyroscopic instrument was used with good success. All anchors surveyed were found to be within the specification alignment of one in thirty, and most were around one in one hundred. Anchor misalignment ranged from 100mm to 470mm both towards and away from the tunnel alignment, but outside the tunnel exodus.

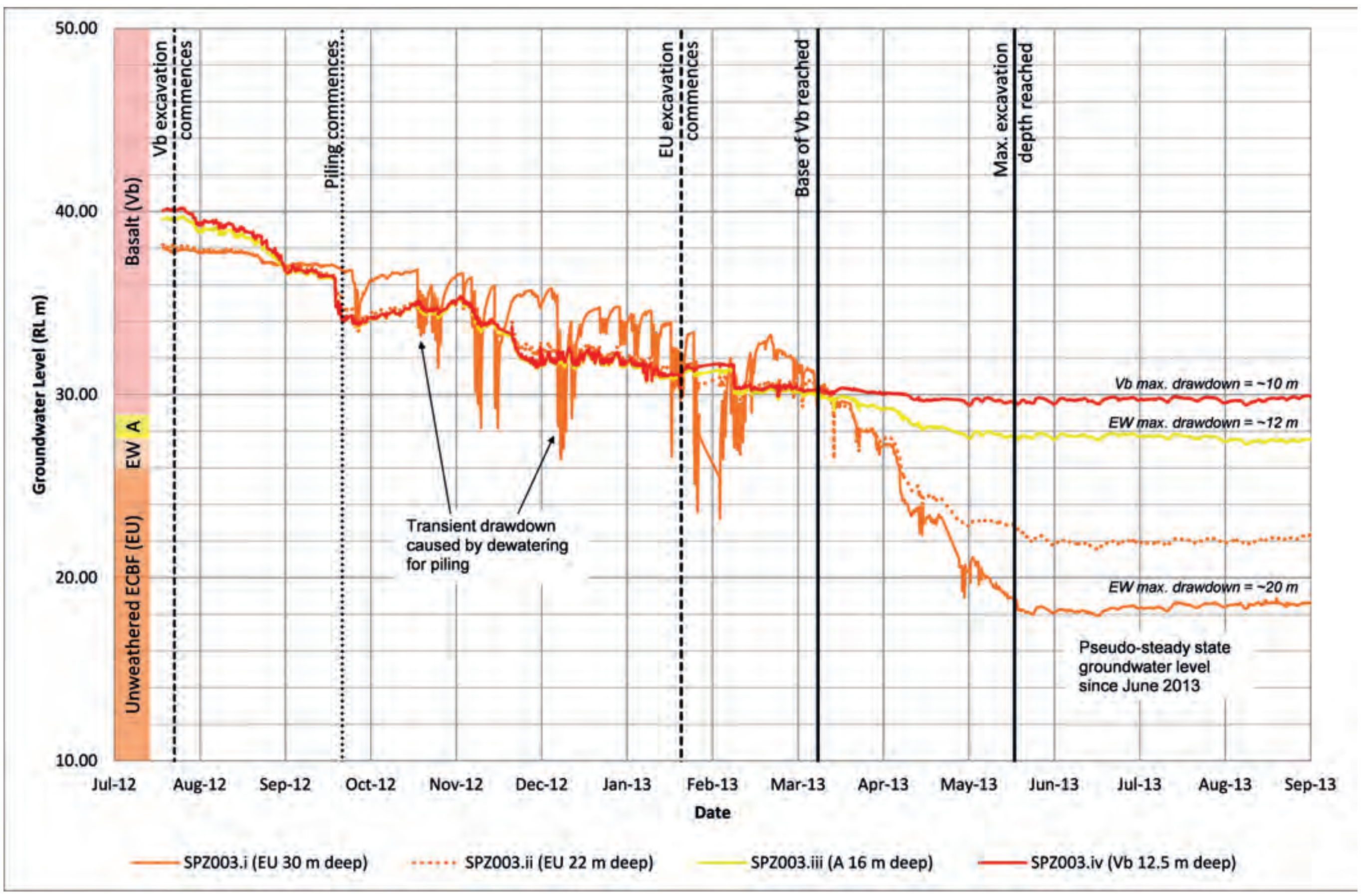

Figure 21: Typical response to dewatering (as seen in SPZ003, located 5 m from SAT)

Groundwater Inflows During Excavation

During construction, total groundwater inflows have typically been of the order of 300 m^3/d to 600 m^3/d (3.5 l/s to 7 l/s) with short term peaks of up to 900 m^3/d (10 l/s) during rainfall events. This compares well to the groundwater modelling which suggested inflows were likely to be of the order of 700 m^3/d. Approximately 60 % to 70 % of this flow is sourced from the basalt, with the remaining from the ECBF.

Despite the groundwater table being within 2m of the ground surface, significant groundwater inflows were not encountered until the excavation was almost 9m below ground level. When groundwater was encountered it tended to "chase" the excavation, with water typically discharging near the base of each excavation and from within the floor of the excavation. The majority of groundwater in the basalt is presently discharging from two bored drains located on the eastern side of the SAT within the deepest part of the basalt lobe infilling the paleovalleys.

Where groundwater was seen to discharge from the ECBF this was typically along bedding planes as can be seen in Figure 9 where groundwater discharges at the contact between a bed of Parnell Grit and an underlying siltstone bed.

Drawdown and Settlement in Response to Dewatering

In addition to the wider network of groundwater piezometer monitoring installed for consent compliance, nested vibrating wire piezometers were installed in close proximity to the walls to confirm the response of units to dewatering, and also to confirm the groundwater pressures on the drained walls.

At the start of works, the water table in the basalt and alluvium was found to be about 2m higher than that in the underlying weathered and unweathered rock, likely the result of perched groundwater on the low permeability alluvium whilst the water table in the ECBF tended to be at about the same level as the creek.

Groundwater levels measured as excavation progressed are shown on Figure 21. From the commencement of basalt excavation and dewatering, the water table in the basalt and underlying alluvium dropped coincident with the excavation level. A more gradual drop in level was also observed in the underlying weathered and unweathered rock, though the water table did not lower by as much or as quickly. For a period of time, the water table in the basalt and alluvium was then lower than that in the underlying ECBF.

Dewatering for piling was found to have a marked effect on groundwater levels in the ECBF with drawdown of the water table during the week days, and then recovery overnight and in the weekends (when pumping was not occurring). This was recorded in piezometers at significant distances of 50 m to 100 m away from the pump locations. This rapid response to pumping is a result of the low storativity in fractures within the ECBF.

When the base of basalt was reached the water table in the basalt reached a steady state condition. From commencement of excavation in the ECBF, the water table in this unit began to drop rapidly, stabilising when the excavation reached maximum depth. Groundwater levels have been relatively steady since May 2013.

As expected the greatest drawdown occurs at depth in the ECBF with a reducing magnitude of drawdown as you move up the geological profile. Drawdown in piezometers located to the south of the fault is much less than those located to the north of the fault.

The porewater pressure for the different units, before and after construction is shown on Figure 22. The basalt is fully drained at the excavation face as a result of the bored drains. Whilst there has been a significant reduction in

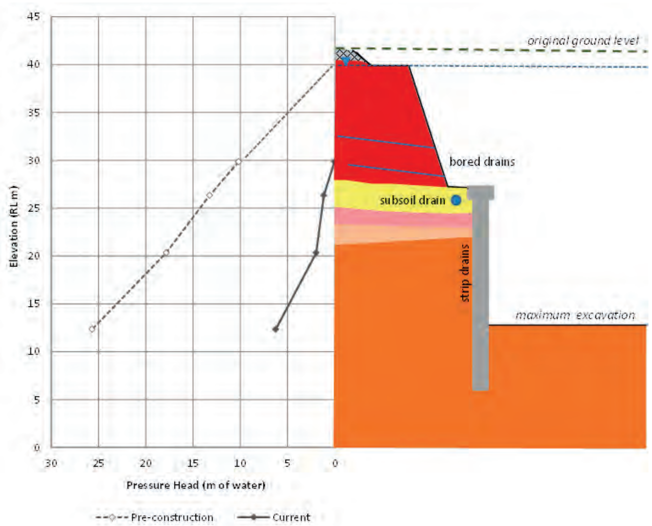

Figure 22: Water pressure pre- and post-construction measured in piezometer SPZ003 located approximately 6m back from RW909

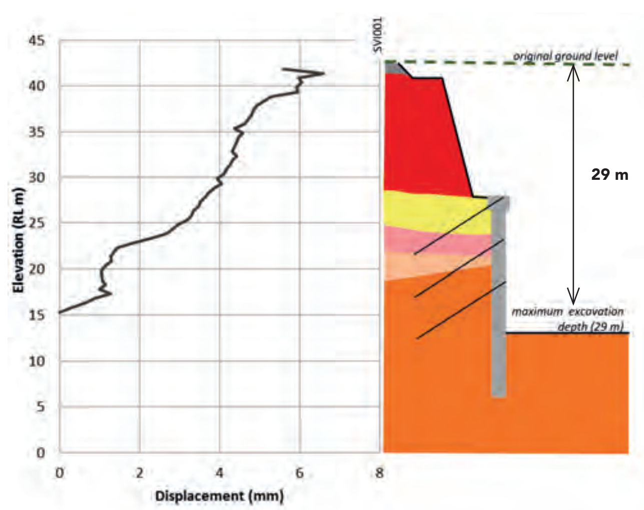

Figure 23: Inclinometer SVI001 displacement

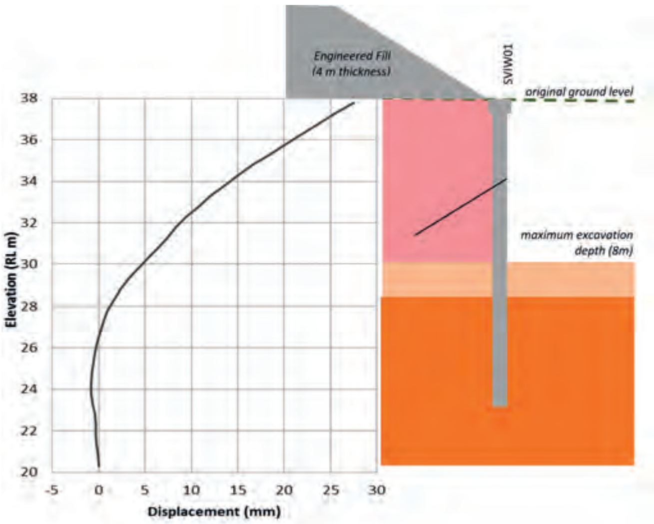

Figure 24: Inclinometer SVIW011 displacement

water pressure in the alluvium and ECBF, the unit is not fully drained at the location of the piezometer, which is approximately 6 m back from the drained retaining wall.

In piezometers located further away from the alignment, drawdown has been observed in the alluvium and ECBF, with the greatest drawdown occurring in piezometers located along the axis of the paleovalley; though it is noted that drawdown has been exacerbated by the exceptionally low rainfall in January to March 2013, and again in July 2013.

The excavation has been dewatered for over a year now and monitoring has indicated no ground movement that could be attributable to consolidation settlement, and significant settlement is considered unlikely given the thickness of basalt at the surface.

Performance of Excavation

Several types of instruments have been installed to monitor the performance of the basalt cuts and side walls and headwall, including inclinometers, piezometers, strain gauges, anchor load cells and survey deflection monitoring. The primary monitoring method was based on the inclinometers, with other instruments playing a secondary role.

As discussed above one of the more critical aspects in terms of performance was of the basalt cuts where the base of basalt interfaces with the alluvium. Five inclinometers were installed prior to excavation of the basalt in boreholes behind the cut face down to the base of the excavation. These inclinometer showed less than 10 mm of movement of the basalt toward the excavation, and no evidence of a discontinuity of displacement at the base of the basalt. Figure 23 shows a typical inclinometer profile of one of these inclinometers.

Twelve inclinometers were installed within sidewall piles to measure displacement of the sidewalls. One of the key factors considered to affect displacements of the sidewalls was the timing and position of ground anchor installation. Some parts of the wall had anchors installed through the capping beam, and through walers, whereas for other parts only waler anchors were used. The highest wall displacements were in one of the areas where only waler level anchors, generally 4 m below the top of the wall were installed, on inclinometer "SVIW011" as shown on Figure 24. The displacements occurred during the initial cantilever stage of excavation prior to anchor installation. During this time approximately 4 m height of fill was placed as a berm behind the wall.

Overall, performance of the excavation has been within acceptable levels set prior to excavation to date. Monitoring will continue until the end of the construction phase of the project, with more intensive monitoring planned during launch and breakthrough of the TBM.

Closing Remarks

Tight construction deadlines set by a race to beat Alice's arrival on site together with the inherent geotechnical risks at the site drove innovative geotechnical design solutions. Geologists and geotechnical engineers worked closely with each other and the construction team to develop, scrutinise and challenge the design and construction methodologies employed. This followed through into the construction phase during excavation and retention as the geology was exposed and site conditions assessed. The SAT design and construction represented a great example of what can be achieved in Alliancing by utilising cross-company resources and experience, to deliver a significant below ground structure in a challenging geological setting.

Acknowledgements

The authors would like to acknowledge the New Zealand Transport Agency, for permission to showcase this project, and the Well-Connected Alliance, for the numerous accompaniments within the text.

Neil Korte (Senior Engineer, Tonkin & Taylor) is acknowledged for his management of the geotechnical design team and his thorough review and inputs into this article.

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Ground Anchors and Soil Nails Provide Retention on the Waterview Connection Motorway Project

GROUTING SERVICES PLAYED a significant role in providing specialist ground anchoring and soil nailing services as part of the retention works for the Waterview Connection Project.

The $1.4b Waterview Connection Project was awarded in August 2011 to the Well-Connected Alliance (WCA). The Alliance comprises NZTA, Fletcher Construction, McConnell Dowell Constructors, Parsons Brinckerhoff, Beca Infrastructure, Tonkin and Taylor, and Japanese construction company Obayashi Corporation. Being the last segment of Auckland's south-western motorway, the Waterview Connection is the final key link for the 48km Western Ring Route which will provide an alternative to the Southern and Northern motorways (SH1), bypassing central transport corridors.

The Southern Approach Trench (SAT), the eastern approach to the tunnel and initial launch area of the 14.5m diameter tunnel boring machine (TBM) is approximately 45m wide and up to 30m deep near the front face. There is a great variance in the geology of the area, which dictated the construction methodology and the design of the trench retaining system.

Design

The design approach adopted to anchoring the reinforced concrete bored pile walls was to install over 400 double corrosion protected multi-strand anchors through concrete waler beams. Anchor capacities ranged from 250T to 320T and lengths varied from 15m to 34m.

The specified working loads for each anchor type and anchor geometry requirements dictated minimum free and bonded lengths. Hole diameters were specified as 200mm and all anchors were inclined. All anchors were designed as permanent Class 1 protection (commonly referred to as double-corrosion protection) and constructed in accordance with BS EN 1537:2000 with a minimum design life of 100 years.

Double corrosion protection requires the full encapsulation of the anchor tendon (strand) strand within cement grout inside a single corrugated plastic duct. The corrugated ducting serves two purposes. First, the duct allows transfer of the capacity of the multi-strand tendon from the inner grout to the outer grout without cracking, and secondly, the duct provides a continuous impermeable barrier to moisture for the lifetime of the anchor. The outer grout solely provides the bond mechanism for the anchor. It does not provide any corrosion protection.

Construction Constraints

Construction of the anchoring works proceeded in tandem with numerous other activities and this required a high level of co-ordination and project management of multi-disciplinary teams. This, coupled with a demanding programme and restricted access associated with working in the trench dictated the use of multiple drilling rigs and crews to ensure target programmes were achieved.

With space being at a premium and anchors weighing in excess of 500kg (meaning manual lifting was not an option), alternative methods for lifting the anchors had to be considered in-lieu of traditional craneage techniques. Grouting Services (GSL) utilised in-house expertise to develop an anchor carousel frame and hydraulically-controlled rotational drive unit to allow the anchors to be safely installed without the use of a crane. Photograph 1 shows the anchor installation equipment.

Photograph 1: Anchor carousels and hydraulically-controlled rotational drive unit

Drilling

Anchor holes for the SAT walls were drilled by conventional rotary wash drilling techniques through the weaker Tauranga Group (TG) into the underlying East Coast Bays Formation (ECBF). Drilling fluids comprised fresh water and all holes were flushed clean using compressed air and water.

The specification required the drill holes to be within 2-degrees of the theoretical centre-alignment over their full length and this was easily achieved. In some locations, it was necessary to install temporary PVC casing to deal with low strength material encountered that was not self supporting.

At the front face (portal entry) strand anchors were installed through the reinforced concrete bored pile wall, with a 7m wide stabilised block immediately behind and anchored into ECBF. An additional challenge was the need to ensure the installed path of the anchors in the upper section did not intersect the path of the TBM as it tunnelled the initial 30m section. Upon completion of the drilling a specialist gyroscopic device was lowered down the hole to take co-ordinate readings at 2m intervals. Readings were taken on the devices during its decent and ascent then averaged and compared to the future path of the TBM. Of the 10-No anchors analysed by this method the maximum deviation was 240mm, at a depth of 20m metres. The allowable tolerance was 700mm.

Overlying the TG and ECBF was a basalt layer of varying competency. To support the gantry crane used to re-build, and eventually dismantle the TBM within the SAT, 32 double corrosion protected multi-strand anchors were installed at a 5° inclination into the basalt layer. The anchor holes were drilled to 15m with a 200mm down-the-hole hammer. Photograph 2 shows the proprietary anchoring rig designed with a swing boom, super high pull back (15tons) and high-torque rotator (up to 4000kgm torque). The double rotation head facilitates continuous double drive drilling with casing and rods.

Anchor Manufacture

The anchors were specified as double corrosion protected and the tendon fabrication was conducted off site in a controlled environment to minimise the risk of damage to the tendon and corrosion protection system. Multi-strand tendons are greased and sheathed over the free length. Critical to the performance of a multi-strand anchor is the requirement of the strand to be fully greased within the lateral sheath. This is not only to allow the tendon to satisfactorily elongate during tensioning, but also to ensure no voids are present within the sheathing which would compromise the integrity of the anchor.

Each individual strand is run through a specialist greasing and sheathing machine that first opens the individual wires of the strands prior to immersion in a grease bath before completely encapsulating the strand in the outer sheathing thus ensuring no voids are present. The individual greased and sheathed strands are configured into the design arrangement complete with plastic centralisers over the bare strand in the bond length to create a basket weave, and, internal grout hose and nose cone prior to insertion into the corrugated ducting.

Photograph 2: Specialist anchor drilling equipment (HD180 anchoring rig)

The completed anchors were coiled onto a carousel ready for delivery to site for installation. The use of the carousel at this stage of the project added a further quality control step in the process and ensured all 431 anchors were the correct length, contained the correct number of strands and were installed in the correct location.

Anchor Installation and Grouting

The use of the anchor carousel system developed specifically for this project eliminated the need for heavy straining and reduced the time taken to install anchors. Anchors were installed to termination depth, and prior to any grouting taking place, all grout lines were checked to ensure they were clear. Grouting took place simultaneously internally and externally via grout lines that extend to the bottom of the anchor to maintain an equal pressure and minimise the risk of collapse to the corrugated duct. Grouting comprised neat cement grout with a maximum water: cement ratio of 0.4. Grout bleed requirements were less than 2%. Reconciliation of grout volumes in conjunction with visual checks on the level of the top of the grout was required to ensure adequate anchorage along the bonded length was maintained. Grout samples were required to be taken and tested to validate the design strength of the grout. The anchor carousel is shown in photograph 1.

Anchor Stressing and Head Protection

The stressing operations included acceptance testing and residual load testing. The testing criteria for the anchors saw every anchor receive an acceptance test equivalent to 150% of the working load over 2-loading cycles. An initial load equal to 10% of the working load was applied to bed in the anchor and testing system, and allow the jack to stabilise. The load was held steady at each increment (and decrement) and maintained to ensure a variation of no more than 2kN was observed from the specified load.

Displacement at each load increment was recorded using a dial gauge atop a remote tripod and at the end of the second load cycle. The anchors were locked off at the prescribed working load. Five percent of the anchors installed were subjected to a residual load test to effectively prove that the anchor system met the specified criteria with respect to creep under constant load. This involved monitoring the load on the anchor and the deflections of the anchor head over a maximum 150 minute period. The maximum test load placed on a single anchor was 2600kN (260 Tonne) on the 15m long anchors supporting the TBM gantry crane. Long-term monitoring was incorporated into a number of anchors with the inclusion of strain gauge load

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Photograph 3: Stressing set up

cells. Photograph 3 shows typical stressing set up.

To meet the requirements for a 100year design life, all bearing plates were galvanised and included a trumpet that lapped with the corrugated ducting. The anchor heads were protected by a fibre reinforced polymer cap fixed to the bearing plate and filled with grout.

Soil Nails and Additional Works

The tendered programme was revised significantly as the project advanced and regular meetings were held between GSL and WCA to look for and realise risks and opportunities. Multiple drilling rigs and installation crews were required on the project as the anchoring works proceeded and the scope increased to include soil nails and bored drainage relief holes, not included in the initial competitive tender. High level co-ordination was required to work in tandem with the numerous additional activities being undertaken by many contractors within the confined space. With an exemplary safety and quality record GSL were extremely pleased with the efforts made by their team and greatly appreciated the assistance provided by many members of the WCA team.

Prepared by: David Sharp
Engineer
Grouting Services

The Use of CFA Piling Technology for Ground Improvement at Waterview

BRIAN PERRY CIVIL is delivering significant specialist foundation and ground improvement works as part of the Waterview Connection project's preparations for the start of tunnel boring in October 2013.

The $2.4b NZ Transport Agency project is being constructed by the Well-Connected Alliance and is the largest and most expensive roading project ever undertaken in New Zealand. It will link State Highways 20 and 16 to complete Auckland's Western Ring Route. This will be a continuous 48km motorway route that bypasses central Auckland to the west, therefore providing a reliable alternative to State Highway 1 and the Auckland Harbour Bridge for travel through Auckland.

The new link includes construction of twin, 2.4km, 13.3m diameter bored tunnels which will each carry three lanes of traffic below Avondale and Waterview. These will be bored over two years by the tenth largest tunnel boring machine (TBM) in the world, which arrives in July and begins work in late October 2013.

At the portals for the twin tunnels ground conditions comprise Tauranga Group (TG) alluvial deposits overlying East Coast Bays Formation (ECBF). The latter has a residual soil zone and becomes less weathered with depth.

While the tunnel alignment is below a basalt flow at the southern end, the tunnel face at the portal is in both soft TG and weak ECBF zones. A stabilised block is required to facilitate the portal construction and permit safe access and egress for the TBM. This will extend through the TG and the completely weathered ECBF and will be sized to ensure a suitable seal can be by the achieved by the TBM.

Photograph 1: CFA Piling at the Southern Portal for Ground Treatment

A number of options were considered for stabilising the TBM's entry and exit at the southern tunnel portal including in-situ soil mixing, jet grouting and full replacement. The use of continuous flight auger (CFA) piles to form the majority of the treatment block was selected as the most cost effective solution. It would also minimise the duration of the works and offer the benefits of a full replacement methodology by providing consistent and reliable strength and diameter compared to soil mixing and jet grouting in variable soils.

The CFA methodology has enabled the construction of a block with strengths in the required 5 - 10MP range using 750mm and 900mm diameter piles. The strength targets encompassed the normal minimum requirements but also a maximum to ensure TBM operations are not unduly affected. Furthermore, the portal face piles were constructed with the addition of fibre reinforcement to provide the necessary structural capacity.

The CFA pile methodology enabled the construction of up to 25 of the required 342 900mm diameter piles or placement of 200cu.m of concrete in a shift; which has seen the successful completion of the works ahead of programme.

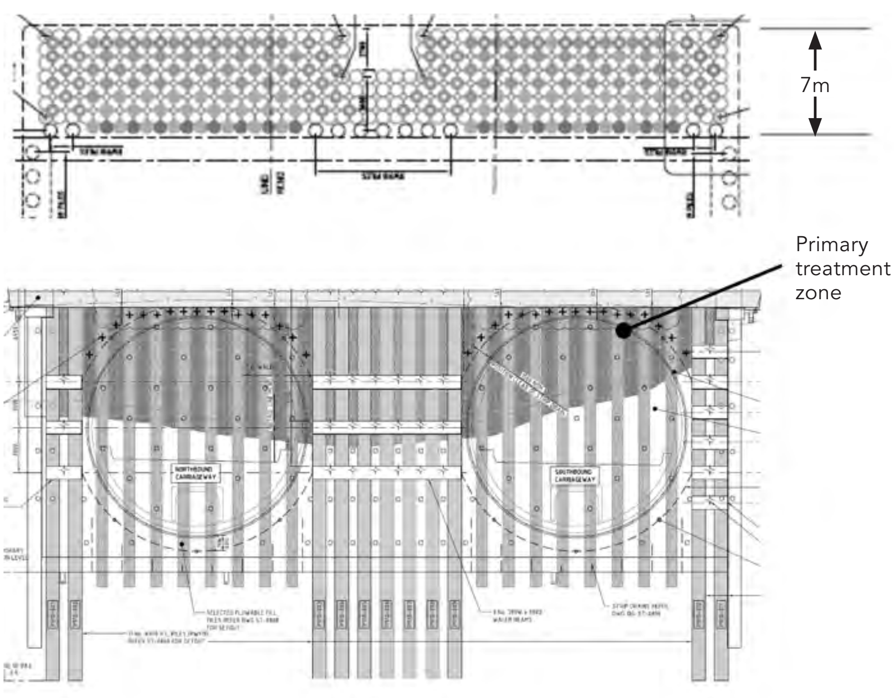


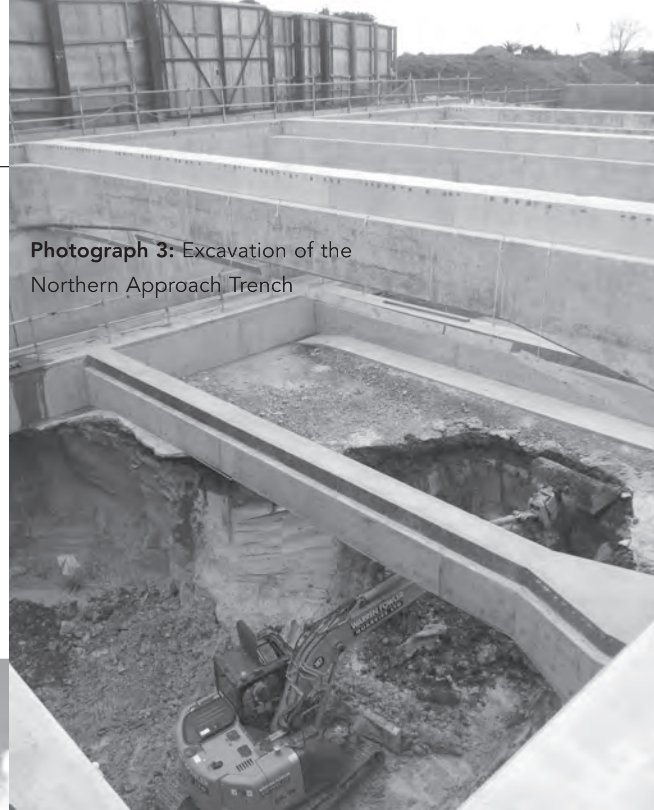
Figure 1: Tunnel Portal Ground Treatment

Prepared by: Nick Wharmby
Brian Perry Civil Ltd

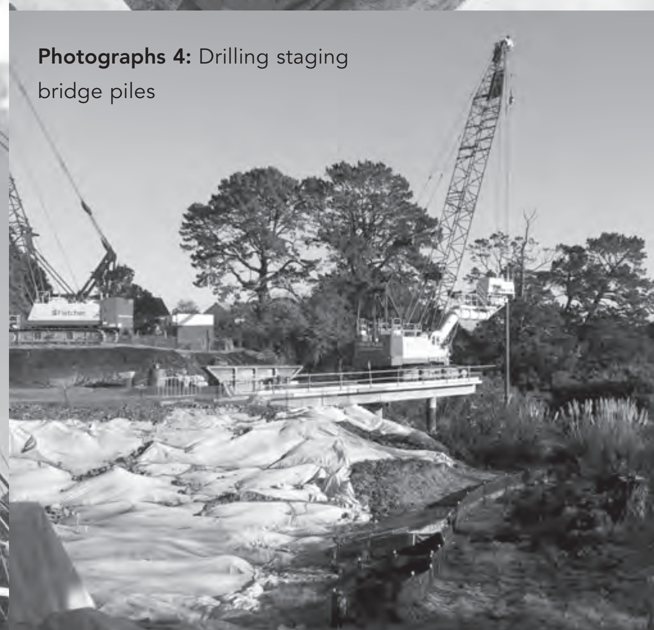
Waterview Connection Northern Area – Photograph Essay

The launch of Alice the tunnel boring machine has attracted much attention to the southern end of the Waterview Connection Project. However, significant works at the northern end of the project on the Northern Approach Trench and the Great North Road Interchange are well underway. This area of the site needs to be ready in to receive the TBM in twelve months and send it back on its return journey to the South.

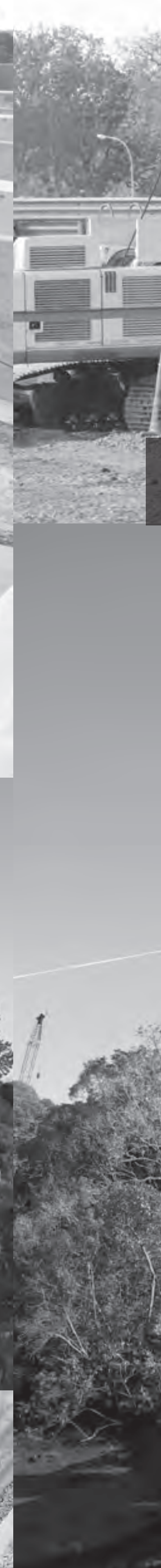
Photograph 2: A reinforcing cage being placed in the diaphragm retaining wall for the Northern Approach Trench

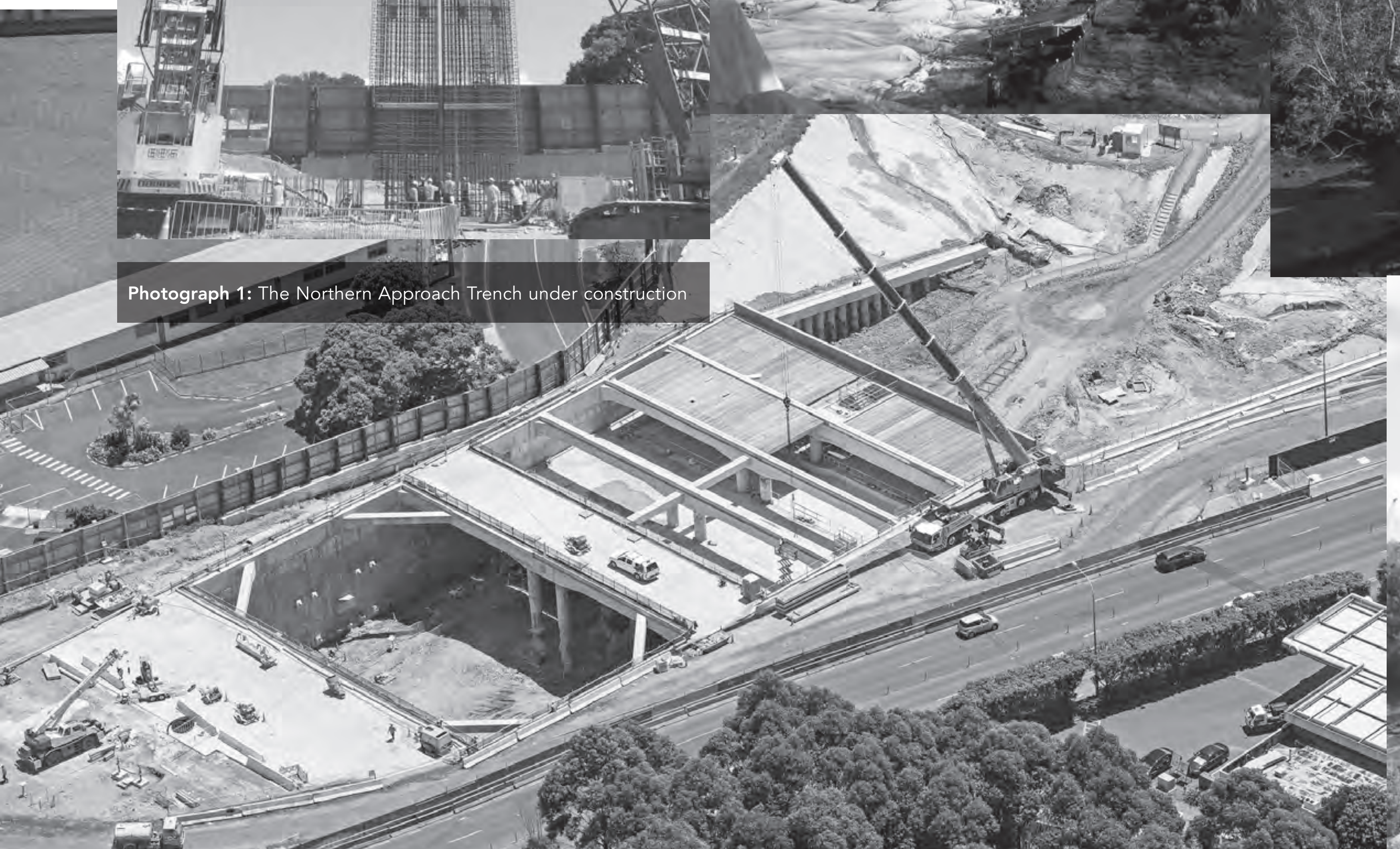


Photograph 3: Excavation of the Northern Approach Trench



Photographs 4: Drilling staging bridge piles

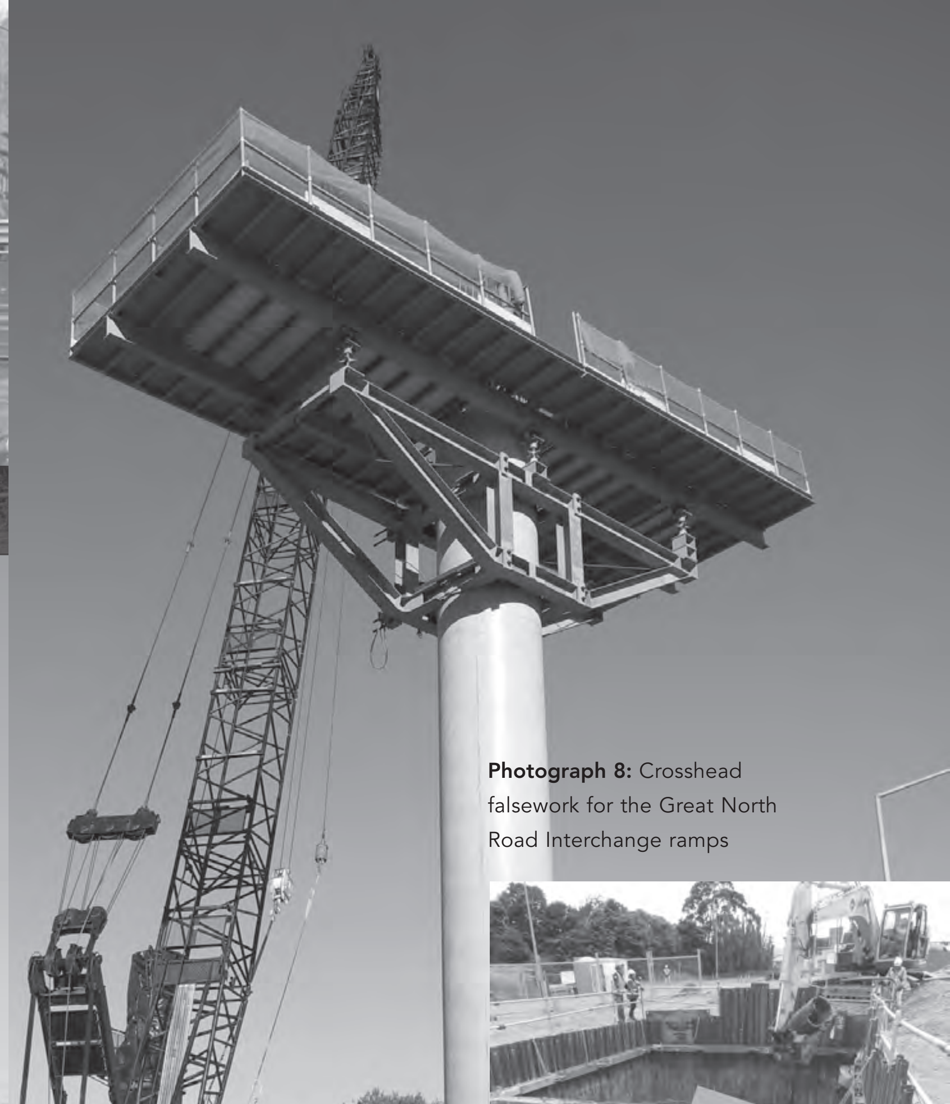




Photograph 1: The Northern Approach Trench under construction

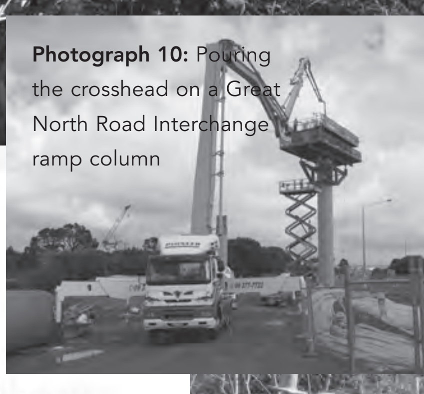
Photographs 5: Drilling foundation piles for construction of the Great North Road Interchange ramps

Photograph 6: Column falsework for the Great North Road Interchange ramps



Photograph 8: Crosshead falsework for the Great North Road Interchange ramps

Photograph 9: Excavation for service relocation



Photograph 10: Pouring the crosshead on a Great North Road Interchange ramp column

Photograph 11: Steel fixing for a ramp abutment beam

Left Photograph 7: A 'forest' of columns for the Great North Road Interchange ramps

Prepared by: Hamish Maclean, NZ Geomechanics News Co-Editor

Photographs 1, 5, 7, 9, 11 courtesy of Well-Connected Alliance. The remaining supplied by Hamish Maclean

TECHNICAL ARTICLES

Empirical Design and Numerical Modelling of Timber Piles Employed as a Liquefaction Countermeasure

– Michelle Willis University of Auckland / Earthtech Consulting

1. Introduction

The New Zealand Geotechnical Engineering Industry is well aware of the effects of soil liquefaction on the built environment. The extensive liquefaction observed in the aftermath of the Canterbury Earthquake sequence, and other earthquakes worldwide, has highlighted the need for liquefaction countermeasures. One available method of liquefaction mitigation involves the installation of a grid of vertical piles. The use of piles, micropiles, or other high modulus columns as a liquefaction countermeasure is not uncommon. Piles may consist of steel, timber, reinforced concrete, grouted or mixed columns (Fujii et al., 1992; Hayden and Baez, 1994).

The design of piles as a liquefaction countermeasure can be carried out through the use of empirical stress- or strain-based methods. These methods are typically developed from the equations presented by Baez and Martin (1994), and are based on a number of assumptions. Little information is available in literature regarding the justification of these assumptions. This raises a question for the Geotechnical Engineer/Engineering Geologist: are these assumptions valid for a particular site, and if not, what effect will they have on the design solution?

In response to this question, the author carried out three-dimensional finite-element (FE) numerical modelling to assess the validity of one of the empirical design assumptions. The research was conducted as part of the Master of Engineering degree at the University of Auckland in 2011 and 2012, and only considered vertical timber piles. The piles do not carry any load, such as from a superstructure, and function solely as a liquefaction countermeasure.

The purpose of this article is to present some of the more imperative findings to the New Zealand Geotechnical Engineering Industry, and to inform those who may consider installing timber piles as a liquefaction countermeasure in the future. Overall, the findings of this research indicated that one of the empirical design assumptions may be inappropriate. As a result, empirical design methods may significantly overestimate the pile effectiveness as a liquefaction countermeasure.

2. Background

Liquefaction is caused by the generation of excess pore water pressure during the rapid loading of cohesionless soils. The increasing pore water pressure results in decreased effective stress, and the loss of shear strength. The ratio of excess pore water pressure to the initial vertical effective stress (r_u) is generally employed to describe the degree of liquefaction, with $r_u = 1$ indicating full liquefaction, or a state of zero effective stress (Kramer, 1996).

The installation of a grid of piles is intended to control excess pore water pressure generation by reducing the soil cyclic shear strain generated during earthquake loading.

Empirical design methods of piles as a liquefaction countermeasure may be stress- or strain-based. These require the assumption that the soil and pile deform with equal strain during loading, essentially that the soil and pile are perfectly bonded together and vertical slip or lateral gapping does not occur at the soil-pile interface. The increased stiffness of the composite system reduces the cyclic shear stresses or strains (depending on whether the stress- or strain-based design method is employed) in the soil, thereby reducing excess pore water pressure generation. This 'equal strain' assumption is examined in this article.

Other assumptions employed by empirical design methods are that the structural capacity of the pile is not exceeded, and that increases in soil density due to pile driving may be conservatively ignored. The empirical design methods only apply to a square grid of round vertical piles.

The following sections of this article apply the strain-based method to timber piles. This methodology is summarised by the following process:

1. A maximum allowable r_u value is selected. A target value of 0.5 is usually adopted based on recommendations presented by Iai and Koizumi (1986), essentially allowing a factor of safety of two.
2. The target r_u value is converted into a target cyclic shear strain (γ) incorporating the effect of soil fines content on excess pore water pressure generation (Hazirbaba and Rathje, 2009). The analyses presented in this article assume clean sand and employ the conversion presented on Figure 1.
3. The soil profile is divided into equal depth increments and the soil small strain shear modulus is calculated at each depth based on available information. The soil shear modulus at the target strain γ is then calculated based on the soil total stress (Ishibashi and Zhang, 1993).
4. An initial pile center-to-center spacing is assumed and the composite system shear modulus is calculated utilising the following equation given by Baez and Martin (1994):

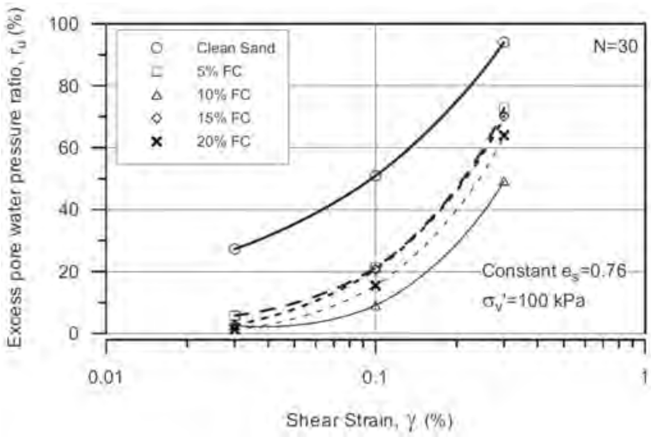


Figure 1: Conversion between r_u and γ (Hazirbaba and Rathje, 2009).

$$G_{comp} = G_{soil}\,(1 - A_{rr}) + G_{pile}\,A_{rr} \qquad (1)$$

where:

G_{comp} = composite shear modulus (kPa);

G_{soil} = soil shear modulus at the target γ γ (kPa);

A_{rr} = the area replacement ratio (ratio of the pile area to the soil tributary area);

G_{pile} = pile shear modulus, taken as 580,000kPa for normal density Radiata pine (NZS3603, 1993)

5. The resulting composite cyclic shear strain is calculated employing Equation 2.

$$\gamma_{comp} = 0.65 \; \frac{a_{max}}{g} \; \frac{\sigma_0}{G_{comp}} \; r_d \qquad (2)$$

where:

γ_{comp} = composite cyclic shear strain (decimal);

a_{max} = peak ground acceleration (g);

g = gravitational acceleration (g);

σ_0 = initial soil vertical total stress (kPa);

r_d = stress reduction factor.

6. Steps 4 and 5 are iterated trialling different pile spacings until the calculated γ_{comp} is equal to, or slightly less than, the target cyclic shear stain. The final γ_{comp} profile is converted back into r_u through Figure 1.

Use of the equal strain assumption implies that the pile deforms in pure shear with the soil. If the pile deformed with some degree of flexure then soil-pile interface displacements would also be present, as interface displacements and pile flexural deformation are related (Olgun and Martin, 2008).

Some researchers state that the equal strain assumption is valid as there is no inertial loading, such as from a superstructure, acting on the piles to cause them to displace in a direction other than the ground motion (Baez and Martin, 1994). However, other studies found that the reduction in soil cyclic shear stress provided by the piles decreased as the proportion of pile flexural deformation increased, and that the piles deformed mostly in flexure (Goughnour and Pestana, 1998; Martin and Olgun, 2007; Olgun and Martin, 2008). These studies employed three-dimensional linear-elastic FE analyses and did not consider liquefaction.

3. Numerical Modelling

Based on this background information, the equal strain assumption appears to be the "weak link" in the empirical design method. This work sets out to examine the validity of the equal strain assumption, and whether it introduces inaccuracy to the empirical solution.

These objectives were examined through three-dimensional FE modelling within the Open System for Earthquake Engineering Simulation (OpenSees, Ver. 2.2.2.f) (Mazzoni et al., 2011). OpenSees is a fully nonlinear object orientated code which provides a platform for the modelling of coupled geotechnical and structural systems under static and dynamic loading.

The numerical analyses were limited to a square grid of vertical piles in level ground scenarios, as the empirical design methods only apply to these conditions. The analyses presented in this article also ignored densification of the in-situ soils due to the pile driving process. Normal density Radiata pine timber was considered as a pile material. Half the model was analysed along the line of symmetry. An illustration of the OpenSees model geometry is presented on Figure 2.

For the sake of the continuity of this article and anyone who may have queries about the numerical modelling, the model properties are briefly described in the following paragraphs. Further details are presented in the thesis by the author (Willis, 2012).

3.1 Numerical Model Properties

OpenSees employs a phase transformation soil constitutive model (Yang et al., 2003) based on the theory of multi-surface plasticity for frictional cohesionless soils (Prevost, 1985) within the bounds of the elastoplastic Mohr-Coulomb yield model (Vermeer and de Borst, 1984). The phase transformation surface defines changes in the soil contractive or dilative behaviour, with the volume of contraction given by a hardening rule (Mroz, 1967; Parra, 1996) and the volume of dilation given by a non-associative flow rule (Prevost, 1985).

The base of the FE mesh is controlled by an energy absorbent boundary condition termed the Lysmer-

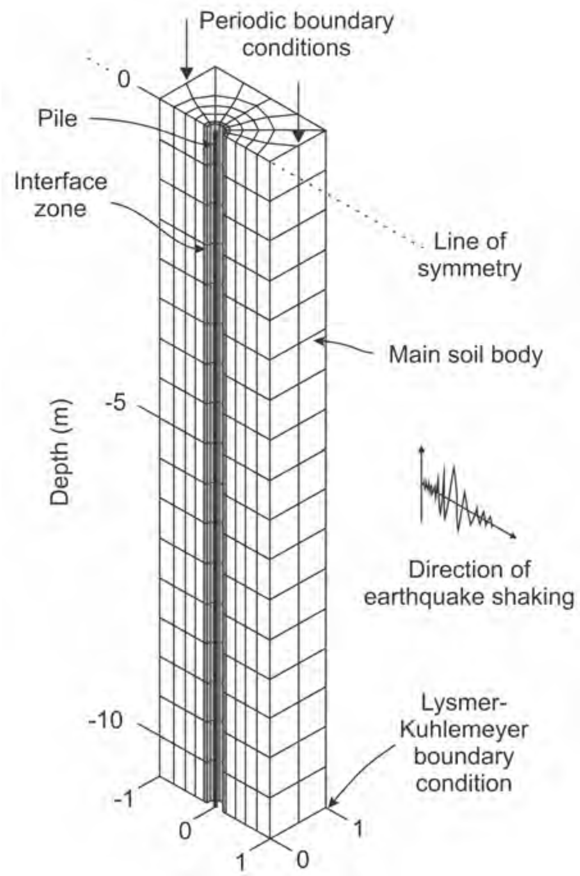


Figure 2: Example OpenSees numerical model geometry and FE mesh.

Kuhlemeyer boundary (Joyner and Chen, 1975; Lysmer and Kuhlemeyer, 1969). The lateral boundaries perpendicular to the line of symmetry are controlled by the 'periodic' boundary condition, which simulates a square grid of piles extending infinitely laterally (Elgamal et al., 2010). The lateral boundaries parallel to the line of symmetry are restrained against out of plane displacements only.

Seismic excitation is applied parallel to the line of symmetry employing an algorithm which allows for acceleration amplification or attenuation through the soil profile.

Both Rayleigh damping and numerical damping are applied to the OpenSees models. The Rayleigh damping mass-proportional and stiffness-proportional coefficients are set to 0.1244 and 0.000788 respectively. These values were calculated employing the method presented by Chopra (2007) assuming that the model predominant frequencies fall within a range of 0.2Hz to 20Hz with a damping constant of 5%. Numerical damping is added to the OpenSees Newmark integrator with the degree of damping set to 0.65, representing minor numerical damping (McKenna, 2010).

The pile is considered to be linear elastic, and is perfectly bonded to the soil by 'rigid-links'. Each rigid-link is specified a length equal to the pile radius. The rigid-links rigidly bond the pile circumference to the surrounding soil allowing pile rotation to take place, but do not allow soil-pile interface nonlinearity in the form of vertical slip or lateral gapping.

3.2 Accounting for Soil-pile Interface Displacements

The possibility of soil-pile interface nonlinearity in the form of vertical slip and lateral gapping is incorporated into the OpenSees model through the use of interface elements. These are essentially weak soil elements adjacent to the pile shaft. The interface elements are indicated on Figure 2, and enlarged on Figure 3.

The properties of the soil-pile interface were determined through the review of information available in literature, sensitivity analyses and the model verification process. An interface friction angle 67% of the soil friction angle, an interface shear modulus 45% of the soil shear modulus, and an interface thickness 16.7% of the pile diameter were selected for these models.

However, analyses showed that the soil ru response recorded greater than 1m from the pile was relatively in-sensitive to the interface properties, due to the distance from the pile and effects of the soil-pile interface. So for all practical purposes, the properties of the interface elements are irrelevant, however their existence allows soil-pile interface nonlinearity to take place (recorded as deformation across the interface elements).

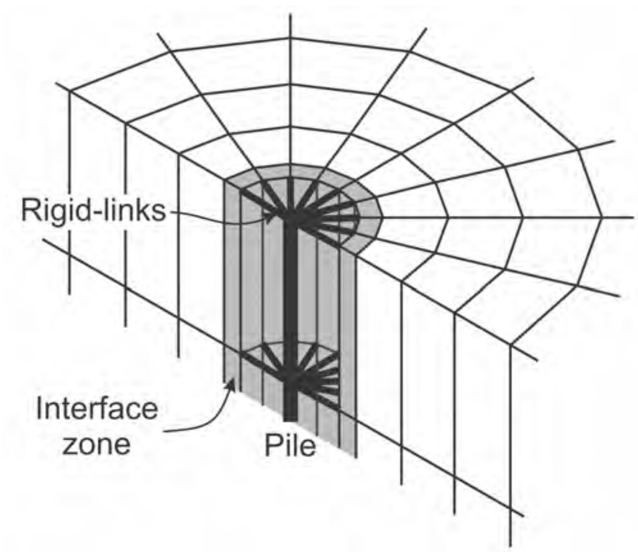


Figure 3: Enlargement of the soil-pile interface

3.3 Soil Input Parameters

User-defined inputs are presented in Table 1, and are correlated to the relative density and void ratio of Nevada Sand (Yang and Elgamal, 2003). The OpenSees default hyperbolic shear modulus reduction curve is assumed.

It is acknowledged that use of Nevada Sand correlations represents a limitation of these models when applied to New Zealand soils. However the soil constitutive model inputs have only been correlated to the properties of Nevada Sand to date.

3.4 Numerical Model Verification

Essential to this process is the verification of the OpenSees model response and assessment of the model limitations. The verification process was carried out by replicating static loading field tests, shaking table experiments, and dynamic centrifuge experiments to assess if the OpenSees predicted responses matched the experimentally recorded data (Willis, 2012).

Results indicated that OpenSees consistently underestimated soil vertical displacement, lateral displacement and shear strain. Underestimation of soil vertical displacement was expected, as this is a known limitation of the program (Yang et al., 2003).

Pile displacements and bending moments were accurate in some verification models, but underestimated in others. These results may be partly due to the underestimation of soil displacements.

Soil acceleration was accurately predicted prior to liquefaction, but underestimated during liquefaction due to overdamping.

Importantly, OpenSees was able to accurately predict the soil r_u response at all the depths considered. Therefore the following analyses rely on r_u when assessing the soil liquefaction response.

These assessed model limitations are dependent on the methodology employed to determine the user-defined input parameters (soil and pile properties, damping parameters and boundary conditions) and might be mitigated by modifying these parameters. Regardless, some limitations are to be expected, as it is not possible for constitutive models to capture the entire soil response, and may reproduce only a few aspects accurately (Waterman, 2011).

4. Comparison Between the Empirical and Numerical Liquefaction Responses

This section compares the r_u responses estimated by the empirical method described in Section 2, against the numerical model at different A_{rr} values.

The pile spacing and diameter in these models varies to achieve the different A_{rr} values. Pile diameters range from 150mm to 450mm, and pile center to center spacings range from 1m to 4m.

A 0.25g 2Hz sinusoidal input excitation is applied to the numerical models, which is equivalent to a random motion peak ground acceleration of 0.385g in the empirical method (Seed et al., 1975).

4.1 Loose Sand Models

The analyses in this example consider 10.3m long piles in loose sand (0m to 10m depth), founded 300mm into dense sand (10m to 10.3m depth). Relative densities are 25% and 75% for the loose and dense sands respectively. The input soil parameters are presented in Table 1. The groundwater table is located at ground level.

Figure 4 presents the average numerical r_u values recorded at the maximum distance between piles. These values are recorded at the depth of the most severe liquefaction response for each model. The maximum r_u values are not presented as these values only occur for brief instants in time and would be inconsequential to the field response of a site.

For this case, the empirical method shows r_u to decrease as the pile A_{rr} increases. An r_u value of 0.77 is predicted for the free-field response without ground improvement (A_{rr} = 0), which decreases to r_u = 0.36 at large Arr values. Conversely, the numerical model predicts r_u = 0.94 for the free-field response, with essentially no improvement as A_{rr} increases.

These numerical results suggest that the pile has not restrained r_u to the degree predicted by empirical design.

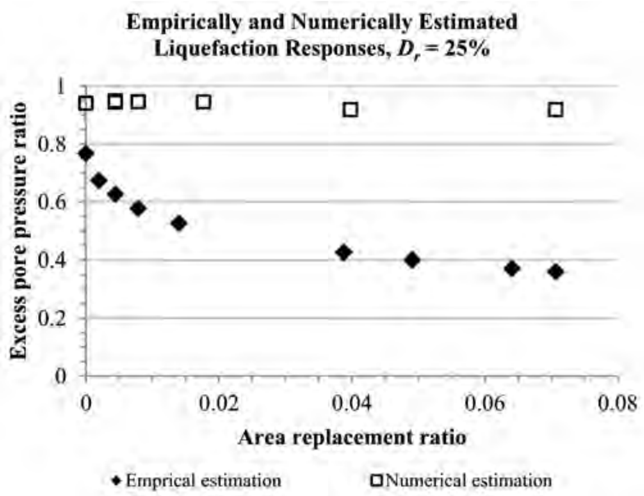


Figure 4: Comparison between numerical and empirical excess pore water pressures, relative density = 25%

4.2 Medium-dense Sand Models

A reader may comment that the ground conditions represented by the above models are perhaps too severe. In answer to this question, a similar series of models were run with medium-dense sand (0m to 10m) overlying dense sand (10m to 10.3m). The medium-dense sand had a relative density of 40%, with the properties presented in Table 1. The groundwater table was lowered to 1.5m depth.

Figure 5 presents the average r_u values recorded at the maximum distance between piles for the medium-dense sand models. Again, these values are recorded at the depth of the most severe liquefaction response for each model.

Similar to the previous scenario, the empirical method shows ru to decrease as the pile A_{rr} increases. An r_u value of 0.64 is predicted for the free-field response, which decreases to r_u = 0.37 at large A_{rr} values. The numerical model predicts r_u = 0.66 for the free-field response, with essentially no change as A_{rr} increases.

Again, the empirical method significantly overestimates the effectiveness of the grid of timber piles as a liquefaction countermeasure when compared to the numerical results.

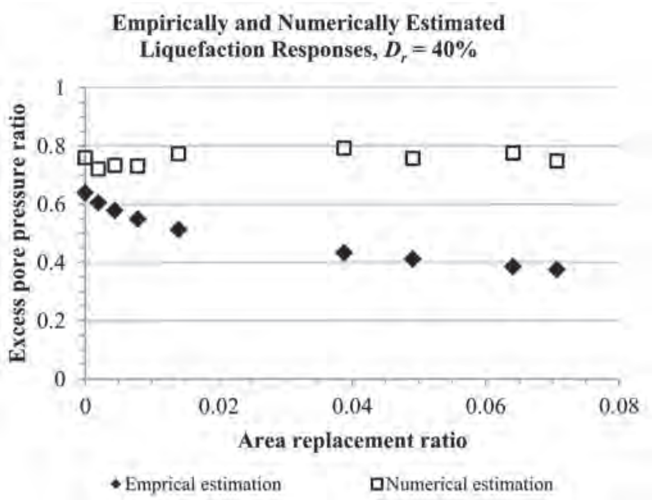


Figure 5: Comparison between numerical and empirical excess pore water pressures, relative density = 40%

4.3 Discussion of Results

A reader may ask if the response differed for other scenarios. A suite of sensitivity models was considered as part of the research, although only two scenarios are presented in this article. The sensitivity models examined sand relative densities ranging from 25% to 75%, and sinusoidal input acceleration time histories ranging from 0.2g 1Hz to 0.3g 3Hz.

For all the models, the empirical method showed ru to decrease as A_{rr} increased, whereas the numerical model indicated little to no change as A_{rr} increased. These numerical results suggest that the grid of piles is relatively ineffective as a liquefaction countermeasure and has a negligible effect on excess pore water pressure generation for the model properties considered.

Additionally, all the numerical models recorded some soil-pile interface displacements. In some cases, up to 30mm of vertical slip and 40mm of lateral gapping occurred. While the author does not intend to place too much weight on these displacements (the magnitudes of which could not be verified), they suggest that pile flexural deformation is occurring. The presence of flexural deformation reduces the pile effectiveness as a liquefaction countermeasure, and suggests that the equal strain assumption may be inappropriate.

Another important point to note is that both the empirical and numerical r_u predictions are inherently different, as would be expected for empirical and FE solutions. This is evident as the methods provide different r_u predictions for the free-field response when the A_{rr} is zero. OpenSees does not provide options to control the pile to deform in pure shear. Instead, the pile is allowed to deform with a natural combination of shear and flexure. As a result, the differences between the empirical and numerical r_u

Soil property	Loose sand (0-10m depth)	Medium-dense sand (0-10m depth)	Dense sand (10-10.3m depth)
Relative density	25%	40%	75%
Density	1.7t/m^3	1.83t/m^3	2.0t/m^3
Small strain shear modulus	55 000kPa	66 700kPa	100 000kPa
Small strain bulk modulus	150 000kPa	178 000kPa	300 000kPa
Friction angle	29°	31°	37°
Octahedral shear strain at the maximum shear stress	0.1	0.1	0.1
Reference effective stress	80kPa	80kPa	80kPa
Pressure dependence coefficient (moduli distribution with depth)	0.5	0.5	0.5
Phase transformation friction angle	27°	25°	20°
Contraction parameter	0.21	0.11	0.05
Dilation parameter 1	0	0.2	0.6
Dilation parameter 2	0	1	3
Liquefaction parameter 1	10	10	5
Liquefaction parameter 2	0.02	0.013	0.003
Liquefaction parameter 3	1	1	1
Permeability	6.96×10^{-5}m/s	6.62×10^{-5}m/s	4.08×10^{-5}m/s
Soil-pile interface small strain shear modulus	24 750kPa	30 000kPa	45 000kPa
Soil-pile interface small strain bulk modulus	67 500kPa	80 000kPa	135 000kPa
Soil-pile interface friction angle	19°	21°	25°

Table 1: OpenSees soil input parameters and soil properties employed in the numerical analyses.

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estimations are partly due to the equal strain assumption, and partly due to other simplifications and correlations employed by the respective calculation methods.

Also, as previously mentioned, the OpenSees program has a number of limitations. In view of these limitations, the author does not hold that the OpenSees predicted liquefaction response is entirely accurate, but should be considered as more of an indicative response as opposed to a quantitative response.

5. Pile Strain Energy

In view of the numerical model limitations, the author sought to verify the above conclusions through independent empirical calculations. This was carried out by calculating the pile shear and flexural strain energy (the energy required for the pile to deform in pure shear or pure flexure).

If the pile is divided into multiple small segments, then the strain energy required for each segment to deform as a shear beam (pure shear deformation) or as a bending beam (pure flexural deformation) can be calculated from Equations 3 and 4 respectively (Megson, 2005).

$$U_s = \frac{1}{2} G_{pile} \gamma^2 A_{pile} \Delta z \qquad (3)$$

where:

U_s = element shear beam strain energy (kNm);
γ = element shear strain (decimal);
A_{pile} = pile cross-sectional area (m^2);
Δz = element height (m).

$$U_b = \frac{1}{2} \frac{M^2}{E_{pile} I_{pile}} \Delta z \qquad (4)$$

where:

U_b = element bending beam strain energy (kNm);
M = element bending moment (kNm);
E_{pile} = pile Young's modulus, 8,700,000kPa for normal density Radiata pine (NZS3603, 1993);
I_{pile} = pile second moment of inertia (m^4).

These calculations were carried out for the loose sand model described in the previous section. The timber pile in these analyses is taken to be 300mm in diameter, with a 2m center-to-center spacing (A_{rr} = 0.0177), and 10.3m long.

The pile element shear strains required for Equation 3 are calculated from the pile lateral displacement profile at 0.26sec, when $r_u \approx 0.5$. The pile element bending moments required for Equation 4 are also calculated from the pile lateral displacement profile at 0.26sec. Equations to calculate pile shear strain and bending moment from lateral displacement are presented in Megson (2005). The distribution and magnitude of pile strain energy would vary with time during earthquake excitation, and would also be dependent on the peak ground acceleration, pile properties and soil geology.

The results of the model verification against experimental data indicated that the OpenSees predicted soil lateral displacements and pile bending moments could be unreliable. To overcome this, both the shear and flexural strain energies were calculated from the same pile lateral displacement profile. In this way, any errors in the OpenSees pile lateral displacement prediction are carried through both calculations.

The pile strain energies required for shear and flexural deformation at 0.26sec are presented on Figure 6. These are integrated over the length of the pile, and indicate that the total energy required for flexural deformation in this case is only 2.6% of the energy required for shear deformation. Naturally, the pile would deform in the mode which requires the least energy.

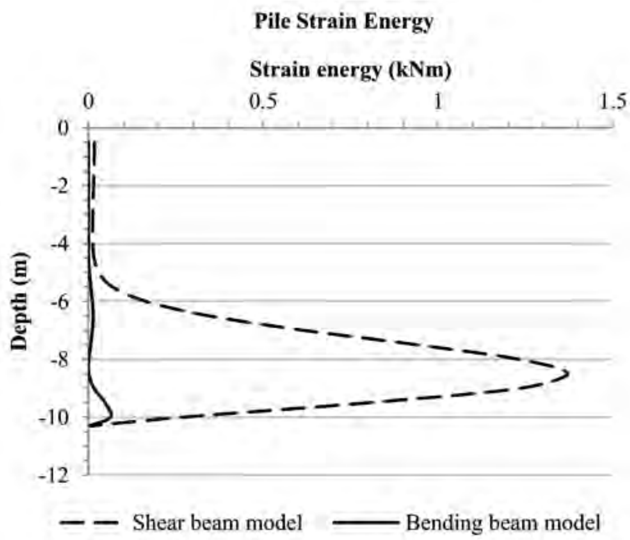

Figure 6: Comparison between the strain energy required for shear and flexural deformation.

These results are another indication that the equal strain assumption (which requires the pile to deform in pure shear) employed by empirical design methods may be inappropriate.

6. Relation to Successful Case Histories of Piles Mitigating Liquefaction

The observations presented thus far indicate that a grid of timber piles is relatively ineffective as a liquefaction countermeasure. Yet there are case histories and experimental data that indicates high modulus columns to have successfully mitigated liquefaction (Charters, 2005; Martin et al., 2004; Martin and Olgun, 2007; McManus et al., 2005). How does such data relate to these results? This question is addressed by the following paragraphs, each representing a possible application (and a topic for further

research) where high modulus piles could be effective as a liquefaction countermeasure.

Shaking table experiments conducted at the University of Canterbury have shown that two piles installed at opposing inclined angles to form an 'X' in cross-section were effective in reducing the cyclic shear strain of dry sand by up to 50% (Charters, 2005; McManus et al., 2004; McManus et al., 2005). It is possible that the pile inclination in these experiments provided additional benefit as the strong axis of the pile was more aligned with the direction of soil shear displacement. However, such an approach would have limited success in field applications where the direction of future earthquake shaking is unknown.

In another case, 600mm diameter jet grouted columns succeeded in reducing, if not eliminating, the surface effects of liquefaction at the Carrefour shopping complex site during the 17 August 1999 Kocaeli Earthquake. Martin and Olgun (2007) and Martin et al. (2004) interpret the data collected from this impromptu field scale experiment, and comment that the jet grouted column installation process would not have increased the in-situ soil density. It is possible that permeation of the grout into the soil immediately surrounding each column, coupled with the large pile diameter, effectively reduced the potential for soil-pile interface displacements, thereby increasing the proportion of pile shear deformation, and increasing the pile effectiveness as a liquefaction countermeasure.

Numerical modelling employing the programs OpenSees and OpenSeesPL (a sister program to OpenSees) have also shown vertical micropiles to be effective in reducing the lateral spreading of medium-dense sand (Elgamal et al., 2009; Lu et al., 2009). These analyses did not incorporate a soil-pile interface layer to account for interface nonlinearity, and still showed ru > 0.85 throughout the soil profile. However, the results could indicate that piles may be effective in providing a barrier to resist lateral spreading displacements, although they may be insufficient to prevent liquefaction.

The numerical analyses presented in this article also ignore any in-situ soil compaction occurring due to the pile driving process. As part of the research, the author also investigated the effect of including a compacted zone around the pile. Results indicated that including pile driving compaction decreased ru values through the soil profile, in some cases causing the numerical and empirical ru predictions to match (Willis, 2012). In field applications, the degree and extent of pile driving compaction can be difficult to predict in advance, but could be confirmed by field testing following pile driving.

The possibility also exists to use alternative pile materials such as steel, reinforced concrete or in-situ mixed columns instead of timber. Additionally, there may be scenarios where only a thin liquefiable layer is present in the soil stratigraphy, allowing the soil layers above and below to constrain the pile against rotation and flexure. These are all potential situations where piles may be more effective a liquefaction countermeasure than the cases presented here.

7. Conclusions

A summary of the important conclusions presented in this article are as follows:

- The empirical design of high modulus piles as a liquefaction countermeasure requires the equal strain assumption. This implies that the pile deforms in pure shear with the soil, and that soil-pile interface nonlinearity in the form of slip or gapping does not occur.
- A square grid of vertical timber piles can be modelled in the three-dimensional FE program OpenSees. Verification of the OpenSees program showed it to have numerous limitations, although the excess pore water pressure response was shown to be reliable.
- The empirical design method indicates the degree of liquefaction to decrease as the pile area replacement ratio increases. In comparison, the numerical model indicates the piles to have little to no effect on the liquefaction response.
- All of the numerical models record some soil-pile interface displacements, suggesting that the pile is not deforming in pure shear (as required by the equal strain assumption) but in a combination of shear and flexure.
- Pile strain energy calculations indicate that pile flexural deformation is the preferred mode of pile deformation, as it requires significantly less energy than shear deformation.
- Laboratory, numerical, and field studies have indicated that the effectiveness of piles as a liquefaction countermeasure may be improved by installing piles at opposing inclined angles, or reducing the potential for soil-pile interface displacements. Additionally, the piles may still be effective as a barrier to resist lateral spreading or as a means of increasing the in-situ soil density.

In answer to the original question, it appears that the equal strain assumption required by empirical design methods may be inappropriate. As a result, empirical design methods may significantly overestimate the pile effectiveness as a liquefaction countermeasure.

Further details relating to any part of this article are presented in a thesis by the author (Willis, 2012).

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Vibrocone CPTu testing as a tool for enhanced definition of liquefaction-prone soil layers

– Vicki Moon[1], Tobias Moerz[2], Ehsan Jorat[2], Stefan Kreiter[2]

Introduction

Intercoast is a joint research initiative between Bremen University in Germany and the University of Waikato. As part of this programme, a high resolution, offshore CPT tool (GOST) was brought to New Zealand for a field campaign in the Tauranga area in February 2012. The sea floor CPTu tool GOST has been invented and developed at Bremen University (MARUM – Center for Marine Environmental Sciences) and is commercially operated and available via Geo-Engineering.org GmbH. GOST (Figure 1) incorporates a small (5 cm^2) cone, and has the facility to undertake vibratory measurements as well as typical static CPT traces. In a vibratory measurement the tip vibrates vertically at a frequency of approximately 15 Hz and an amplitude of a few millimetres around the average push velocity, giving the ability to directly assess the impact of oscillatory motion on the sediment resistance and pore water pressure. A typical test sequence will include two CPT traces recorded less than 1 m apart; the first being a standard static test, and the second a vibratory test. Comparison of the results from the two indicates changes imposed on the materials by the dynamic activation of the grain fabric.

During our field campaign in February 2012 a large number of CPT soundings were undertaken within the sea floor sediments of the Stella Passage, the main shipping channel for the Port of Tauranga, as well as several on-land traces being undertaken near recently active coastal landslides in the area. Our research is concentrating on the pore pressure response of the sensitive clay materials that comprise a significant component of the local geology. However, in several of the traces it is apparent that

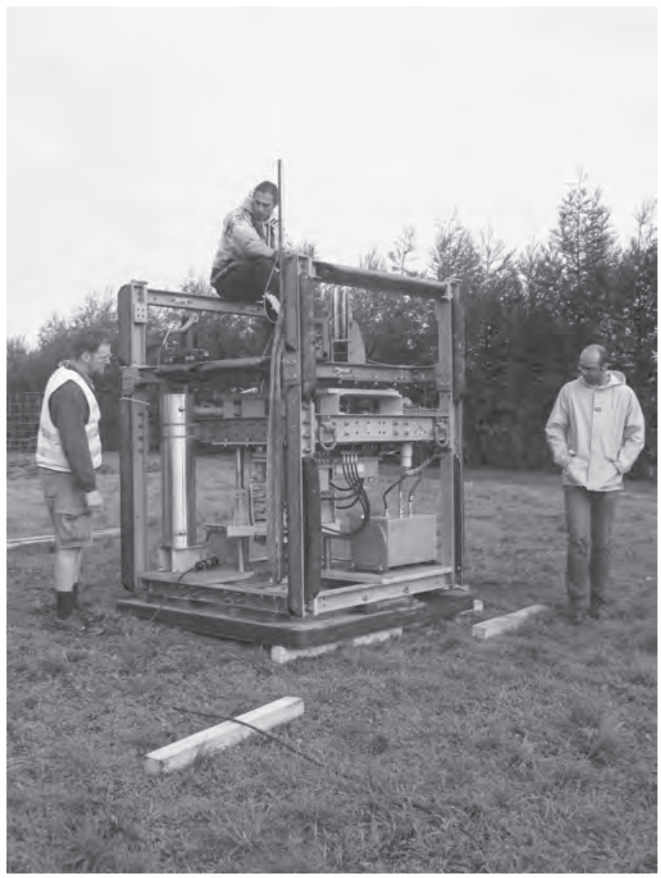

Figure 1: GOST is a frame-mounted CPT designed for deployment on the seafloor, but can also be used for terrestrial applications. Here GOST is being launched from a barge in Stella Passage, Tauranga Harbour (left), and is in operation on a landslide site near Pyes Pa, Tauranga (right).

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potentially liquefiable layers exist within the stratigraphy. This paper outlines the benefits of using a vibratory cone to more precisely identify the liquefaction-prone zones and hence obtain a better estimate of likely settlements.

Previous use of vibrocones

Sasaki *et al*. undertook initial development and testing of a vibropiezocone in 1984 in Japan. In this work they showed that the tip resistance of liquefaction-prone materials is significantly reduced by the tip vibration, accompanied by a dramatic rise in the induced pore water pressure (Tokimatsu 1988). Mitchell (1988) also recognised this effect in vibratory testing of sandy soil and suggested this as a rapid means of liquefaction potential assessment; similar results were reported by Moore (1987) in silty soils, and Wise et al. (1999) in sands. The methodology was further investigated by Bonita et al. (2004) who undertook laboratory tests on a variety of sandy materials and inferred that effective stresses within the influence of the cone were at or near liquefaction conditions; in effect, local liquefaction is induced in the sediment by the vibrating cone.

To quantify this effect, Sasaki *et al*. (1986) introduced the reduction ratio (RR) given by:

$$RR = 1 - q_{cv}/q_{cs} \qquad (1)$$

where

q_{cv} = vibratory cone penetration resistance, and
q_{cs} = static cone penetration resistance.

Reduction ratio values near unity indicate a marked drop in vibratory tip resistance compared with static tip resistance, whilst reduction ratios near zero indicate little to no change in tip resistance between static and vibratory modes.

However, since this early research the procedure seems to have been largely forgotten, and as far as we are aware GOST is now the only operating rig of this type presently available and in operation.

Field data

One example of a trace with liquefaction-prone sediments is shown in Figure 2. This is a sea floor trace immediately south of the Sulphur Point wharves in Stella Passage. Figure 2A shows the tip resistance and pore water pressure measurements from a static test, Figure 2B shows the estimated liquefaction potential determined by the method of Robertson and Wride (1998), and plotted as a cumulative Liquefaction Potential Index (LPI) using CLiq (www.geologismiki.gr/Products/CLiq.html) software. Vibratory tip resistance data from a neighbouring test are shown in Figure 2C, and the calculated reduction ratio derived from a comparison of the static and dynamic tip resistances is given in Figure 2D.

Interpretation

From Figure 2A we can identify three parcels of poorly-draining sediments separated by zones of higher tip resistance and much lower induced pore water pressures; intuitively, these might be interpreted as parcels of silty / clayey materials separated by free-draining sandy layers with

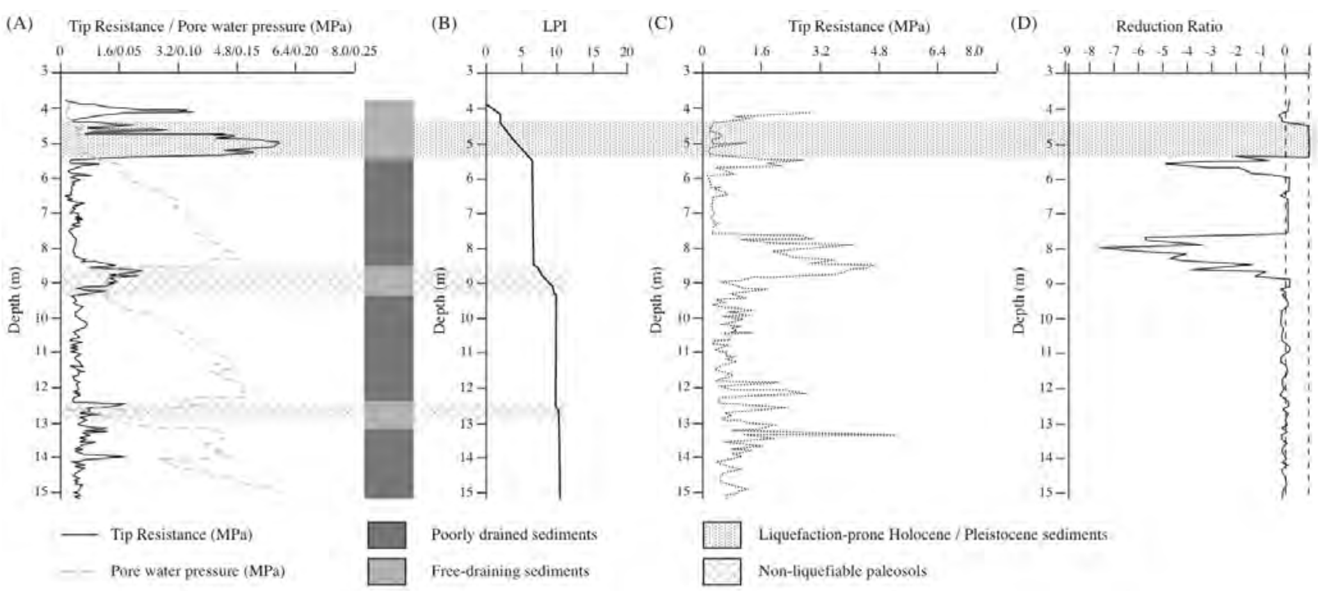

Figure 2: CPT results and interpretation from a test site in Stella Passage, Tauranga. (A) Static tip resistance and pore water pressure curves and broad interpretations. (B) Calculated liquefaction potential index (LPI) using the method of Robertson and Wride (1998). (C) Tip resistance from vibratory testing of the same sequence. (D) Reduction ratios for tip resistance calculated using equation (1). Note, depths are from a local datum, the traces start at the sea floor (approximately 3.9 m below datum).

increased tip resistance. If the LPI for these static results is determined using standard methods (Figure 2C), two zones of increasing liquefaction potential are identified: one at the top of the sequence (~ 4.5 – 5.5 m depth); and a second at approximately 9 m depth. A third, much smaller, increase in the LPI corresponds with the zone of reduced pore water pressure at approximately 12.5 – 13 m; this zone does not show a large increase in tip resistance.

Comparison with the vibratory tip resistance values (Figure 2C) shows that the tip resistance dramatically reduces in the upper part of the profile. Using the resistance ratio (Figure 2D) a zone from 4.5 – 5.5 m depth is identified as liquefiable, with a reduction ratio of close to 1 (almost complete loss of tip resistance in the vibratory mode). For the bulk of the profile, the reduction ratio varies only slightly about zero, indicating that the static and vibratory tip resistances are essentially equal. Note that we are ignoring the two zones of increased tip resistance with vibration (negative reduction ratio) for this purpose as these zones are clearly not liquefaction-prone.

In the stratigraphy of nearby boreholes, we see an upper layer of Holocene to Pleistocene sandy sediments extending to approximately 1.5 – 2 m below the sea floor. This is the material of relatively high static tip resistance and low induced pore water pressure that is identified as potentially liquefiable by both the standard LPI calculation and the reduction ratio from vibratory testing. At the 9 m depth however, the static LPI calculation suggests liquefiable materials, but this is not borne out by the reduction ratio results. From comparison with the local stratigraphy and on-land CPT traces, we infer that the material at this depth is a paleosol formed on ancient tephras. In this case the soil structure, which includes well-developed pedological features including cracks, leads to an increase in tip resistance (increased density) yet allows more ready drainage than the surrounding materials. This material is unlikely to liquefy. The zone at approximately 12.5 m is believed to be another paleosol, and hence does not show as liquefiable in the vibratory test procedure.

Conclusion

Prediction of liquefaction-prone materials by means of static CPT traces is recognised as being a simplified method based on empirical calibration and a number of assumptions; it is useful in providing an estimate of liquefaction potential in sandy soils (Robertson and Wride, 1998). In the example here, the addition of vibratory measurements has confirmed the likelihood of liquefaction in the sandy materials at the top of the profile (~ 5 m depth). The results of the two methods are in very close agreement.

However, other materials can show CPT results that mimic those from sandy soils; in the case considered here, layers believed to be structured paleosols show the same static CPT characteristics as the sandy layers, but have high resistance and free-draining characteristics due to a well-developed soil structure. Static CPT traces will identify these as liquefiable materials; by also considering the effect of vibration it is clear that liquefaction is unlikely in these layers.

Vibratory CPT provides a means of gaining a more precise indication of the extent of liquefaction-prone materials as the vibration effectively induces liquefaction, causing a dramatic drop in the tip resistance. Our experience so far indicates that static methods alone may overpredict liquefaction potential and thus provide a conservative estimate of the likely extent of liquefaction in a profile. Addition of a vibratory trace can more precisely discriminate those areas that are genuinely liquefaction-prone, avoiding unnecessary remediation costs.

GOST has returned to Germany at present, but will be back in NZ for a second research campaign in 2014 and is available for other applications. Contact Prof. Dr Tobias Moerz (tmoerz@uni-bremen.de) for further information.

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FOREIGN CORRESPONDENT

Naomi Mason

Occupation
Engineering Geologist
EBA, Vancouver

NEW ZEALAND WILL always be home but travelling is in my blood, so it was only a matter of time before I would leave. A secondment with SKM to Australia in 2010 really got the ball rolling and in 2011 I left New Zealand on a nine month trip, which included Hong Kong, the UK and Canada, to see where I wanted to end up. Vancouver was last on the list and it turned out to be the last stop in March 2012. With the help of a good friend and colleague I had a job within a week of landing, as an Engineering Geologist with EBA (a Tetratech Company) in the Pacific Geotechnical Group.

The experience I had gained in New Zealand such as working on stormwater and wastewater networks, being involved in tunnelling projects, and exposure to some of the Christchurch earthquakes assessment work all proved invaluable as these skills have definitely helped for what I do now, although the projects are much larger in Canada, along with everything else. I am lucky enough to have been sent to a variety of locations all over the British Columbia Province. Last winter I spent four months drilling for a new rapid transit line in Greater Vancouver. This summer I was out walking through forests (some areas which had never been explored before) conducting a Natural Hazards Assessments for a gas pipeline in Terrace, two hours flight north of Vancouver. Other projects have included investigating a highway widening in the Shuswap area, a quarry feasibility assessment in Fort Nelson, and a pavement assessment of a historic downtown area. A highlight of the job so far has been helicopter rides through glacier carved valleys and over tablelands still smouldering after a lightning storm. Occasionally I get a bit of ribbing about how small the towns I'm going to are, but usually they turn out to be huge compared to some of the smaller places in New Zealand.

Among the biggest differences I've had to get used to are the drilling methods for geotechnical investigations, some of which I'd had limited exposure to before coming to Vancouver. HQ/PQ core drilling in soil is almost never used in Vancouver; instead Mud Rotary and Auger drilling are the preferred methods. Becker hammer and sonic drilling are also frequently used and seismic studies are a big focus in Vancouver as the city is in a delta, so numerous SCPTs are commonly incorporated into the drilling programs. Standard core lengths are measured in inches and feet because most of the equipment comes from the US, yet Canada operates under the metric system so everything has to be converted. Luckily for me, all the drillers and contractors have been very patient and understanding as I de-evolve to their systems.

The other major change is wildlife awareness, something we don't really have to think about in New Zealand. Here, black, grizzly, and polar bears, cougars, lynx, wolves, coyotes, moose, and occasionally even snakes all pose a danger. The only wildlife training that is widely enforced is Bear Awareness training. There doesn't seem to be instruction on what to do if encountering other animals. Bear Awareness training teaches us that "Whoah Bear" is a necessary phrase one must learn while out on site, letting the bear know you mean it no harm if you accidentally find yourself in its space, and allowing you to back out. It turns out Canadians have been hiding the fact they have 'Drop Bears'. I recommend looking up Black and Grizzly bears climbing trees on YouTube, they're pretty quick! There is some solace in carrying bear spray (basically mace for bears), however having accidently spraying myself, and suffering limited effects I personally think it would just aggravate the bear further. If I encountered one I'm not confident that I would avoid getting eaten even armed with "whoah bear", bear bangers, bear spray and training. Hopefully I will just be able to outrun one of my colleagues.

Overall, Vancouver has been the right move, with adventures and work challenges that motivate me to stay here. I just wish I could get a decent mince pie for lunch!

Above: Field work in Canada can be as impressive as home in New Zealand

BOOK REVIEWS

Hydrogeological Conceptual Site Models – Data Analysis and Visualization – Neven Kresic and Alex Mikszewski

AS THE AUTHORS state in the preface "while this book is technical in nature, equations and advanced theoretical discussions are minimized, with the focus instead placed on key concepts and practical data analysis and visualization strategies". Nevertheless the subject is covered with a strong technical and theoretical foundation.

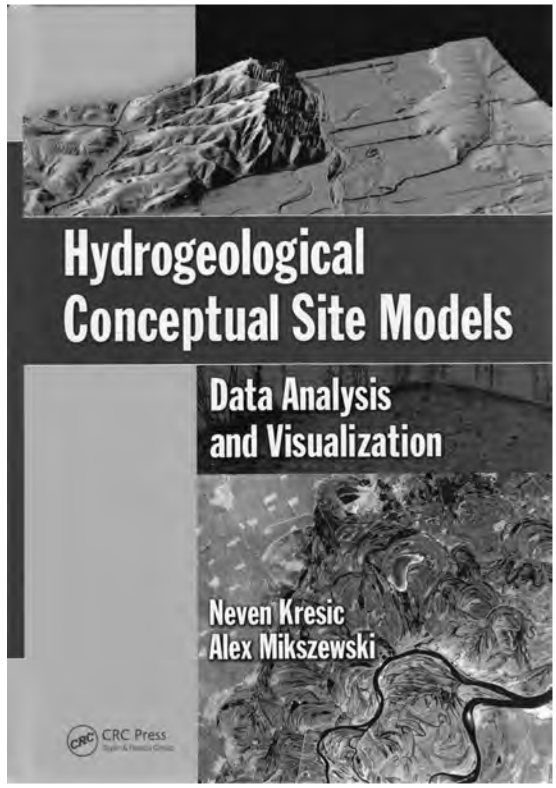
The book is addressed to a very wide audience. As an extensive range of topics is covered, both young and more experienced consultants, regulators, and attorneys will benefit from the clear text and illustrations. The topic selection is excellent and well structured, while the transition between the topics is logical and comfortable as it follows the natural course of most hydrogeological projects.

The purpose of the book ... is to fill the gap between theory and practice

The authors, using the familiar story of Dr. Snow, the "father" of modern epidemiology, give a particularly good illustration of how the use of simple visualization methods coupled with the proper analysis techniques can "bolster a study and potentially win over even the most ardent miasma believers". Using a highly structured layout the reader is guided through the definition and applicability of conceptual hydrogeological models, data management through the use of GIS, contouring methods and techniques, groundwater modeling and visualization, site investigations tools and practices, groundwater remediation methods and the development of a groundwater source for water supply.

Throughout the book, the use of new computer techniques is illustrated in examples using a variety of popular commercial software. On a number of occasions the authors demonstrate very successfully how the misuse of very powerful software by inexperienced practitioners may result in questionable and potentially disputed results and set out the steps that should have been followed in the analysis. A very useful companion DVD includes animations, reference material, modeling software, and more.

The purpose of the book is not to become, or even replace one of the standard hydrogeological textbooks. It is to fill the gap between theory and practice; to identify and explain key concepts that the professional hydrogeologist, the client or the regulator face in their everyday practice; to guide them through practical examples and clear illustrations over the majority of hydrogeological applications and to demonstrate available tools and applicable methods that will be essential to the success of any project. Overall the authors deliver on their promises, and I am confident that most practitioners in geotechnical engineering will greatly benefit from reading this book.



Reviewed by Theo Sarris
Senior Hydrogeologist, Beca Ltd

Title	Hydrogeological Conceptual Site Models – Data Analysis and Visualization
Author	Neven Kresic and Alex Mikszewski
Publisher	CRC Press
Year Published	2012
Hardback	600 pp
ISBN	978-1-4398-5222-4
Web shopping	www.crcnetbase.com/isbn/9781439852286 (ebook)
Price	USD 150

Active Faults of the World – Robert Yeats

THIS IS A high quality book, well-written with many informative diagrams, useful maps and geological sections. There are many references and a comprehensive index. It is suitable for both academics and students and is particularly good for engineering geologists, civil engineers and planners. It is also useful reference for analysts and consulting firms.

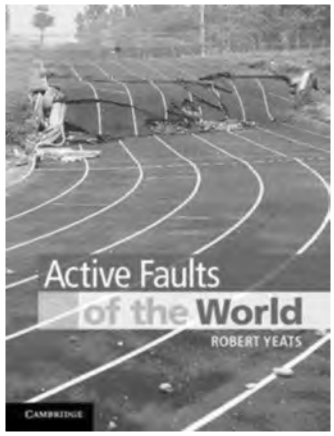
The author notes that the study of faults requires the understanding of several areas of geology, including tectonics, plate tectonics, structural geology, geodesy etc. He considers regional seismic hazards and active earthquake faults worldwide and then puts them into a regional seismic and plate tectonic context.

Many faults and types of fault from around the world are discussed as are their associated volcanoes and microplates. All the usual suspects turn up: San Andreas, East African Rift, Indonesian thrust, Red River etc. Followed by a discussion on how the earthquakes can be dated; e.g. from historical records to 14C dating to correlation with dated tephra. Histories of some plate movements are given (e.g. the Caribbean, N and S American plates) together with their associated faults.

This fascinating and comprehensive survey for the general geological reader, the planner and engineer facing the daunting challenge of construction in unstable regions.

The book treats South America and the Andes at some length, but does not neglect the other remnants of Pangea, including Europe. The Dead Sea Fault is discussed in considerable detail. The author focuses on the effects of earthquakes (including tsunamis) and on faults with the potential to destroy large cities both in the developed and developing world.

It then moves on to the value of planning for large-scale projects such as nuclear power plants, hydroelectric dams and oil pipelines. It provides an important basis for upgrading building standards and other laws in developing nations.



It looks at the impact of major quakes on social development through history. Two examples are the 1505 earthquake in Kabul, which killed hundreds of people, and the quake which probably caused the destruction of the biblical cities of the plain, Sodom and Gomorrah.

This fascinating and comprehensive survey provides interest for the general geological reader, the planner and the engineer facing the daunting challenge of construction in unstable regions. Highly recommended.

Reviewed by Steve Rowlatt

Reprinted courtesy of the Geological Society of London. This review was originally published in Geoscientist Volume 23 No 7, August 2013

Title	Active Faults of the World
Author	Robert Yeats
Publisher	Cambridge University Press
Year Published	2012
Hardback	621 pp
ISBN	978-0-521-19085-5
Web shopping	www.cambridge.org
Price	GBP 50

MEMBERSHIP PROFILE

A NUMBER OF people have expressed interest in who makes up the membership of the NZGS. The Secretary has provided the following information to help. Some caveats must be noted; not all NZGS members state the organisation with which they are affiliated, so the numbers here do not include these individuals. The data is based on the most recent membership lists, so will not take into account any recent movements of individuals between branches or organisations. This data should not be taken to show the total number of geo-professionals in each organisation listed, as there will be people who are not NZGS members or for whom we do not know their correct affiliation.

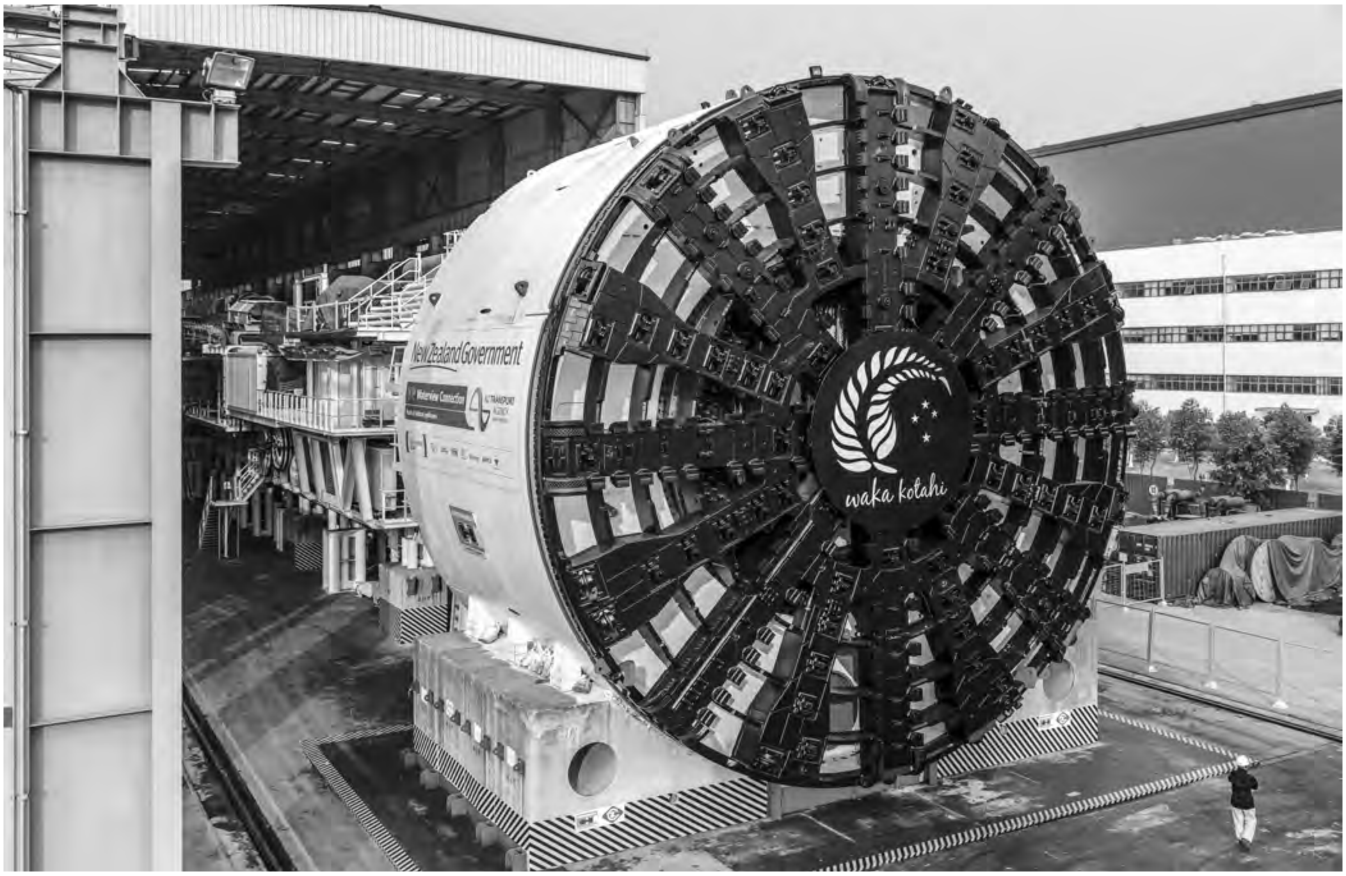
	Auckland	Canterbury	East Coast	Hawkes Bay	Manawatu	Nelson-Marlborough	Northland	Otago	South Canterbury	Taranaki	Waikato-Bay of Plenty - Hamilton	Waikato-Bay of Plenty - Rotorua	Waikato-Bay of Plenty - Tauranga	Wanganui	Wellington	West Coast	Total
Branch Total	404	246	1	13	4	26	15	27	1	7	55	5	36	3	113	7	963
Breakdown by organisation																	
Tonkin & Taylor	48	17				3		3			1		3		16		91
Opus	11	10		4			1	6		1	9			1	12	1	56
Beca	30	11											3		4		48
Coffey	13	22									3		6		2		46
AECOM	21	8									4				3		36
Aurecon	3	12									1		2		8		26
MWH	2	6			1	5		2			4				3		23
Riley	11	8															19
Geoscience Consulting Ltd	3	10													5		18
Auckland University	18																18
SKM	13	4															17
Soil and Rock	9	5															14
Golder	2	6				2		1									11
URS	8	2															10
University of Canterbury		9															9
KGA Geotechnical	7	2															9
Geotechnics	4	1											1		2		8
Fletcher	4	2									1				1		8
GNS Science								2							5		7
Pattle Delamore Partners	5	1													1		7
Geoconsult	5	2															7
Damwatch															6		6
Auckland Council	6																6
Fugro		5															5
Engineering Geology	5																5
GHD	2	1													2		5
4 people or fewer *	184	105	1	9	3	16	19	13	1	6	32	7	23	2	47	6	471

• Due to space limitations organisations with four or fewer known NZGS members have been amalgamated. If there is demand for more in-depth industry information, please contact the editors. We can facilitate the collation and publication if there is wide interest and buy-in from the organisations involved.

Prepared by: Ross Roberts, NZ Geomechanics News Co-editor

COMPANY PROFILE

Parsons Brinckerhoff



Above: Alice the TBM, Waterview Connection

Above: Te Mihi Geothermal Plant, Taupo

PARSONS BRINCKERHOFF WAS founded by William Barkley Parsons more than 125 years ago in the United States. Parsons designed New York City's first subway which was completed in 1904. Today Parsons Brinckerhoff is a global multi-disciplinary consulting firm with over 14000 staff and 150 offices across all five continents. In 2009, Parsons Brinckerhoff became the professional service division of Balfour Beatty, the international infrastructure group operating in professional and construction services and infrastructure investments. The acquisition brings to Parsons Brinckerhoff a whole new range of capability and services that can be offered to clients.

Parson Brinckerhoff operates on a regional operation model. The NZ geotechnical business is part of the Australia-New Zealand region, which also covers the Pacific. The geotechnical business has a relatively short history in New Zealand, but has already played a key role on some of the country's most high profile projects in recent years.

Perhaps Parson Brinckerhoff is best known in New Zealand for its role on the Waterview Connection Project. The Waterview Connection is the final link in the completion of a motorway ring route around Auckland –

and is one of the government's seven Roads of National Significance. At the heart of this 5 km link will sit twin 2.5 km, 14 metre diameter bored tunnels. The Tunnel Boring Machine (TBM) used for this project is affectionately known as Alice, having been through a competition run for local schools. Alice is the world's 10th largest TBM, and the largest ever to have been used in the southern hemisphere. Parsons Brinckerhoff has undertaken the civil, geotechnical, structural and M&E design for the tunnels as part of the Well-Connected Alliance – a multi-disciplinary team consisting of the NZ Transport Agency, Fletcher Construction, MacDow, Obayashi, Beca, Parsons Brinckerhoff and Tonkin & Taylor.

Locally we have a strong team of geotechnical and tunnel engineers and geologists with solid knowledge of New Zealand conditions and geology, and a blend of international experience. The team recently completed the geotechnical design for the 166 MW twin turbine geothermal power station at Te Mihi. Our role encompassed the development, from additional site investigations, through the detail design of the earthworks and the foundations which included over 400 piles. As part of the joint venture we also took responsibility for the construction monitoring.

Other recent multi-disciplinary projects include the Ngatamariki Geothermal Plant, Ngaruawahia Bypass Project and SH16 Causeway Tender Design.

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NOTICE BOARDS

Around the Office

'Around the Office' is a collection of humorous snapshots spotted around the offices of NZGS members. Please send any suitable material to the Editors.

Above: A stuffed predator sabotaging the office garden (look closely)

Above: Innovative ground anchor protection technology

Above: "Honestly boss, it was the biggest roller they had"

PERRY
GEOTECH

MEMBER PROFILE

Ross Roberts

Occupation
Engineering Geology Team Leader
SKM
Auckland

NZ Geomechanics News Co-editor,
ross.c.roberts@gmail.com

HOW MANY SCHOOL kids aspire to be engineering geologists? I haven't met any yet, which I like to think makes me unusual; engineering geology became of interest to me at the tender age of 14. I was lucky enough to be at a school with two qualified geology teachers, quite a rarity in the UK, and one of them had a passing interest in engineering. When choosing my study subjects the prospect of spending my school days walking the hills and coastlines of northern England and Wales appealed far more than my other option of turgid Tudor history, and so the path for the rest of my life was set. Eight years later I had a qualification in engineering geology, slightly frostbitten hands as a result of four years in the frigid winds of Edinburgh, and somehow avoided liver damage as a result of a year in Newcastle, the drinking capital of England.

Since then I've enjoyed the variety that engineering geologists often take for granted. Every day I feel a little sorry for our friendly traffic engineers forever watching imaginary cars on their computer screens, and our tunnelling engineers with their pallid faces many too years spent in the dark.

By contrast I've managed to be involved in slope stabilisation in Java, wharf construction in Darwin, piling in Ireland, ground investigation in Sumatra, bridge building in England, and surfing in New Zealand. Sometimes I manage to fit work in around these activities. I'm now team leader for a great group of engineering geologists at SKM in Auckland, and have no regrets about coming here.

In my experience the phrase 'a picture tells a thousand words' is even applicable to geotechnical work, so rather than write too much more I've put in photographs of some of my personal highlights. I hope you enjoy them.

Above: Installing the first beam on one of four new viaducts built to bypass the existing grade-separated roundabout between the A2 and M25 in Kent, near London. We also widened the A2 to three lanes each way, and the M25 to four lanes each way. This required chalk earthworks up to 30 m high, ground improvement using lime and cement, CFA piling, sheet piling, reinforced earth structures, soil nailed cuttings, and micro-tunnelled drainage. The scheme covered approximately 10 km of motorway grade road. I was designers' site representative responsible for monitoring, redesigning (in some cases!) and approving all geotechnical aspects of the scheme. Those rusty old beams are brand new - it's a design feature.

Above: Jungle investigation in Sumatra for a new geothermal field. It was a four hour off-road hike to this borehole, and the crew had to carry the plant in by hand.

Right: Tramping and rock-fall assessment in the Port Hills. It was fun work, but really drove home the impact of getting geotechnical decisions wrong.

Above: This large wharf in Darwin will eventually have to cope with ships to service the offshore gas industry. The 8 m tidal range means the wharf wall is 17 m high, hence the very large capping beam and anchors.

Above: Studying landslides in Java was a fabulous experience, even in the wet season. Designing stabilisation measures for such tricky slopes was challenging, but very liberating being free of design codes! As you can see, previous attempts had failed so in this instance I scoured the terrain for alternative alignments for the damaged infrastructure.

Riley Gerbrandt

Occupation
Geotechnical Engineer
Opus International Consultants Ltd, Napier

NZGS Hawkes Bay Branch Coordinator

STEPPING OFF THE airplane in Auckland with my wife and 18 month old daughter in tow was quite surreal. It was early in the morning on a late October day in 2011, and we had just survived the flights from San Francisco to LA to Auckland with hundreds of other bleary-eyed passengers. With some merciful assistance from a nice Auckland airport worker who told the x-ray operator that he had some "new Kiwis" to guide towards the exits, we successfully collected our seven large bags and stepped into the International Arrivals greeting area and our new home in New Zealand.

As many others in the New Zealand geotechnical industry can attest, moving your family to your new home overseas is quite an experience. We have met some wonderful people along the way during our past two years here in New Zealand, not the least of whom is our wonderful former pastor who drove up from Hamilton to collect us and our mound of bags from Auckland that October morning in 2011. But it has been well worth it!

Working in New Zealand has been a wonderful professional experience. Whilst it took me many months to get up to speed with the local geology, different codes/standards and some innovative Kiwi designs, my eyes have been opened to a different world than I had previously experienced. Whilst working from Hamilton, I had the opportunity to work on some very diverse projects and encounter some very unique geology. I encountered volcanic deposits whilst working on new transmission line foundations being constructed in Taupo, compressible peat deposits near Te Awamutu and Hinuera formation deposits along the Waikato River in Hamilton. I worked hard alongside my other team members, and will fondly remember many of those experiences.

Earlier this year, an opportunity came my way to join the Opus geotechnical team here in Napier, and my family shifted (yet again … haha) to the "sunny" Hawke's Bay. The opportunity has been great for my professional growth, as I have been able to learn from some very skilled engineers and to engage in a wide variety of projects, such as liquefaction studies, fault investigations and land development assessment.

Seismic analyses and land development assessments bring me back to my earlier professional experiences back in California. I started my professional career back in 2006 on the California Central Coast. I worked largely on smaller residential land development projects and commercial structures. After meeting and marrying my wonderful wife, I moved up to the San Francisco Bay Area amidst the global financial crisis and worked for a medium-sized geotechnical consultancy. While practicing in what is known to locals as the "East Bay", I had the opportunity to work on some larger land development and public infrastructure projects, which broadened my professional experience.

During the past several months, I have had the pleasure of starting up the Hawke's Bay Branch of NZGS. The new Branch has started off well with the goals to connect local geo-professionals, engage in vibrant discussions on issues affecting the geotechnical industry, to enhance our professional knowledge and to present opportunities for networking and fun. Our first Branch meeting, which was graciously hosted by RDCL last month, was a huge success. RDCL staff presented on Multichannel Analysis of Surface Waves (MASW) seismic testing as well as downhole geophysical testing. I am proud to announce that we are planning to hold our second meeting prior to Christmas, and I hope to foster more positive engagement between the local geotechnical industry and the wider public. Hawke's Bay Branch information and news, including upcoming Branch meeting information and recaps on recent Branch meetings, is available at http://www.nzgs.org/branch/HawkesBay.htm.

Growing up the redwood forests and sand hills of the Santa Cruz Mountains (on the coast about 1.5 hours southwest of San Francisco), I never would have imagined the journey my career would take. While I was interested in "geotechnical engineering" from a young age and fondly remember building dams in mountain streams and examining fascinating rocks with my grandfather, it would be hard to imagine that I would earn a BSc and MSc in that field and practice in geologically active New Zealand. But 30 years on, here I am!

EVENTS DIARY

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

2014

28 January – 7 February, 2014
University of Auckland, Auckland
Engineering Geological Mapping
http://web.env.auckland.ac.nz/course_pages/geology701/

28-30 March 2014
Sofia, Bulgaria
Balkan Speleological Conference
http://www.balkan-speleo-2014.eu/eng/home.html

7-9 May, 2014
Afyonkarahisar , Turkey
ROCKMEC'2014-XIth Regional Rock Mechanics Symposium
http://www.kayamek.org/
CPT'14 International Symposium on Cone Penetration Testing

12-14 May, 2014
Las Vegas
http://www.cpt14.com/

21-23 May, 2014
Stockholm, Sweden
International Conference on Piling and Deep Foundations
http://www.regonline.com/builder/site/Default.aspx?EventID=1221506

27-29 May, 2014
Vigo (Spain)
Rock Mechanics and Rock Engineering: Structures on and in rock masses
http://www.eurock2014.com/

1-4 June, 2014
University of Minnesota, Minneapolis, USA
48th US Rock Mechanics/Geomechanics Symposium
http://www.armasymposium.org/

2-6 June, 2014
Beijing, China
World Landslide Forum 3 (WLF3)
http://www.wlf3.org/

17-26 June, 2014
Albena Resort & SPA, Bulgaria
14th INTERNATIONAL MULTIDISCIPLINARY SCIENTIFIC GEOCONFERENCE & EXPO SGEM2014
http://www.sgem.org/

9-11 July Sky City Auckland
12 July in Christchurch
Second Australasian Structural Conference on Earthquake Engineering
http://www.asec2014.org.nz/

NEW ZEALAND GEOTECHNICAL SOCIETY INC.

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

ADVERTISERS DIRECTORY

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	$100
Students	Free
Annual IPENZ service centre fee applies to all NZGS members who are not members of IPENZ	$43.70

Affiliation fees for International Societies are in addition to the basic membership fee:

International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE)	$35.00
International Society for Rock Mechanics (ISRM)	$35.00
International Association of Engineering Geology & the Environment (IAEG)	$35.00
(with bulletin)	$80.00

All correspondence should be addressed to the Management Secretary. The postal address is:

NZ Geotechnical Society Inc, P O Box 12 241, WELLINGTON 6144

The Secretary
NZ Geotechnical Society Inc.
The Institution of Professional Engineers New Zealand (Inc)
P.O. Box 12-241, WELLINGTON 6144

NEW ZEALAND GEOTECHNICAL SOCIETY INC

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APPLICATION FOR MEMBERSHIP

(A Technical Group of the Institution of Professional Engineers New Zealand (Inc))

FULL NAME Dr/Mr/Mrs/Ms/Miss (Underline Family Name): ....

HOME POSTAL ADDRESS: ....

Phone No: ( )........................... **Cell Ph:** ( ) .... **Fax No:** ( ) ....

E-MAIL: Home... **E-MAIL:** Work..

DATE OF BIRTH ....

ACADEMIC QUALIFICATIONS: ....

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

PRESENT EMPLOYER: ....

WORK POSTAL ADDRESS: ....

OCCUPATION: ....

EXPERIENCE IN GEOMECHANICS: ....

STUDENT MEMBERS: ....

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

SUPERVISORS SIGNATURE: ....

Preferred email (please circle): home/work **Preferred address:** home/work

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 ....

ADVERTISING INFORMATION

NZ Geomechanics News is published twice a year and distributed to the Society's 800 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.

BLACK AND WHITE	COST	SIZE
Full page	$310/issue	210mm x 297mm high
Half page	$240/issue	90mm wide x 265mm high
Quarter page	$210/issue	90mm wide x 130mm high
COLOUR		
Full page A4	$550/issue	210mm x 297mm high
A3 position	$1035/issue	420mm wide x 297mm high
Half page (vertical)	$275/issue	90mm wide x 265mm high
Half page (horizontal)	$275/issue	210mm wide x 148mm high
COVER PLACEMENTS		
Inside front & back and outside back cover	$830/issue	210mm x 297mm high
FLYERS/INSERTS		
Insert to be posted with magazine	$275/ for A4 single page flyer	

(For other size inserts the price is dependent on weight of insert. Price provided on request)

(advertiser to provide the insert at their own cost)

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.
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: secretary@nzgs.org